UNIVERSITY OF NOVA GORICA
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CONSERVATION OF HISTORICAL MASONRY ARCH BRIDGES A PROCEDURE OF MODELLING AND STRENGTHENING

DISSERTATION

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Table of contents

Abstract

Keywords: *Conservation, Masonry arch bridge, Structural modelling, Multi-scale analysis, Strengthening*

Masonry arch bridges are a remarkable evidence of the engineering progress and the technological achievement and skills the mankind has developed over the centuries: they are an essential part of the architectural historical heritage. Their presence is a characteristic feature of the Italian and European landscape. The European railroad networks include thousands of masonry arch bridges, mainly built during the XIX century, that are still in exercise. The present rail traffic is heavier than in the past. While in case of monumental bridges, which its historic value is recognised, the performance requirements may be sacrificed in order to ensure the conservation, in case of masonry arch railway bridges their conservation is guaranteed by their functioning. The capacity of historical masonry arch rail-bridges to carry the actual traffic, as well to respect the actual standard, must be verified. Considering the great number of those bridges, the aim of the thesis is to propose a methodology of analysis that must be reliable and fast.

An evaluation of safety and durability of masonry arch bridges with the aim of their conservation and restoration is here presented. The thesis provides a state of art regarding the methods of analysis and the techniques of structural modelling of masonry arch bridges, outlining the different approaches, the fields and the limits of applicability. An overview of the frequent damages and the relative common repair and strengthening interventions is given, in order to discuss problems and opportunity of conservation. Then a procedure for structural analysis based on a multi-scale approach is applied to a case of study, the Venice Trans-Lagoon rail Bridge. Different types of models realised at diverse scales and with various details are analysed in order to evaluate their applicability and reliability. The thesis focuses on the structural behaviour under service conditions, discussing the response of the bridge due to normal traffic loads and the methods of assessment.

Povzetek

Ključne besede: *Konservatorstvo, Zidani ločni mostovi, Strukturno modeliranje, Večnivojska analiza, Ojačevanje*

Zidani ločni mostovi so izjemen dokaz napredka v gradbeništvu in tehnološkega dosežka ter spretnosti, katere je človeštvo razvilo skozi stoletja: so bistveni del arhitekturne zgodovinske dediščine. Njihova prisotnost je karakteristična za italijanske in evropske pokrajine. Evropska železniška omrežja vključujejo na tisoče zidanih ločnih mostov, ki so zgrajeni predvsem v XIX stoletju in so še vedno v uporabi. Sedanji železniški promet je težji kot v preteklosti. Medtem ko je, v primeru monumentalnih mostov, zgodovinska vrednost že priznana in se zahteve učinkovitosti lahko žrtvujejo za zagotovitev ohranjanja le teh, v primeru ločnih železniških mostov, njihovo ohranjanje jamči prav njihovo delovanje. Zmogljivost zgodovinskih ločnih železniških mostov za potrebe dejanskega prometa, kakor tudi za zahteve trenutnih standardov, pa je potrebno še preveriti. Glede na veliko število takšnih mostov, je cilj disertacije izdelava metodologije analize, ki mora biti zanesljiva in hitra.

Disertacija predstavlja oceno varnosti in trajnosti ločnih zidanih mostov, ki zagotavlja predvsem njihovo ohranjanje in obnovo. Disertacija predstavlja trenutno stanje na področju analitskih metod in tehnik strukturnega modeliranja zidanih ločnih mostov, s povdarkom na različnih pristopih, področjih in mejah uporabnosti. Disertacija podaja pregled najbolj pogostih poškodb, popravil ter ojačitvenih posegov, ki so potrebni za razparavo o problemtikah in priložnostih za konservatorstvo. V nadaljevanju je podan postopek za strukturno analizo, ki temelji na večnivojskem (*multi-scale*) pristopu, ki je izveden na študijskem primeru vstopnega železniškega mostu v Benetkah, ki prečka laguno. Analizirane so različne tipologie modelov za oceno njihove uporabnost in zanesljivosti, ki so izvedene na različnih ravneh in z različnimi podrobnostmi. Disertacija se osredotoča na strukturnem vedenju v pogojih obratovanja, z analizo odziva mostov kot posledica normalnih prometnih obremenitev in izbranih metod ocenjevanja.

Introduction

The conservation of masonry arch railway bridges

The masonry arch bridges are among the most ancient and best preserved historical structures. Their construction dates back to the dawn of the history, their development has gone hand in hand with the technological advancement. They were the most advanced structures until the twentieth century, when the new structural materials - steel and concrete - definitely superseded the masonry. They are a remarkable evidence of the engineering progress and the technological achievement and skills the man has developed over the centuries: they are an essential part of the architectural historical heritage. Their presence is a characteristic feature of the Italian and European landscape, as well as of principal cities, which often born and developed in correspondence of bridge. For this reason masonry arch bridges may have a considerable cultural value beyond their immediate functional purpose, that should be recognised and need a special stewardship by those responsible for their upkeep.

An evaluation of safety and durability of masonry arch bridges with the aim of their conservation and restoration is here presented. The purpose is to design interventions of strengthening in the respect of architectural and cultural values. In case of historical masonry structures there are many uncertainties and the risk is to do not properly understand the structural behaviour. This could bring to unnecessary or over dimensioned interventions. Therefore it is necessary to define a methodology of analysis able to consider all the aspects.

The thesis provides the state of art of the analysis and modelling of masonry arch bridges, outlining the several approaches, the fields and the limits of applicability. An overview of the frequent damages and the relative common repair and strengthening interventions is given, in order to discuss problems and opportunity of conservation. Then a procedure for structural analysis based on a multi-scale approach is applied to a case of study, the Venice Trans-Lagoon rail Bridge. Models with various details and realised at diverse scales are analysed in order to evaluate their applicability and reliability. The target is to propose guidelines for the structural analysis of masonry arch bridges.

The subject is topical. The Italian railroad network includes thousands of masonry arch bridges, mainly built during the XIX century, that are still in exercise. This is also common in other European countries, such as in the United Kingdom, France, Spain and Germany[1]. The present rail traffic is heavier than in the past. In particular, starting to the second half of the XX century the number and the frequency of trains running on the railroad has considerably growth, as well as their weight and velocity had been consistently increased. For these reasons technical regulations about rail bridge[2] increased the overloads that must be considered in the design of new rail bridges. These overloads have to be applied to all the bridges belonging to the Italian railroad network, thus they have to be taken into account even for the evaluation of existing bridges. Present loads and dynamic stresses were not expected in the XIX century. Moreover, at the time of their construction masonry arch bridges have been designed relying on geometrical and empirical rules. The capacity of historical masonry arch rail-bridges to carry the actual traffic, as well to respect the actual standard, must be verified. Considering the great number of those bridges, the aim of the thesis is to propose a methodology of analysis that must be reliable and fast.

While in case of monumental bridges - such as the historical bridges of many European towns, or any bridge which its historic value is recognised by statutory designation, such as listed building or scheduled monument status - the performance requirements may be sacrificed in order to ensure the conservation, in case of masonry arch railway bridges their conservation is ensured by their functioning. Their survival is related to their adequacy to serve their original function: to carry the current loading providing the required structural performances. It is necessary to find an equilibrium between the diverse needs: on one hand the preservation of the historical value, on the other the satisfaction of the actual standards. It is a very wide

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[1] A report about the number of masonry arch bridges belonging to the railways company as been provided by [UIC, 2005]. In italy the percentage of masonry arch bridge and culverts in the railways network is about 63% of the overall bridge stock [Weber, 1999]. More information about this topic will be given afterwards in the introduction and in the first section of the thesis.

[2] In Italy regulation n° I/SC/PS-OM/2298 provided in 1995

issue, which includes several aspects: from the functional adaptation to the repair and strengthening, from the seismic retrofitting to the protection from natural hazards. A key aspect is the conservation state: bridges having a bad state of conservation or affected by severe damages has to be tackled with particular attention. However, the 85% of the masonry arch bridges belonging to the different European railroads network are in a good or average state of conservation[3].

The thesis focuses on the structural behaviour under service conditions, discussing the response of the bridge respect to normal loading and the methods to assess them. Only with a complete understanding of the structural behaviour of the bridge it is possible to design and plan appropriate strengthening interventions sensitive to the important heritage features of the structure, in order to reach that compromise between the preservation of the historical character of the bridge and its adequacy to remain in service, which is the only guarantee of its conservation.

[3] UIC Report, 2005.

Structure of the thesis

The thesis consist of three sections:

Section 1 - The morphology of masonry arch bridges

The first section concerns the masonry arch bridges and describes their morphology. A brief history of the evolution of this typology of bridges and their constructive features is provided. The attention is focused on railway bridges. The number of masonry arch railways bridges that are still in service is given, in order to outline the dimension of the problem of their conservation. Then structural elements of a masonry arch bridge and materials which are made of are described.

A deeper knowledge of technology and constructive typologies of masonry arch bridges is fundamental to understand their structural behaviour, to identify damages and structural problems that may affect them, to take appropriate measures for their strengthening and conservation.

Section 2 - Modelling and analysis

In the second section a complete state of art of the modelling of masonry arch bridges is given. The aim is to evaluate which approach may be more appropriated for the analysis of masonry arch railway bridges with the purpose of their study and conservation. The section is organised in three part corresponding to the different structural levels which a masonry arch bridge may be analysed:

- at the local level - the material;
- at the substructural level - portion of structure, the masonry arch;
- at the structural level - the masonry arch railway bridge.

The idea is to connect the different analysis of the diverse levels in order to perform a consistent multi-scale analysis. The structural behaviour of each level and the global behaviour of the whole bridge are discussed, with an overview of the main strategies of analysis and modelling in literature.

Section 3 - Damages and strengthening

The third section a complete state of art of the strengthening of masonry arch bridges is given. The aim is to have an overview of the available techniques of consolidation in order to evaluate which are more appropriate for the conservation of masonry arch railway bridges. Therefore it is necessary to outline the most common damages and the main problems which may affect bridges. The section is divided in two part:

- Deterioration, damages and performances decay - regards the problems of ageing and adjustment of masonry arch bridges and includes a catalogue of the most common damages;
- Maintenance, repair and strengthening - describes the treatments to repair the damages and to improve the performances of bridges, in order to ensure their serviceability in the respect of the conservation needs.

A complete overview of problems and solutions are on one hand essential to evaluate treats and opportunity for conservation and on the other useful in the choice of the strategy of modelling and analysis.

Case of study - Multi-scale analysis of Venice Trans-Lagoon Bridge

An example of modelling and analysis of masonry arch bridge is proposed in the case study. History and the constructive features of the historical Venice Trans-Lagoon Bridge which connects Venice to its mainland are investigated. A multi-scale approach is applied to the modelling and analysis of the bridge. The purpose is the evaluation of its service behaviour and the reliability of the different modelling approaches. An evaluation of strengthening applied to backfill is presented. Multi-scale analysis is coupled with an homogenisation procedure, to better characterise the mechanical characteristics of masonry material. An evaluation of the effect due to the real structural form on the global behaviour of bridge is presented, investigating the effects due to the presence of external stone arch rings.

Section 1
Morphology of masonry arch bridges

Introduction

This section concerns the typology of masonry arch railway bridges. The aim is to provide an overview of their principle characteristics: the constituent elements and the materials which are made of. In fact, the knowledge of the historical structures is the first fundamental step in order to be able to correctly model them an to reach a complete understanding of their behaviour.

First it is outlined the historical framework of masonry arch bridges, giving a brief excursus of the constructive evolution of this bridge typology. The history of masonry arch bridges is very long and interesting: wonderful bridges have been built by the Romans, subsequently their construction suffered a decline after the disintegration of the Roman Empire and then begin again from the middle age with a constant increase form the renaissance to the 1700. In the eighteenth century the french and english engineering provided great technological development, and in the nineteenth century masonry arch bridges reached their maximum progress. The attention will be precisely focused on the modern masonry arch bridges built in the period from the first half of the nineteenth century to the 30's of the twentieth century in order to realise the Italian railway network.

In fact, although in that period the new structural materials - concrete and steel - were becoming to overcome the masonry for the construction of structures, the great majority of railway bridges have been realised with the structural principle of the masonry arch. In the 30's of the twentieth century, when the railway network was almost completed, masonry structures were superseded by steel and concrete structures. This led the study of masonry structures, and in particular the study of stone arch bridges, to a state of neglect for several years. As a result, the technological knowledge developed during their long history have been lost. The modern theory of structures focused is attention to the study of the new material, steel and concrete, which were more suitable to be studied with the new instruments provided by the theory, in the academies the study of masonry was disregarded.

However, their study has been considered again essential when, in the decades after the II World War the rail traffic considerably increased and the trains became more weight and fast. The evaluation of their safety and load-bearing capacity began of fundamental importance, because of the elevated number of masonry arch bridges belonging to the network. It is estimated that there are about 200’000 masonry arch bridges in service in Europe. More detailed number will be provided in this section in order to emphasise the dimension of the problem of their conservation.

For this reason the study of masonry arch bridges is first of all the knowledge of this structural typology, of its morphology, materials, geometrical dimensions, and technical aspects. The topic is very wide and the literature is huge, in this first section just an overview of this aspect is provided. Attention is paid to masonry arch railway bridges.

1.1 History and typological evolution of masonry arch bridges

The birth and development of the bridges are important moments in the evolution of civilisation. Bridges play a key role in the relationships between people and social groups, they are a core element of the civil development and the road network of an area. Their diffusion testifies the wealth in trade and communication and the technological progress, it is an indicator of the economical and social boundary condition of an area and of an epoch. It is assumed that early bridges, although primordial and consisting of a single trunk placed to cross a stream, have been built very far in the history, in the same period in which men made their first shelter from the elements.

The historic bridge *par excellence* is the stone arch bridge, which is a characteristic element of Italian and European landscape. The majority of existing bridges are masonry arch bridges, their presence is typical in many towns both Italian - Roma, Firenze, Torino, Verona, Venezia - and European - Paris, London, Madrid, Budapest and many others. The masonry arch bridge born as a durable solution when "*(men) began to worry about the immortality of their name ...*" considering that the bridges made of stone were "*...more lasting and give greater glory to their makers*" [*Palladio*].

The durability of masonry bridges is guaranteed by the fact that they can usually carry the actual loads. In fact the majority of the historical masonry arch bridges are still in service, even if today are no longer being manufactured. This ability is related to the ratio between the own weight and the applied loads: in masonry arch bridges the self weight is very big respect to the applied loads, therefore even significantly increase of loading may be not so relevant such as in light structure bridge, for instances steel bridges, suspended or cable-stayed bridges. In general, if masonry bridges can bear their self-weight they may be also able to carry applied loading.

The longevity of masonry arch bridges is due to a prefect mix between material and structural typology: on one hand the masonry material is a very durable material, on other hand the stone arch has an exceptional aptitude to preform both the structural and the constructive duties of a bridge. The perfect combination of material and structural typology led to a configuration which has remain equal, although many progresses have been reach during the centuries, and has guaranteed the success of this type of bridges.

Masonry is a composite material made of units, blocks of stone or bricks, juxtaposed with or without mortar joints. Bridges made of bricks and bridges made of stone have to been studied together: they both belong to the same category of masonry arch bridge and have the same morphology. The potential of the two materials are the same, brick may be considered as an artificial stone. Moreover in the majority of bridges both materials are present. It is not intended any difference between the terms masonry arch or stone arch bridge in the thesis. It is possible to summarise the mechanical behaviour of masonry material:

- Good compression strength;
- Poor and uncertain tensile strength;
- Shear strength dependent by compression;
- Diagram stress / strain is linear only for very low loads and tends to become non-linear for loads already far from ultimate ones.

The materials, especially in the early stages of a technology, play a crucial role in the configuration: the characteristics of the masonry led to a configuration of bridges based on the structural typology of the masonry arch. The arch, thanks to its shape, is a structure that supports the vertical loads through a resistance mechanism in which the predominant stress is that of compression. Therefore the arch is the structure more suitable for materials that have not tensile strength and thus it is perfect for masonry[1]. For this reason all the bridges made of masonry are based on the static principle of the arch[2].

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[1] Complete information about the mechanical characteristic of masonry and the structural behaviour of masonry material and masonry arch will be provided in the section 2 of the thesis.

[2] Few examples of bridges made of stone based on the static principle of trilith can be found in China, for instances the Wah'an Bridge on the Luoyang river, the Anping bridge or the Hanchou Bridge. However, this type of bridges made with stone beam are rare and usually of small dimension, with length of span of few meters. [Troyano, 2006].

The application of arches and vaults for bridging space is very old, probably several thousand years old. Short span barrel vaults were already built about 5000 years ago in Mesopotamia, also Sumerian and Egyptian probably knew the vault. There are many different theories on how this type of structure has been invented. Ancient Greeks knew the arch structure, however they did not use it, their architecture was based uniquely on horizontal and vertical structural elements. The first major step in the development arch was during the time of the Etruscans, which are considered as the inventor of the wedge stone arch[3]. The second step was done during the Roman Empire. The Romans on one hand improved the quality of the placement of the stones on the other invented the mortar. Moreover they introduced pentagonal-shaped stones to link arch and spandrel walls, obtaining the improvement from wide vaults, built by the Etruscans, to wide-spanned arch bridges [*Proske and Van Gelder, 2009*].

The Romans can be considered without any doubt the greatest bridge builders of the antiquity. They perfectly understood their utility for the control and the management of the territory, the Roman Empire was based on a very extensive road network, of which bridges were an essential part. Before Romans, masonry arch bridges had short spans and the piers were completely or partially underground, while roman bridges have an elegant shape, solid piers and long span arch, usually made of odd number of arches. The Roman bridges were always made with half circular round arches, and only in rare cases with small lowering, anyway with minimum a ratio between the rise and the span R/S equal to 1/3[4]. The ratio rise to span is a very useful parameter to describe the evolution of masonry arch bridges: during the time this value reached very low values, up to 1/15. However the mastery of Roman in bridging was really great and the very high quality of Roman bridge remained unsurpassed for several centuries. The *Alcàntara Bridge*, in Spain over the Tago River, is an example of the perfection level reached by roman bridge engineers.

Roman arch were characterised by a geometric perfection: all the voussoirs have the same dimension both in elevation, from springing to keystone, and in the

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[3] In wedge stone arches, every stone has a wedge shape, which allows a better shape of the arch compared to normal ashlar-shaped stones.

[4] In round arches the ratio R/S is 1/2.

arch width, and the generally the geometry of the arch precisely reproduces a circumference. The most famous Roman bridges were the great aqueducts that supplied the cities of the empire overcoming the wide valleys they met on their way. To build aqueducts and bridges with a great height was very complex, in particular the realisation of high piers, the Romans solved this problem "overlapping" a bridge above the other, in order to create two or three levels of arches which have the function of stiffening and upwind. The *Pont du Gard*, built in the 15 b.C. in the south of France, still amazes for its majesty and the graceful of its arches.

Piers were usually very wide, however during the time Roman bridges were subjected to great evolution: early bridges have usually a ratio between the width of pier and the length of span P/S about 1:1, while subsequently this ratio reach a value of 1/4.3 in *Bibey Bridge*, in the north of Spain, with an average value of 1/2. The solid appearance of the bridge was given also by the high thickness of arch. The average value of the slenderness of arch, which can be measured by the ratio between the thickness of arch voussoir and the span V/S, was about 1/12.5, with maximum value of 1/18 in the *Bibey Bridge*. When possible, Romans increased the span in order to clear the river avoid the construction of piers in the bed. The Roman bridge still existing with the longest span is the *San Martino Bridge*, in Val d'Aosta, in the north of Italy, which has a span of 35.5 meters.

Fig. 1.1 - Bibey bridge, Galizia, Spain

Fig 1.2 - The Pont du Gard, France.

Fig 1.3 - The Alcàntara Bridge on the Tago River, Spain.

Fig 1.4 - San Martino Bridge, Val d'Aosta, Italy

After the fall of the Roman Empire, the construction of bridges almost completely stopped for several centuries, during the whole early middle age, because of the drastic reduction of trade. The revival of the construction of bridge began in the high middle age, in the twelfth and thirteenth centuries, and during the late middle age, fourteenth and fifteenth centuries several very interesting bridges have been built. In fact, although the perfection reached by roman bridges were not surpassed until the eighteenth and nineteenth centuries, some important technological progress have been achieved in that period. On one hand, the knowledge developed by Romans were lost and the medieval bridges are of lower quality than those of the Romans, on the other hand the mastery in the construction of arches and vaults typical of the Romanesque and Gothic, which medieval cathedrals are clear examples, was applied to medieval bridges.

The medieval bridges are generally more slender than Roman ones. The slenderness regards both piers and arches: the ratio P/S has an average value of 1/5 and the ratio V/S has average values between 1/15 and 1/30, with peak of 1/35 in the gothic period. The arches of the medieval bridges had not the same geometric perfection of Roman arches but the resistance mechanism becomes more clear. The springing begin to be built with square stones curved at the edges to give the right starting angle to the arch, typically such stones started from the base of pier or abutment and reach a certain height so to arrive at an angle of about 30°, then began the real arch, which is limited to 120°, which is the sector that really works as an arch. Arches were usually round arches or pointed arches, however early shallow arch bridges have been realised. For instances *Ponte Vecchio* in Firenze has a ratio R/S equal to 1/7: This bridge is worldwide famous because it is lived-in bridge, however at the time several bridges were inhabited[5], hosted market or defence system, mainly towers. Also span of medieval bridges were usually bigger respect to Roman bridges: span of 40 metres were quite common, the *Trezzo Bridge*, on the Adda river in the north of Italy, had a span of 72 metres, which was the longest until the twentieth century. Unfortunately this bridge was destroyed, the existing medieval

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[5] Inhabited bridges were present in Paris and London, but houses have been distroyed in the XVIII Century to enlarge the roads and improve the traffic.

bridges with longest span are the *Ponte Scaligero*, in Verona, and the *Pont Grand de Tournon*, in France.

Fig 1.5 - Typical medieval bridge, Puente de la Regina, Navarra, Spain

Fig 1.6 - Ponte Vecchio, Firenze.

Fig 1.7 - Ponte Scaligero, Verona.

With the Renaissance there was a return to classical scheme, which interested also the bridges. Leon Battista Alberti defined some rules for the design of bridge, on the base of the *Roman Bridge of Augusto*, in Rimini, Italy [*Alberti, 1483*]: the slenderness of the arch should be between 1/10 and 1/15 (ratio V/S) and the slenderness of the piers between 1/4 and 1/6 (ratioP/S). The span of renaissance bridges are usually shorter respect to the medieval bridges, however in that period several innovation have been developed, which lasted until the nineteenth century. Moreover many famous renaissance architect dealt with the construction of masonry arch bridges, such as Palladio, Alberti, Leonardo da Vinci, and some of the most beautiful masonry arch bridges have been built, for instances the *Rialto Bridge*, in Venezia, and the *Trinità Bridge*, in Firenze.

Fig 1.8 - Rialto Bridge, Venezia, Italy.

Fig 1.9 - Trinità Bridge, Firenze, Italia.

The birth of modern masonry arch bridge is due to the french school starting form the seventieth century. In the eighteenth century began the distinction between architecture and engineering. The corps of engineers of Pont et Chaussées[6] was established in 1716 in France, and subsequently, in 1747, the famous "*Ecole de Pont et Chaussées*" was founded. In the first half of the century, the French engineers built a series of stone bridges of very high quality, whose main features were the use of poly-centric or elliptical shallow arches and the presence of triangular rostrum end with a cover at the level of the crown. A typical examples of this typology is the *Pont Royale* in Paris.

Fig 1.10 - Pont Royale, Paris.

The real revolution in masonry arch bridges occurred in the second half of eighteenth century, thanks to Perronet. Founder of the Ecole des Pont et Chausses and first modern engineer, he introduces the greatest innovation in masonry bridges since the Roman times: all bridges built after him were inspired by his magnificent bridges. The main innovation provided by him were:

- Reduction of the ratio P/S from 1/5 to 1/10, taking advantage of the compensation of the thrust of two adjacent arches;
- Increasing of the lowering of the arch, with ratio R/S up to 1/15, using of mono-centric shallow arches instead of poly-centric or elliptical;
- Introduction of a clear discontinuity between the arch directrix and vertical facing; in some bridges, connect the arch with the piers through the *corn de vache*;

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[6] Bridges and roads

- Invention of a system that allow removing simultaneously all the centering;
- Invention of the pier-abutment, although he never used it.

Fig 1.11 - Perronet, Pont de Neumurs, on the Loing River, France.

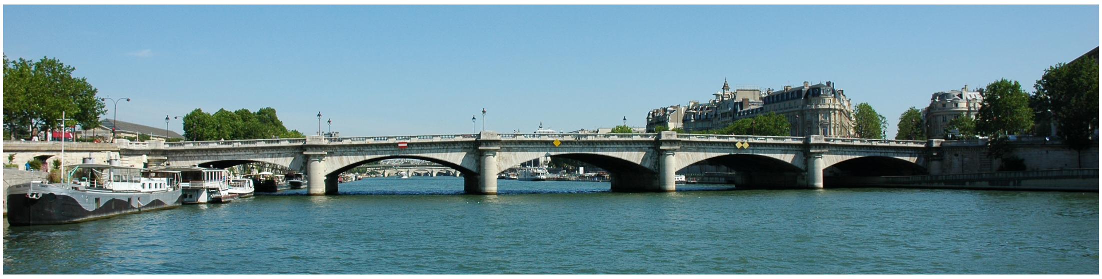

Fig 1.12 - Perronet, Ponte de la Concorde, Paris, France.

Fig 1.13 - Perronet, pont de Meumurs, detail of the connection between pier and arches.

The nineteenth century was the period of greatest development of bridges, but at the same time it was also the period in which masonry bridges become to be obsolete. The new structural material, steel and concrete, start spreading and at the beginning of the twentieth century masonry material have been superseded by them. Despite the progressive abandonment, between the ‘800 and the early ‘900 several very interesting masonry bridges have been built. The lesson of the French school spread across Europe. Thanks to the contribution of the theory of structures, masonry arches with considerable span were realised. The European stone bridge with the largest span was built in Germany in 1903: the *Plauen Bridge*, on the Syros River, with 90 meters of span.

The biggest contribution has been provided by the great English engineers of the nineteenth century, who developed the first steel bridges but mastered also the technique of masonry and wood, such as John Rennie and Thomas Telford. The 800 was also the century of the railway, which had a significant impact on the bridges. In those years large railway viaducts in masonry have been realised, on the base of the Roman aqueducts, but introducing important innovations. In particular the height and slenderness of piers has been considerably increased. The last great engineer and builder of masonry arch bridges was Sejournè, also author of a very famous treatise [*Sejournè, 1913*], which as the one provided by Rondelet [*Rondelet, 1831*] are fundamental references for the knowledge of the stone bridges. He built bridges with important span and tried out new and interesting forms.

Fig 1.14 - Telford, Over Bridge, on Severn Bridge, UK.

Fig. 1.15 - Sejournè, Pont des Amidonnier, Touluse, France.

Fig 1.16 - Rail viaduct of Port Launay, France

Fig. 1.17 - Dean Viaduct, Edinburg, Scotland.

Fig 1.18 - Sejournè, Fontpederuse railway viaduct, France.

In the ‘900 the masonry material were definitely abandoned in favour of the new structural material, and also the construction of masonry arch stopped. However, although their are not more being built, there are thousand of masonry arch bridges in service. Exhaustive information of the history of masonry arch bridges can be found in [*Troyano, 2006*]. Further information about the number of the existing masonry arch railway bridges are given in the next paragraph.

1.2 Masonry arch railway bridges in service

Starting from the nineteenth century masonry material has been superseded by the new structural materials, steel and concrete, for the construction of buildings and structures. At that period began the end of the history of masonry arch bridges. However, that was the time in which thousands of masonry arch bridges have been built all around Europe, due to the birth and diffusion of the railway. The Italian railway network, as well as the European, has been almost completely built in one century, from the 1825, year of the first railway[7], to the 30's of the twentieth century. The first Italian railway has been realised in 1939[8] and the great part of bridges have been built in the fifty years from the 1860 to the 1910, subsequently to the unification of Italy.

Although the new structural materials began spreading, almost all the railway bridges were realised with the typology of masonry arch bridge, which was at its maximum technological level, as testify by many masonry arch railway bridges built at the time, for instances the ones realised by Sejournè[9]. Therefore, masonry arch bridges reached such a level of safety that have been considered the most appropriate for railways. Their construction relied on a well established practical experience coupled with the great advances in the knowledge of their structural behaviour reached in that period by the modern theory of structures. The typology ensured a great load-bearing capacity and stiffness, moreover it was particularly suitable to realise viaducts, which allowed to overcome irregular grounds with the required low slope. Finally, masonry arch bridges are very durable structures, which is a fundamental aspect in the realisation of the railroads network. Thanks to their qualities, thousand of masonry arch railway bridge have been built and, thanks to the durability of their materials, a high percentage of them are still in service.

An evaluation of the number of masonry arch railway bridge has been provided by many authors and railway organisations. Specific studies regarding the

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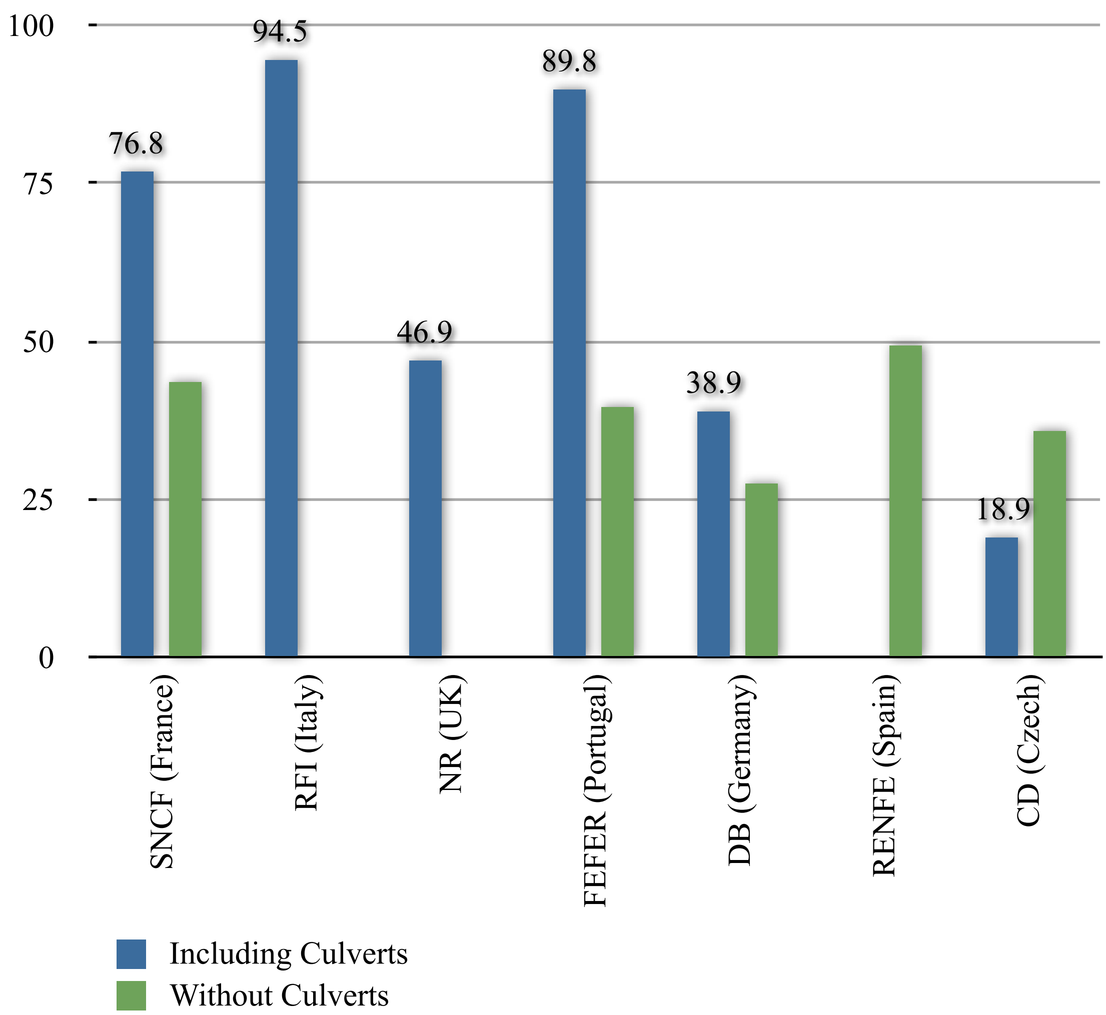
[7] The Stocktown and Darlington Railway, built in the Unitet Kingdom.

[8] The Napoli - Portici Railway, built under the Borbone's Kingdom.

[9] The Lavaur Bridge, the Castelet Bridge and the Antoniette, at Veilmur, on the Castres-Montauban Railway are eixamples of the high technological skills reached in the 80's of the XIX century.

dimension of bridges stock and the percentage of each typology of bridge belonging to the network have been provided almost for each country, both from national railway companies and researcher. A systematic analysis of the masonry arch bridge stock in the international railway system has been carried out by the International Union of Railways, involving thirteen different railway organisations and more then 200,000 railway arch bridges [*UIC, 2005*]. The report outlined that the percentage of masonry arch bridge is about the 60% of the overall bridge stock of the railway organisations which participated to the survey. The proportion of masonry arch bridges respect to the overall railway bridge stock of some of the main European railway networks provided by the UIC Report is reported in the next graph:



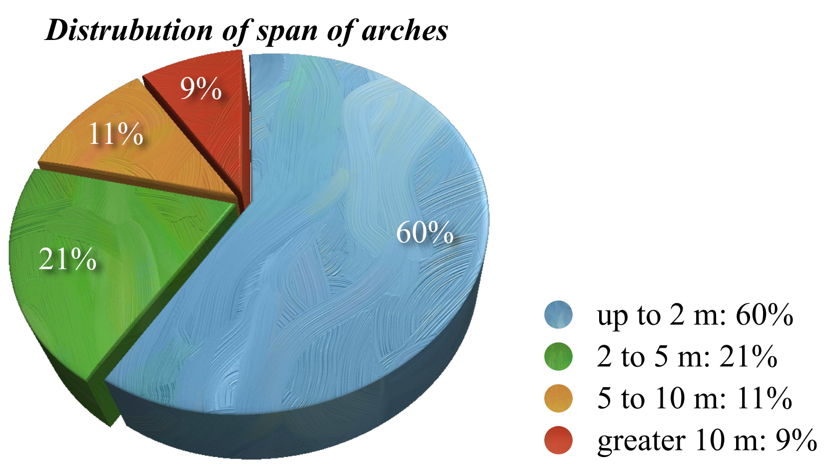
Graph 1.1 - Proportion in percentage of masonry arch railway bridges on the overall bridge stock of some of the main European railroad network [UIC, 2005]

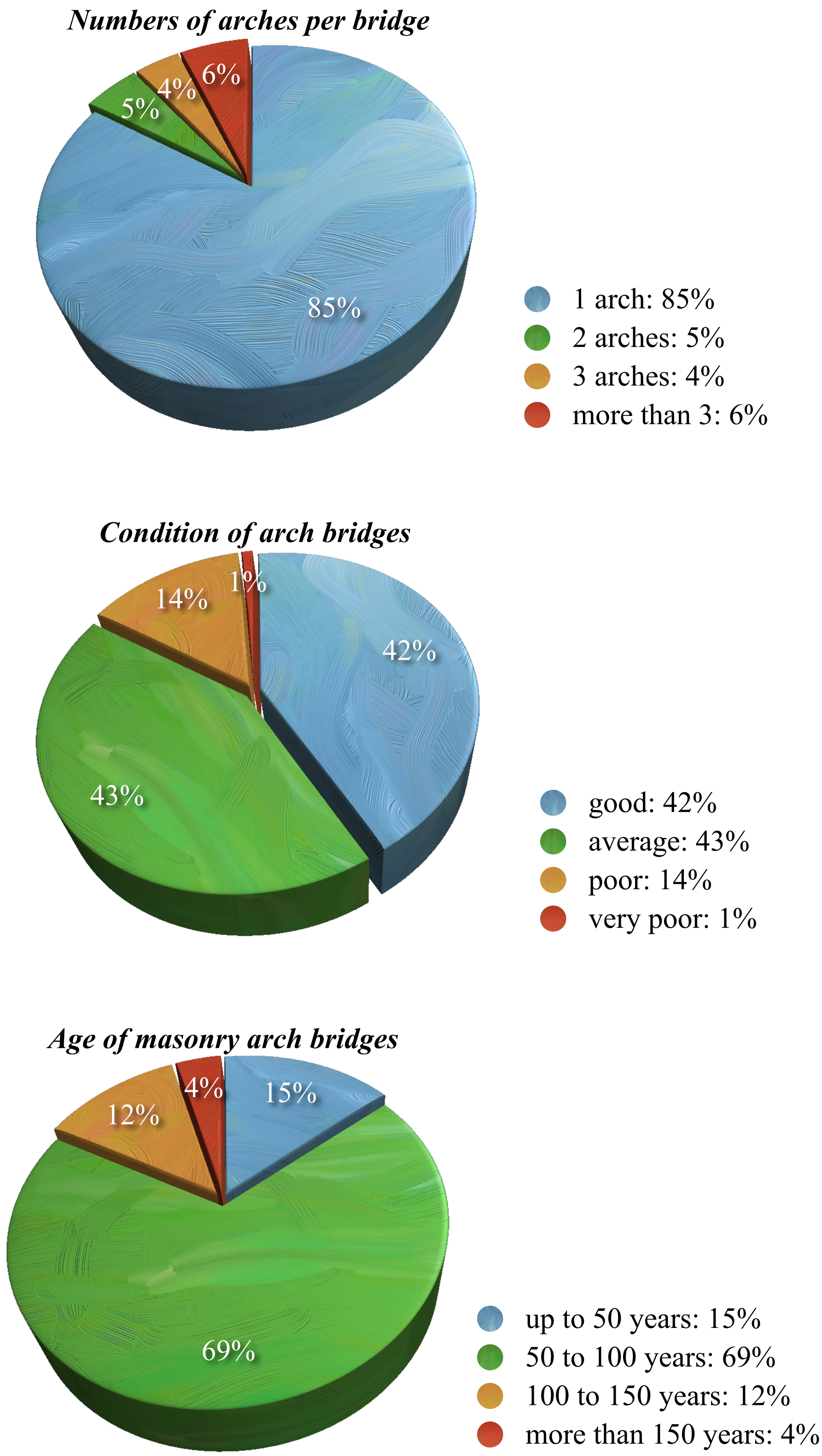
The report provide also the number of bridge corresponding at the percentage reported above:

Railway organisations and country	Number of stone arch bridges and culverts	Number of stone arch bridges
SNCF France	78000	18600
RFI Italy	56888	
NR United Kingdom	17867	16500
REFER Portugal	11746	874
DB Germany	35000	8653
RENFE Spain		3144
CD Czech	4858	2391

Table 1.1 - Numbers of stone arch bridge in the main Europan railways network

The UIC report provided also some interesting information: the majority of the masonry arch railway bridges are single-span bridges, having a span shorter than 10 m, built between 50 and 150 years ago and have a good, or sufficient, state of conservation. The distribution of the characteristics of masonry arch railway bridges in Europe are reported in the following graphs [*UIC, 2005*]:





Graph 1.2 - Distribution of characteristics of masonry arch railway bridges in Europe [UIC, 2005]

The estimated number of italian masonry arch bridges without including the culverts is of 37400, equal to about 63% of the overall stock, has been provided by

[*Weber, 1999*]. In Italy more than 7000 stone arch railway bridges with a span greater than 8 m have been built in the second half of the nineteenth century in the Italian railway system, as at least 7,000 [*Brencich and Colla, 2002*]. The Italian railway includes 12000 masonry arch bridges with a span greater than 2 meters, of which about the 80% has a span smaller than 5 meters [*Cavicchi and Gambarotta, 2004*]. Having been built in a relatively short period and being the typology at its maximum technological level - and at the same time in decline, in 1930 when the network was already built masonry was definitely superseded by steel and concrete - it is possible to find quite an homogeneity in the typologies of bridge belonging to the network. Many of the geometrical characteristics of masonry arch railway bridges, such as span, number of arches, thickness at crown, radius of curvature, ration rise to span, are comprised in recurrent intervals.

1.3 Morphology of masonry arch railway bridge

The modern masonry arch bridges, and especially the railway bridges, were built over a period of about 100 years, roughly from 1830 to 1930. Despite the construction is placed in a period of time rather limited, this type of bridge presents some common structural solutions and some structural and constructive features that varies on the base of the time of construction and the geographical area and by the calculation method adopted and the designer. However, many common aspect may be outlined.

The construction of stone bridges is fairly simple, the construction technique used is the vault made by ashlars, which mainly remained unchanged by the Romans until the '800: In the construction, the stone elements have a very much smaller dimension compared to the span to overcome: the voussoir arch is the perfect structure for this type of material: therefore, as long as bridge have been built up in masonry, the technique had not substantial changes.

The construction of masonry arch bridge is realised through falsework, called centering, usually made of wood. Centering may be supported by the ground or may be attached to the piers or abutments and span as cantilever beams. The techniques of construction and the centering have been subjected to several technological innovation during the history. Briefly, temporary frameworks are realised and placed to overcome the span, between piers and or abutments. Centering are shaped on the base of the arch profile, usually their initial shape is defined in order to accommodate further movements due to the weight of the arch before its completion. The voussoirs are placed on the extrados of the centering, starting from the springing and arriving to the crown. The construction of the arch end with the placing of the keystone. Once the arch has been realised the centering is removed. The removal is performed elimination provisional supports placed under the centering, usually wood wedges which are destroyed when the arch is done, or through other systems, such as sand bags which are punched in order to be emptied. An innovative method was developed by Perronet, who invented a system to remove at the same time all the centering placed under adjacent spans. More information about centering may be found in [*Torre, 2003*].

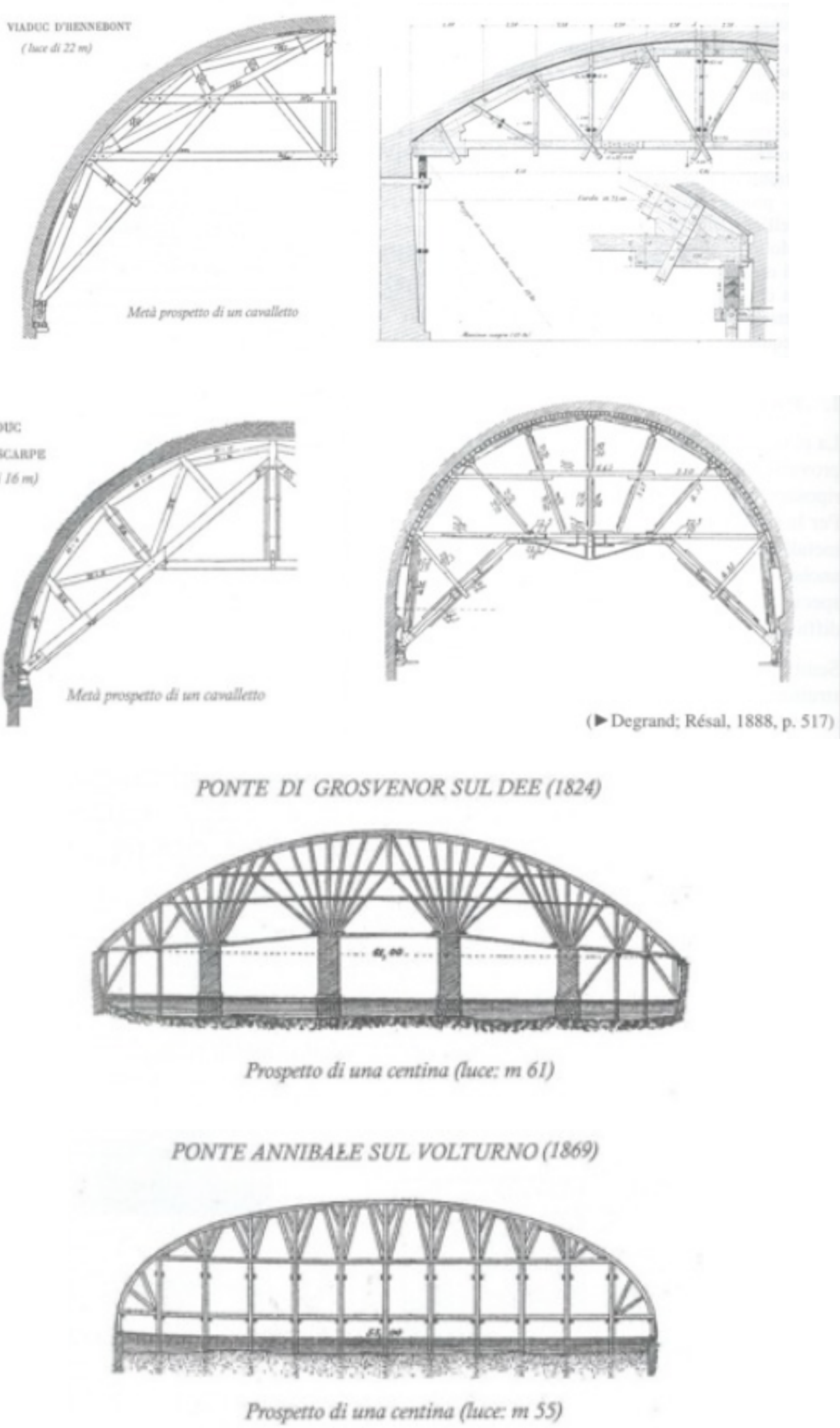


Fig 1.19 - Examples of centering: above cantilever centering, below supported centering, taken from [Torre, 2003]

The geometry of the bridge is strongly dependent by the obstacle that has to be cleared. In general, the orography of the valley determines the main typologies of masonry arch bridges:

- Wide and deep valleys are crossed by viaducts, multi-span bridges on high piers. Viaducts were built by Romans for their aqueducts, then the typology spread with the railway, in fact aqueduct are particularly suitable to overcome height difference with low slope and/or wide curvature radius, as required by railway network.
- Wide and shallow valleys are crossed by multi-span bridges on low piers. This are the proper bridges, typical in case of rivers.
- Minor valleys and small rivers are usually crossed by single-span bridges.

The main elements of a stone bridge are [McKibbins et al., 2006]:

- The arch, which is the main structural element, which allow clearing the obstacle and carrying the deck.
- The support structures of the arches:
  - Abutment, a body, usually of masonry, which provides the resistance to the vertical forces and the thrust of the arch;
  - Pier, an intermediate support between adjoining bridge spans;
  - Pier-abutment, a mix between a pier and an abutment, that is a wider pier that, thanks to its dimension, allow avoiding the global collapse of the bridge due to the collapse of a span with the consequent a-symmetric thrust. For the same reason, it can be very useful in the construction of bridge.
- The area overlying the arch barrel under the road surface (or equivalent), occupied by the spandrel walls, fill material or voids, and occasionally hidden elements such as internal spandrel walls. It consists of two main elements:
  - Backfilling (also called backing or filling), the material, usually low quality fill, used to give support behind a structure. For a masonry arch bridge, backfill material is placed in the spandrels between the arch barrel and the road surface and retained laterally by the spandrel walls and/or wingwalls.

It normally consists of granular material eg gravel or building debris, which may have been excavated for the foundations or is waste from the construction.

- Spandrel walls, masonry walls that sits on the edge of the arch barrel and that limits the extent of, and retains, the backfill. Sometimes "internal" spandrel walls may be present at other locations on the arch.

Beside the external structure of the bridge there are the foundation. Typically made by wooden piles, inserted in the ground, and massive stones. They are the part of the bridge that is not visible, so any information on the consistency of the foundation works shall be deducted from the historical bibliography on construction techniques.

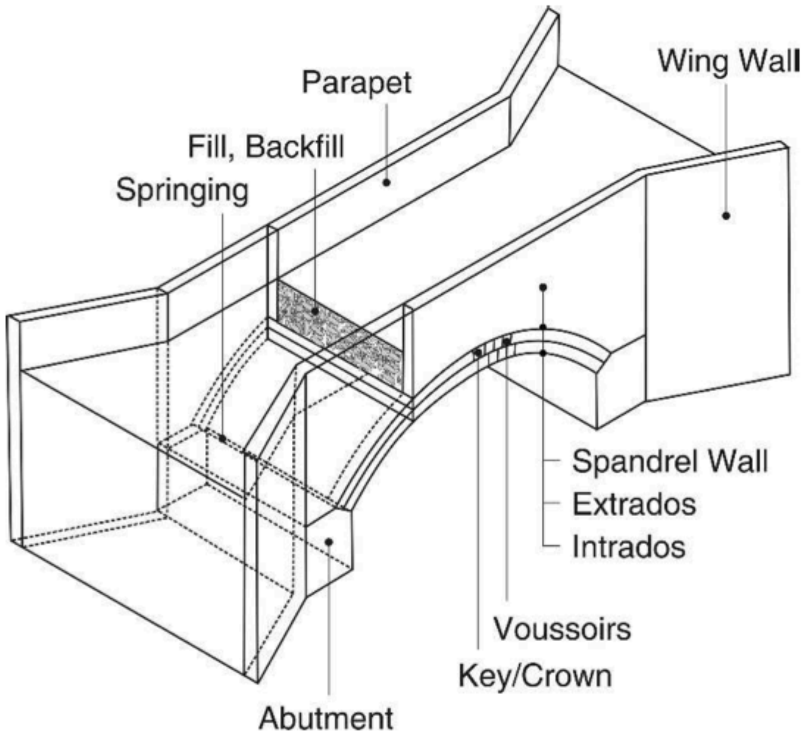


Fig 1.20 - The main element of a masonry arch bridge according to [Hughes and Blackler, 1997], taken from [Proske and van Gelder, 2009]

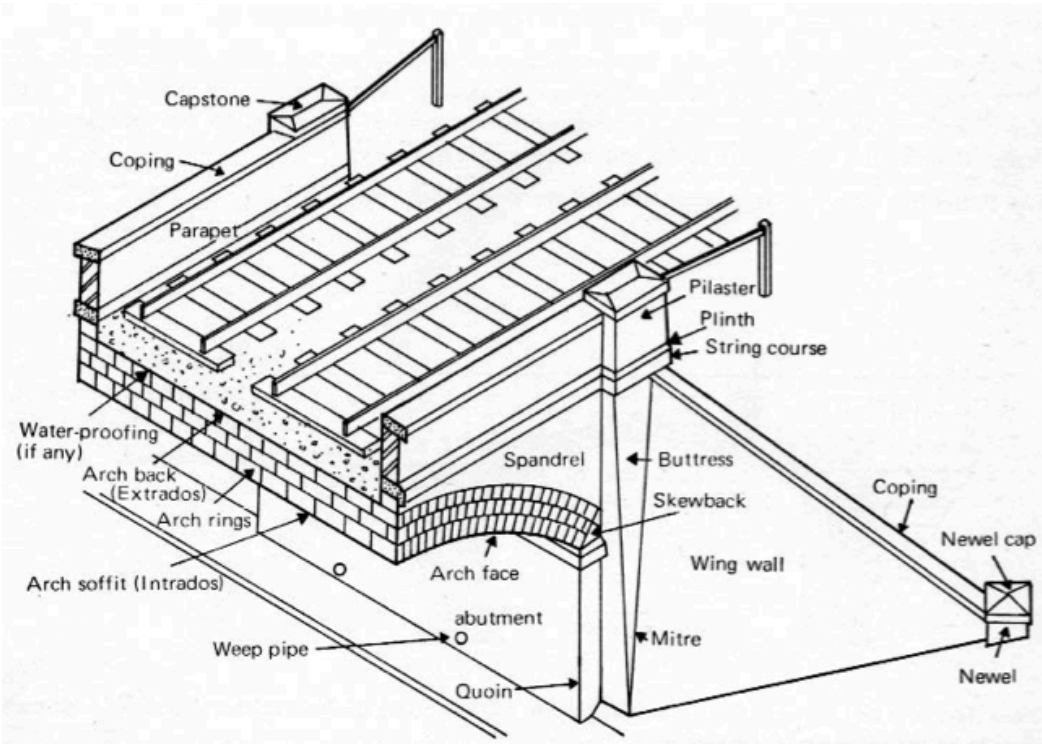


Fig 1.21 - The main element of a masonry arch railway bridge ,
taken from [McKibbins et al., 2006]

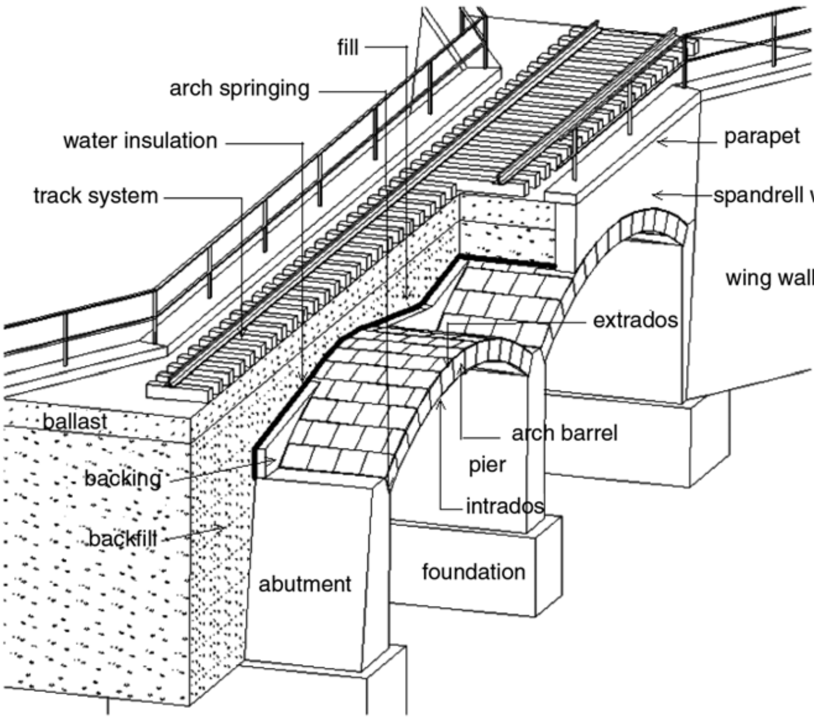


Fig 1.22 - Typical structure of a double-span masonry arch railway bridge,
according to [Orbàn and Gutermann, 2009].

The arch is the main structural element of a masonry bridge. The arch is a curved structural member capable of supporting vertical loads across an opening and transferring these loads to piers or abutments. The arch barrel is the load-bearing part of the arch. It is generally made with barrel vaults, having a cylindrical intrados and a straight plant. Instead skewed arches are used when the road or railway axis passes through the river or valley along a path that is not perpendicular to the axis of the valley. Skewed arch barrel may be realised with different construction patterns [*Melbourne and Hodgson, 1996*].

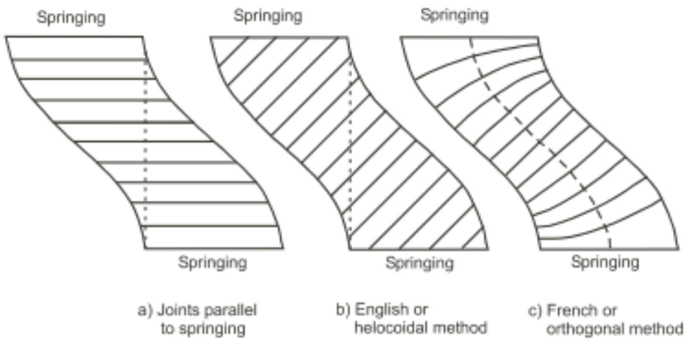


Fig 1.23 - Different pattern in skewed arch barrel according to [Melbourne and Hodgson, 1996], taken from [McKibbins et al., 2006].

The intrados of the arch may be define by a circular directrix, by an elliptical directrix or may be poly-centric. The shape of the arch is determined by the ratio between rise and span R/S:

- R/S = 1/2, round arch;
- R/S < 1/2, shallow arch;
- R/S > 1/2, pointed arch.

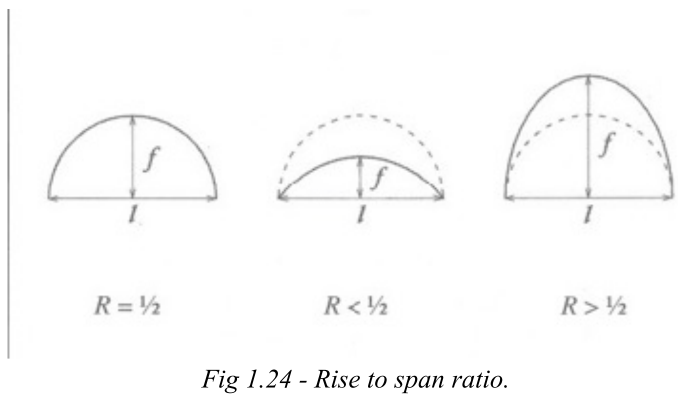


Fig 1.24 - Rise to span ratio.

The principal parts of a masonry arch are [*Mckibbins et al., 2006*]:

- The springing: plane from which an arch springs, such as the junction between the vertical face of the abutment and the arch barrel.
- The haunch, the lower section of the arch barrel towards the springing.
- The keystone or crown: the highest and last-placed stones in an arch. In the arch barrel of a bridge there are a series of keystones at the crown, across its width, which are often left projecting on side elevations.
- The extrados: in an arch or vault is the top surface of the arch barrel, the outer (convex) curve of an arch.
- The intrados: in an arch or vault is the inner surface of the arch barrel ie the inner (concave) curve of the barrel.

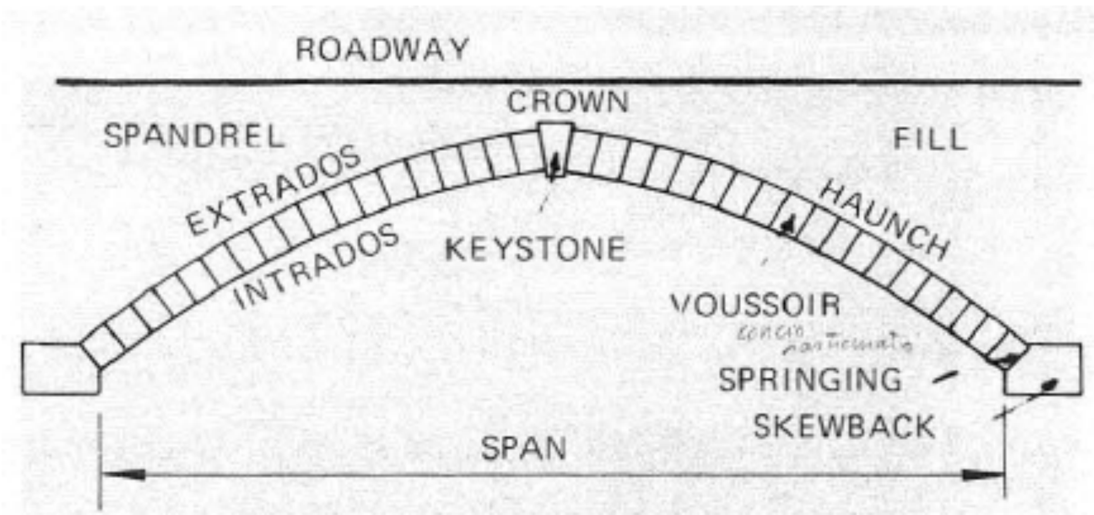


Fig 1.25 - The main element of a masonry arch,
taken from [Heymann, 1982].

The material used for almost all the structural element is the masonry, made with stones, bricks or both the material. Piers, arches and spandrel walls are generally made with high quality masonry. Backfilling is made with incoherent filler, sand, stones and bricks. Backfilling often consists of two layers, separated by the waterproof system, usually realised with a concrete saddle. The lower backfill plays also a structural role, while the upper one simply fill the space between spandrel to reach the surface road. Further information about the materials and their properties of each element will be provided in the second section of the thesis.

The typical configuration of modern masonry arch bridge and an exhaustive description of its elements and material has been provided by [*Torre, 2003*], here some of the illustrations given by her masonry arch dictionary are reported. Illustrations regards typical configuration of masonry arch bridges built in the nineteenth century in Italy, in particular in Piemonte region.

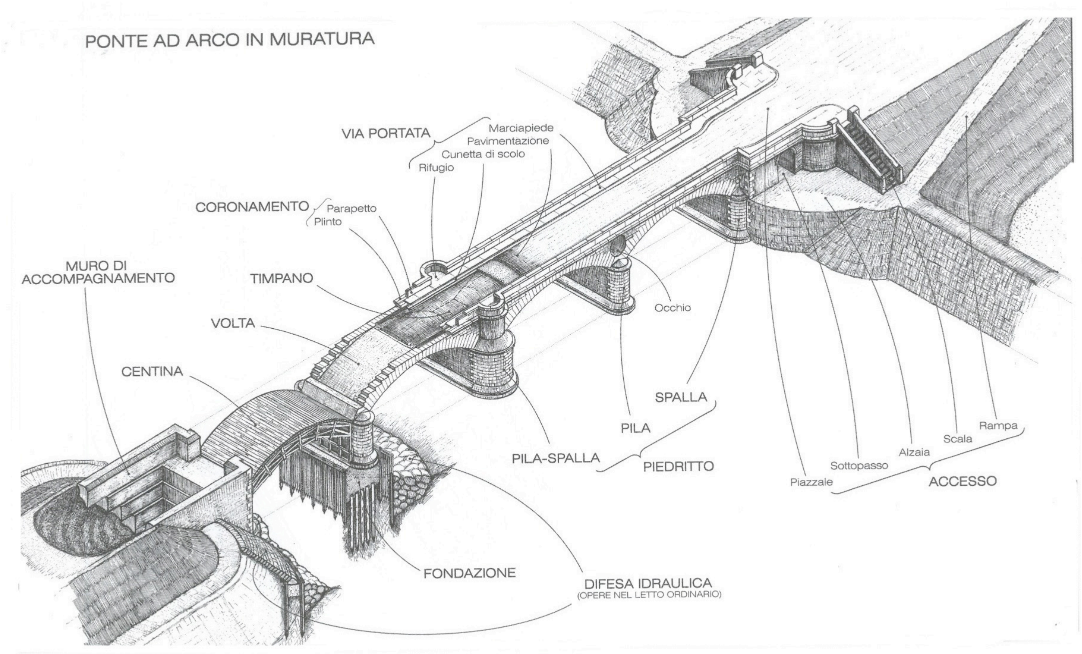


Fig 1.26 - Typical configuration of modern masonry arch bridge, taken from [Torre, 2003].

In the figure reported above is possible to notice:

- The different typologies of supports (*piedritto*): piers (*pila*), abutments (*spalla*) and pier-abutment (*pila-spalla*);
- Arch barrel (*volta*) both in construction and completed;
- Centering (*centina*);
- Spandrel (timpano) and backfill and the road surface;
- Foundations and protective systems of hydraulic defence;

In the following illustration elements are described more in detail.

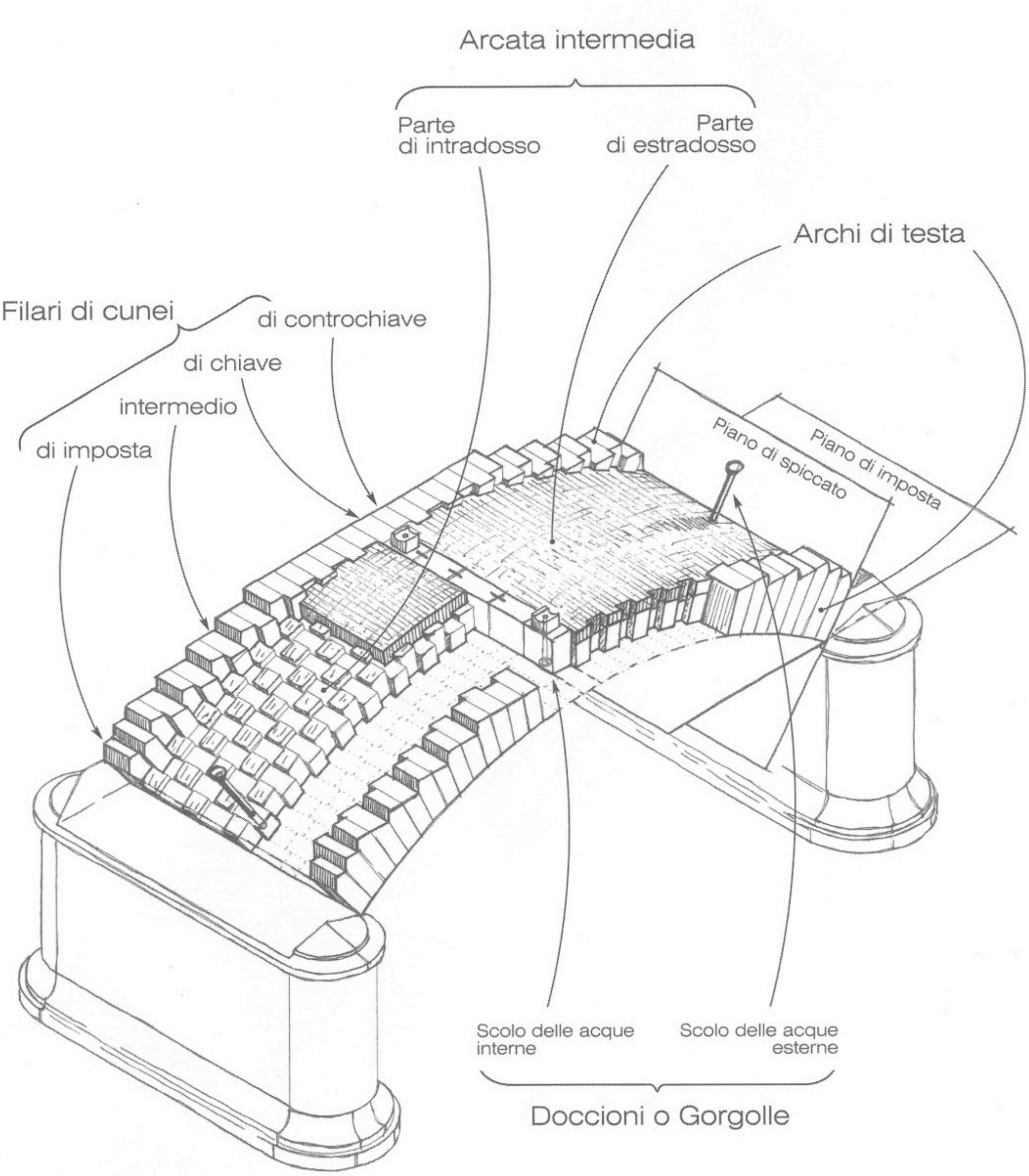


Fig 1.27 - Typical configuration of an arch barrel, taken from [Torre, 2003].

In the figure reported above is possible to notice:

- The front arches and the barrel, with the difference between the intrados, made of blocks, and the extrados.
- The different typologies of voussoirs (*cunei*)

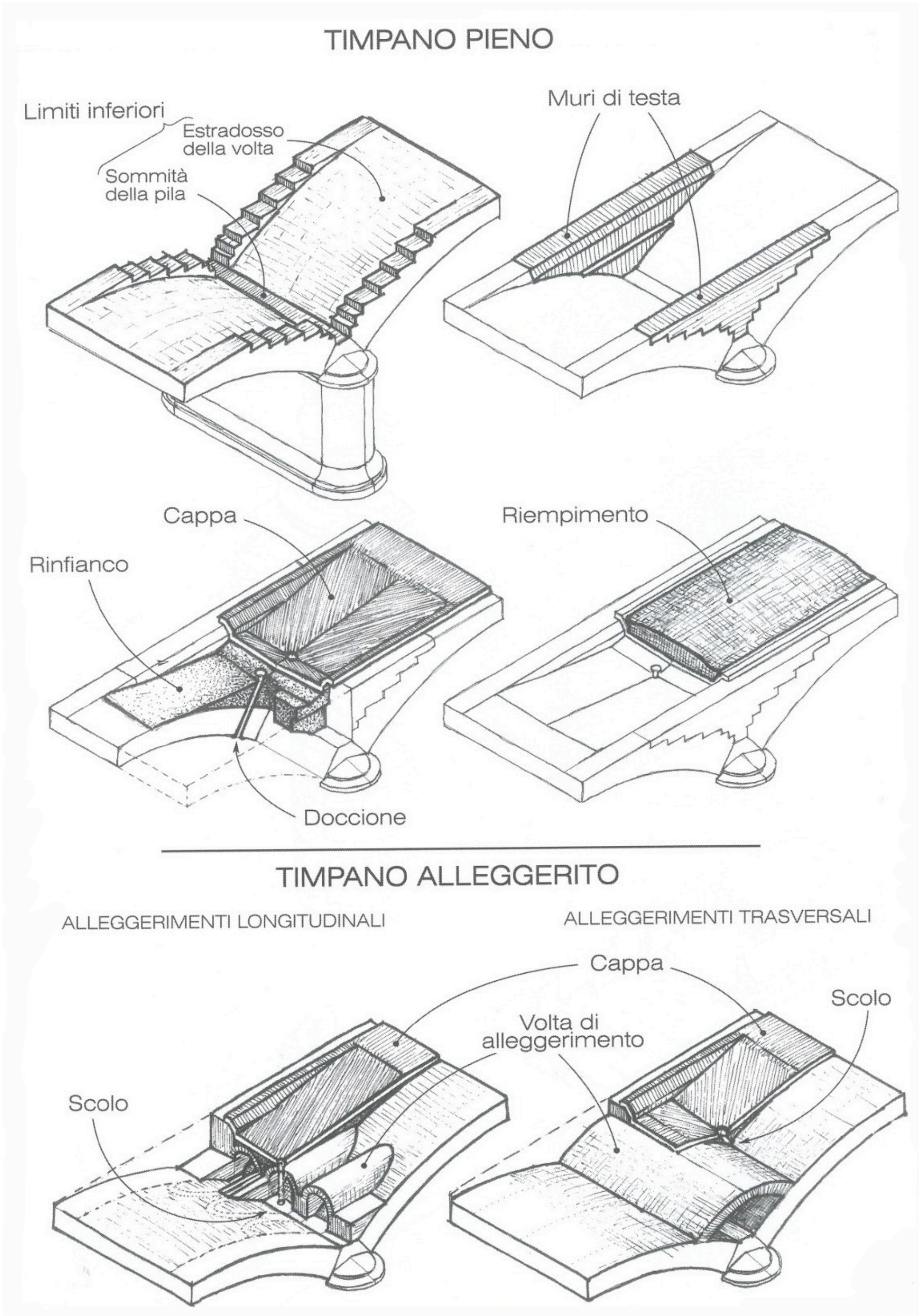


Fig 1.28 - Different typologies of spandrel and backfill, taken from [Torre, 2003].

In the figure on the top are represented the element of a typical spandrel: spandrel walls, upper and lower backfilling and the waterproof system. In the figure down is represented an enlightened spandrel, made with transversal or longitudinal lightening arches.

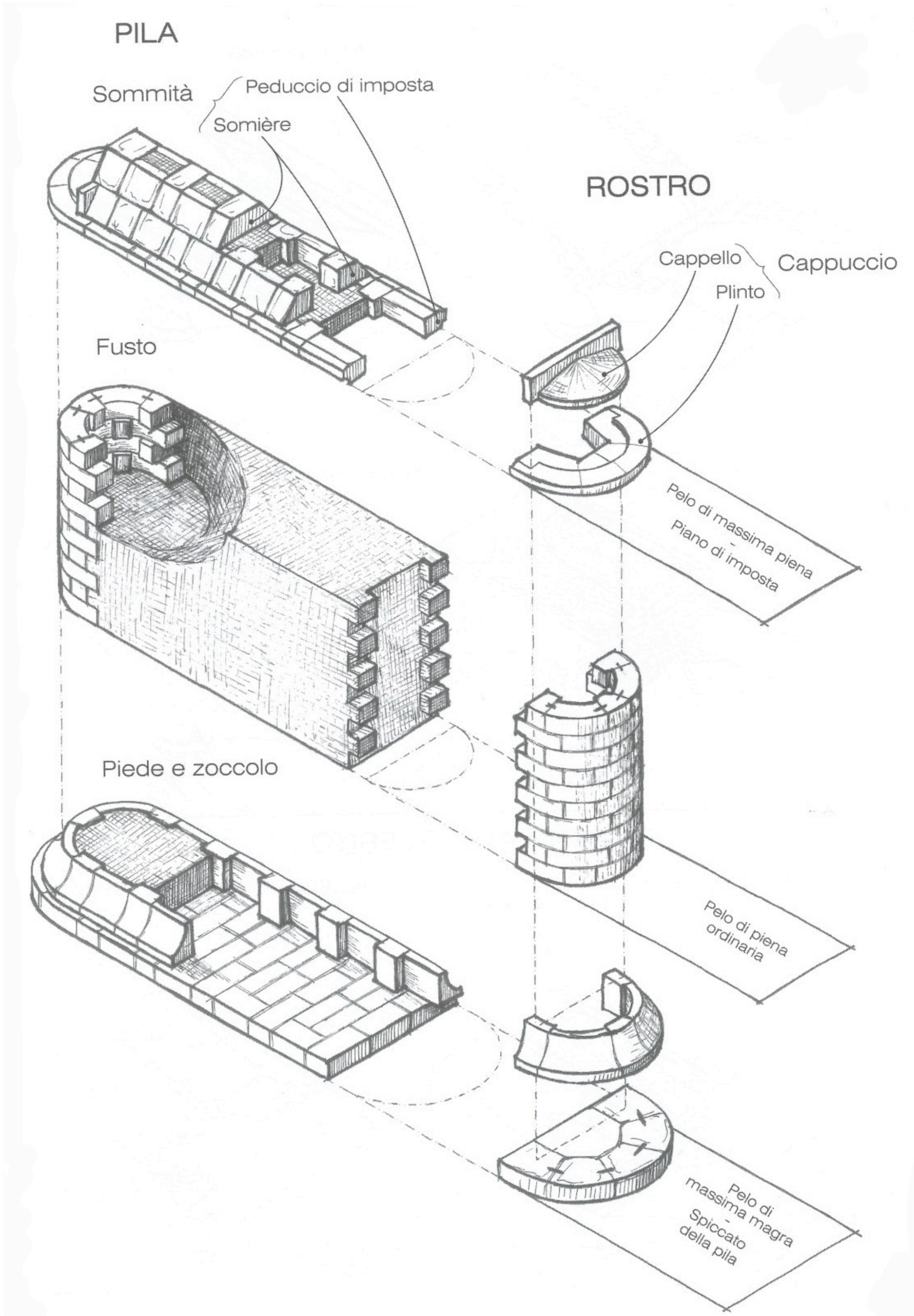


Fig 1.29 - Typical configuration of pier,
taken from [Torre, 2003].

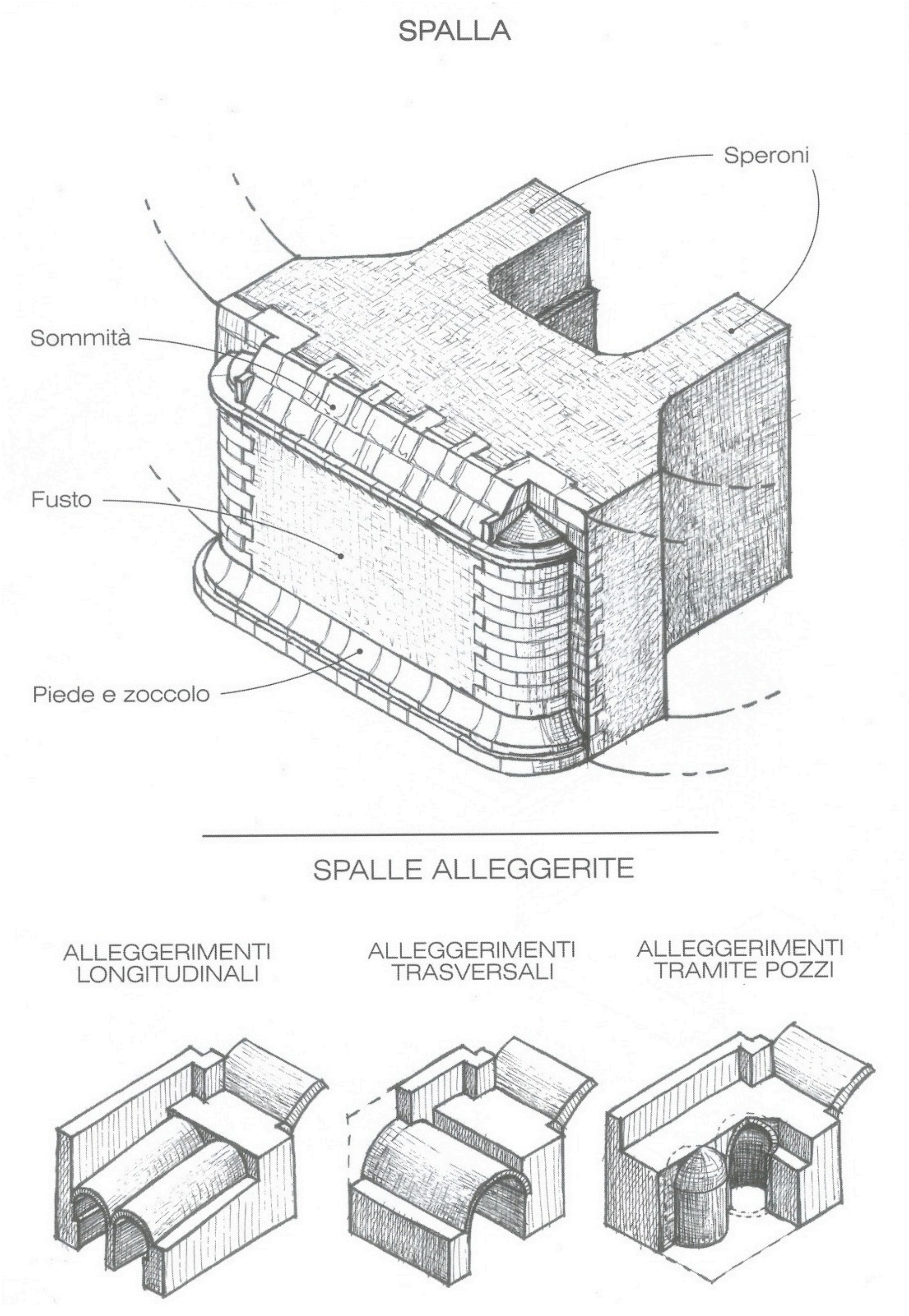


Fig 1.30 - Typical configuration of abutments, taken from [Torre, 2003].

SECTION 2

Modelling and analysis of masonry arch bridge

Introduction

This section deals with modelling and analysis of masonry arch bridge. The aim is to define which strategy is considered more appropriate to be used in the analysis of masonry arch railway bridges with the purpose of their study and for their consolidation. For this reason it is necessary to have an overview of the principle methods of modelling and analysis, with a critical comparison between the different approaches. It is convenient to subdivide the analysis in different levels:

1) Local level: the material;
2) Sub-structure level: portion of structure - the masonry arch;
3) Structure level: the masonry arch railway bridge.

1. Local level. First it is necessary to study the materials constituent the bridge. Structural elements constituting masonry arch bridge - arches, piers and spandrel walls - are usually made of masonry, made of bricks or squared blocks of stones, while filling in spandrel and hollow piers is made with incoherent material, stone, sand and bricks. The characteristics and problems of historical masonry and its mechanical properties and structural behaviour will be briefly described in the following paragraphs. Moreover it will be provided an overview of the main methods of modelling masonry in literature and of the principle techniques of assessment and tests for the evaluation of mechanical properties of historical masonries.

2. Sub structure level. This part of the section concerns the basic structural element of this typology of bridges: the masonry arch. Its structural behaviour and the models to represent it will be discussed. Masonry arch is the basic element that constitutes masonry arch bridge: the study of its structural behaviour is essential

to understand the global behaviour of bridges. Strategies of bridge modelling became from their models.

3. Structure level. Finally will be given the state of art of the principle methods of modelling and analysis of masonry arch bridges, with a critical comparison between the different approaches. It implies also considerations about the capability of models to properly represent the situation before and after consolidation, so to evaluate efficacy of strengthening. Particular attention will be paid to the role of fill and spandrel respect to the structural behaviour of bridges.

At the end of the section some brief conclusions are reported, with a critical comparison between the different approaches to modelling and analysis.

SECTION 2 – Part 1
The local level: the material

2.1.1 Characteristics and problems of historical masonry

Masonry is the main constituent material of masonry arch bridge. Here is given an overview of its characteristics and of the methods of analysis and modelling of masonry material and masonry structures, in particular the masonry arch. The research will focus on existing masonry structures mainly built until the end of the XIX century, in fact object of study are small span masonry arch rail bridges. Therefore it is necessary to investigate the ordinary masonry, made by natural or artificial blocks and mortar joints, without steel reinforcement.

Masonry buildings are deeply influenced by the specific characteristics and mechanical behaviour of masonry material. It implies several differences respect to the analysis of modern structural materials, such as reinforced concrete or steel, which can be defined as continuous materials and so are investigated through the mechanics of continuum. Instead masonry is the result of the union of blocks and mortar put together to compose a structural element that is not continuous. It is a material heterogeneous and anisotropic with a non-linear behaviour. The behaviour of the material depends by kind and quality of its components, by the way it has been built, by the dimension of blocks and the thickness of joints, by the pattern which blocks laid inside wall, by the position of joints, and generally by the geometry. Its configuration has a prominent role in its behaviour; therefore it is difficult to define standard characteristics suitable for all kind of masonries.

Actually there are some typical recurring aspects that are common in all the masonries and in all the masonry structures. These aspects have to be outlined and considered to study the masonry. Briefly it is possible to point out the main mechanical characteristics of masonry:

- Low and uncertain resistance to tensile stress;
- Quite good compressive strength;

- Shear strength depending by compression, on the base of Coulomb’s law on friction;
- Diagram load/displacement elastic-linear for very low loads, but that turns to non-linear behaviour just for loads that are far away from the last ones.

As told before, these aspects can considerably change in base of specific characteristics of each masonry. The difference between historical masonries and the masonries built at the moment is that the second ones are designed responding to the actual standards and with standardisation. It means that they are designed responding to the current requirements and on the base of the present knowledge; but especially we know how they are built, with a “regular” configuration, and we know the qualities of material utilised. Instead historical masonries may present some randomness about their real texture, their materials, the pattern which blocks laid, their connections. Moreover during time they were probably subjected to many events, such as modifications, damages, or even earthquakes, which have determinate their “mechanical history”. Their mechanical properties strongly depend by the state of conservation. Each case has to be analysed in order to recognize its specific characteristics and how to evaluate them. When possible experimental tests are essential to establish mechanical properties of existing masonries, otherwise it is necessary to utilize suitable values for historical masonries in literature. Only after considering all this variables it could be possible to decide which kind of model is suitable to carry out the needed analysis and which values give to mechanical parameters.

A very relevant aspect in the behaviour of historical masonry is the pattern in which blocks laid inside the wall, usually called bond. If masonry is built in a “correct” way its structural behaviour reaches higher performances and its mechanical properties increase. Right typology of bond has linear, plane and regular horizontal joints, staggered vertical joints and presence of transversal connections. Other aspects contributing to the rightness of bond are the regularity in dimension of blocks and the thickness of mortar joints. When masonry is built respecting the so called “*regola d’arte*”[1] [*Giuffrè, 1991*] it shows a monolithic behaviour of walls and panels. Instead masonry realised in a not correct way, with a wrong bond, reaches lower performances, its mechanical properties decrease and may shows a non-monolithic behaviour, with choking in separate portions and layers. Reduction of performances increases with the number of defects.

Masonries made respecting the “*regola d’arte*” are usually present in important historical buildings. Indeed it has to be pointed out that in case of masonry arch bridges, and in particular in case of railway bridges built in the nineteenth and twentieth centuries, masonry is generally built in the correct way. Regarding the structural elements constituting masonry arch bridge, usually arches, piers and spandrel walls are made of good quality masonry, made of bricks or squared blocks of stones, while filling in spandrel and hollow piers is made with incoherent material, stone, sand and bricks. As regards to its structural role, filling could be considered as an isotropic material with brittle breaking. Instead mechanical behaviour of masonry is more complex and there are different approaches to its characterisation.

Afterward are given more information about mechanical properties and structural behaviour of masonry and are illustrated the principles methods of modelling and analysis.

[1] The ensemble of rules that have to be respected in bond to realize a correct masonry.

2.1.2 Mechanical properties of masonry

The masonry, as has been said, is a composite material formed by an ordered set of interconnected blocks, joined together by means of dry or mortar joints. The properties of masonry are strongly dependent on the properties of its individual constituents. Its components are identified in three main entities: the blocks, the mortar and the joint interface. The mortar and the blocks, stone or brick, are independent materials with their own mechanical properties. Instead the joint interface is an abstraction to represent interaction between mortar and blocks. Before describing the mechanical properties of the masonry is convenient to outline that of its components.

2.1.2.1 Properties of constituent materials

Numerous experimental studies deal with the response of mortar and blocks, whose results have been used in international regulations. The property of individual constituents that has been most investigated is the compressive strength, while in the literature there are few contributions on the tensile and the shear strength. The literature about compression tests of mortar and blocks in order to determine the compressive strength of masonry is almost exclusively related to the design of new buildings. This type of testing is useful to define the ultimate strength of the material, while provides little information about response linear and non-linear fields.

The study of the deformability of material has great significance in relation to the mechanism of breaking of wall due to compression, caused by the different deformability of mortar and blocks [*Hilsdorf, 1969*]. The greater deformability of mortar respect to blocks is prevented by friction that brings to an effect of confinement of mortar. This phenomenon, on one hand generates a state of three-axial compression on the mortar joints, while on other hand produces bi-axial tensile stresses orthogonal to the load in blocks. The three-axial compression state of mortar increase its compressive strength [*McNary and Abrams, 1985*]. Anyway compressive strength of blocks is higher respect to mortar, while deformability is lower. Blocks show an elastic-brittle behaviour; while mortar has a tensile strength considerably lower than that of blocks, breaking occur in the elastic-plastic phase and its behaviour under shear and compression actions is non-linear, with large inelastic deformations [*Page, 1978 and 1981*].

Tests regarding the behaviour of blocks and mortar under shear and tensile actions confirm very low tensile strength of mortar and brittle behaviour of blocks. At the same time the low tensile and shear strength of masonry depends mainly by the interface rather than qualities of material [*Van Der Pluijm, 1992*]. The joint interface is usually the weakest point of masonry, in which non-linear behaviour is more evident. Values of tensile and shear strength of the interface are very low. Breaking due by shear is one of main causes of collapse of buildings, especially under seismic or cyclic actions [*Atkinson, 1989*]. Shear tests under different level of compression shows a linear relation between compression stress and shear strength [*Van Der Pluijm, 1993*]. Cohesion of the interface is related to the quality of mortar, while friction does not depend by material. For this reason there are several models that represent interfaces with Mohr-Coulomb relation.

It has to be noticed that tests on constituent materials of masonry previously cited are usually performed in lab on specimens of modern masonries. Instead only few experimental campaigns of tests about compressive, tensile and shear strength have been carried out on historic masonries [*Binda, 1995*]. Afterward will be provided an overview of the principal techniques of tests *in situ* for assessment of mechanical properties of masonry.

2.1.2.2 Properties of masonry

The last Italian technical regulations about construction[2] gives values of the two main mechanical characteristic of masonry that have to be considered during the design: the characteristic compressive strength f_k, and the characteristic shear strength f_{vk}. Starting from these values it are also defined the modulus of normal

[2] NTC2008 “Nuove norme tecniche per le costruzioni”, Decreto Ministeriale 14/01/2008, Gazzetta Ufficiale n°19, 04/02/2008, Supplemento Ordinario n°30.

elasticity $E = 1000\, f_k$ and the modulus of secant elasticity $G = 0.4\, E$. The values of the characteristic compressive strength f_k, and of the characteristic shear strength f_{vk} depend by the kind and quality of blocks[3] and by the kind of mortar[4]. The European regulation[5] gives a semi-empiric relation for the characteristic compressive strength of masonry: $f_k = f_b^{\alpha} \cdot f_m^{\beta}$ in which f_b is the strength of blocks, f_m is the strength of mortar, k, α and β are corrective coefficients that have to be evaluated for each case, depending by elements, materials, geometry, texture. Such as Italian regulation, the characteristic shear strength f_{vk} is related to the characteristic compression strength f_k.

These values are provided for the design of new masonry buildings. In case of historical masonry it is necessary to do some relevant considerations in order to evaluate its right mechanical properties. Intrinsic randomness characterising historical masonries make difficult to have both reliable values of its mechanical properties and models suitable for each case: the observation has a preeminent role and theoretical models may be not able to describe correctly to the reality. Any way it is possible to refer to analytical models of the mechanical characteristic of a "general" masonry, related to its component and to its behaviour. Compressive, tensile and shear strengths of masonry are described, mainly referring to the studies carried out by [*Tassios, 1988*]. It is important to point out that object of the study is the ordinary masonry, meaning without any kind of reinforcement and built with a correct bond without constructive mistakes or damages.

The aim is to put in relation the aspects of historical masonry previously described, that are the result of empiric observations, to these analytic mathematical considerations, based on theoretical models and experimental tests. Theoretical models may not be directly applied to historical masonries, but provide some parameters useful to describe the characteristics and the behaviour of historical masonries too.

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[3] If blocks are made by natural stones or artificial bricks, the material, the dimension of the blocks..

[4] Mortars are classified on the base of their composition and compression strength in four categories: M1, M2, M3, M4.

[5] Eurocode 6, UNI ENV 1996-1-1:1998, specific for masonry structures.

2.1.2.3 Compressive strength

There are some parameters that influence the compressive strength of masonry, besides both mechanical and physical properties of its components, but also the modality of execution. Briefly:

- strength and the geometry of blocks;
- strength of mortar;
- type of bond and constructive system;
- thickness of mortar joints;
- strains of blocks and mortar;
- hygrospicity.

Compressive masonry strength f_{wc} depends by strength of blocks and mortar, therefore changes on the base of the kind and quality of materials used, and on the base of their configuration, so it is difficult to establish a "standard" strength. Actually there are several empiric relations for the compressive strength of masonry that combine it with the compressive strength of blocks f_{bc} and mortar f_{mc}. In literature can be found several empiric relations obtained by experiments:

- For good quality materials:

  $f_{wc} = \sqrt{f_{bc}}$; $f_{wc} = \sqrt[3]{f_{mc}}$; $f_{wc} = \sqrt[4]{f_{mc}}$ [*Hendry, 1981*]

  $f_{wc} = f_{bc}/6 + \sqrt{(f_{bc} \cdot f_{mc})}/4 - f_{mc}/20$ [*Tassios, 1983*]

  if $f_{bc} < f_{mc}$: $f_{wc} = (1 - 0.8 \cdot \sqrt[3]{\alpha}) \cdot f_{bc}$

  if $f_{bc} > f_{mc}$: $f_{wc} = (1 - 0.8 \cdot \sqrt[3]{\alpha}) \cdot [f_{mc} + 0.4(f_{bc} - f_{mc})]$

  in which α is thickness of joint [*Tassios, 1985*]

- For middle quality materials:

  $f_{wc} = 0.7\sqrt{f_{bc}} \cdot \sqrt[3]{f_{mc}}$ [*Bröcker, 1961*]

- For low quality materials:

  $f_{wc} = (2/3\, f_{bc} - f_0) + \delta\, f_{mc}$ [*Tassios, 1988*]

  in which f_0 and δ are coefficients depending on the blocks material

The parameters that influence the cubic compressive strength of masonry can be divided in two groups. The first group consists of those parameters that affect the real mechanics of collapse: the kind and the quality of blocks and mortar, the thickness of joints and the adherence. To the second group belong the parameters that influence the distribution of stresses inside blocks, thus the static behaviour: the geometry of blocks, the kind of support and the way they are realised.

In any case it is important to remark that the compressive strength of masonry is always less than that of blocks, for any kind of material. There are some particular cases on which masonry can reach value of strength bigger than blocks. It was observed that in case of steel plate connectors used for the linkage between blocks the strength of masonry was greater than the blocks strength, because of the three-axial state of compression due to the contrast against lateral deformation given by the toothing [*Hendry, 1981*]; but it refers to reinforced masonries. Anyway the quality of material used for the linkage and the way as bond is realised is very relevant for the whole behaviour of masonry. That consideration may confirm the importance of a correct bond for the strength and the behaviour of masonry, as demonstrated by the observation of historical masonries [*Giuffrè, 1992*].

In any masonry observed it is possible to notice that under compressive stress masonry gives way with cracks, due to the tensile stresses orthogonal to compression, parallel to the principle direction of load. As previously said, the reason of break through vertical cracking is due to the different characteristics of the strains of its components [*Hilsdorf, 1969*]. In fact respective movements of materials are not allowed because of the adherence that stops the relative displacements: bricks are under bi-axial tensile stresses and mortar under three-axial compression stresses. The crisis of the bricks because of tensile correspond to the break of masonry due to compression. On the base of these considerations some researchers - Hilsdorf, Hendry, Tassios - developed theoretical elastic models to evaluate the relation between stress and strain and the compression strength of masonry.

To better understand the mechanics of breaking under compression stress it is convenient to consider a little cube made by bricks and mortar under a mono-axial stress of compression σ_z. The hypothesis of this model implies the elastic behaviour of material, presuming both mortar and bricks homogeneous and isotropic. The consequent lateral stresses due to the compression in direction z are: tensile stress in direction x and y for bricks, σ_{bx} and σ_{by}, and compressions stress in x and y for mortar, σ_{mx} and σ_{my}, due to the friction between mortar and the faces of bricks:

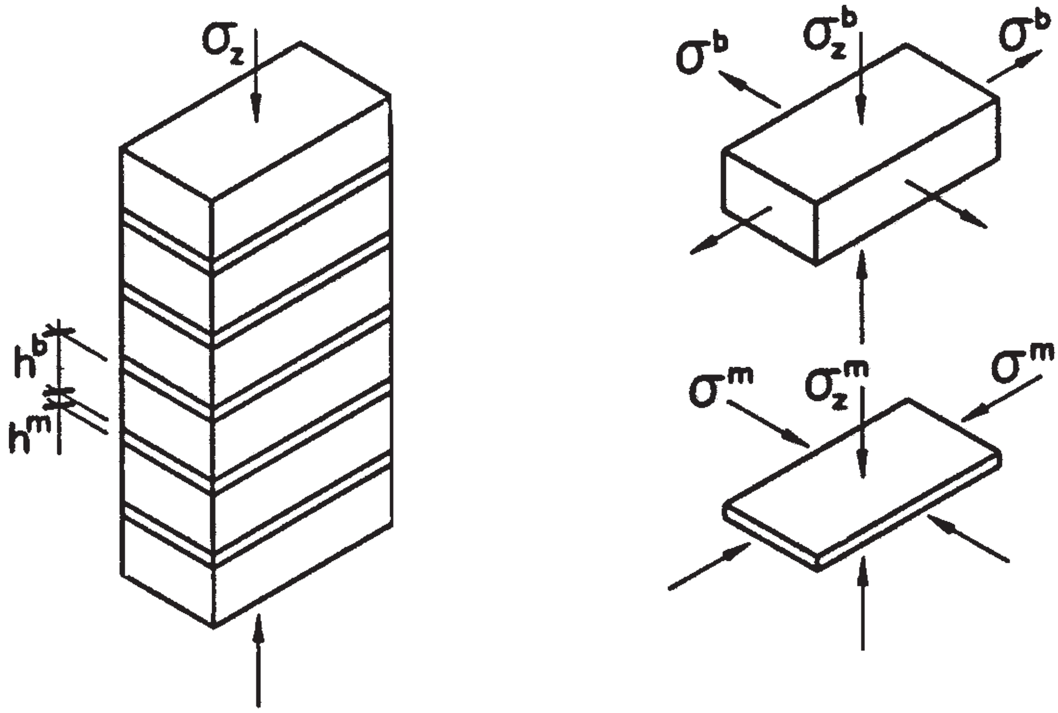


Fig 2.1 - Stresses inside unites and mortar

Imposing the equilibrium of stresses and the congruence of deformations it is possible to obtain the tensile stress of bricks $\sigma_b = \sigma_{bx} = \sigma_{by}$. Because of the Hooke's law, strains of bricks along x and y, ε_{bx} and ε_{by} , are:

$$\varepsilon_{bx} = 1/E_b \cdot [\, \sigma_{bx} + \nu_b \,(\, \sigma_z - \sigma_{by})]$$

$$\varepsilon_{by} = 1/E_b \cdot [\, \sigma_{by} + \nu_b \,(\, \sigma_z - \sigma_{bx})]$$

The strains of mortar along x and y, ε_{mx} and ε_{mx}:

$$\varepsilon_{my} = 1/E_m \cdot[- \sigma_{mx} + \nu_m (\sigma_z + \sigma_{my})]$$
$$\varepsilon_{my} = 1/E_m \cdot[- \sigma_{my} + \nu_m (\sigma_z + \sigma_{mx})]$$

In which E is the Young modulus and ν is the Poisson's coefficient. The congruence implies that strains of bricks and mortar are equal:

$$\varepsilon_{bx} = \varepsilon_{mx} \text{ and } \varepsilon_{by} = \varepsilon_{my}$$
$$1/E_b \cdot[\sigma_{bx} + \nu_b (\sigma_z - \sigma_{by})] = 1/E_m \cdot[- \sigma_{mx} + \nu_m (\sigma_z + \sigma_{my})]$$
$$1/E_b \cdot[\sigma_{by} + \nu_b (\sigma_z - \sigma_{bx})] = 1/E_m \cdot[- \sigma_{my} + \nu_m (\sigma_z + \sigma_{mx})]$$

To have the equilibrium, the resultant of compression forces of mortar have to be equal to the resultant of tensile forces of blocks, in both directions x and y:

$$\sigma_{bx} \cdot d \cdot t_b = \sigma_{mx} \cdot d \cdot t_m \quad \rightarrow \quad \sigma_{bx} = \alpha \cdot \sigma_{mx}$$
$$\sigma_{by} \cdot d \cdot t_b = \sigma_{my} \cdot d \cdot t_m \quad \rightarrow \quad \sigma_{by} \quad \alpha \cdot \sigma_{my}$$
$$\alpha = t_m / t_b < 1$$
$$(d = width;\ t_b = block\ thickness;\ t_m = mortar\ thickness)$$

Defining β as the ratio between Young's modulus of blocks and mortar, multiplying everything for E_b and revising the relations about deformations and tensile:

$$\sigma_{bx} + \nu_b\, \sigma_z - \nu_b\, \sigma_{by} = \beta \cdot[- \sigma_{bx} / \alpha + \nu_m (\sigma_z + \sigma_{by} / \alpha)]$$
$$\sigma_{by} + \nu_b\, \sigma_z - \nu_b\, \sigma_{bx} = \beta \cdot[- \sigma_{by} / \alpha + \nu_m (\sigma_z + \sigma_{bx} / \alpha)]$$

The relation of the tensile stress in bricks due to compression is:

$$\sigma_{bx} = \sigma_{by} =[\alpha\, (\nu_m - \beta\nu_b) / (1 + \alpha\beta - \nu_m - \alpha\beta\, \nu_b)]\cdot\sigma_z$$

In which $\beta = E_m / E_b < 1$ and it represent a coefficient of homogenization. This relation expresses the value of the tensile stresses to which the brick is subject, in function of the compression that acts on the prism, σz.

It is important to notice that the presence of the tensile stresses σ_{bx} and σ_{by} implies a decreasing of the value of compression stress σ_z that bring to failure, σ_{zu}. Tensile strength of block is related to the compression strength of block: $f_{bt} = \lambda f_{bc}$. Presuming a linear relation between tensile and compression related to the breaking under compression:

$$\sigma_{zu} / f_{bc} + \sigma_t / \lambda f_{bc} = 1$$

The compression strength of masonry f_{wb} is equal to σ_{zu}, therefore:

$$f_{wc} / f_{bc} = 1 : \{1 + [\alpha (\nu_m - \beta \nu_b) / \lambda (1 + \alpha\beta - \nu_m - \alpha\beta\, \nu_b)]\}$$

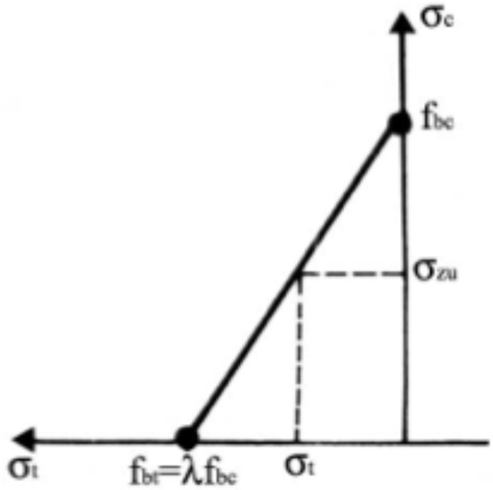


Fig 2.2 - Strength domain of masonry subjected to bi-axial compression – tensile stresses

Applying this relation it is possible to outline the influence of the thickness of joints in the compression strength, in fact the strength decrease as the mortar joint thickness increase. This model allows also pointing out the relationship between masonry strength and the resistance of the material of its components. Applying the same relation but utilising different values for bricks and mortars compressive strength of masonry varies: f_{wc} raises up when also f_{bc} raises [*Hendry, 1981*]. The velocity of increasing depend by the quality of mortar: if the quality of mortar is good, the strength of masonry increase rapidly, otherwise slowly, referring to the proportionally. At the same time the increase of f_{wc} is not linear respect to the increase of f_{mc}: to double f_{wc} it needs to increase fivefold f_{mc}.

Masonry is generally treated as a linearly elastic material, although tests indicate that the stress-strain relationship is approximately parabolic. This assumption is considered reliable for the calculation of normal deformation because under service conditions masonry is generally loaded up only to a fraction of its ultimate load. The deformation behaviour of masonry, the relation σ - ε between stresses and strains can be expressed as:

$$\sigma / f_{wu} = 2(\varepsilon / \varepsilon_u) - (\varepsilon / \varepsilon_u)^2$$

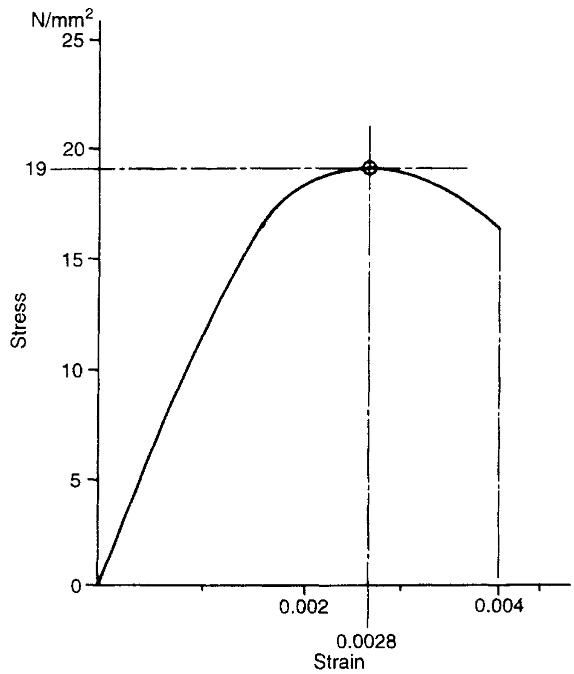


Fig 2.3 - Typical stress-strain curve for masonry, taken from [Hendry et al. 2004]

The diagram is based on tests [*Hendry et al., 2004*] and shows the relation between stresses and strains: ε_0 is the maximum strain of a specimen under compression loads, with a value between 0.25 % and 0.35 %; f_{wu} is the ultimate compressive strength; E_{w0} is the Young's modulus for the initial portion while E_{wu} is the one at maximum compression. This relation describes correctly the initial rising portion of the diagram while is not completely reliable regarding the second portion. The analysis of the deformation behaviour of masonry is extremely laborious and suffers from uncertainty, especially when trying to determine the value for the elastic modulus E. The elastic modulus is considered equal to the tangent to the curve σ - ε only for values of f_w less than $0.4f_{wu}$. Without direct experimental measurements it is possible to refer to empirical relations [*Wesche, 1974; Hendry, 1981; Mauerwerk Kalender, 1982*]. Values provided belong to those ranges:

- Young's modulus of blocks: $E_b = (300 \div 400)\, f_{bc}$
- Young's modulus of mortar: $E_m = 900\, f_{mc}$
- Young's modulus of masonry: $E_w = (500 \div 1000)\, f_{wc}$
- Poisson's coefficient: $\nu = 0.1 \div 0.2$
- Fluage: $\varphi = 0.75 \div 2.5$

2.1.2.4 Tensile strength

It is not possible to evaluate the tensile strength of masonries referring to an evident behaviour. In fact, the reaction of masonry to tensile stress varies with the angle of slope of cracks. When tension is applied in vertical direction the breaking affects mortar, provoking the detachment between blocks and mortar, indeed the cracks are horizontal. For this reason the "vertical" tensile strength of masonry can by expressed as a percentage value of the tensile strength of mortar:

$$f_{wt} = \xi \cdot f_{mt}$$

in which ξ is a coefficient that depends by the compactness of mortar and by its conservation state. In "ordinary" cases, ξ has a value close to 2/3.

The tensile strength of masonry in horizontal direction is related both to the resistance opposed by joints to the sliding between blocks and mortar and to the tensile strength of blocks. It implies two mechanisms of cracking: in the first case the cracks occur only in the joints – horizontal tensile strength depends only by mortar strength - while in the second one cracks occur also in the blocks – tensile strength depends also by blocks strength.

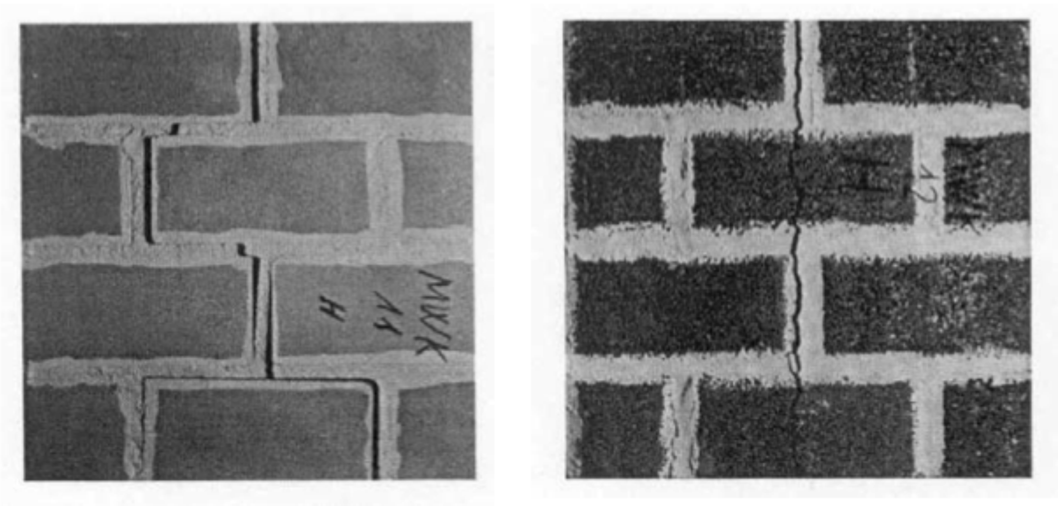


Fig 2.4 - Different mechanism of cracking due to horizontal tensile [Bakes, 1985]

The problem is that is not known how to calculate the tensile strength of masonry respect to the variation of the angle of the main tensile stress. Moreover, it should be taken into account even the role of the lateral compression stress, which plays a negative role. Masonry has a tensile strength very low and uncertain. These considerations imply that usually masonry is considered as a material without tensile strength and that it is not possible to evaluate its shear strength on the base of its tensile strength. This is the reason why masonry is frequently modelled as NRT (no tension resistant material).

2.1.2.5 Behaviour under complex stress states

All the combinations of effort that can involve all three spatial dimensions give complex stress states. Even more than in the previous case, in which properties of masonry have been described referring to its mono-axial behaviour, the anisotropy of masonry makes the study of these stress states quite onerous. However, the results obtained from experimental tests denote that the anisotropic behaviour of masonry can be reduced to an orthotropic behaviour. For this reason, the bi-axial tests are very interesting in order to understand the global behaviour of masonry. In fact, tests highlight several aspects of the response of masonry for actions normal to the planes of the mortar joints, but they also allow understanding the shear response.

Considering the behaviour of masonry under tensile stress, usually the bi-axial tests have been carried out to investigate the behaviour of masonry in relation to the variation of the angle of horizontal mortar joints respect to the principal stresses of compression/tension. In this field the works of Page represent the main reference [*Page 1981 and 1983*]. Three types of bi-axial tests have been carried out: tensile – tensile, compression – compression and tensile – compression. The results allow defining strength domains of resistance related to generic stress states in the plane. In the case of tensile – tensile tests the strength domain is strongly dependent by the angle of horizontal mortar joints. Same relevance of this aspect is highlighted by tests in the case of tensile – compression. Several mechanism of cracking can occur, involving both horizontal and vertical mortar joints, depending by the inclination of the joints. It is interesting to point out that when the angle of inclination is equal to 0, so that tensile acts horizontally while compression acts vertically, there is an increment of the tensile strength of masonry, due to the effect of vertical compression that do not allow sliding and opening joints. Instead in the case of compression – compression tests the behaviour of masonry is not so strictly dependent by the inclination of joints.

Considering the deformation due to bi-axial stresses is interesting to point out that in case of compression – compression tests the response of masonry is strongly non linear, while in the case of tensile – compression tests the breaking occur in the linear field. Moreover during the linear phase the behaviour of masonry could be considered isotropic, while in the non-linear phase is anisotropic, due to the weakness of mortar respect to blocks [*Page, 1983; Dhanasekar et al., 1985*].

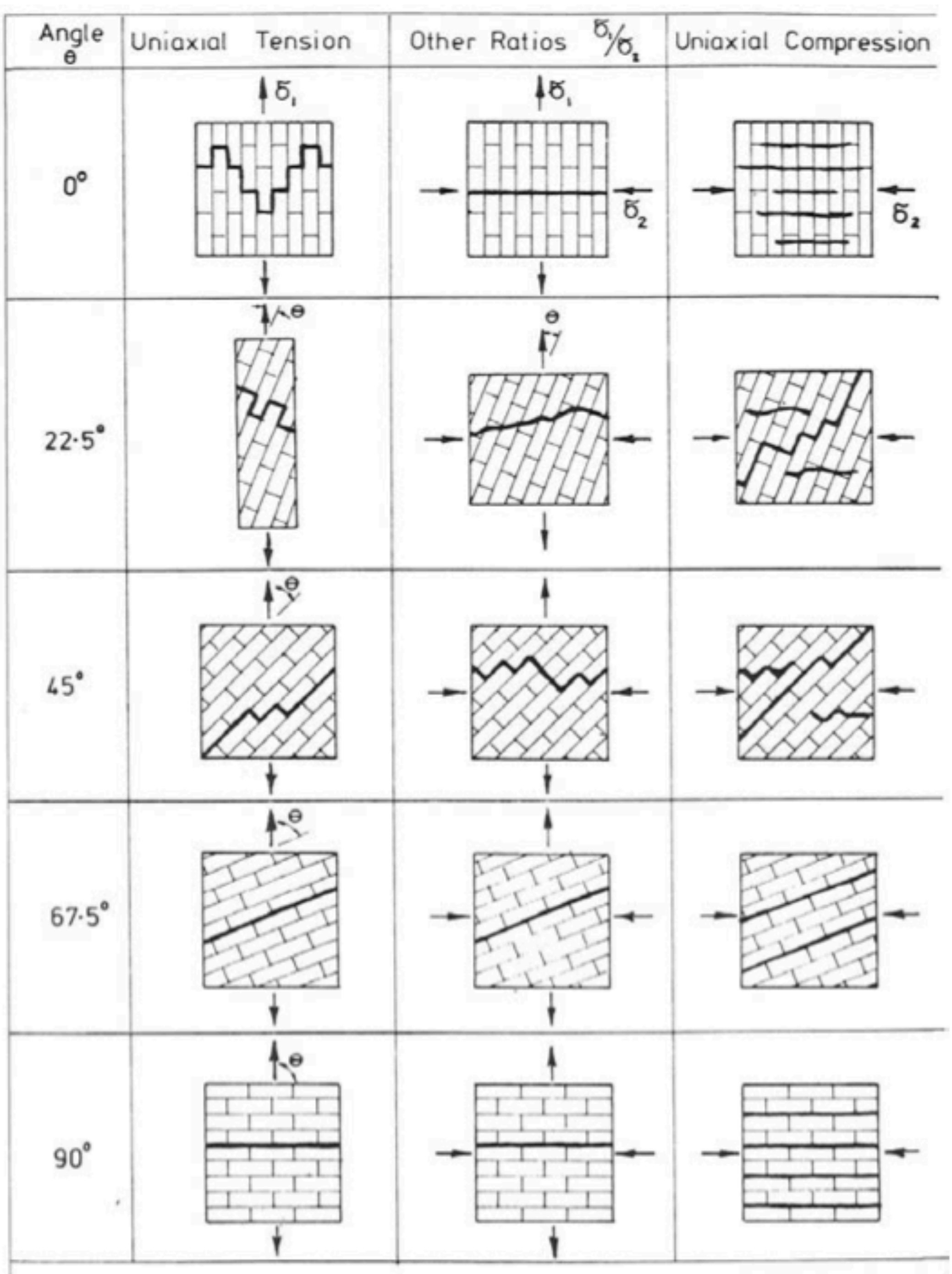


Fig 2.5 - Mechanisms of cracking due to tensile – compression tests [Page, 1981]

2.1.3 Modelling of masonry

Considering the different types of masonry structures and the wide variety of issues to be addressed, static and dynamic modelling of masonry historic buildings represents today one of the most important topic of research in the field of civil and conservation engineering. There are many reasons that may motivate the modelling of an historic building: the level of stresses under service loads; the evaluation of a structural subsiding; the assessment of the safety of a structure that has been modified for functional or performance requirements; the verification of the effects of changes in the environment surrounding the structure; the monitoring in order to understand the evolution of the damage over time and the residual safety of the building; the seismic vulnerability assessment; the plan of consolidation; and many others. For this reason the definition of objectives, the recognition of problems and the identification of the structure are the essential requirements for the modelling of masonry. Moreover the strategy of masonry modelling deals with important choices:

- The scale level: the dimension of the object of the analysis may considerably vary, from an ensemble of buildings until the architectural detail, going through the building or parts of it; moreover, at the material level, the masonry can be modelled with different levels of detail: from a micro-scale - in which the constituent elements of masonry are taken into account – to a macro-scale – in which object of modelling is a whole portion of structure.

- The structural schematisation: the choice of the structural scheme is the synthesis of geometric and mechanical structure; in masonry buildings there are inherent difficulties in identifying the structural scheme, later will be described the specific case of masonry bridges.

- The constitutive law: in order to idealise the mechanical behaviour is necessary to formulate a number of assumptions and to define a constitutive law that allows to briefly describe the material behaviour.

- The type of analysis: linear or non-linear or limit analysis, either static or dynamic.

These choices are interconnected whit each other and denote the main problem of modelling: the mediation between the accuracy of the model and the need of synthesis. The accuracy is needed in order to properly describe the reality, but the synthesis is essential in order to have understandable and verifiable results. The right proportion between accuracy and synthesis depend on type of analysis that has to be carried out and by the complexity of problems to face.

The difficulty in modelling such structures depends on three fundamental problems:

- The composite nature of masonry, made up of a complex system of blocks and joints, assembled with several possibilities of bond and realised with different constructive techniques and materials, as described in the previous paragraphs.

- The size of heterogeneity respect to the size of masonry structure, which strongly influence the scale of model.

- Several geometric complexities typical of masonry constructions, and the relative difficulties on the structural schematisation, which impose the adoption of 2D and 3D modelling approaches.

In the study of masonry structures the use of local models, describing parts of the structure, or global models, representing the structure in its whole, is a hard topic. In fact the preparation of a global model is time consuming and, because of the big dimension of the model, in the analysis of the results some important aspects could be lost in sight. From one point of view, it's preferable modelling some structural parts and details instead of modelling great and complex structures. More in general, a global model is worth because it is able to implicitly catch the interactions between the different parts of the building, but usually it is too complex from the conceptual and operative point of view in an historical construction. On the other point of view local models tend to simplify sometimes the analysis through rough hypothesis; nevertheless they have the value of using intuitive calculus schemes and easy interpretability of the results.

In literature, many models and tools of analysis have been developed. They may be distinguished by the scale of the problem faced, constructive features, type of masonry, acting forces. It is possible to divide the different approaches to masonry modelling on the base of the scale:

- The micro-scale: masonry is described by modelling separately its constituent elements; this type of modelling is fit to analyse of structures of small dimension or made by huge blocks.

- The meso-scale: masonry is considered as an equivalent continuum material and constitutive equations are formulated through homogenisation procedures - obtained from the micro-scale - or phenomenological models; these constitutive models usually are implemented in finite element procedures and may be used for complex masonry structures.

- The macro-scale: it is used for constructions in which a characteristic behaviour may be a-priori recognised, modelled through structural elements of bigger dimensions, called macro-elements; for instance, such approach is adopted in codes for the seismic analysis of buildings.

Here an overview of modelling in literature is given, on the base of the scale of model.

At micro-scale, masonry is modelled as a discrete system of elements: blocks, joints and/or interfaces. Many contributions have focused on micro-polar modelling of periodic masonry [*Masiani et al., 1995; Sulem and Mühlhaus, 1997; Stefanou et al., 2008*] based on an idealisation of the masonry as an assemblage of rigid blocks interacting through linear elastic interfaces and represented as a Lagrangian system. To overcome the limits deriving from the assumption of rigid blocks, Casolo proposed a *Cosserat* homogenisation based on a heuristic evaluation of the mean local rotation of the brick units [*Casolo, 2006*]. The *Cosserat* homogenisation technique has been proposed first for continuously deformable heterogeneous media [*Forest and Sab, 1998*] and then has been extended to periodic masonry [*Bacigalupo and Gambarotta, 2011*] and by Addessi the last contribution to include elastic damage constitutive equations at the micro-scale [*Addessi et al., 2010*]. In Bacigalupo and Gambarotta [*Bacigalupo and Gambarotta, 2011*] an evaluation of the reliability of *Cosserat* homogenisation has been carried out by analysing a boundary shear layer problem concerning masonry walls.

At micro-scale, the main approaches developed in the literature are based on the equilibrium limit analysis. In general, blocks are supposed rigid and infinitely resistant, while the non-linearity of the material is concentrated in joints [*Livesley, 1978; Gilbert and Melbourne, 1994; Baggio and Trovalusci, 2000; Ferris and Tin-Loi, 2001; Orduna and Lourenco, 2005*]. Other approaches are based on the distinct element method [*Cundall, 1976*]. They require a dynamic incremental analysis, performed through the explicit integration of the equations of motion [*Azevedo et al., 2000; De Felice and Giannini, 2001; Lemos, 2007*]. Most of these approaches have been developed to model periodic regular masonries. The issue of modelling irregular masonries is still open.

At meso-scale, masonry is modelled as an equivalent continuum. The constitutive model may be defined either through a phenomenological approach - smeared cracking or NRT[6] models [*Lourenco et al., 1998*; *Pietruszczak and Ushaksaraei, 2003*] - or through homogenisation or direct identification techniques. Advanced homogenisation techniques have been developed in order to define in-plane [*Anthoine, 1995; Lourenço and Rots, 1997; Cecchi and Sab, 2002a*] and out-of-plane [*Cecchi and Sab, 2002b; Cecchi et al., 2005*] elastic properties of the material and its failure domain [*Corigliano and Maier, 1995*; *De Buhan and De*

[6] Not Resisting Tension.

Felice, 1997; Sutcliffe et al., 2001; Sab, 2003; Milani et al., 2006; Cecchi et al., 2007; Cecchi and Milani, 2008]. These techniques present the advantage of keeping memory, at meso-scale, of the main characteristics of masonry at micro-scale. However currently their complexity does not allow the formulation of an evolutive non-linear constitutive law. An interesting approach to face this problem is the Transformation Field Analysis (TFA) method, recently applied to masonry by Sacco [*Sacco, 2009*].

Further methods are based on multi-scale approaches, in which the microstructural behaviour of masonry (micro-scale) is related to the continuum (meso-scale) through a micromechanical analysis [*Gambarotta and Lagomarsino, 1997; Luciano and Sacco, 1997; Pegon and Anthoine, 1997; Massart et al., 2004; Calderini and Lagomarsino, 2008*]. All the above cited constitutive models have been developed by describing masonry as a Cauchy continuum, for which two main drawbacks may be pointed out: it does not allow to keep into account the absolute size of the microstructure, and to describe scale effects; the macroscopic fields of the RVE are supposed non-uniform. In order to overcome such drawbacks, various authors have proposed models based on generalised continua. Particular attention has been paid to the Cosserat continuum, in which an internal scale parameter is considered [*Masiani et al., 1995; Trovalusci and Masiani, 2003; Casolo, 2006; Brasile et al., 2007*].

In the case of meso-scale, such as in the case of the micro-scale, the modelling techniques developed in literature mainly refer to regular periodic masonries. Indeed in the common practice, in order to analyse non-periodic and/or irregular masonries, phenomenological NRT or smeared cracking constitutive models have been adopted most frequently. Recently, however, these masonry types have captured the attention of the researchers. In this field is interesting the work carried out by Cluni and Gusella [*Cluni and Gusella, 2004*], oriented to define the elastic properties of the material. Starting from studies on non-periodic bodies homogenisation techniques, Cecchi and Sab have recently proposed the elastic homogenisation of non-periodic regular masonries through a perturbative approach [*Cecchi and Sab, 2009*].

At the macro-scale there are two relevant modelling approaches: the structural element modelling and the equilibrium limit analysis of "macro-blocks". Both these techniques have been developed for the analysis of buildings subjected to horizontal forces, in particular seismic forces.

The first approach aims at evaluating the overall response of masonry structures made up of walls with regular openings, describing with adequate accuracy the in-plane behaviour of single structural elements. The technique is based on the identification of macroscopic structural elements (portions of structure such as "piers" or "spandrels"), defined from a geometrical and cinematic point of view through finite elements (shell or frame) and described from a static point of view through their internal forces.

A first class of models is based on the use of mono-dimensional elements, such as "variable geometry" struts [*Calderoni et al.,2007; Braga and Dolce, 1982*] or shear-deformable beams [*Tomazevic, 1978; Tomazevic and Weiss, 1990; Braga and Liberatore, 1990*]. Other models consider the walls as "equivalent frames", in which deformable elements - piers and spandrels - connect rigid nodes - parts of the wall which are not usually subjected to damage. Masonry panels, in which the non-linear response is concentrated, may be described both through detailed models or through more simplified ones, like as non-linear beams [*D'Asdia and Viskovic, 1994; Magenes and Della Fontana, 1998; Brencich and Lagomarsino, 1998; Magenes et al., 2000*]. By concentrating damages, sliding and rotations in predefined sections of the structural elements, these models allow performing non-linear incremental collapse analyses of entire buildings. The modelling of the whole structure is obtained assembling masonry walls, idealised as 2D frames, and horizontal floors, not necessarily assumed as rigid.

It is worth noting that the above described macro-scale approaches are oriented to evaluate the overall response of masonry constructions by considering the response of structural elements to only in-plane forces. Since in complex masonry structures, the lack of connections between its parts may induce partial collapses due to out-of-plane actions, a further macro-scale modelling approach is present in the literature: the equilibrium limit analysis of macro-blocks. It may be useful to evaluate the response of masonry structures, which may be reasonably assumed as monolithic. The latest approach can be successfully adopted for large-scale models [*Abruzzese et al., 1992; D'Ayala and Speranza, 2003; Casapulla and D'Ayala, 2006; Curti et al., 2006*].

In this thesis the modelling of the masonry will be addressed using mainly models at the meso-scale (with reference to the micro-scale), while the macro-scale will be considered only in specific cases of modelling of masonry arch bridges.

2.1.4 Analysis of masonry

On masonry structures it is possible to carry out numerous analysis types. It is possible to divide them in three groups: linear analysis, non-linear analysis and limit analysis.

Linear analyses are the simplest type: they assume the elastic behaviour of materials, obeying to the Hooke's law. Indeed it is necessary to know the elastic properties of masonry and the maximum allowable stresses. This kind of analyses allows obtaining the deformed shape and the stress distribution in the structure. In order to take into account the possibility of cracking of masonry and the consequent redistribution of stresses, it is possible to assume a reduction of stiffness in correspondence of the cracked areas. Linear analyses are useful to understand the behaviour of masonry structure under service loads, when the material still shows an elastic behaviour, but they are not suitable to establish the collapse limits. It is convenient to use of this type of analysis to study the whole structure in order to identify its global behaviour and to find out the areas in which tension can produce cracking.

There are two different kinds of linear analyses:

- Linear static, in which a forces system is distributed along the building assuming a linear relationship between loads and induced responses. Forces applied are usually vertical, self-weight and dead loads, but is also possible to apply horizontal static forces.

- Modal analysis, to evaluate the natural frequencies of vibration of the structure. Modal analysis, associated with the design response spectrum, can be performed on bi- and three-dimensional structure to evaluate the stresses values in the elements. In this analysis it needs to take into account all the vibration mode with a participating mass bigger than 5% and summing them in order to reach at least the 85% of the whole mass.

The verifications of security at the ultimate limit state consist in the comparison between the strength of each structural element and the actions due to combined compressive and bending stress in and out of plane, plane shear and sliding.

Non-linear analyses allow studying the complete behaviour of the structure: elastic field, cracking and post elastic field, until the collapse. There are two different types of non-linear behaviour: *mechanical*, due to the non-linearity of the material; *geometrical*, due to the fact that the application points of load change with the increase of actions. Non-linear analyses are very useful to investigate affected by damages, in order to identify the loss of stiffness. To carry out this type of analyses it is necessary to know both elastic and inelastic properties and the strength of materials. Non-linear analyses can provide as results the stress and strain distribution and the damage in constitutive function until the collapse of the structure.

Non-linear analyses may be performed in static or dynamic field:

- Static non-linear analyses, known as "pushover" analysis, apply to the structures vertical and horizontal loads, monotonously increasing them until the collapse. The method can be used both to evaluate the bearing capacity of existing buildings and to perform seismic analysis; it is provided by regulations. The analysis is frequently performed on bi-dimensional portions of building extrapolated from the whole structure.

- Dynamic non linear analyses, known as "time history", allows to carry out a dynamic analysis in the time domain to evaluate strains and stresses due to actions variable during time, such as seismic forces. This type of analysis is suitable for both linear and non-linear field and allows simulating the complete behaviour of the structure during the length of an earthquake, but because of its complexity it is not frequently used in practice.

The verifications of security at the ultimate limit state, in this case of non-linear analyses, consist in the comparison between the capacity of ultimate displacement of the structure and the demand of displacement.

Limit analyses have the purpose of determine the collapse load, identifying a multiplier of loads that provokes the collapse. Limit analysis refers to two different theorems [*Drucker et al., 1952*]:

- The static theorem (lower bound): assumes that the plastic collapse multiplier load is the largest of the entire multipliers correspondent to the static admissible set (a stress distribution in equilibrium with the external forces, which respect the plastic conditions in any point of the structure).

- The kinematic theorem (upper bound): in which the plastic collapse multiplier load is the smallest of the entire multipliers correspondent to the kinematic admissible set (a kinematic mechanism, related to the distribution of plastic hinges, which respects the kinematic condition).

Therefore there are two possible methods of limit analyses:

- The static method, which assumes a static admissible distribution of stresses in order to find the maximum multiplier of load;

- The kinematic method, which assumes a kinematic admissible distribution of displacements, in order to define collapse mechanisms depending by geometrical parameters in order to find the minimum multiplier that activate the mechanism.

According to the uniqueness theorem, a multiplier that is statically and kinematically admissible coincides necessarily with the collapse multiplier.

Limit analyses are very helpful in the analysis of masonry buildings, because it is difficult to establish the real values of stress, while it is possible to study their structural behaviour through the identification of the possible mechanisms of collapse. As previously said, the masonry constitutive model is of fragile type, with a high value of collapse in compression compared to tension. The collapse tension stress is not only small but is also characterised by a high uncertainty of evaluation. Therefore this type of analysis is suitable to be applied on NRT models and macro-blocks models. The applicability of limit analysis to masonry structures has been firstly investigated by [*Coulomb, 1773*] and afterwards was object of study for many researchers, in particular regarding the strength of masonry arch. Heyman provided the main contribution in this field [*Heyman, 1966 and 1982*], stating the basic hypotheses on the mechanical behaviour of masonry. He gave way to the modern limit analysis, coupling the traditional pre-elastic theories with the *limit design* principles developed during 50's for steel structures. This argument will be faced more in detail in the next paragraph, about the behaviour of masonry arch.

SECTION 2 – Part 2

The sub-structural level: the masonry arch

2.2.1 The masonry arch

The masonry arch is the main element of the historic bridge, which connotes its form and structure and defines its architectural and engineering characteristics. The structural behaviour of the whole bridge is strongly dependent by the behaviour of the masonry arch, which provide the main contribution to the load-bearing capacity, although not the only one. For this reason the comprehension of its behaviour is fundamental to the study and the understanding of the global behaviour of masonry arch bridges.

The masonry arch is a structure made of wedge shaped blocks - stones or bricks – called *voussoirs*, placed one next to the others, with or without mortar joints, in order to precisely create an arch ring. In large-span arch voussoirs are usually stones cut with very precision and assembled without mortar, or just with a minimum of it. Ancient arches built by Romans were usually made like this, such as many bridges built from renaissance to the XIX century. Instead small arches may be realised using stones roughly cut or bricks assembled with mortar joints. Masonry arch is built upon temporary false-work, called centering, generally made of timber, that are removed once the arch has been completed with the laying down of keystone, by means of wedges or similar provisional devices developed during the history.

In bridges, when the arch has been completed and the centering removed some filling is placed on the arch in order to create an horizontal extrados at the desired height to carry the road. Often part of the filling is placed on abutments before the removal of centering, in order to stabilise the arch ring. Filling could be made with different materials and is retained by spandrel walls, built on the arch rings on the two face of the bridge. In large bridge a series of parallel masonry walls may be realised to carry the road. The larger part of the self-weight of the arch is due to the backfill and spandrel, that even if are not “real” structural elements have a stabilising effect on the arch, distribute the loads maintaining the symmetry and gives a contribution to the load bearing capacity. This topic will be faced with more details in the next part of this section, regarding the global behaviour of masonry arch bridges and the role of each structural element.

The barrel may have the same height of the external arches or could be lower, that is a situation quite common. Moreover usually the masonry of the barrel is realised in a different way respect to the external surface: voussoirs might be cut with less precision, because they are not visible, or barrel may be made of bricks while the external arch is in stones. Not in every case is possible to consider a constant radial thickness. The parallel arch rings that compose the barrel may be independent but very often the arch voussoirs have different axial lengths in order to interlock each other and create a continuous prismatic arch barrel.

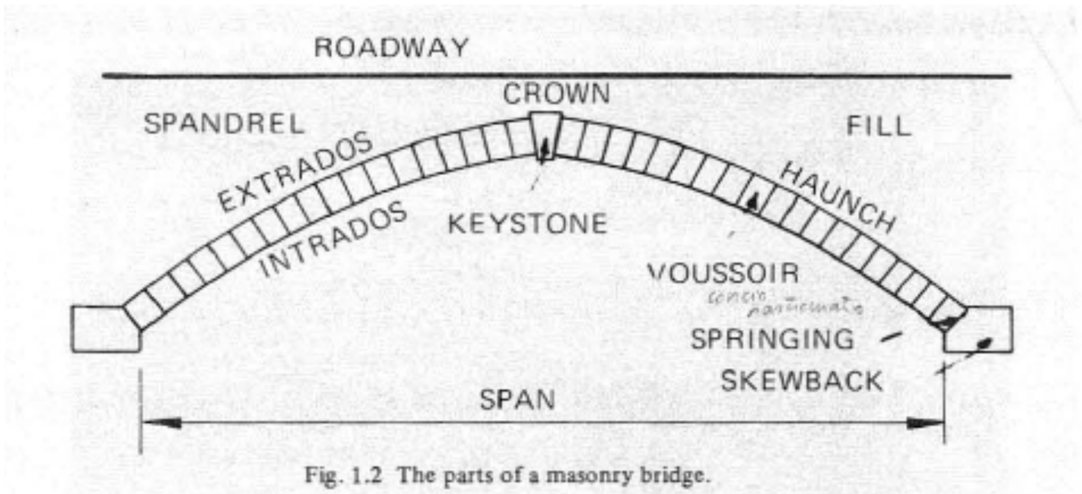


Fig. 1.2 The parts of a masonry bridge.

Fig 2.6 - Parts of a masonry arch, taken from [Heyman, 1982]

2.2.2 Structural behaviour

The arch is a structure that transmits the loads applied and the self-weight to the abutments or to the piers through compression. Masonry arch is a structure whit a very long history, the study of its behaviour began since the middle age, however the complete understanding of its behaviour has been reached only in the last century. For thousands of years its behaviour was supposed on the base of experience and practice, without any theoretical consideration. Starting from renaissance, many authors dealt with this topic providing geometrical and empirical methods for the design and assessment. In the XVIII century with the born of the theory of structure the approach become more theoretical. The study of masonry arch can be considered as the first step in the development of this new subject. However the development of the new materials – steel and concrete – has taken over to the masonry, indeed even the theory has taken a different path. The use of masonry became obsolete, even if for all the XIX century and even in the beginning of the XX century several masonry arch bridges have been built around Europe. As a consequence the study of masonry arch was neglected. Only in the XX century its study has begun to arouse again interest among researchers who have tried to mend the rift between the theory of structures and the study of the masonry arch, applying the modern principles to this ancient structure. A complete overview of the history of arch analysis and the evolution of the structural theories has been provided by Kurrer [*Kurrer, 2008*].

Many authors dealt with this issue. A complete and exhaustive dissertation about the behaviour of masonry arch has been given by Heyman. Heyman's contributions are so fundamental that it is difficult to imagine today's state of the art without his work [*Kurrer, 2008*]. Initially in its "Stone skeleton", 1966, and later in "The masonry arch", 1982 - which can be considered as a milestone and the main reference to the study of the behaviour of masonry arches – Heyman discussed the application of ultimate load theory to for masonry structures and voussoirs arches [*Heyman, 1966 and 1982*].

The simplest tool for the analysis of arches is the funicular polygon, which is a graphic method that allows determining the resultant, its direction and the application point of a system of vectors applied in a plane. This method may be used to determine a possible line of thrust and to find the equilibrium. The values of the horizontal reactions have to be known or assumed, some preliminary statics has to be done in order to ensure that the system is in equilibrium. By the way the funicular polygon provide the line of thrust in an arch subjected to a certain load, but it is the thickness of the voussoris surrounding the line of thrust that give the stability of the arch [*Heymann, 1982*].

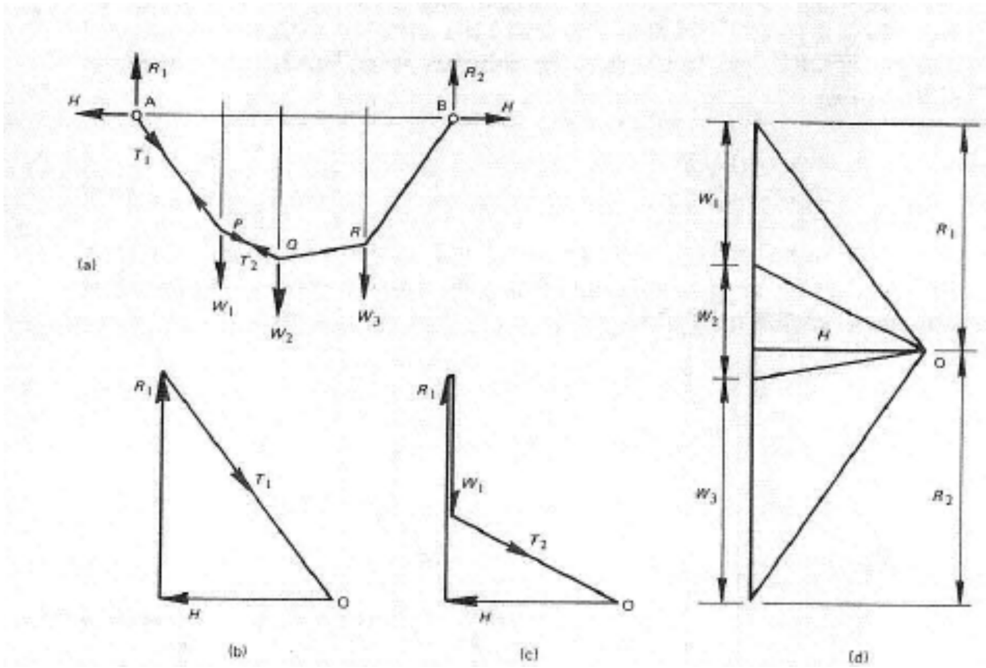


Fig 2.7 - Funicular polygon, taken from [Heyman, 1982]

To explain it is useful to consider the centre line of a three-pins arch loaded by a series of vertical forces. Frictionless hinges are unable to transmit moment; therefore the funicular polygon, corresponding to the line of thrust, has to pass through them. At the same time, the line of thrust does no coincide with the centre line of the arch except at the three hinges. In fact, considering a section of the arch obtained through a cut of the arch rib at a distance x from the hinge, in order to guarantee the equilibrium it is necessary to introduce a bending moment M in addition to horizontal and vertical forces. By simple statics, the bending moment in the arch ring is equal to the horizontal component H multiplied for the distance between the line of thrust and the centre line of the arch, as showed in the following picture. The line of thrust equilibrates the loads applied.

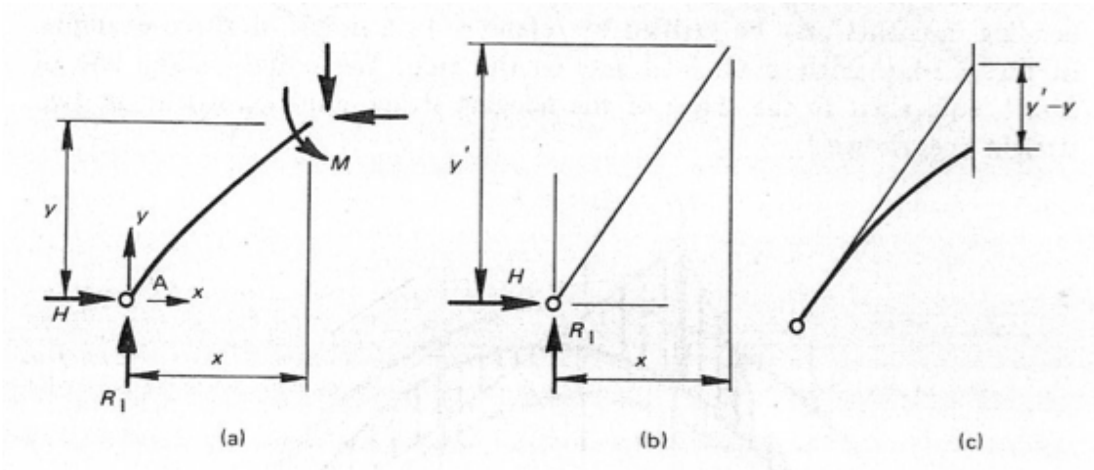


Fig 2.8 - Line of thrust and moment in the centre line of three-pin arch, taken from [Heyman, 1982]

In a voussoir arch having the same centre line of the previous arch, in any joint between two blocks, in order to maintain the equilibrium, it is necessary to apply thrust among the line of the funicular polygon. Thrust does not necessary be perpendicular to the joint, in any section there are a normal and a tangential components. The tangential component tends to make slide the two adjacent voussoirs, however its value is enough small that is possible to assume that sliding between voussoirs is not allowed [*Heyman, 1982*]. The normal thrust and its position is important to describe the behaviour of the arch.

The distribution of stresses varies in base of the position of the line of thrust respect to the centre of the section. According to elasticity theory, when the thrust is applied in the central point of the section the voussoirs are equally compressed with a uniform distribution of stresses. While, when the thrust moves from the centre of the section to its hedge the distribution of stresses changes. When the load is at between the centre and one third of the section the distribution is linear with the maximum value of stress in the hedge close to the load and the minimum at the opposite hedge. The limit value is reached when the load is applied at one third of the section: in this case the value of stress at the opposite hedge is equal to zero. Moving further the load off the centre, a part of the section is not more compressed and should transmit tensile tension. However it is assumed that the arch, and in general masonry, is not able to transmit tensile, whether assembled with or without mortar joints – even if mortar is present its tensile strength is very low and uncertain. The distribution of stresses is still linear, but in the areas in which tensile acts the voussoirs tend to separate.

Indeed when thrust line lays in the "core" of the section, the stress in the voussoirs is of compression. In an arch, which usually has a rectangular section, this core coincides with the middle third. The respect of the so-called "middle third rule" has been considered until the 60's of the XX century as a fundamental criterion that an arch has to satisfy. The following figure shows the distribution of stress previously described, in a pile made of stone slab, assumed elastic and assembled without mortar, subjected to a load applied in different position moving from the centre to the hedge.

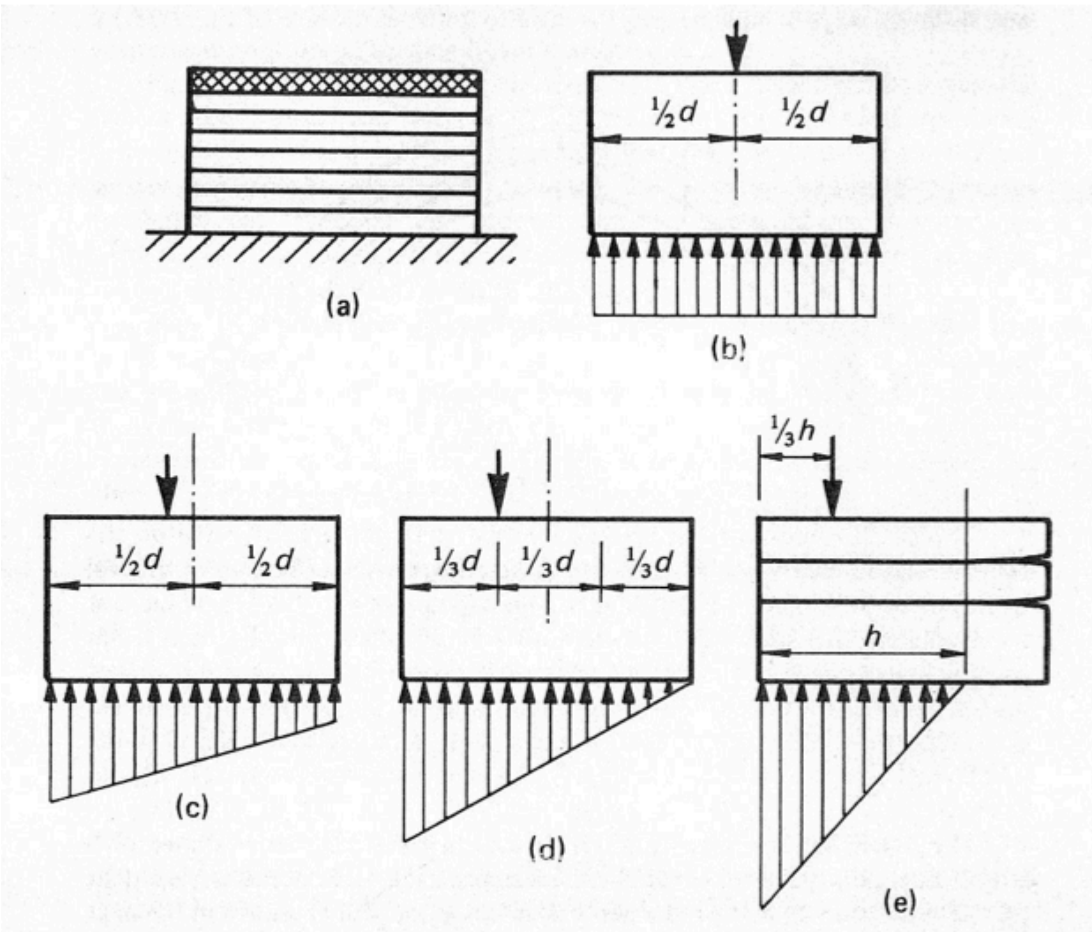


Fig 2.9 - Distribution of stresses and "middle third rule", taken from [Heyman, 1982]

The middle third criterion implies that the line of thrust has to lie in a thinner imaginary arch ring having a depth of a third of the real arch. In the reality linear elastic behaviour does not occur. The idea that cracks in the mortar joints are dangerous has been overtaken that the observation of real masonry arch. Moreover the position of the line of thrust is arbitrary: it possible to find different lines of thrust that equilibrate the same given loads. In one way is possible to say that the satisfaction of the middle third criterion guarantee that the real arch has a geometrical factor of safety respect to the thinner arch.

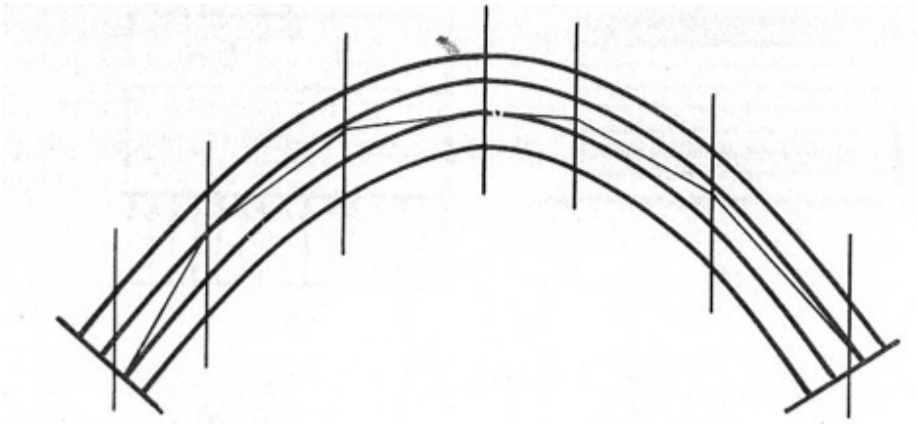


Fig 2.10 - Real arch and middle third rule, taken from [Heyman, 1982]

Ultimate load theory was developed initially for steel structures, but can be applied to masonry structure if the masonry material complies with certain conditions. Drucker was the first author that suggested the use of ultimate load analysis for the study of the equilibrium of masonry arch, subsequently followed by others authors, such as Koorian, Onat and Prager, which described the material conditions the voussoirs have to satisfy so that ultimate load theory can be rigorously applied and the corresponding yield surfaces drawn [*Drucker, 1953; Koorian, 1953; Onat and Prager, 1953; Prager, 1959*]. As previously said, here we refer mainly to Heyman's works [*Heyman, 1966, 1982 and 1995*]. In order to deal with ultimate load theory masonry material has to satisfy three conditions:

1. The compression strength of the masonry is infinite; this assumption may seem not safe but is realistic, in fact in masonry structures usually compression stresses are so low that there is not danger of crushing of material.

2. The tensile strength of the masonry is zero; stones and bricks have a tensile strength, but the joints between voussoirs may be dry or realised with very weak mortar, therefore the tensile stresses are not transmitted within masonry.

3. Adjacent masonry units cannot slide on one another; although in practice occasionally is possible to observe slipping in arch, in general the friction between voussoirs is enough to avoid this phenomenon, considering also a limited value of the tangential component, as previously mentioned.

These conditions have been already discussed in the previous part of the section, regarding the modelling of masonry as NTR material. When these conditions are satisfied, the component of the resultant of the stresses acting perpendicular in any section must be a compression force *N*. Hinge forms when the force *N* is applied at the hedge of the section. This leads to a yield surface bounded by two straight lines. The moment *M* is the product of the normal force *N* for the eccentricity *e*: $M = N \times e$; the eccentricity must be lower than half depth of the arch. For pairs values of *M* and *N* that are included in the yield surface the force *N* acts inside the section and therefore the line of thrust lies in the arch profile: the masonry arch is stable. For pairs values of *M* and *N* that lie on the lines defining the yield surface the force N acts on the hedge if the section and hinge forms. In case of pairs values of m and N that are not comprise in the yield surface the force N is outside the arch: the masonry arch is not stable.

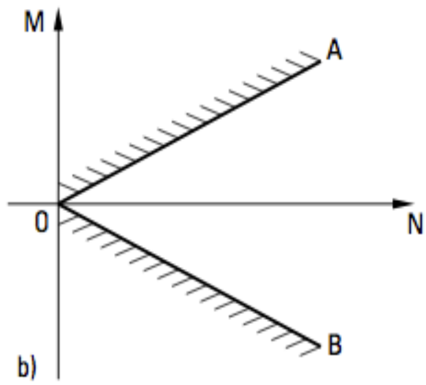


Fig 2.11 - Moment-normal force diagram with yield surface in rigid unilateral masonry, taken from [Heyman, 1982]

While the knowledge of the position of the line of thrust was fundamental for the engineers of the XIX and first half of XX century, that could calculate it through elasticity equations, instead in the ultimate load theory the knowledge of its position is not so relevant. In the reality the arch press against abutments that are not rigid: they are subjected to small movements and for elevate value of thrust may yield. The pressing of the arch make the abutments spread so the span of the arch increases. In order to reply to this the arch adapts itself changing its geometry: cracks occur to allow the necessary movements. A crack forms in the intrados at the crown and two crack occur at the extrados at the abutments. The arch became a three hinges arch, a structure statically determinate, the three hinges determine the position of the line of thrust. Movements may be asymmetrical, abutments movement could be both horizontal and vertical, and only one abutment may yield. For every possible movement there is a different cracks pattern: the arch reply to the changes in the boundary conditions opening and closing cracks. Therefore cracks are not dangerous; actually it is an ability of the masonry arch to adapt to changes in the boundary conditions, that is possible thanks to the NRT material properties.

The distribution of cracks defines the position of the line of thrust, which must pass through the hinges developed. When the cracks pattern changes the position of the line of thrust changes too. Movement may be large, but often they are small and not visible, however the effect is the same. In the reality it is not possible to know or predict the cracks pattern, therefore it is impossible to know the true line of thrust. Two extreme positions of the line of thrust are possible, corresponding to the maximum or the minimum horizontal thrust. It has been assumed that the compression strength is infinite, thus the collapse of the arch occurs in relation to the development of a kinematic failure mechanism. When the line of thrust touches the hedge of the arch a hinge forms and rotation is allowed. The three hinges arch is a statically determinate structure, but the development of one or more hinges make it become a kinematically permissible hinges mechanism that may provoke the collapse of the arch without material crushing.

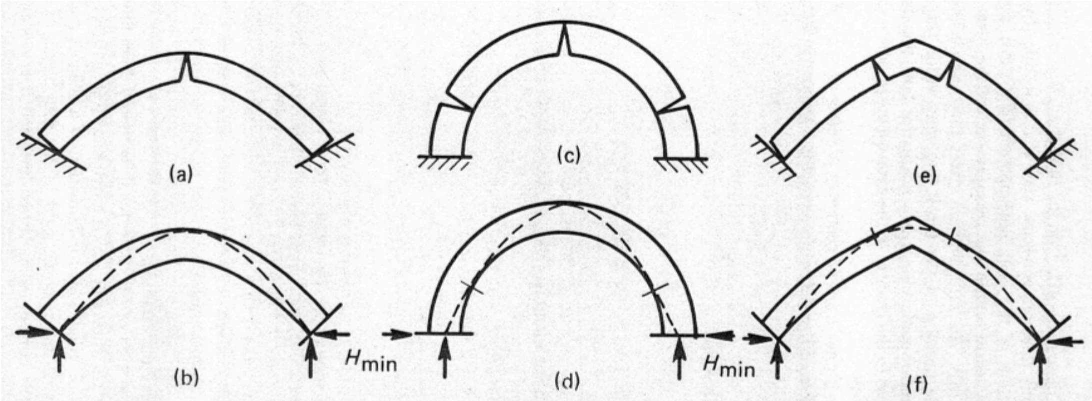


Fig 2.12 - formation of hinges mechanisms in masonry arch, taken from [Heyman, 1982]

2.2.3 The safety theorem

If it is possible to draw a line of thrust for the complete arch that equilibrates the loads, both external and self weight, lying within the profile of the arch, then exists at least one possibility that the arch is able to resists to the given loads. Therefore the arch can be considered safe when is possible to find an equilibrium that does not infringe the hinge condition. This condition is statically admissible and corresponds to the lower bound of the ultimate load. Instead the upper bound of the ultimate load is given by a kinematic maximum load resulting from a permissible and inevitable kinematic mechanism, quantifiable through the principle of virtual displacements: this condition is kinematically permissible.

In case of masonry arch, every line of thrust drawn for a given load satisfies the equilibrium conditions. But also the material conditions have to be respected: masonry has to resist to compression stresses. This implies that the stress resultants have to act inside the voussoirs in each cross section. In this case the line of thrust lies completely inside the arch profile and the arch is stable and will not collapse under the given loads. Instead, the safety theorem does not provide any statement about the boundary conditions: cracks occur in the arch in response to the support movements. When the boundary conditions change the arch find a new equilibrium. It means that the line of thrust changes its position but always lying inside the arch profile, therefore does not form enough hinges to transform the arch in a failure mechanism.

The importance of the safety theorem is that it is not important to find the "actual" line of thrust: to demonstrate that the arch is safe it is necessary only to find at least one satisfactory internal forces system. Having found a any one satisfactory thrust line the arch will not collapse under these loads. the safety of the load-bearing structure can be assessed without the necessity of have assumptions about its actual state.

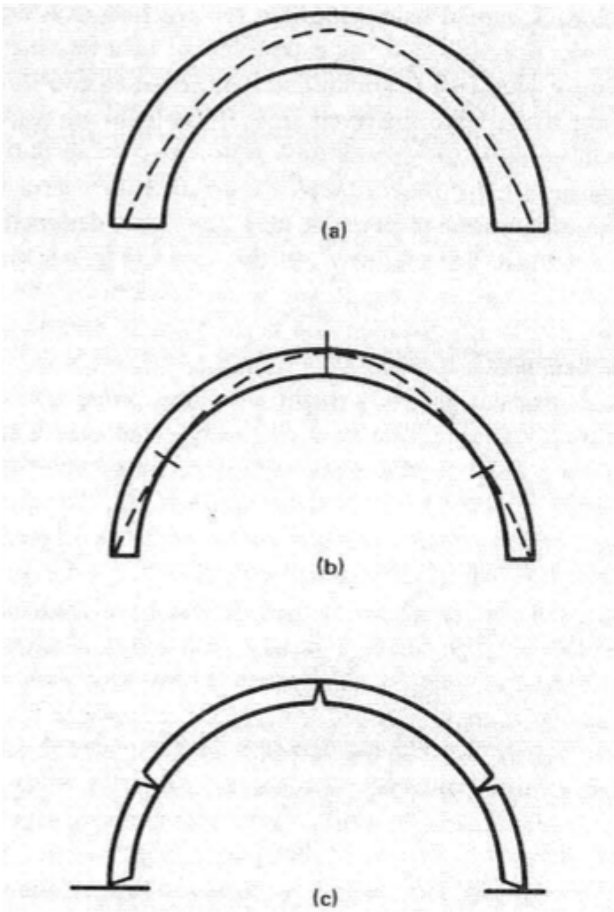


Fig 2.13 - Geometrical factor of safety, taken from [Heyman, 1982]

The safety of masonry arches can be assessed with the ultimate load theorems, upper and lower bound. It is possible to define a factor of safety through the comparison between the geometry of the real arch with the one of an arch that has the minimum thickness necessary to carry the given loads. The arch is safe if exists a line of thrust equilibrating the loading that completely lies inside its profile. The arch having the minimum necessary thickness can be found reducing the thickness of the real arch until it is possible to find only one single line of thrust lying within it. Comparing the two arch thickness is possible to define a geometrical factor of safety. Heyman recommends a value of 2, meaning that the thickness of the real arch is the double of the one of the minimum arch, for the most unfavourable loading case [*Kurrer, 2008; Heymann, 1982*]. The evaluation of the real value of the geometrical factor of safety may be not immediate, however is quite easy to determine a lower bound.

2.2.4 Mechanisms of collapse

Here the principle mechanisms of collapse are described. The figures show graphically the possible kinematic mechanisms and are followed by a brief description.

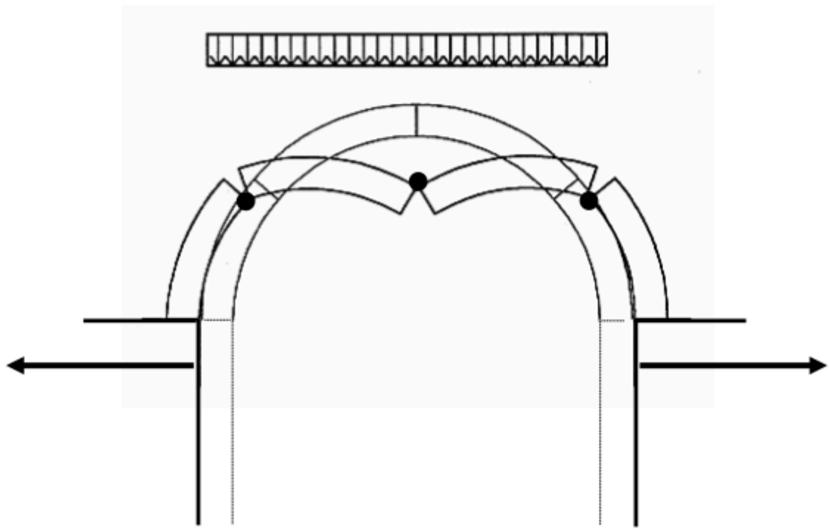


Fig 2.14 - Opening of springing

The kinematic mechanism of collapse with opening of springing is due to a rotation and/or translation of the piers or abutments, or a part of them. Three hinges form: one at the key in the extrados, the other two at intrados of the haunches.

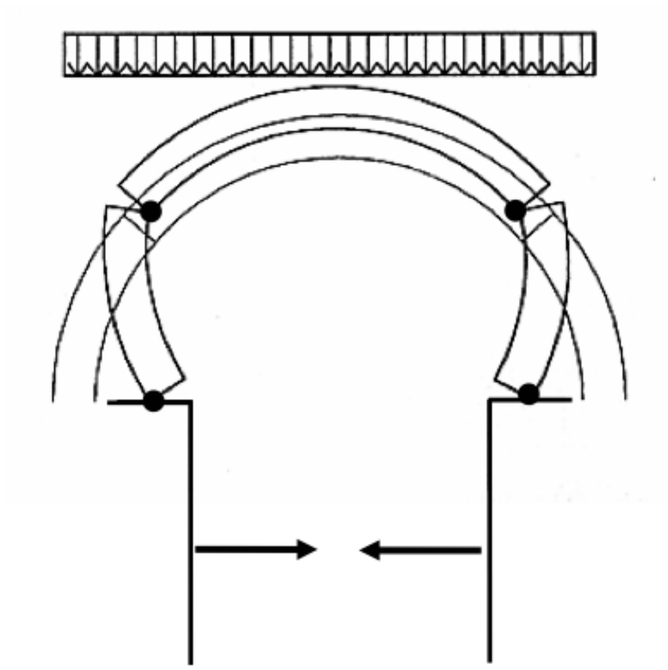


Fig 2.15 - Closing of springing

As the previous one, also the kinematic mechanism of collapse with closing of springing is due to a rotation and/or a translation of the piers or abutments, or a part of them. Differently to the mechanism with opening of springing, in this case four hinges form: two at extrados of the springing plus two at the intrados of the haunches.

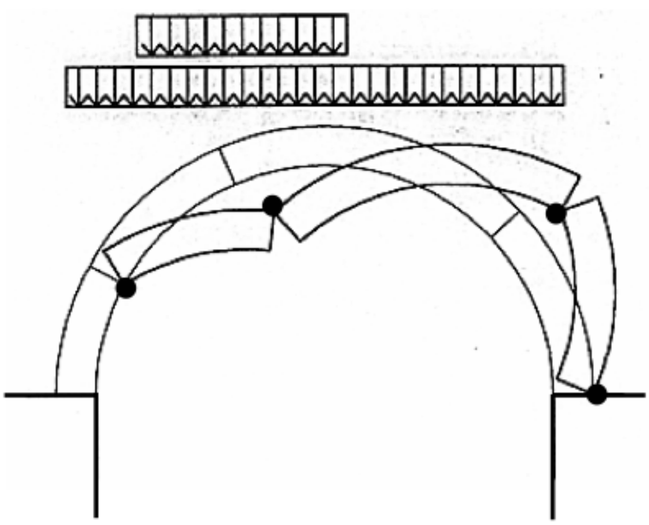


Fig 2.16 - A-symmetric mechanism with fixed springing

The a-symmetric kinematic mechanism with fixed springing leads to the formation of four hinges. Hinges developed alternatively at the intrados and the extrados. Usually, the last hinge in the side of the arch less loaded occurs at the extrados of the springing. The other hinge at the extrados of the arch occurs in the most loaded side of the arch and tends to develop in correspondence, or sometimes only near, to the line of action of an eventual concentrated force.

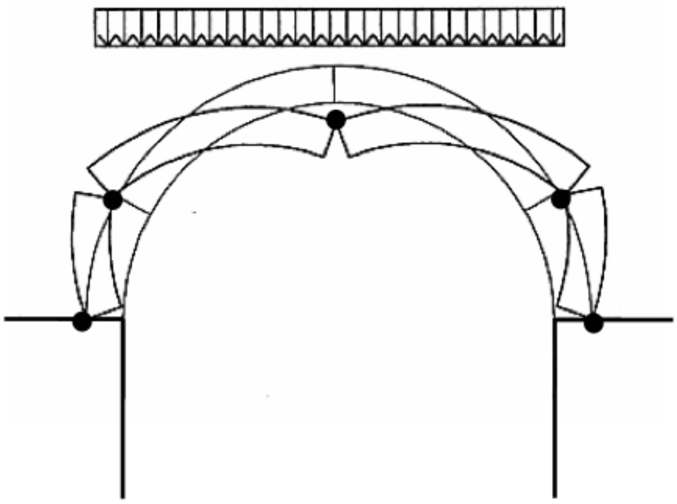


Fig 2.17 - Symmetric mechanism with fixed springing

The symmetric kinematic mechanism with fixed springing leads to the formation of five hinges. Symmetrical hinges develop alternatively at the extrados of the springing, at the intrados of the haunches and at the extrados at the crown.

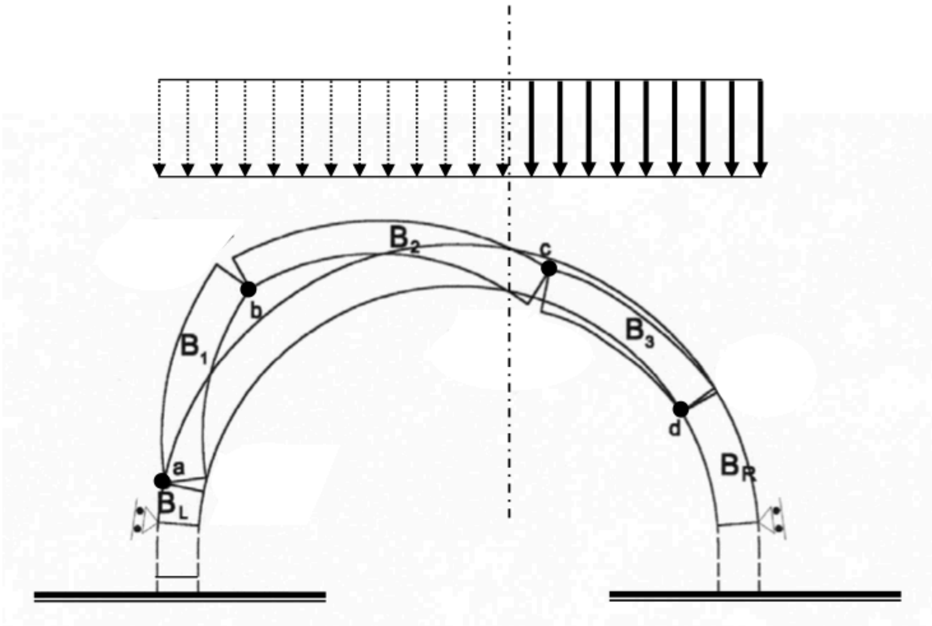


Fig 2.18 - Positive and negative work

Considering the loading acting on an arch, the right part, drawn with the solid line, does a “positive” work: the load activates a possible mechanism. Instead the left part of the load, drawn with the dashed line, does a “negative” work: the load resists to the mechanism provoked by the right part of the load. Indeed, if the negative work is bigger then the positive one, this mechanism of collapse cannot occur. However the arch may collapse with a different mechanism of collapse under the same load, that may be weaker then this one. The arch is safe and can carry on the applied load only if, for all the possible kinematic mechanisms eligible, the absolute value of the total negative work is greater than the positive one.

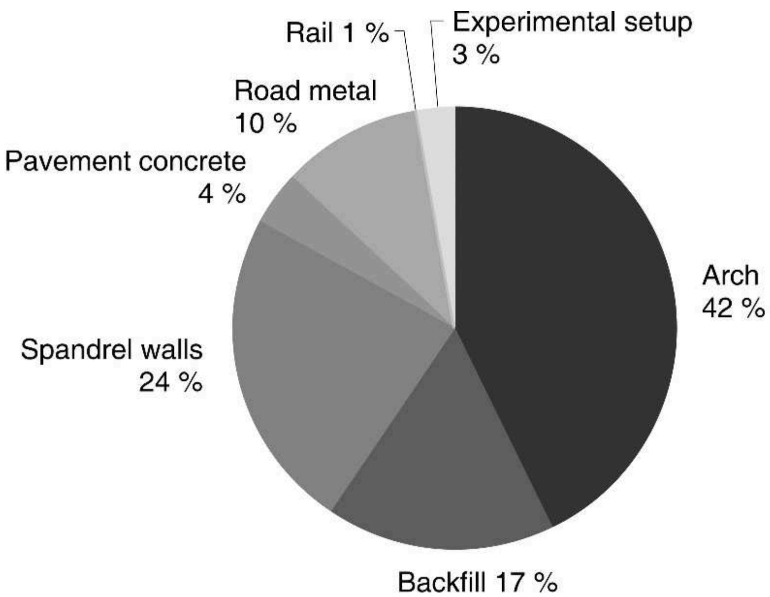
SECTION 2 – Part 3

The structure level: masonry arch bridges

2.3.1.1 The global behaviour of masonry arch bridges

The structure level concerns the global behaviour of masonry arch bridges. In order to understand it, it is important to analyse which resisting mechanisms are activated under loads and how failures take place. The behaviour of masonry arch bridges under loading is considered under static loads; afterwards this behaviour is adapted to consider its response under dynamic loading. The structural behaviour is described referring to the different typologies of bridges: square bridges (non-skewed arches) - single or multi-span - and skew bridges.

First it is necessary to outline the role of each structural element. Although the arch provides the main contribution to the load-bearing capacity of arch bridges, also other elements have a role in the load bearing capacity of arch bridges. A quantification of the various contributions due to each elements has been provided by Weber, which proposed an increase factor of 1.5 to the load-bearing capacity from the single span pure arch to the single span arch bridge.



Graph 2.1 - Contribution of different structural element to the load-bearing capacity of masonry arch bridges, taken from [Weber, 1999]

2.3.1.2 The role of backfill and spandrel

The behaviour of masonry arch bridge is strongly influenced by the behaviour of the masonry arch, which has been described in the previous paragraph. However, although the arch gives the biggest single contribution to the load-bearing behaviour, the contribution of further structural elements in arch bridges has been known for a long time. In the 40's of the last century authors noticed a change of the line of trust caused by the backfill in arch bridges [*Cramer, 1943*], studied the shear stress between the arch and the backfill [*Fischer, 1940, and 1942*], proved an increased load-bearing capacity of arch bridges when shear forces were transferred to the backfill [*Jäger, 1938*]. The role of backfill became clearer in the 60's, when authors showed an higher load-bearing capacity thanks to the contribution of backfill [*Herzog, 1962; Bienert, 1959-1969: Bienert et al., 1960-1962*]. Herzog found a lower eccentricity when backfill was acting, however, he only considered the same Young modulus for the arch and the backfill. Bienert and his co-authors discussed the effects on the load-bearing behaviour of arch bridges caused by backfill.

A very intensive list of references can be found in Gocht, which has developed also models representing the effects of backfill [*Gocht, 1978*]. He discussed the effect of an active height of backfill, considering a solid joint or a sliding joint between the arch and the backfill and whether a joint at springing exists. The presence of a solid joint may allow the development of a flat arch inside the backfill. This phenomenon can change the span of the arch, as showed in the collapse of Traversa Railway Bridge in the Italian railway between Torino and Genova [*Brencich and Colla, 2002*]. An evaluation of the increase in the load bearing capacity due to backfill can be found in Smith [*Smith et al., 2004*].

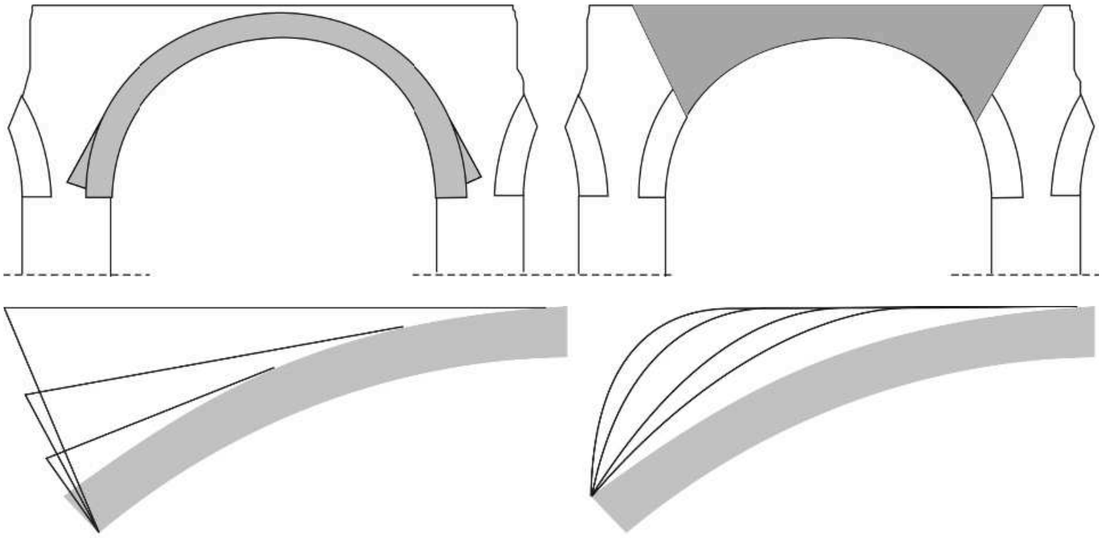


Fig 2.19 - The effect of active height of backfill, taken from [Gocht, 1978]

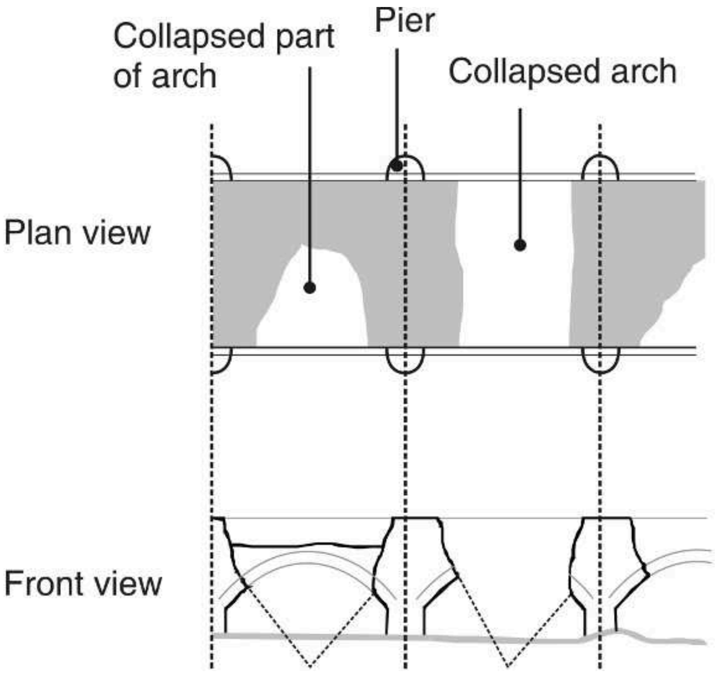


Fig 2.20 - Partial collapse of Traversa Bridge, taken from [Brencich and Colla, 2002]

The role of the other structural elements has been studied by Molins and Roca, which showed an increase of the failure load when modelling not only the arch itself but also the other elements. They compared the results obtained through numerical simulations with the values of experimental tests carried on two bridges. When models considered the spandrel the failure load has tripled, however reaching around the half of the experimental failure loads; to obtain the experimental values, models had to considered also tensile tie bars [*Molins and Roca, 1998*].

Similar experiments with comparable results has been preformed by Cavicchi and Gambarotta. They have studied the influence of the backfill carrying out tests at real scale at the Prestwood Bridge, in UK. The bridge, with a span of 6.55 m and a rise of 1.428 m, had a thickness of 0.22 m at the crown and the height of backfill at key was 0.165 m. It was loaded over the entire width of 3.0 m at 1/4 point. The failure load measured was 228 kN, that provoked a four-hinges mechanism of collapse. Authors assumed the masonry compression strength to be 4 MPa and modelled the bridge, but considering only the arch without the backfill: they obtained a failure value in the range of 46 kN, corresponding to around the 20% of the overall load-bearing capacity [*Cavicchi and Gambarotta, 2004*]. In another example provided by the same authors the contribution of the arch has been estimated in the range between the 33 and the 50% of the overall load-bearing capacity [*Cavicchi and Gambarotta, 2005*]. Other studied have been published by the same authors, which developed on the base of these results, a model that consider the backfill [*Cavicchi and Gambarotta, 2006 and 2007*].

The influence of backfill, spandrel walls and masonry backup on the load-bearing capacity of bridges have been studied by Royles and Hendry, which reached an improvement of the load bearing in a range between the 8 and the 50% respect to the pure arch [*Royles and Hendry,1991*]. An improvement equal to the 33% has been given by Becke, which make a distinction between effective and not-effective backfill depending by its stiffness: the backfill could be considered only if its stiffness is more than 1/100 of the arch, otherwise it do not give contribution [*Becke, 2005*]. Another recent work regarding backfill has been published by [*Harvey et al., 2007b*].

Not only the backfill, but also the also the spandrel walls play a role in the behaviour of masonry arch bridge [*Schreyer, 1960*]. Voigtländer provided a decrease of about 20% in stress of the arch due to the collaboration of spandrel walls [*Voigtländer, 1971*]. This decrease of stress occur at springing, while usually at the crown the bond condition of spandrel walls do not guarantee its active functioning.

In term of deformation and stresses in serviceability levels even roadway and pavement concrete give contribution and increase the load-bearing capacity. The influence of these elements has been experimentally assessed: asphalt layer increases the load-bearing capacity of about 3%, the pavement concrete of about 12%, and the protection concrete up to 10% [*Gutermann, 2002*].

2.3.1.3 Behaviour of square bridges

Once described the single contribution of each elements of a masonry arch bridge to the load-bearing capacity, is useful to outline the global behaviour. Single span square arch bridges are very representative of the general behaviour of arch bridges. For square arch is intended an arch not skewed, meaning that longitudinal and transversal axes are perpendicular. The resistant mechanisms activated by loads and the failure modes are very useful to define the global behaviour of masonry arch bridge.

The main resisting mechanism depends by the geometry of the arch. The behaviour of masonry arch has been previously described. Summarising, an arch is the anti-funicular geometry of a set of loads. Moreover, gravity pre-stresses the arch allowing it to resist even to loads having a different anti-funicular respect to the arch. The geometrical nature of the way of the resistant mechanism of the arch means that its capacity is dependent on the whole arch shape, not just its span and rise.

In the case of multi-ring arches, the arch behaves as a unit only if ring separation between rings does not occur. In case of separation the arch become a stack of thin independent arches with a reduction its load carrying capacity [*Melbourne and Gilbert, 1995*].

The weight of spandrel, acting as a permanent uniform load, has a stabilising effect on the structure. Moreover it confines the arch reducing tensile stresses, creating a sort of pre-stress condition. It must be noticed that the positive effects of spandrel walls are due to their profile: if the weight at haunches is too big respect than at the crown the effect of spandrel could be negative.

Backfill can be considered as a structural material in contact with the arch: when the arch moves under external actions its strength is mobilised against these movements. This effect increase normal and shear stresses applied at the arch extrados respect to those induced only by the weight of the backfill.

Similar to the backfill, the spandrel walls constrain the arch movements with their stiffness. Moreover, an interaction between backfill and spandrel walls is generated, giving a further contribution to reduce the arch movements. This interaction is complex and it is not completely clear if it is sufficient to guarantee the functioning of this resistant mechanism in case of transverse spandrel wall-arch separation.

The wing walls reduce the rotation of spandrel walls around their bases: it gives a contribution to the strength of masonry arch bridges because increases the in-plane stiffness of spandrel walls. In the same way the movements of the backfill are constrained by the surrounding soil. The lateral pressures of the surrounding soil may be very important to the stability of the structure under dead loads, in particular in case of deep arches.

The spread of load through the backfill is another factor that contributes to the strength of masonry arch bridges, but the extent of this is unknown [*McKibbins et al., 2006*].

Three main modes of failure have been observed for square masonry arch bridges [*Hughes, 1995a; Page, 1995*]. These failure modes are an idealization of the real ones, that are more complex because are usually the result of a combination of different failures in addition to a general loss of performance. By the way, their description is very useful to understand the global response of masonry arch bridge and to outline the problems that may affect the serviceability. The main failure modes are:

- Failure by formation of a hinge mechanism. This failure, which is typical of the masonry arch, provokes the formation of hinges and/or “sliders”. Due to tensile stress, openings may occur between the ashlars where the line of trust is too eccentric, so that the structure can rotate as if it were an articulation. It makes the arch behaving like a kinematic chain of blocks connected by hinges. The number of hinges required to activate this mechanism is four, while in case of central symmetric load five hinges are required. When the abutments contract or spread enough, mechanism of “slider”, a three-hinge mechanism may occur.

- Snap-through failure: it is a mechanism in which, due to great rotations occurring in one of the hinges, a local instability failure happens before the complete formation of the hinges mechanism of failure. Concentrated rotations usually develop in the hinge under the load, in particular when the arch is highly constricted. This failure accelerates the global mechanism of collapse.

- Crushing failure: it occurs when compression failure of the masonry in a certain zone of the structure results in local damage that instigates the global failure. High levels of compressions affect masonry in the hinge areas because of the reduction of section due to the opening of cracks. This failure may be brittle, therefore it is important to intervene immediately when signs of crushing appears in the arch.

In addition, local failures - such as ring separation in multi-ring arches, spandrel walls-arch separation and radial shear failure between the ashlars, and in general a loss of performance and/or a decrease of material properties – may bring to a global failure when combined to one of the three main failure modes. The critical position of load depends by the shape of arch: in shallow squared arch it is usually the quarter point, while it is around the 1/3 point in deep arches.

Respect to single-span, multi-span square arch bridges show a different behaviour, related to the slenderness of the piers. If the piers in between arches are too slender the load applied on one span can mobilise the adjacent one. The four-hinges mechanism in the loaded arch can transmit a horizontal pressure to the adjacent one that may bring to the collapse. Even in case of stocky piers the interaction between adjacent spans could make some problems, in particular in case of concrete haunches or, sometimes, in presence of compacted backfill. The interaction between adjacent spans is important not only for the effect that can have n the failure modes, but also for its influence on the long-time performance behaviour.

A parametric study regarding the multi-span mechanism has benne carried out by Hughes to determine when single rather than multi span behaviour occur [*Hughes, 1995b*]. When multi-span behaviour does not take place, the failure load is equal to the one of the single-span bridge, while in case of multi-span behaviour the load-bearing capacity may be assessed multiplying the load failure of the single-span for a coefficient, which is the result of a parametric equation. In case of multi-span the value of the coefficient is always less then 1, therefore the load-bearing capacity of multi-span square bridge is lower respect to the single-span.

The typical failure mode of multi-span square arch bridges is a seven-hinges mechanism. This mechanism is activated by rotations occurring around the base of the pier. In case of slender piers, only three hinges on the loaded span are necessary to activate the mechanism: due to the slenderness of the pier, outward movements of the intermediate support induce failure. For this particular failure mode the critical load position is near the centre of the span [*Melbourne et al, 1997*].

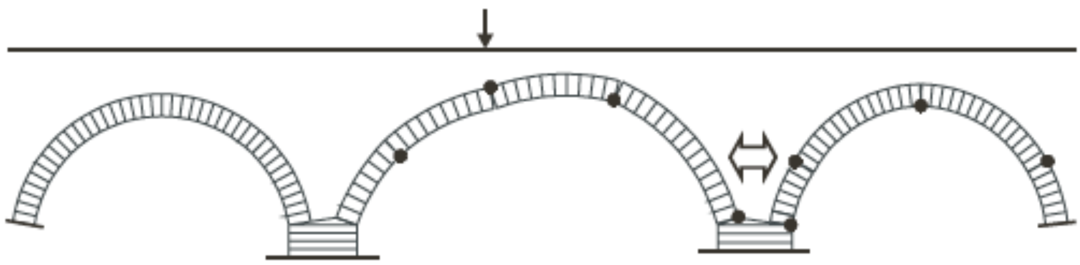


Fig 2.21 - Transmission of pressure due to load to adjacent span in multi span bridges, taken from [Melbourne et al., 1997]

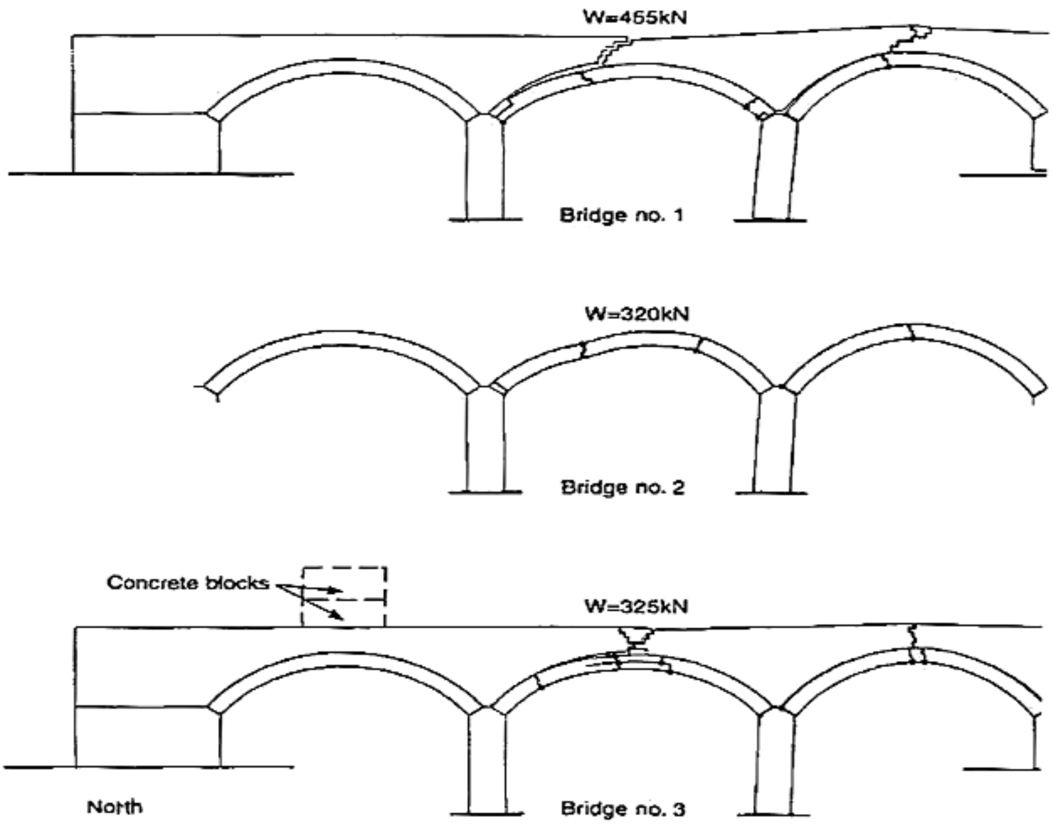


Fig 2.22 - Multi span failure mechanism, taken from [Melbourne et al., 1997]

2.3.1.4 Behaviour of skewed bridges

A skewed arch is an arch in which the longitudinal and transverse axes are not at right angles. The behaviour of all masonry arch bridges has a three-dimensional component: in a square arch the main component is the longitudinal one, while in skewed bridges the transversal component has more relevance. Indeed its behaviour is strongly influenced by the three-dimensional component and is more complex respect to square bridges and not yet completely understood. The clearest consequence of this is that the stiffness of the arch varies considerably across its width [*Melbourne and Hodgson, 1996*].

In structures loads always choose the shortest ways to reach the ground, in case of an arch they are transferred across the shortest span available. When the arch is skewed usually the shortest span does not coincide whit the longitudinal axis. As a result of this, torsional moments are applied to the abutments and piers, provoking a rotation or a leant backwards. This problem may be very dangerous on a slender pier supporting two arches skewed in the same direction [*Page, 1993*].

Due to these characteristics, mechanisms of collapse of skewed bridges are different respect to the ones of square bridges. Usually failure occurs with complex three-dimensional hinge patterns. The orientation of the hinges can change across the width of the structure, giving way to diagonal cracks that separate the barrel in isolated acute corners. Stresses concentrate in the obtuse corners, increasing the torsional moment transmitted to piers and abutments. Moreover the hinges that develop are also more diffused than in square spans.

A particular case is the multi-ring skewed arch bridge. In case of ring separation the rings will slide transversely over each other. Multi-ring tests [*Melbourne, 2001*] showed a very relevant interaction between the skew barrels and the piers when piers have a failure due to torsion transmitted by the arch. The initial kinematism was a five-hinges mechanism, but after the torsion failure of the pier, it changed and the barrel behave forming initially a four-hinges mechanism and finally a three-hinges mechanism. These modes have to be taken into account in order to assess the load bearing capacity with sufficient safety. Moreover the bond used for the masonry of arch and barrel has a significant effect on the stiffness and strength of skewed bridge [*Melbourne and Hodgson, 1996*]. In case of multi-ring, this aspect has to be taken into account. It is necessary to establish if each ring is connected or not to the next one. A good constructive technique should show headers incorporated in the barrel every third course. Finally is important to evaluate additional positive features in the connection between the arch and piers and abutments.

2.3.1.5 Behaviour under cyclic and dynamic loading

The structural behaviour of masonry arch bridge under dynamic and cyclic loading is not very different respect to the static one. Dead loads are usually considerably bigger respect to live loads, indeed dynamic effect is not so relevant such as in other typologies of bridges. Their high mass and damping is sufficient to prevent significant accelerations or dynamic amplification of the displacements.

Fatigue has not been identified in this type of structure, due to the low level of stresses developing in masonry structures and to the limited influence of strength of materials in the load-bearing capacity. Pippard studied the effect of cyclic loading founding that the load that producing the first cracks decrease when repeated [*Pippard, 1948*]. More recently other researchers have investigated the effect of repeated loadings, but without significant results [*Peaston and Choo, 1997*]. Experimental tests suggest an endurance limit of 50% respect to the static load strength [*Melbourne et al., 2004; Roberts et al., 2004*].

Thanks to the maintenance carried out in European countries, the experience accumulated in the last 50 years showed that the cyclic application of heavy loads might accelerate the deterioration of masonry arch bridges. An important aspect to be considered in the conservation of masonry arch bridge is that even if fatigue failures are not usual, the increases in the heavy traffic loads in this period suggested that full potential consequence of fatigue related to this new level of loads are not yet manifested. Although this topic is not completely acknowledged, usually in regulations suggest to prudently reduce the loads to around the half of the ultimate load, so to prevent fatigue problems.

2.3.2.1 Modelling and analysis of masonry arch bridge

In order to perform structural analysis it is necessary to develop an appropriate numerical model. Many authors have studied the evaluation of the load-bearing capacity and of the safety of masonry arch bridges. In each method of analysis are different specificities, related to the purpose of the methodology, that mainly depends by the type of action with respect to is necessary to evaluate the safety of the structure. Several models have been developed to analyse the structural behaviour of masonry arch bridge. The choice of the model depends on the respective questions and the provided resources. The report prepared for the EU Commission during the COST-345 in 2004 provides a list of analyses methods recommended for different levels of assessment.

Description level	Models
1	Empirical or two-dimensional-model, linear-elastic arch frame
2	Two- or three-dimensional, linear elastic or elastic-plastic, allowing for cracking
3	Two- or three-dimensional, linear or non-linear, elastic or plastic, allowing for soil-structure interaction, cracking, site-specific loading and material properties
4	FEM analysis of specific details of the structure being assessed not considered in the previous levels
5	Reliability analysis based on probabilistic models

Table 2.1 - Methods of analysis and levels of assessment - COST-345 (2004)

However, a simplified rule for choosing an appropriate model cannot be given. Even very simple empirical rules have shown to be a solid basis for bridges with ages of centuries and millenniums.

The main analyses that can be performed on masonry arch bridges are basically the same previously listed in the section regarding modelling and analysis of masonry (paragraph 2.1.3): linear static, natural frequencies, non linear static, non linear dynamic. In this thesis the analyses will be performed to assess the safety of the bridge and to evaluate the stability, the strength and the stiffness, which are the normal serviceability requirements of bridges. The principal methods of analysis to evaluate the structural capacity of masonry arch bridges are:

- Semi empiric methods;
- Limit analysis methods;
- Solid mechanics methods.

In this paragraph a state of art of principal approach to this issue is given. Particular attention will be paid to the role of filling and spandrel respect to the global behaviour of masonry arch bridges. At the end of the section a critical comparison between the different approaches is provided.

2.3.2.2 Loads

Bridges are exposed to several types of loading:

- Dead loads: the weight and distribution of the bridge and its superimposed dead loads are essential for the stability of masonry arch bridges. In case of maintenance works, strengthening or restoration these loads are temporarily changed: it is very important to consider this aspect because it may affect the stability of masonry arch bridges. However, it should be taken into account that dead loads have a beneficial effect, but they can also have a negative effect if the pattern of dead weight loading in relation to the shape of the arch is inappropriate.

- Traffic loads – static, dynamic and cyclic – consist of all the vertical, longitudinal or lateral loads due to the passage of trains (or vehicles or pedestrian, if any), possible accidental loads and, in case of maintenance, strengthening or restoration, even the equipment loads during works. Particular attention has to be paid respect to load magnitudes, positions and frequency. Anyway, it has to be pointed out that, due to the high mass of masonry arch bridges, the effects of traffic loads are not so relevant such as in other types of bridges, steel bridges for instances. When determining the loaded lengths masonry arch bridges do not behave as elastic structures and therefore approaches based on lines of influence are not valid.

- Environmental effects: wind can be ignored as a result of the high mass of masonry arch bridge. According to the geographic location of the bridge, the same consideration may be done regarding the load due to snow. However, other environmental effects that have to be considered with attention are floods and droughts, which can have significant effects on the foundations, that are often one of the weakest points of masonry arch bridges. Although masonry is a durable material, deterioration may affect its performances: the effects of weathering and of other different deterioration mechanisms can ruin its state of preservation. The thermal properties of masonry can vary quite significantly between the different types of masonries, however, generally they are not considered to have significant effects on the integrity of masonry arch bridges.

- Ground movement: the weakest points of older masonry arch bridges are very often piers and especially foundations, as previously mentioned. This specific aspect of masonry arch behaviour will not faced off in this thesis. However foundation movements may occur when the centring was removed and during its service life. The arch reply to these movements adapting its geometry to the new conditions: it provokes cracking. These types of cracks are found in many masonry arch bridges and in most cases their effects on the structure can be neglected.

In summary, all the loads that can act on bridge are:

- Dead loads.
- Vertical traffic loads;
- Initial drive forces;

- Breaking forces;
- Centrifugal forces and nosing;
- Wind loading;
- Snow;
- Loads due to variations of temperature.
- Impact forces;
- Settlements and ground movement.

In this thesis the attention is focused mainly on the dead loads, which are essential to assess the stability of masonry arch bridge, and on the vertical traffic loads, which represent the service loading conditions.

Historical arch bridges, in contrast to newly constructed bridges, require further considerations of loading. As said in the first section of thesis, first railways have been introduced in early years of the nineteenth century for goods traffic, and have been used since the 30's of the same century for passenger traffic. In the following 100 years railroads experienced an incredible growth that yielded to a huge demand for bridge construction. Therefore, between 1845 and 1890 especially, many railway bridges were constructed as arch or vault bridges. Due to the rapidity of increase of the number of passengers and goods transported, locomotives were constantly improved with a consequent increase of the overall weight. From 1830's to 1920's the weight of locomotives expanded from 10 to 175 tonnes, corresponding to an increase in uniform load from 2.5 to 13.67 tonnes per metre [*Beyer, 2001*]. In parallel also the velocity of trains increased: from an initial maximum speed equal to 40 km/h to the current high velocity trains.

In the beginning of railway technology railway loads design was a train with real axle loads. Afterward railway load patterns have been introduced. During the second half of the nineteenth century the UIC (*Union Internationale des Chemin de fer*) defined a series of load models. The load models do not describe real trains: they have been identified so that their effects are representative of the effects of real trains. On December 1994 the third part of the European standard ENV 1991 "'Eurocode 1" was issued (ENV 1991-3). In addition to taking the loads UIC as models of vertical load for railway bridges, the code identified in a systematic manner all other actions to take into account in the design and / or verification of railway bridges. The regulation framework was fragmented, indeed on January 1997, the Italian State Railways provided a specific regulation with the aim to bring together and harmonise all the technical regulations issued during the years for railway bridges and to incorporate the contents of the European standard ENV 1991-3 (*instructions n° I/SC/PS-OM/2298, first version in June 1995, updated in January 1997*). These instructions have to be applied for design, implementation and testing of new railway bridges of the Italian State Railways, as well as for all the existing bridge in case of static restoration, adaptation and restructuring.

In this standard are listed all the actions and their combinations that should be considered in the design and in the verification of railway bridges. The dead loads are given by the self weight of the bridge plus a series of permanents loads relative to the weight of the ballast and sub-ballast, and of reinforcement and waterproof (including protection). Conventionally, for a straight line, they are assumed as a load equal to 18.0 kN/m^3 applied over the entire width to an average height between top of rail and extrados deck equal to 0.80 m. This value increases up to 20.0 kN/m^3 in case of curved line. The vertical loads are defined by means of two load models: the first representative of normal traffic, the train load *LM 71*, the second representative of heavy traffic, the train load *SW*. The values of these loads must be multiplied for a coefficient of adjustment α variable because of the type of infrastructure. Three typologies of loads are defined.

The LM71 load model consists of:

- Four axes of 250 kN arranged at intervals of 1.60 m;
- Distributed load of 80 kN / m in both directions, starting from 0.8 m from the axes of its ends and for an unlimited length.

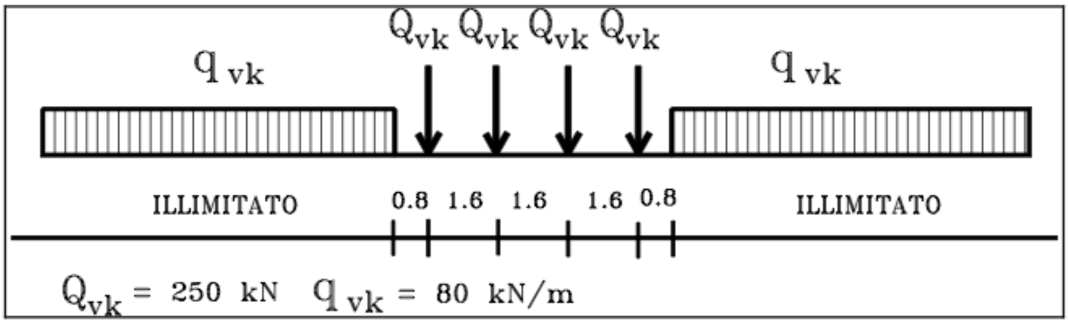


Fig. 2.23 - Load model LM71

For the SW load model two different configurations are considered, the SW/0 and the SW/2:

- SW/0: q_{vk} = 133 kN/m, a = 15.0 m and c = 5.3 m;
- SW/2: q_{vk} = 150 kN/m, a = 25.0 m and c = 7.0 m.

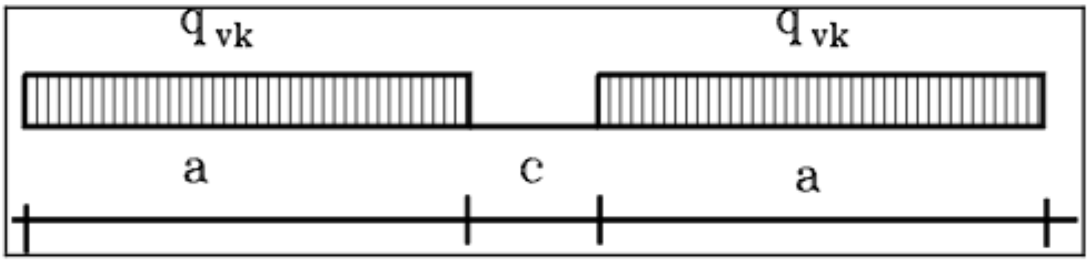


Fig. 2.24 - Load model SW2

Further information about the application of load models and about the other loads will be provided in the case of study. A complete resume of loadings is provided in national and international regulations.

2.3.2.3 Historical rules for dimension

During the history, the construction of masonry arch bridge has been carried out using simply and empirical models, based on geometrical rules derived from the experience. Starting from L.B. Alberti, different empirical rules for the dimension of the arch have been provided by famous engineering and bridge builders of the seventeenth and eighteenth centuries, such as Dupuit, Gautier, Perronet, Dejardin, Lesguillier, L'Eveillè, Gauthey, Minchon and many others [*Corradi, 1998*]. These methods described the ultimate load-bearing capacities related to the dimension of the elements of the arch and the bridge. Five task have been defined by Gautier for the design of masonry arch bridge [*Heymann, 1998*]:

- The shape of the arch;
- The arch thickness at the key;
- The thickness of the foundation and abutment;
- The thickness of the piers depending on the design of the arch;
- The thickness of the wing walls.

These empirical criteria were different from country to country and often showed several inconsistencies between them. The issue of the evaluation of the safety respect to the collapse of the bridge was not dealt with a scientific approach. Moreover, starting from the nineteenth century, masonry bridge were considered as a thing of the past, destined to be supplanted by more modern facilities in steel or reinforced concrete. However, given the high number of existing masonry bridges it became necessary to provide for their verification. It implied the awareness that it needed to study not only the behaviour of bridges under service loads, but also to evaluate their safety.

The historical methods of design and assessment are not the subject of this research. Although, the knowledge of the method utilised in the design of bridge may be useful to establish the presence mistake during the design or the construction of the bridge. For this reason, in the case study, the Scheffer's method, which has been used for the design of the Venice Trans-Lagoon Bridge, will be discussed.

Castigliano provided the first modern approach to the statics of masonry arch bridge in 1879. He developed a method of calculation and verification in order to assess of the safety of the Mosca Bridge on the Dora River in Turin, built in the 30's of the nineteenth century. The method was inspired by the configuration of the

Mosca Bridge, which is made by big blocks of granite without mortar. Castigliano provided an iterative method based on two hypotheses:

- Joints cannot transmit tension;
- Collapse of bridge occurs when compressive strength of material is reached in any section of the bridge.

This constitutive model represents a material with no tensile strength and elastic compressive strength; it could be considered as no tension resistant model used for masonry (NTR). The method developed by Castigliano had a big spread around Europe, and it was also a source of inspiration for several methods developed afterwards.

Although it is a method dating back to the nineteenth century, although currently there is a Castigliano's method proposed by regulations in UK. It is based on the classic method and analyses one single arch utilising the Finite Elements method. The assumptions of the method are that the material has no tensile strength while has an infinite compressive strength (NRT). The analysis determines the areas of the arch in which there are tensile stresses. In these areas cracks occur and the section is chocked. The geometry of the arch varies in every section in order to consider only the compressed parts. The analysis has to be repeated until the reaching of the convergence value of the reagent height. The method does not provide immediately the load-bearing capacity of the bridge: it is necessary to perform several analyses to find the position of the load that determines the minimum admissible load. The method is rigorous but it is difficult to implement it in software. UK regulations provide two calculation programs that are based on this method, the *CTAP* and the *MAFEA*.

An evolution of the Castigliano's method has been developed by the University of Genova [*Brencich and Di Francesco 2004*] in order to define a simplified method for the load bearing assessment. It consists of a bi-dimensional model that schematises the arch through straight beams. The material is considered as no tensile resistant (NRT) and elastic-plastic in compression and taking into account the ductility. It is a model that considers several aspects of the mechanical behaviour of masonry, even if with some simplifications which, anyway, are in favour of security. The iterative procedure is equal to the one of the classic Castigliano's method. It is easy to implement into commercial software and it is able to represent also multi-spans bridges. Being a bi-dimensional model it does not take into account neither the contribution of spandrels nor the interaction between arch and filling, which is regarded simply as a weight-stabilising effect.

These methods are suitable to perform collapse analyses applying monotonous incremental static loads, while are not fit to carry out dynamic analyses. They can be used to study the response of bridge respect to vertical loads due to trains.

2.3.2.4 Modern rules for load-bearing assessment

These methods have been used starting from the first half of the XX century. The first one was a semi-empiric method developed by Pippard in the 30's, based on the assumption of linear elastic behaviour of masonry arch. It has been widely utilised during the II World War in order to assess the load bearing capacity of historical masonry arch bridges respect to military loads, considerably greater than usual ones. The massive use of tanks - never before the existing masonry bridge had to support the passage of vehicles weighing several tons – make the necessity of a more rigorous evaluation of the safety of masonry arch bridges became more urgent. Many methods for the assessment of the load bearing capacity have been developed starting from the Pippard's method.

The main one is the *Mexe-Mot Method* (Military Engineering Experimental Establishment – Minister of Transport) that consists of a review of the Pippard's method. The NATO developed the early MEXE method after the II World War. The campaign of testing carried out in UK during the 50's provided newer results that were not considered in the first generation method. Therefore a review of the method has been provided during the 60's, because of the increase of live loads. The MEXE-MOT method is widely used in UK, it is still regularly updated and provided by regulations [*Department of Transport, 1993*]; it spread also in many other country since it was included in UIC-Codex in 1995. The application of the method is simple and fast. The boundary conditions of the method are:

- Span of the arch smaller then 20 meters;
- Rise greater then ¼ of the span;
- Height of the filling above crown between 30 and 105 cm.

The admissible load Q_{adm} is obtained multiplying the load applied Q_p for a coefficient f that consider a variety of parameters, such as the shape and numbers of arches, the state of conservation and the materials. The simplicity and velocity of the method explain its wide application; by other hand the method has several approximations that make it not completely reliable. For this reason, some authors criticised the method outlining the fact that it could provide unsafe results in some circumstances [*Brencich et al., 2001*] and not recommending its use in the future [*Hughes and Blackler, 1997*]. For this reason, in the last few years many authors carried out research in this field with the purpose of develop successors of the MEXE-MOT method. Here a brief list of some of those methods is reported.

The *FILEV Method*, which has been developed in 2004 by Martin-Caro and Martinez [*Martin-Caro and Martinez, 2004*], is based on a database of results obtained by nearly 800 bridge models with different parameters. The load bearing assessment of those bridges has been provided with FE program in order to establish the ultimate load. The database has been used to derive approximated equations. Thanks to the high number of bridge analysed the method is robust, however equations have a variety of simplifications.

Harvey provided a new substitute of the MEXE Method [*Harvey et al. 2007*]. The method proposes a simplified approach to allow first level assessment by artisans. Inspectors should consider the geometry of the bridge, in particular the shape of the arch. Through sound observation and measurement it is possible to know span and rise of arch and to evaluate the fill depth. These measurements have to be inserted into a spreadsheet in order to provide a rapid check for rationality.

Knowing these parameters is easy to look up the required ring depth for a particular load. The method provides a simple system of graphs. The method is robust and very simple to apply and does not require the use of calculators or computers. However it is intended that the method is suitable only for a very first assessment.

Many authors developed ultimate load-bearing curvatures. Purtak proposed curvatures based on finite element simulations, considering both non-linear behaviour of masonry and opening of joints [*Purtak et al., 2007*]. Martinez developed curvature for the crown thickness depending by compression strenght of stones and geometry of the arch [*Martinez et al., 2001*]. Aita compared two methods of assessment of the stress level in arch, which are based on either geometrical or material parameters, obtaining stability areas curvature [*Aita et al., 2007*].

2.3.2.5 Simplified methods based on limit analysis

To this category belong the methods that consist of the application of the equilibrium limit analysis. They can be categorised in base of the application of the fundamental static and kinematic theorems of the limit analysis:

- Graphic application of the static theorem;
- Analytical application of the static theorem;
- Analytical application of the kinematic theorem.

Traditional methods for the application of the limit analysis consisted in the graphic application of the static theorem [*Campanella, 1928; Albenga, 1953*] and the method was widely used in the first half of the twentieth century during the design of the railway net[7]. This type of analysis studies the hypothetic lines of trust at the ultimate state. It means that line of trust has to lie inside the arch thickness but not necessarily inside the middle third part of the section to avoid the chocking of

[7] The method was provided by the Italian Minister of Trasport for the design of railway bridge since 1924.

section. This method simulates the collapse due to incremental static loads, providing a lower bound collapse limit.

Analytical application of the fundamental theorems had a great diffusion in the last thirty years. The typical application of those methods to masonry implies the schematisation of the structure as a rigid body with no tensile resistance (NRT) and infinite compressive strength in which sliding is not aloud. In the last few years some of those approximations have been reviewed in order to better describe the real masonry behaviour. In the case of masonry bridge the structure object of the equilibrium limit analysis is the arch. In the beginning the analysis was usually limited to one single arch, while recently limit analysis has been performed on multi-span bridges too. In this case piers are considered as part of the kinematic mechanism. In case of single arch typical mechanism involves four or five hinges, while they become seven or more in multi span arches.

The limit analysis with a static approach[8] aims to determine whether, under the applied loads, the equilibrium conditions are met and if in any section of the arch the internal actions do not exceed the plastic limit. It is complementary to the kinematic analysis and allows easily representing the inelastic response of masonry through appropriate mechanical models.

Different constitutive relationships have been proposed for masonry, on which the limit domain is based. A first class of models assess the ultimate load when the stress is close to the domain, defined by boundary surfaces $N – M$[9], on the base of simplified assumptions of the distribution of stresses in mortar joints [*Clemente et al., 1995; Boothby, 1997*]. In fact, knowing the resultant N, its position and the eccentricity e, or the bending moment $M = N \cdot e$, for each collapse point, it is always possible to substitute the actual stress distribution with an equivalent uniform stress diagram. Taking into account the material factor its possible to deduce the equivalent design compression strength. The yield surface of a rectangular cross-section in the

[8] Based on the static theorem.

[9] Normal force and bending Moment due to eccentricity.

plane (N, M) is formed by two parabolic arcs. The limit domain can also be written in terms of eccentricity.

More recently an admissible strength domain has been defined on the base of results of experimental tests and considering mechanical properties and constitutive law [*Boothby, 2001*]. The safety of the bridge respect to loads is guaranteed when distribution of stresses is compatible with the defined strength domain. This method allows establishing the lower bound collapse limit.

In some models friction interfaces Mohr-Coulomb have been developed, avoiding the basic assumption of sliding not allowed [*Livesley, 1992*]. The same authors applied the static limit analysis to bridge exhibiting a three-dimensional collapse behaviour whit transversal torsion, but in this case the value of the load multiplier obtained is less reliable then in the two-dimensional case, depending from the distribution of stress on the interfaces and of the value of the friction coefficient.

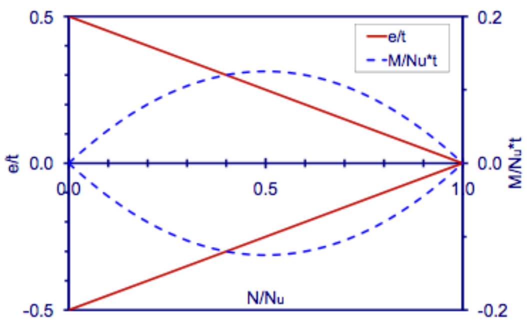


Fig 2.25 - Limit domain M – N

The limit analysis with a kinematic approach (based on the kinematic theorem) aims to find the mechanism that occurs for the minimum value of the loads multiplier. It considers the arch as a set of rigid bodies connected through hinges: the purpose is to find the position of the hinges in order to reduce the load necessary to activate the mechanism. Hinges occur when the line of trust is tangent to the intrados or to the extrados of the arch. This approach is widely used nowadays, and thanks to the research carried out in the last 20 years is very reliable and gives safe result. For this reason it is provided by UK regulations. Many authors paid effort on this method

[*Harvey, 1988; Gilbert and Melbourne, 1994; Blasi and Foraboschi, 1994; Como, 1998*].

Other relevant aspects that affect the load-bearing capacity of the bridge have been introduced, such as the soil interaction, the effect of filling, and the presence of multi-level rings [*Gilbert and Melbourne, 1994; Falconer, 1994; Hughes, 1995*]. Other authors considered also other possible mechanisms, due to the limited compressive strength of masonry [*Crisfield and Packam, 1988*]. This implies the adoption of a constitutive law rigid-plastic for the masonry, which means that the line of trust cannot be tangent to the arch but has to lie at a certain distance from the extrados or the intrados, therefore hinges are not points. However this method allow establishing only the upper bound collapse limit. For this reason the procedure is able to assess the safety of the bridge while not always is suitable to evaluate the safety margin [*Resemini, 2004*].

It has to be pointed that kinematic limit analysis is a reliable method for arches that show a low level of stresses under service loads while it could provide unsafe results in arches showing a higher level of stress, such as flat arches for instance. Regardless the level of stress, this method assumes infinite an-elastic strains that are not compatible with the real masonry behaviour and that are not verified by tests [*Brencich et al., 2002*].

2.3.2.6 Analytical models made with beam elements

Models made with beam elements have been introduced since the development of analytical models. Initially beam models had to be very simple in order to be dealt through hand computation. For this reason first beam models realised represent only static determined structures – two or three hinge or at least fixed arches – and did not considered the structure as a whole, loosing several important aspects of the bridge behaviour. Nowadays, thanks to the possibilities given by automatic calculation, these types of models have been extended to consider many effects of the arch bridge load behaviour, with very precise results. At the moment many beam models also consider the backfill, elastic foundation, and roadway structures [*Voigtländer, 1971; Model, 1977; Gotch, 1978*].

However, considering the different contributions given by each structural element to the global behaviour of masonry arch bridge, the development of beam models encountered several difficulties and in many cases models are heavily disputed [*Proske and Van Gelder, 2009*]. Some beam models have been previously cited in the paragraph 2.3.1.2 regarding the role of backfill and spandrel [*Molins and Roca, 1998; Cavicchi and Gambarotta, 2006 and 2007*].

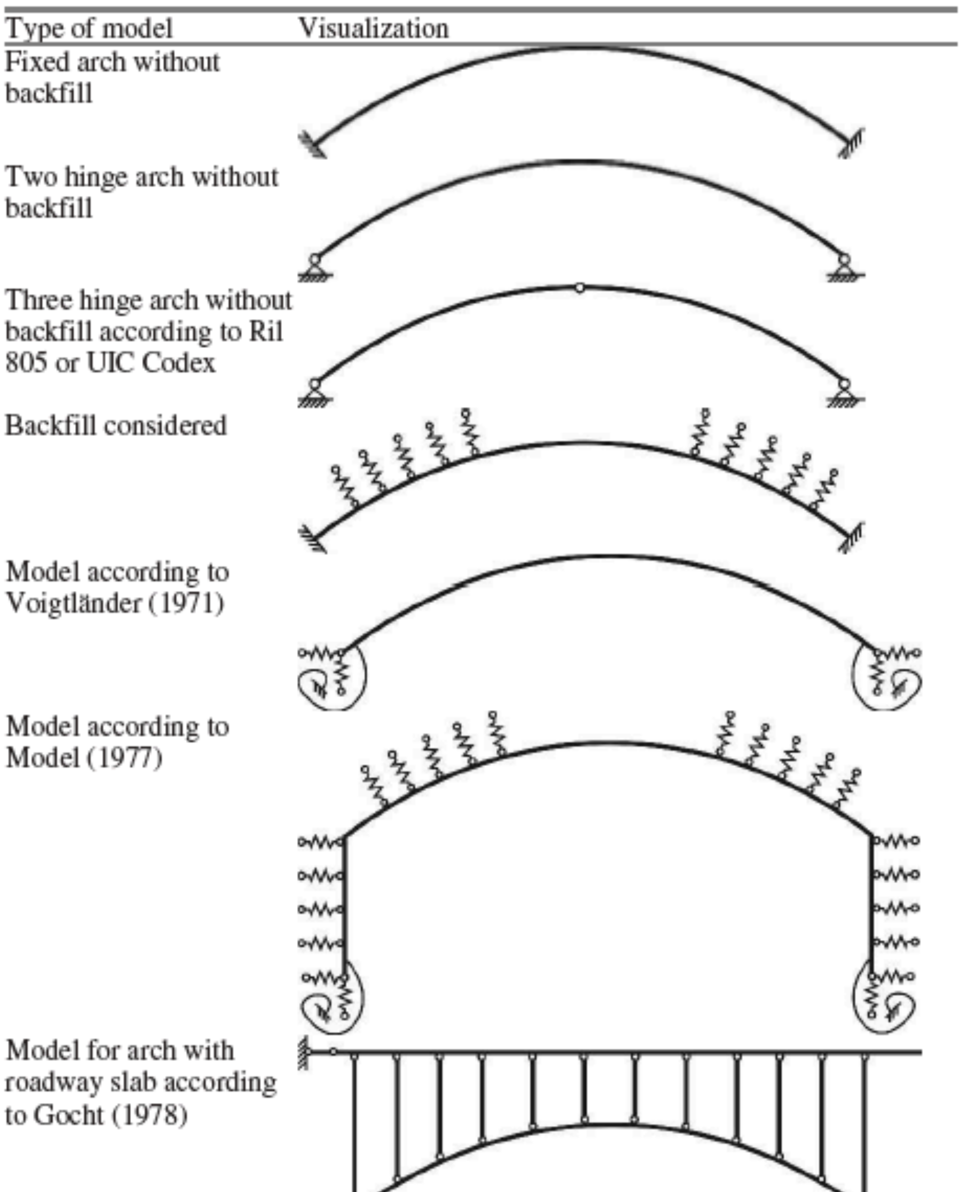


Fig 2.26 - Evolution of beam models, taken from [Proske and Van Gelder, 2009]

Recently beam models of the arch have been realised in order to allow the development of hinges, on the base of plasticity theory for masonry [*Heymann, 1966*]. In a similar way to what happens in the limit analysis, hinges represent the areas in which cracking may occur. These types of models allow finding or the equilibrium inside the arch or to define the kinematic chains of blocks and the relative kinematism. For this reason, beam models may belong to the category of models useful for limit analysis, previously described, however several beam models are more complex and may provide further information. A comprehensive review of beam models for the arch has been provided by Gilbert [*Gilbert, 2007*]. New highly sophisticated programs, such as RING [*Limit State Ltd, 2008*] and Archie-M [*Harvey, 2008*] specific for the analysis of masonry arch bridges, utilises non-linear beam elements.

Compound beam models represent a particular category of beam models. First studies on this topic have been carried on recently [*Hannawald, 2006*]. Compound beam models born to describe the behaviour of beam made by two different part, such as mixt structure steel-concrete or wood-concrete. Recently their application has been proposed for masonry arch bridges. The cross section of the beam is not only the arch itself but is increased with further part on the extrados to represent the effect of backfill. The properties of the two materials may be characterised differently, for example varying the Young modulus. A partial load is transmitted to the backfill, thus the position of the line of thrust varies on the base of the properties of the section and is more reliable respect to a normal beam model. A relevant aspect is the distinction between solid or sliding joints inside the compound cross sections: when relative sliding is prohibited the joint is considered solid while in case if sliding allowed the contribution of the backfill to the load-bearing decreases. Sliding is more probable than solid joint and depends by the constructive features of the extrados and the backfill, in particular the type of bond. The properties of the compound beam cross section are defined easily on the base of the geometry and the stiffness of the single cross section, providing a factor that is used to define the properties of the materials. However in case of arch bridge the thickness of backfill change considerably during the length of the arch and the factor has to be changed.

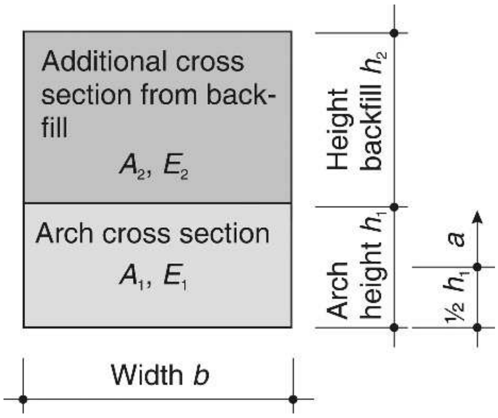


Fig 2.27 - Variables of compound section, taken from [Proske and van gelder, 2009]

2.3.2.7 Finite Elements method

Detailed modelling through the Finite Element method (FEM) is a widely used method in numerical structural analysis. FEM is usually adopted to achieve sophisticated simulations of the structural behaviour and it is a powerful tool to study stresses and displacement in bodies. It can describe the structural response of a structure in great detail but with high computational costs. The method can be used to perform both for static and dynamic analyses, on mono- two- or three-dimensional models, depending by the type of element chosen and the constitutive law adopted. Applied to masonry structure may be used to analyse localised areas or specific elements and with the complement of other techniques, may help in the structural assessment. Finite element models of masonry and concrete arch bridges have become more and more popular since the 80's, in which first finite element analyses of arch bridge have been carried out [*Crisfiled, 1985; Towler, 1985*].

Using FEM models it is possible to assess the safety of bridge respect to several conditions, from the traffic loads to the seismic actions. The analyses can be performed both in the elastic field and in the non-linear field. Analysis performed in the elastic field is very useful in order to represent the behaviour of an historical bridge under service loads or to evaluate the safety margin respect the original design. It can provide a detailed distribution of strains and stresses while it is not suitable to describe comprehensively the ultimate strength of the bridge. On the other hand the use of non-linear constitutive law [*Pegon and Anthoine, 1997; Gambarotta and Lagomarsino, 1997; Alpa and Monetto, 1994*] may not be easy due to the high number of parameters to be entered, in particular in case of three-dimensional models of the whole bridge. Even if very powerful, due to computational costs, to the choice of parameters and to the difficulties in the evaluation of results, this type of models did not have a wide diffusion. However they are utilised in case of monumental structures, which require a deeper analysis [*Podestà, 2001*], or by some authors to study the seismic behaviour of masonry arch bridge [*Karaesmen et al., 1996; Oliveira, 1995*].

It is important to choice both the elements to be used and the scale of model, from the entire bridge to specific parts of it. Such as in the other masonry structures, the geometry can be idealised in different ways, namely, by considering the structure to be made of linear elements, two-dimensional elements, shell elements or fully three-dimensional elements. As a first impression, it would seem reasonable to use of three-dimensional elements. However, fully three-dimensional models are usually very computationally onerous with respect to preparation of the model, to perform the actual calculation and to analyse the results. The results of models incorporating shell elements are reasonably difficult to analyse due to the variation of stresses along the thickness of the elements. In addition, the large thickness of the structural elements might yield a poor approximation of the actual state of stress. Increasing the details and size of the model might result in a large amount of information that may blur the important aspects.

For this reason in literature it is possible to find more simple FEM models that reduce the bridge in a mono-dimensional model [*Crisfield, 1984 and 1985; Bridle and Highes, 1990; Choo et al., 1991; Molins and Roca, 1998.a*] or in more detailed two-dimensional models [*Loo and Yang, 1991; Falconer, 1994; Boothby et al., 1998; Lourenço and Rots, 2000*] and three-dimensional [*Rosson et al., 1998*]. Codes usually advised to utilize simple mono-dimensional models also for complex structures [*Molins and Roca, 1998.b]* and two-dimensional models for simple structures. However, thanks to the increase of the capacity of calculation and to the efforts paid by many authors in the last years, nowadays an increasing number of professional finite element programs include modules for realistic material description of masonry and are used for the simulation of arch bridges[10].

Lourenço provided a summary of different computation strategies [*Lourenço, 2002*], including Discrete Element Method, which will be described in the next paragraph. He suggested that, in the analysis of masonry historical structures, it is better to use two-dimensional models than three-dimensional models, to avoid using shell elements in areas important for the global behaviour of the structure and to model structural parts and details instead of modelling complete and large structures. An important aspect regarding FE Modelling of masonry arch bridge is that very complicated simulation techniques are characterised by an high level of uncertainty due to an increasing number of input variables. Defining a numerical safety factor for the different computation strategies, its value increases with their complexity, reflecting the increasing of uncertainty [*Lourenço, 2002*].

Approach/analysis type	Safety factor
Allowable stress (f_{ta} = 0.2 MPa)	0.31
Kinematic limit analysis	1.8
Geometric safety factor	1.2
Physical nonlinear and no tensile strength	1.8
Physical and geometrical nonlinear and no tensile strength	1.7
Physical nonlinear and tensile strength of 0.2 MPa	2.5
Physical and geometrical nonlinear and tensile strength of 0.2 MPa	2.5

Table 2.2 - Safety factors for different computation strategies according to [Lourenço, 2002]

Although FEM modelling provide very reliable results at the same time the richness of details could make the results not so clear. Considering the computational costs, complex FEM models are not always suitable to perform analyses of masonry

[10] An interestin example to be cited is the model proposed by Ford that has been realized with Straus7, the same program which has been used in the case study [Ford et al., 2003]. Further information will be provided in the chapter about the case of study.

arch bridges. However, simple FEM models are able to give easily the distribution of stresses and strain.

2.3.2.8 Discrete Elements method

The most widely used method in computational solid mechanics is the Finite Element Method. In recent decades a set of computational methods have been developed to deal with particulates, jointed rock, granular flows and problems where the so called emergent properties of a system are a result of interaction between large numbers of individual solid particles. The most widely used method for a large class of these problems is the Discrete Element Method (sometimes called distinct element method). Under great cracks FEM can shows convergence problems, therefore the advantage of assuming homogenous material properties over certain space regions cannot hold anymore: the application of DEM is a valid alternative.

The DEM provides a consistent procedure to study masonry structures thanks to the possibility of creating models made of separated blocks. In particular, these models can properly represent the behaviour of historical masonry constructions, which could be considered as made of dry stone blocks exhibiting a periodic pattern. Discrete models to investigate masonry behaviour are proposed under the hypothesis of rigid block connected by mortar interfaces. These assumptions are justified from the observation that, in the case of historical masonry, mortar is much more deformable than blocks and its thickness is often negligible when compared to block dimensions. Hence the blocks are modelled like rigid bodies connected through Mohr-Coulomb interfaces (i.e. mortar thin joints). In other words, masonry is seen as a *molecular skeleton* in which the interactions between the molecules (rigid blocks) are represented by forces and moments, which depend on their relative displacements and rotations [*Lourenço and Rots, (1993); Lofti and Bensons Shing, 1994; Markov, 1999*]. This assumption seems particularly valid in case masonry arch bridge in which the arch is made by stone. In general their application may be very useful to the study of masonry arch bridge [*Maunder ,1993; Lemos, 1995; Owen et al., 1998;*

Roberti and Calvetti, 1998; Thavalingam et al., 2001; Brookes and Collings, 2003; Bićanić et al., 2003; Jackson, 2004; Schlegel 2004; Rouxinol et al., 2007].

Although DEM is a very general and robust method, the problem for practical application is still an extensive computation time and a great multitude of different material parameters that are often unknown or difficult to measure on the structure. Its application may be very useful for the study of single arch, while in case of complex structures it could be too complicated and do not provide a synthetic model. Moreover DEM could show cracks and mechanisms of collapse, but they can be deeply mesh influenced: to avoid this problem it need to realise a very refined mesh, increasing the computational costs. However, in case of stone arch this problem is not relevant, on the contrary the stone ashlars of the arch may be perfectly modelled through DEM.

In the early 1990s the two methods FEM and DEM have been combined and the resulting method was termed the combined FEM-DEM [*Munjiza, 2004*]. It is in essence a discrete element method with individual elements meshed into finite elements. Finite elements allow to model elastic deformation (if any), while discrete element algorithms allow to model interaction, fracture and fragmentation processes. The combination of DEM and FEM allows studying both linear and nonlinear masonry behaviour. Nowadays, there is a big development of new methods for the study of masonry structures based on combinations between DEM and FEM.

Conclusion of the second section

Some considerations about the different approaches to the structural modelling on masonry arch bridge are here provided. The main advantages and limitation of the different methods of analysis are outlined in the following tables:

MEXE and other modern rules for load-bearing assessment	
Applicability	• Span of the arch smaller then 20 meters; • Rise greater then ¼ of the span; • Height of the filling above crown between 30 and 105 cm.
Advantages	• The simplicity and velocity of the method
Weak points	• Un-reliability: approximations may give unsafe results; • Take into account only the arch and the weight of backfill.

Table 2.3

Castigliano's method and its evolutions	
Applicability	• Single span bridges; • Only two-dimensional model; • Masonry modelled as NRT material;
Advantages	• The simplicity and velocity of the method • The method is rigorous
Weak points	• It need an iterative procedure • Difficult to be implemented in software (however, some of the recent Castigliano's method have been implemented in softwares); • Does not take into account the contribution of spandrel and backfill.

Table 2.4

Methods based on limit analysis	
Applicability	• Static approach allows determining whether the equilibrium conditions are met under applied loads and if internal sections do not exceed the plastic limit; • kinematic approach allows finding the mechanism that occurs for the minimum value of the loads multiplier; • can be difficult to apply to shallow or deep arch and in general to complex geometry (however recently there are been many advances in this direction); • Recently beam models of the arch have been realised in order to allow the development of hinges, on the base of plasticity theory for masonry
Advantages	• Static approach allows establishing the lower bound collapse limit; • Kinematic approach provide reliable and very safe results; it allows establishing the upper bound collapse limit, assessing the safety of the bridge; • Reliable results for arch showing low level of stress; • Developed by many researchers is widely used; • There are many specific software for masonry arch bridge based on this approach.
Weak points	• May provide unreliable results in shallow arches and in general in arches showing high level of stresses; • Assumes infinite an-elastic strains not compatible with the real masonry behaviour; • Kinematic approach is not always fit to establish the safety margin; • Do not take into account spandrel, however many recent models consider the backfill.

Table 2.5

Finite Elements method	
Applicability	• Can be used for any kind of bridge and geometry; • Can be used both to perform static and dynamic analysis, linear or not-linear; • Can be use on two- and three-dimensional elements; • May provides results about the service conditions, ultimate loads and seismic behaviour of the bridge;
Advantages	• Can be use to assess the safety of the bridge in almost every condition; • Provide easily the distribution of stresses and strains; • Can be extremely versatile and allows almost any sophistication required; • May be used to consider strengthening and/or repair options and evaluate their benefit; • May provide very reliable results ...
Weak points	• ... however the richness of detail could make them not so clear; • May be difficult to define the right properties of masonry or backfill or the interface between the different structural elements; • Results are very sensitive to input parameters; • The computational cost may be very high when dealing with complex models; • The increasing in complexity of simulation increase also the uncertainty.

Table 2.6

Discrete Elements method	
Applicability	• Same as FEM can be use for any kind of geometry, on two- and three-dimensional models; • Can be used to study the non-linear behaviour also in case of great cracks.
Advantages	• May be very useful for the study of masonry; • May be able to simulate cracking; • The combination of FEM with DEM may provide very powerful method.

Discrete Elements method	
Weak points	• There still problems in the practical application due to extensive computation; • Difficult to be used in case of complex structures; • Model may be deeply influenced by mesh.

Table 2.7

The importance of good interpretation cannot be overstated: the correct understanding of the behaviour of masonry arch bridges is of fundamental importance. The factors which influence performance and behaviour should be identified. The global behaviour of masonry arch bridge is strongly related to the influence of each single element, structural (piers and arch) and non-structural (backfill and spandrel). Therefore model should be able to take into account all the elements of a masonry arch bridge. Moreover, in the masonry structures there is not a clear difference between structural and non structural elements, hence model with consider them as a continuum, such as F.E.Models, seems to be more appropriate to represent the real structure of the bridge. However their problem could be the characterisation of the mechanical properties of masonry material: homogenisation procedures are suggested to overcome this weakness. D.E.Model may be a very powerful method for the study of masonry arch bridge, especially if combined with FEM. However, its practical application is still difficult. Limit analysis is a consistent method for the assessment of the safety of the bridge, however does not provide many information about the service behaviour of the bridge. Considering the availability of different effective methods a combined use of them is suggested, on the base of the needs. In this view multi-scale analysis seems to be very suitable to establish a procedure of analysis.

SECTION 3

Strengthening of masonry arch bridges

Introduction

This section deals with the strengthening of masonry arch bridges. The aim is to have a overview of the available techniques of consolidation in order to establish which are more appropriate to be used in the conservation of masonry arch railway bridges. For this reason it is first necessary to outline the main problems that may affect masonry arch bridges and their causes. It needs to appraise the state of conservation of the structure in order to evaluate if the bridge is able to satisfy the required performances. Attention will be paid also to operations of preventive and planned maintenance, useful to ensure a good level of conservation and to prevent unnecessary or late interventions.

The section consists of two parts:

1) Deterioration, damages and performance decay;
2) Repair and strengthening.

1) Deterioration, damages and performance decay. First it is necessary to define the performance required to the bridge and to evaluate the consequences of loss of performance and the risks: attention is paid to performance under service loads. Then a lists of the principle problems affecting the bridge is provided. Three are the main problems: boundary condition, structural problems, material deterioration. A short version of a catalogue about the most common damages affecting masonry arch bridges is reported.

2) Repair and strengthening. A review of the principal techniques of strengthening of masonry arch bridges is given, with a comparison between the different approaches. It is necessary to evaluate the compatibility of interventions with the need of conservation, so to reduce the impact of consolidation and prevent from the loss of cultural value. With this purpose, routine operations of operation of

preventive and planned maintenance are very suggested to ensure a good state of conservation and to avoid, or to reduce to the minimum, too invasive interventions of strengthening. Consolidation may require a temporary out of service of the structure to allow interventions, with the consequence of disruption of the network. Particular attention will be paid to the techniques that might be used without interrupting, or only for a very short time, the functioning of the bridge.

At the ned of the section some brief conclusions are reported, with a critical comparison between the different approaches to repair and strengthening.

Section 3 - Part 1

Deterioration and decay

3.1.1 Performance requirements for masonry arch bridges

The basic structural performances that a masonry arch bridge has to satisfy are:

- Stability;
- Stiffness;
- Strength.

In addition to these basic structural requirements, there are a series of needs related to the serviceability, for instance adequate clearance, drainage, security for users, appearance, and many others. They vary depending by the typology of infrastructure, the needs of the owner, the use of the bridge and regulations.

It is quite common that bridges do not fully meet all performance requirements. Moreover, although showing some minor problems that may affect their serviceability requirements, they are frequently forced to remain in service, being an essential part of the infrastructure which belong to [*McKibbins et al., 2006*]. If conditions are not properly appraised and interventions do not take place, the loss of performance may proceed slowly but surely. In many cases bridges are not subjected to an optimal - or at least to the minimum necessary - programme of inspections and maintenance.

The progressive reduction of performance level is usually provoked by defects occurred during the construction or in subsequent interventions, while may not be associated to overall instability or excessive movements. In general structural failures are not related to serviceability problems, they are provoked by different causes. On the other hand, structural problem may affect also serviceability requirements. Structural failures may be really dangerous, because they could develop without any advance warning unless first signs of problems appear. Inspections and maintenance operations are highly suggested to have an immediate perception of possible risks so to prevent structural failures. Furthermore, they allow evaluating and reducing also serviceability problems.

Structural failure and/or collapse of a masonry arch bridge occur only rarely, but when it happens the effects are catastrophic. The consequence of structural instability and potential collapse can be extreme and unacceptable. They can provoke heavy disruption to the transport network, damaging not only the bridge itself but also all the service and furniture and involving adjacent properties. Moreover the consequence could be very tragic in case of injury or loss of life of people. Finally, by an economic point of view, emergency interventions of remedial or replacement are very costly respect to planned minor intervention. However it has to be noticed that in many cases, structural failures occurred in the past were related by phenomena that are difficult to identify during normal routine operations of inspection, or have been provoked by external causes, such as scour or heart-quake. For this reason structural analysis and condition appraisal have to be coupled in addition to routine inspections and maintenance.

Respect to structural failures, loss of serviceability has less severe consequence, but it can compromise the usage of the bridge. A lower level of performance may imply restriction of traffic: from the reduction of weight or speed of vehicles to the reduction of traffic lanes, or, in the worst case, even the interdiction of the whole traffic. Moreover it became absolutely necessary to increase inspections and maintenance, to implement some temporary measures for reduce the risk and is strongly suggested to carry on specific monitoring. All these operations imply an increase of costs, that have to be added to the inconveniences, both economical and other, provoked by the restrictions of traffic. Finally, if not appropriated measures are taken, serviceability problems may develop provoking a complete deterioration of the structure and even evolving in structural failure.

For these reasons, it is very important to plan a programme of condition appraisal and maintenance and to intervene in time with remedial measures when it needs. The recognition of the causes, the significance of changes and defects that become apparent during the inspection is essential to ensure the conservation of masonry arch bridges.

3.1.2 Ageing and adjustment of bridges

Bridges and infrastructures during their life have to sustain:

- persistent actions: self weight and dead loads;
- frequent actions: live loads and natural actions;
- extreme actions: exceptional load and natural disasters.

Persistent and frequent actions determine a continuous decay of the mechanical performances of the structures, while extreme actions, assuming that do not lead to the collapse, provoke a discontinuous but progressive decay. Continuous and discontinuous decay depends by the structural typology, the constructive features, the presence of defects, the level and accuracy of maintenance and many other factors. In any case it is a inescapable phenomenon. This is common to all the structures, but, in case of bridges, factors leading to a crisis may have a faster evolution once started and usually give less warnings. Disadvantages are severe both in term of costs for maintenance or remedial works and for the diseases which the users are subjected. Moreover there is a factor of dangerousness and it may be necessary to involve institutions such as the civil protection. Indeed ageing and loss of performance of bridge is a problem of great relevance.

The masonry arch, principal structural element, usually has a very slow ageing. Problems due to constructive features or eventual presence defects play a fundamental role only in the phase of construction during the removal of centering[1] : its longevity is related to a correct realisation of the barrel. Piers and abutment, spandrel and wing walls, filling and other components of the bridge may suffer more of ageing respect to the arch. In general their ageing is related to the decay of masonry material, which may present deterioration of mortar and blocks. Deterioration of bridge materials is usually related to natural agents, however, if decay is not too much severe, masonry continues to resist, thanks to the fact that it works in compression. However, inevitably some parts of a masonry structure could

[1] Falsework used for the construction of masonry arch bridges (see sections 1 and 2).

be subjected to tensile stresses, doe to distribution of loads, concentrate actions, impacts, settlements or hydraulic problems. Tensile strength is subjected to a progressive mechanical decay. Therefore masonry ageing has to be considered in the evaluation of bridge performances, but usually it is a local phenomenon.

Extreme external actions that could affect a bridge are due to combinations of loads, including dynamic effects, destructive natural events and problems of foundations, in particular due to the erosion of the base of piers and in general to hydraulic risks. The safety of the bridge respect to extreme actions should be assessed regularly, when the verification is not satisfied it is necessary to intervene. Safety concerns not only the load-bearing capacity but should be extended to the whole use of the bridge: performances under service loads - usability and longevity - and under ultimate loads - load-bearing capacity. In bridges safety assessment is fundamental.

Modern dead loads of a bridge are usually bigger than original ones: increasing of thickness of deck and ballast, enlargement of road, increasing of number of traffic lanes, new barriers and furnitures. The bridge may not be able to carry the increased dead loads, or, more frequently, the increasing of loads may accelerate the structural decay and the ageing. However, this problem seriously affect steel and pre-stressed concrete bridges and in a minor way reinforced concrete bridges, while usually does not concern masonry arch bridges. On the contrary an increase of dead loads, especially if they are symmetric, may give a contribution to the stability of the arch. Some negative effects may occur in abutments and piers, anyway self weight of masonry arch bridge is very big and the increased dead loads do not have a big impact.

The modern live loads are really bigger than in the past. The current traffic is stronger: number, weight and velocity of trains have considerably grown. As previously mentioned about dead loads, this increase may result in a structural inadequacy. In this case problems are related to not symmetric distribution of the loads applied. Masonry arch railway bridges were built mainly during the XIX and first half of the XX centuries: the traffic was much less intense and trains were much more light. Even if self load is very big, differently to what said about dead loads, in this case masonry arch bridges suffer more then concrete bridge, because of anti-metric loadings.

Regardless deterioration or damages, many bridges are not adequate to the current traffic, their safety has to be evaluated. However these bridges continue to be in working and safety problems usually do not come out under frequent service loads. Safety problems may occur only in particular circumstances in case of severe conditions, with a return time of many years. On the other hand, state of service of a bridge do not provide information about future problems and current level of safety. The safety of historical bridges still in use have to be assessed respect to the modern standards of use and security. An accurate control may lead to adjustment interventions that do not seem necessary considering only service conditions.

In the last years, technical regulations have introduced specific issues regarding the protection from seismic events. A new classification of the Italian territory as been provided, including also areas in which no seismicity was supposed. In parallel new rules have been given regarding both the design of new constructions and the assessment of the existing ones. In the case of bridges, collapse has to be avoid and it is necessary to guarantee the possibility of transit in order to allow the conduit of rescue operations. The level of damage allowed is related to the intensity of the seismic event and to its return time, as provided by regulations [*NTC 2008, Eurocode 8*].

Historical masonry bridges were built on the base of experience and designed through geometrical rules. Although this empirical approach, in many case they are still safe under static loads. Instead, in case of seismic actions, this consideration could not be taken into account. It is not possible to deal with the seismic behaviour of masonry structures without specific mechanics. Moreover the return time of heart-quake is longer than the average life of people, builders could not supply the lack of knowledge with the experience. On the other hand many masonry structures are still existing albeit have been subjected by heart-quake during their life. Builders developed some anti.seismic device. In general, a masonry structure built respecting the “*regola d’rate*”[2] has a greater possibility to resist to seismic actions even if has been realised without anti-seismic construction techniques. However, the longevity of a bridge do not proof its capacity to resist in case of heart-quake, even if it survived to seism during its life. Natural ageing, modification in the structure occurred during years, increasing of structural and not structural masses and other factors may have deeply modified the seismic behaviour of the bridge. Moreover, in case of interventions of functional adjustment it became necessary to realise also interventions for the seismic adjustment. Considering all this reasons, a big number of historical bridges needs intervention of seismic retrofitting.

[2] The ensemble of rules that have to be respected in bond to realise a correct masonry [*Giuffrè, 1991*] (see section 2 part 1).

3.1.3 Loss of bridge performance

Typical defects and problems that may affect masonry arch railway bridges have been outlined by several authors [*Bién and Kaminsky, 2004 and 2007; McKibbins et al., 2006; Proske and Van Gelder, 2009*]. The most common are:

- Deterioration of masonry; two are the principle mechanisms of decay:
    - loss of material from the face of blocks, called spalling, through flaking or de-lamination[3];
    - loss of mortar, with a reduction of joints;
- Movements of the spandrel walls: bulging (formation of bulges in spandrel walls), sliding, overturning and detachment of spandrel walls from arch barrel;
- Discontinuities in material with presence of cracking in arch barrel, piers, abutments and spandrel walls;
- Longitudinal cracking in the arch barrel;
- Transverse cracking in the arch barrel;
- Diagonal cracking in the arch barrel;
- Incompatible deformations of the arch barrel, which yield to changes of the initial geometry;
- Loss of material (falling stones) and destruction of material caused either by chemical or by physical processes;
- Movement of piers and abutments;
- Separation between rings in multi-ring arch barrels (sometimes called de-lamination);
- Damages in parapet or in other auxiliary bridge elements.
- Contamination (natural cover, besmirch)

[3] Further information about masonry deterioration processes and definitions of the specific words will be provided afterwards in the paragraph 3.1.3.3 regarding the material deterioration.

A survey conducted on british masonry arch railway bridges identified a series of common causes of bridges deterioration and loss of performance [*McKibbins et al., 2006*]:

- Loading and overloading;
- Instability of foundations;
- Water percolation, due to an inadequate waterproofing or drainage system;
- Growth of vegetation and other biological attacks;
- Vehicle impact;
- Thermal movements, only in large structures such as viaduct, it not so relevant in masonry arch bridges as in other typologies of brides.

Authors studied the typical damage patterns for arch bridges [*Beinert, 1976; Yanez and Alonso, 1996; Mildner, 1996*].

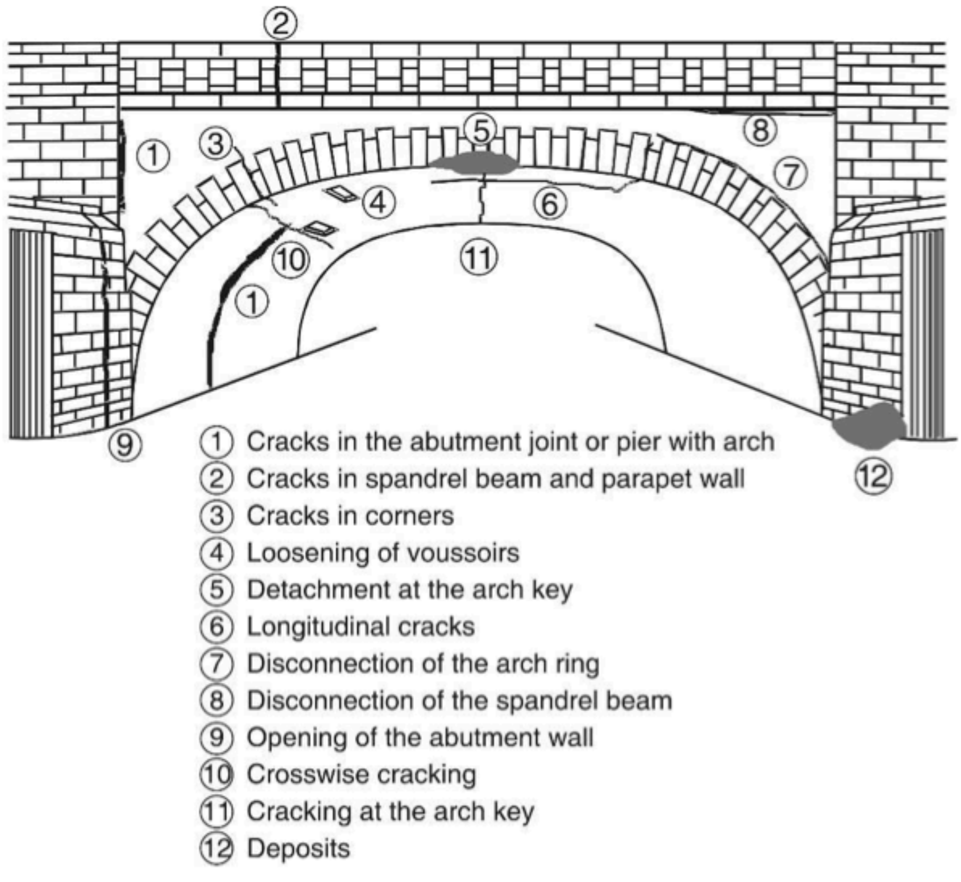


Fig 3.1 - Most frequent damages in masonry arch bridge according to [Yanez and Alonso, 1996], taken from [Proske and van Gelder, 2009]

A classification of damage patterns for historical stone arch bridges of the European railway organisations shown the frequencies and percentage of bridges interested by damages [*Orban, 2004; Proske and Van Gelder, 2009*]. Frequency and percentage of bridges affected by the different types of damage have been provided by the different railway organisations and have been expressed by factors:

- 1: Very frequent, about 50% of all bridges;
- 2: Frequent, about 25% of all bridges;
- 3: Occasional, about 10% of all bridges;
- 4: Rare, about 5% of all bridges;
- 5: Exceptional, less than 5% of all bridges.

Data are summed up in the following table:

Type of damage	Frequency
Damage at sealing	2.1
Deterioration of material	2.4
Separation and movement of wing wall	3.0
Separation and movement of spandrel wall	3.5
Damages at piers, foundation and skewback	4.0
Geometrical problems with the structures	4.0
Other problems (damages caused by plants, earth-quakes, impacts, and wrong maintenance)	4.0
Cracks in arch caused by settlements	4.2
Damages at the road crossing construction	4.3
Damages caused by overload	4.3
Deformation	4.4
Cracks in arch caused by overload	4.5
Damages at the parapet caused by single loads	4.6

Table 3.1 - Types of damages and their frequency according to [Orbán, 2004]

It is fundamental to understand the relationship between causes and problems in order to interpret the mechanisms of deterioration and find the appropriate solutions. However in many cases problems are due to a combination of different factors: it is important to consider every aspect in order to have a reliable assessment.

The problems that may influence the performance of a bridge belong to three categories:

1. Boundary conditions, regarding the conditions in the interface between the structural part of the bridge and the surrounding elements, mainly the foundations. But also the problems in the interface between the backfill and the structural elements belong to this category. In fact, even if is not a real structural element, the backfill plays a key role in the behaviour of masonry arch bridges, restraining the barrel and abutments and spreading the load. Loss of backfill performance is critical to the strength of the bridge as are its foundations and structural elements.

2. Structural condition, regarding the conditions of the different structural elements of the bridge - piers and abutments, the arch barrel, spandrel and wing walls - their capacity to carry on loads and to transfer them to the foundations.

3. Material condition: the state of conservation and the deterioration of bridge materials, typically the masonry.

3.1.3.1 Boundary conditions

A complete list of the potential structural consequences due to loss of support in an arch bridge is provided by [*McKibbins et al., 2006*]. When springing remain parallel, the typical settlements that may occur are:

- Vertical differential settlement between adjacent supports;
- Horizontal spread of support;
- Horizontal inward movement;

In these cases the arch develops three hinges (rarely the arch is able to accommodate support movements with two hinges). If three hinges form and vertical or horizontal settlement continues then this represents a failure mechanism and should be treated immediately.

Settlements may be transversal, springing do not remain parallel and are subjected to rotation:

- Rotation of an abutment or pier, which may cause diagonal cracks in the arch barrel and/or movements of spandrel wall.
- Local differential settlement in an abutment or pier, which imply a redistribution of stresses that may causes cracks in the abutment; when differential settlement occurs along the springing cracks interest the arch barrel.

Often in the reality bridges are subjected to a combination of different settlements that increases the negative effects of the individual settlements. For instances, a combination of translation and rotation of the base of a pier causes severe cracking in the arch barrel and in the spandrel walls; settlements of abutments combined with rotation provoke diagonal cracks in the arch barrel. The worst case is when loss of support produce a distortion of the arch barrel which can seriously reduce the load carrying capacity of the bridge.

The majority of bridges span over rivers, canals, lakes, and other types of watercourse. One of the main cause of settlements in bridges is the scour of foundation due to the action of watercourses. This phenomenon is very dangerous, in fact it has been observed that the scour has provoked severe damage and failure in bridges, moreover without prior warning or sign of distress to the structure. The action of the water may provoke a loss of foundation material with the consequent exposition of the footing of piers. The footing is not protected and became less stable: sliding or lateral deformation may occur. The discover of the footing is due to high velocity of water, especially in case of flood. One other dangerous problem in foundation of pier is the loss of material beside and beneath the base of the footing. In this case the foundation loose part of its load bearing capacity, causing structural failure. The bridge is subjected to a stress redistribution, in the worst case the instability may lead to structural collapse.

A list of the potential consequences of scour in arch bridges can be found in [*May et al., 2002*]. Typically scour may expose the bridge to hydraulic loading, debris accumulation, sediment abrasion, and washout of embankment behind abutment. Severe damages affect piers and abutments, which may be subjected to settlement, due to loss of support, and/or tilting. Moreover, differential movements of abutments or piers provoke twisting of the arch. Local damages may occur to masonry in the barrel intrados and spandrel, due to suction or washout. In more sever cases, scour may lead to a partial or total collapse.

Three different types of scour exist: natural, local or contraction. Natural scour is a long-term phenomenon, associated with erosion and deposition of bed material. It concerns the watercourse itself, which may be more or less vulnerable to scour, and it is related to flood events. Local scour happens in proximity of piers and abutments, due to their effects to the flow. Dimension and orientation of piers and abutments and some characteristics of the watercourse, such as accumulation of debris or bed configuration, may increase the velocity and depth of flow resulting in a faster scour, especially in case of floods. The localised loss of material could be very fast provoking a consistent loss of foundation material. It is one of the major causes of bridge failure and collapse. Contraction scour is due to a reduction of the width of the watercourse in the vicinity of the bridge. Turbulent flow under the bridge creates vortexes, which cause local scour. In case of flood events the width is reduced as the water rises under the arch. The design of the bridge, in particular shape and dimension of piers and presence or not of cut-water, may reduce the risk of contraction scour. The identify of the type of scour is essential to design the appropriate defence systems.

Fig 3.2 - Foundation and piers damaged because of scour

As said in the previous section, the presence of cracks is normal in masonry arch bridges: the masonry arch reply to changes in boundary conditions opening and closing cracks. The presence of cracks do not compromise the structural integrity of the bridge. Anyway the presence of cracks shows movement of piers and abutments, which has to be evaluated: differential settlements, in particular when it is transverse, may reduce the load bearing capacity of the bridge and cause local material damages and failures. For this reason, many masonry arch bridges have been built paying particular attention to the foundation, which are usually shallow, for instances with corbel[4] in the base of piers to reduce the bearing stress or incorporating into the foundation wooden piles when the ground was particularly weak[5].

Common causes of differential settlements are: foundation on weak ground, in particular ground with variable strata which may show different settlement between abutments and piers; changes in hydrostatic water pressure; washout of backfill and foundation with consequent change of volume and movement of

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[4] Horizontal outward masonry projection, in brickwork usually constructed of headers, to provide an outstand from the normal line of masonry.

[5] This technique is commonly used in venetian buildings and in particular in the Venice Trans-Lagoon bridge (see section 1 and the case of study).

structural elements. Movement of the foundations will affect the load bearing capacity of the bridge and may reduce its residual life. The signs of differential settlements develop over a long tome period so it is important to understand if and when foundation has settled. Often partial settlements occur during the construction of the bridge and therefore adjusted in the same phase, the observation of parapets and stringcourse may help to understand it. Previous repair works, especially masonry repointing, hide cracks: it is useful to observe also eventual irregularities in masonry bedding planes in arch and spandrels, which indicate movements. Irregularities in stringcourse and spandrel may indicate a uniform transversal settlement of a pier or abutment in addition to a longitudinal differential settlement. When cracking is present recent movements have interested foundation: in this case it is suggested to identify the causes as soon as possible in order to intervene in time. The extension of settlement and which part of the barrel is supported by the settled part of foundation have to be determined. The typical cracking due to differential settlement is show in the following figure.

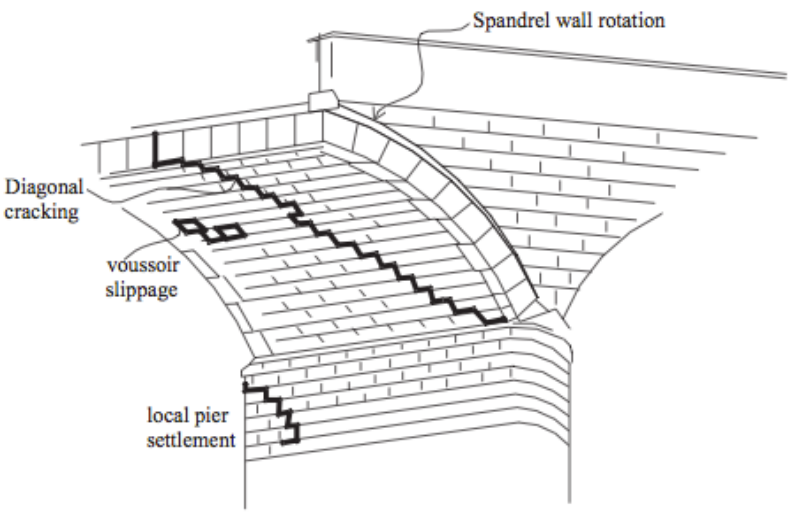

Fig 3.3 - Typical damages due to differential settlements [McKibbins et al., 2006]

Backfill, although is not really a structural element, plays a fundamental role in the carrying capacity of masonry arch bridges. Filling is used to give support behind the structure. In masonry arch bridge, backfill material is placed in the spandrels between the arch barrel and the road surface and retained laterally by the spandrel walls and/or wing-walls. It is common that a bridge is subjected to modifications during its history, and often modifications interest the backfill. Backfill may have been replaced, in case of longitudinal spandrel voids have been filled with incoherent material - stones, bricks, sand - or even with concrete. Changes in backfill pressure modifies the equilibrium of the arch and/or provokes movements in abutments that may affect the structural elements that are supported or retained by backfill material. Increasing or decreasing of backfill pressure has to be considered in case of intervention - realignment of railroad, consolidation, expansion of the structure - or in case of scour or flood. Further information about the effects of changes in boundary conditions due to backfill and the interactions with the structural elements will be given in the next paragraph.

3.1.3.2 Structural conditions

To evaluate the structural conditions of the bridge is useful to consider each structural element:

- piers and abutment;
- the arch barrel;
- spandrel and wing walls.

Piers and abutments

Piers and abutment transfer the vertical load from the arch to the foundation and contain the thrust of the arch. They are massive structures that work in compression with no, or very low, tensile strength. Their stability is due to geometry and equilibrium: every problem that affects their balance may have serious consequences to the whole bridge. Attention has to be paid also to the filling behind and over the abutments and to the material under the footing of piers, which contrasts the horizontal thrust of the arch.

In multi-span bridges, piers are particularly vulnerable imbalance between the thrusts of tho adjacent spans. In skewed bridges, piers may be subjected to torsion, failure due to torsion is possible even in squat piers [*Melbourne et al., 1997*]. In case of large piers, they are usually built with an external layer of high quality material, for instances square stones, while the core may be filled with low quality material, such as random rubble: settlements of the internal core expose the top part of the pier - which is the most stressed by the arch thrust - to risks. Thermic changes due to seasons may cause changes in the lateral earth pressure acting on abutments and wing-walls, provoking movements or damages in supports or changes in the stress of arch barrel.

Arch barrel

The performances of the arch may be affected by a damages, defects and cracking. The formation of hinges, with the consequent incremental loss of statical indeterminacy, is a normal phenomenon in masonry arch, which replay to loading, support movements and changes in boundary conditions, opening and closing cracks. However, hinge formation may lead to the development of kinematic mechanism that may result in failures and collapse. Collapse mechanisms of masonry arch have been described in the previous section. In the evaluation of the structural condition of the arch is important to understand if the present hinges are dangerous or not and establish when and why they have formed. In general the development of one hinge in the quarter-span region may actually herald the onset of a four hinges mechanism. The formation of a four-hinges mechanism implies that the barrel has reached its ultimate limit state and the arch is not safe. Four hinges mechanism is critical especially when support are not stiff, while when support are more rigid five, or more, hinge mechanisms are necessary to lead to collapse. Moreover the formation of cracks may change, or at least influence, the response of the arch to further movements, therefore is important to define the chronological order of formation of hinges to properly interpret the future behaviour and plan the repair works.

The types of cracks that may occur in the arch barrel are usually transverse or longitudinal. Less frequently diagonal cracking, arch barrel distortion and local failures may occur. Comments about the cracks can be found in [*Bienert,1976; Bartuschka, 1995; UIC-Codex, 1995*].

Due the plastic behaviour of the arch, transverse cracks are very frequent. Their relevance in the loss of bridge performance depends on a number of factors. If cracks are very longstanding and there are not signs of recent movements, their formation occurred at the time of construction, due to the redistribution of stress after the removal of the centring and the placing of backfill. Redistribution of stress occurs also in case of reconstruction, widening, adjustments and in general in case of works, with possible formation of cracks in the structure. If there are not actual movements these cracking are not dangerous: their presence has to be taken into account in the structural assessment of the bridge.

Instead, recent transverse cracks are symptom of actual movements, they should be dealt with promptness and causes have to be determined. Position and extension of cracking help the assessment: in single span arch bridges, formation of hinges and development of mechanisms is usually associated to cracks in the quarter-span region. Instead, in multi-span arch bridges, mechanisms develop with formation of hinges at the crown.

Longitudinal cracking may open in every part of the arch barrel. Differently to the transversal ones, they are more dangerous because affect the capacity of the arch to distribute the load regularly in the arch barrel and to abutments and piers. Longitudinal cracks that occur close to the spandrel wall can separate the wall from the arch, reducing the contribution of spandrel to the whole behaviour of the bridge. This type of damage is called front circle cracking and is due to the major flexibility of the arch respect to the stiffness of the spandrel walls.

Longitudinal cracks that interest only the crown region of the arch are usually due to transverse bending, while when they are extended down to the haunches the cause could be the opposite directional flows of the traffic: each half of the arch tries to swing in the direction of the relative traffic flow. This damage is also associated with an incremental permanent deformation and is a sign of low transverse distributional strength. Longitudinal cracks reduce the support of haunches, which is restricted to the segments between the cracks. Moreover, due to the loss of transverse continuity, the lateral pressure of the backfill on the spandrel walls determine its outward movement.

In single span bridges, local cracking in the crown region may be associated with a punching type failure mechanism and may be accompanied by the formation of a “yield-line” type failure. Causes could be the reduction in cover to the extrados at the crown, point loads and overloading, spreading of the abutments. In any case, tensile stresses in the crown intrados that may result in cracking.

Diagonal cracking is always due to differential settlement or spread of abutments or piers, which provoke torsion in the arch. Sometimes individual units of the barrel can move and became displaced giving way to a local failure, usually associated with point-loading and in case of deterioration or washing out of the surrounding mortar joints. The pattern of cracks in skew bridges may be different to those observed in square arches.

When arch barrel distortion in observed, usually due to long-term movements, it is necessary to take action of monitoring because it may seriously reduce the carrying of the bridge.

As previous said, type and intensity of actions and loads acting on the arch barrel may produce cracking, damages and local failure. Summing up, for each action consequence are outlined (however, often damages are consequences of a set of causes):

- Pointing load; it may may result in local failures:
  - pushing some masonry units through barrel, especially if the mortar is deteriorated and/or has been washed out;
  - yield-line pattern failure, often in conjunction with diagonal cracking due to other causes, such as differential settlement.

- Shear loading; it may damage the arch barrel. Typical damages are radial slipping: masonry arches are more vulnerable to this than other forms of construction. In multi-ring arches, shear may provoke longitudinal slipping and/or ring separation, called debunking. The skew arches are more vulnerable than square one because of the longitudinal shear, which is a consequence of the kinematic complexities of the structure.

- Transverse bending; it gives way to longitudinal cracks in the crown region. When it is combined with lateral pressure on the spandrel, because of the longitudinal flexing of the barrel respect to the longitudinally stiffness of the spandrel walls, a longitudinal crack opens in the barrel adjacent to the spandrel walls, with a separation between the barrel and the spandrel. This type of cracking is called front circle crack. When two opposite directional traffic flows pass over the bridge, longitudinal cracking may extend out of the crown region, coming to affect up to 2/3 of the span.

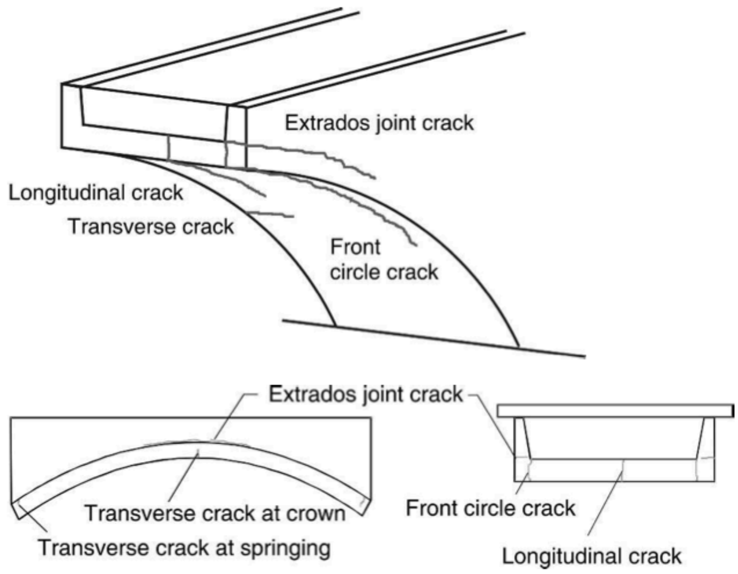

Fig 3.4 - Typical cracking in masonry arch bridge according to [Bienert ,1976], taken from [Proske and Van Gelder, 2009]

Spandrel and wing walls

Spandrel and wing walls show typical damages [*Melbourne, 1991*]: movement of spandrel walls ma result in tilting (or overturning), bulging (or buckling) and sliding, while longitudinal cracking in arch ring may induce a outward movement of spandrel that separate the cracked barrel from the other one, the so called detachment. Failure of spandrel may occur due to overturning after earth-quakes [*Rota, 2004*]. Sever cracking is usually due to pier settlement [*Como, 1998; Fauchoux and Abdunur, 1998*].

The causes of these movements are several. Sometimes the design of the walls was inadequate, due to a lack of knowledge of soil mechanics at the time of construction. In other cases,subsequently modification of the bridge may have affect the strength of spandrel, for instances, in large bridges, the substitution of the covering slab spanned between internal spandrel walls with filling material, which increased the backfill pressure. However, extensive haunching to the arch barrel over the pier or adjacent to the abutments reduces the effective height of the spandrel walls and subsequently the soil pressure [*McKibbins, 2006*].

Vertical live loads push the filling provoking a lateral pressure on the spandrel walls. The increase of loads in railway bridges due to the growth of traffic has worsen this problem. In addition, the rise of train speed transmit centrifugal forces to the filling. Moreover in the last years widening of carriageways and realignment of track made the traffic run very close to the wall, with a further increasing of the backfill pressure.

The absence of waterproofing, which is quite common in masonry arch bridges, coupled with defects or problems in the draining, implies that the bakfill is saturated. The cycle freeze-thaw causes incremental permanent movement, which deform the spandrel and may lead to collapse. It is important to maintain the existing drainage system, if any, and to isolate the carriageway, in order to avoid, or at least to minimise, the accumulation of water in the backfill.

In case of impact of vehicles with with the parapet it is suggested to check eventual damages of spandrel, because in case of repair of parapet without intervention on spandrel may cause problems.

In skew arch bridges attention has to be paid to the rotation of spandrel, which may occur as a reply of the spandrel itself to the oblique position of the arch.

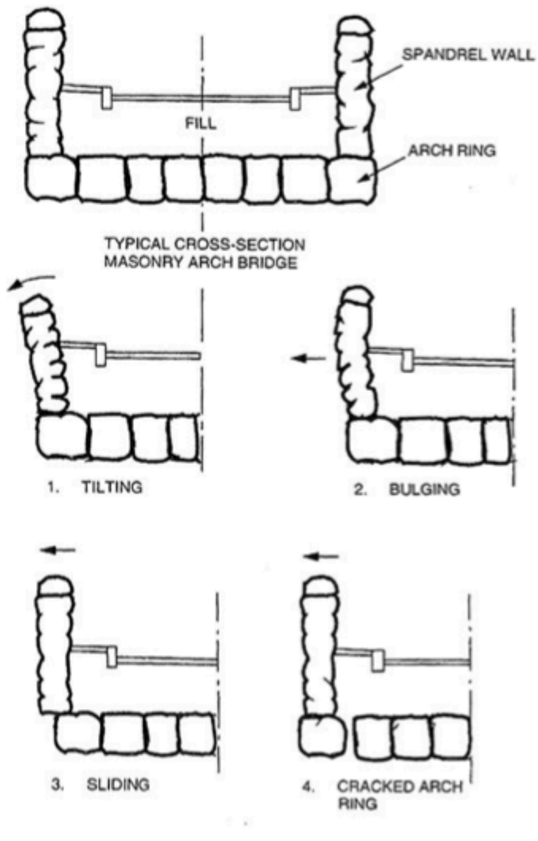

Fig 3.5 - Typical defects of spandrel according to [Melbourne, 1991], taken from [Mckibbins et al., 2006]

3.1.3.3 Material deterioration

Being the masonry the main material which masonry arch bridge consists of, the material deterioration of this type of bridge is typically the degradation and decay of stone, bricks and mortar due to physical, biological or chemical attacks. Although masonry is a very durable material, deteriorative processes are constant and, even if very slow, during the time the incremental weakening and destruction of material become relevant. The majority of masonry arch bridges have been built before the twentieth century, therefore it is frequent to observe evidence of material deterioration.

Deterioration is strongly influenced by the original quality of materials and by the accuracy of design and construction and its progress is related to the in-service environment. However it can be accelerated by lack of maintenance and by modifications to the structure or its use, including wrong works. Moreover, usually deterioration of masonry in bridges is due to the presence of water, it is fundamental to keep masonry dry or to allow it to dry and drain freely.

Problems affecting masonry arch bridges due to deterioration of masonry may occur in a series of circumstances in which the serviceable life of the bridge may be shortened. As a consequence maintenance costs increase while performances decrease: structural problems may occur with the needs of remedial intervention, which may be drastic and disruptive. Such circumstances are [*McKibbins, 2006*]:

- Bad quality of original masonry units: they are not very durable and deteriorate along with the mortar;
- Repointing is not carried out as required: mortar joints begin to deteriorate;
- Particular harsh in-service environmental conditions: jointing mortar or masonry units are affected rather than the pointing mortar;
- Repairs or alterations carried out with incompatible materials;
- Adverse changes in structural behaviour, loading or environment.

Deterioration of masonry, stone, brick and mortar is a very complex and wide topic. A considerable amount of literature has been published on this subject, and is not possible in this theses to completely deal with it. In this thesis only the most frequent typologies of deterioration and damages of masonry material [*Grimmer, 1984; Collepardi, 1989; ICOMOS, 2008*][6] that may affect masonry arch bridges and the relative consequences are reported [*McKibbins et al., 2006; Proske and Van Gelder, 2009*]. They are:

- Moisture saturation;
- Freeze-thaw cycling;
- Leaching of mortar;
- Salt attack and chemical weathering;
- Vegetation and biological attacks;
- Repair with unsympathetic materials;
- Expansion and contraction.

Moisture saturation

Moisture is one of the main cause of deterioration of masonry. Moreover it may affect strength and modulus of masonry and its resistance to fatigue and cyclic loads. Masonry units are vulnerable to the environmental agents, which cause deterioration, moisture saturation increase the exposition of units to deteriorating agents. The nature and extension of saturation depend by the material porosity. The movement of moisture may provoke washout of mortar, weakening the mortar joints. When backfill is saturated its pressure increases, in particular when water freezes, with consequent damages to spandrel walls. Saturated ground provokes movement and instability in abutments and wing-walls.

Freeze-thaw cycling

In climates with cold and wet winters, freeze-thaw cycling it is one of the most aggressive deterioration process. In masonry constant wet, repeated freeze-thaw

[6] The literature about this topic is huge, an interesting report of the specific bibliography about masonry degradation has been provided by [*Carosino and Matero, 1993*].

cycles cause spalling[7] of masonry units and mortar loss from joints. Although freeze-thaw cycling is the most frequent cause of spalling, attention must be paid to other possible reasons, such as de-lamination[8] due to salt-attack or crushing due to wrong repointing. The basic mechanism of freeze-thaw damage is well understood, exhaustive information about freeze-thaw damage of brickwork is available [*Stupart, 1989*].

Leaching of mortar

Leaching is a deteriorative process where moisture movement through or over the surface of a material causes the removal of soluble components from it; the "leachates"[9] may crystallise out of solution elsewhere or be redeposited at surfaces where evaporation occurs causing distinctive staining and discolouration, and gradual build-up of mineral deposits [*McKibbins et al., 2006*].

Some of the components of the mortar[10] are vulnerable to leaching, which has a physical effect on its structure that provokes a loss of soluble components resulting in an increase of permeability. The increase of permeability is progressive, because cause greater water flow and further leaching. When mortar is subjected to leaching becomes more vulnerable to the other mechanisms of physical and chemical deterioration (freeze-thaw cycling and salt attack). Moreover leaching provokes a loss of solid mass with consequent reduction of mortar strength and adhesion. In case of sever leaching mortar becomes weak and friable, joints may be subjected to wash-out or to compression extrusion in the areas most stressed with possible loss of masonry units and local concentration of stresses.

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[7] Spalling is a condition of masonry in which the outer layer (or layers) begin to break off unevenly, or peel away in parallel layers from the larger block of masonry. Differently respect to exfoliation and de-lamination, spalling may affect both natural stones and bricks or other fabricated masonry materials [*Grimmer, 1984*].

[8] De-lamination is a condition of stone in which the outer surface of the stone splits apart into laminae or thin layers and peels off the face of the stone. Because of their layered composition, this may be a natural condition of sedimentary stones such as sandstone or limestone [*Grimmer, 1984*].

[9] The solution resulting from leaching, made of soluble material components, by downward percolating water.

[10] The principal components that are vulnerable to leaching are calcium hydroxide and calcium carbonate from the cement and possibly also from the aggregate [*McKibbins et al., 2006*].

When the water containing leachates passes around masonry the dissolved salt precipitates causing surface staining and deposits. Bricks and stones are more on less vulnerable to leaching on the base of their porosity: porous and weak bricks, limestone and weakly cemented calcareous sandstones are more exposed to leaching. Joints made of impermeable mortar help to protect them by leaching. Voussoirs made with sedimentary stone may suffer of leaching of their natural cementing material, which reduces the strength of the cores of the individual voussoirs. This phenomenon is dangerous because the outer skin of the voussoir seems in good condition, this defect can be detected only in the coring.

Salt attack and chemical weathering

The physical salt weathering of masonry is related to subflorescence[11]. The presence of salt accumulation inside masonry is dangerous during the freeze-thaw cycle: moisture and salts in the wall freeze and expand, building up pressure within the masonry, which, if sufficient, may cause parts of the outer surface of the masonry to spall off or delaminate.

There are different sources of salts, which are in contact with masonry. First of all the groundwater, which going up from the ground in to the pores leave inside them salt deposits. Pollution is an other source of salt: polluted rainwater and contaminated runoff from the bridge surface, and the airborne pollutants, such as traffic fumes and sooty deposits from steam-trains. Finally the bricks, mortar, fill, ballast and other construction materials of the bridge itself.

The mechanism of deterioration of masonry related to the salt attack can vary depending by the type of salt, the type of substrate and its environment. Different theories have been proposed [*Larsen and Nielsen, 1990*] but it is settled that the crystallisation of water-soluble salts near to the surface of stone and brick may cause disintegration. Various mechanisms have been proposed to explain this disintegration [*Neilsen, 1988*]. The transport and precipitation of salts may provoke a series of

[11] Subflorescence is a potentially harmful accumulation, or hidden build-up, of soluble salts deposited under or just beneath the masonry surface as moisture in the wall evaporates. External signs of efflorescence may indicate the presence of subflorescence beneath the surface (subflorescence is some- times referred to as cryptoflorescence) [*Grimmer, 1984*].

masonry damages, both in the units and in the mortar: softening; crumbling[12]; flaking[13]; blistering[14]; laminar spalling of mortar and masonry units.

Hard bricks and durable stones are less vulnerable to physical attack and chemical weathering respect to mortar. Anyway, in case of repointing they can be affected, especially where hard and impermeable mortar has been used to fill joints, because it concentrates moisture movement inside bricks and stones.

Chemical weathering occurs in masonry material subjected to different chemical reaction. It is the result of the natural transformation process of masonry material that, due to the moisture from the environment, may assimilate some chemical elements[15]. Reactions provoke the formation of acids and bases that solve the binding material of the stones and mortar, provoking a series of possible damages: spalling, contour scaling, chipping[16], and flaking.

Sulphate attack mainly affects the mortar, causing flaking and crumbing. It may also attack bricks and some types of stone, provoking similar damages. In particular limestone, marble and calcareous sandstones are vulnerable to acids, especially sulphuric acid coming from acid rain or polluted air. Calcium carbonate, present in the mortar, is attacked by sulphuric acid to form gypsum, which forms a skin on the surface and prevents evaporation, with consequent damages of blistering and spalling.

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[12] Crumbling is a condition of a certain brittleness or tendency of the masonry to break up or dissolve. It may be caused by an inherent weakness of the masonry and gradual dissolution of the binder, or it may be the result of external factors affecting the strength or durability of the masonry, such as salts or moisture entering the masonry [*Grimmer, 1984*].

[13] Flaking is an early stage of peeling, exfoliation, de-lamination or spalling, and is best explained as the detachment of small, flat, thin pieces of the outer layers of stone from a larger piece of building stone [*Grimmer, 1984*].

[14] Swelling accompanied by rupturing of a thin uniform skin both across and parallel to the bedding plane, usually a condition found on sandstone, but also on granite. Because blistering can be caused by deicing salts and ground moisture, it is generally found on a surface close to the ground. Blistering may remain a relatively constant condition scattered over the masonry surface but, more often, it eventually results in greater surface peeling (exfoliation, de-lamination or spalling) [*Grimmer, 1984*].

[15] Such as sulphur dioxide, nitric oxide, or carbon dioxide.

[16] In chipping, small pieces or larger fragments of masonry separate from the masonry unit, often at corners or mortar joints. This may be the result of damage caused by later alterations or repairs, such as use of too hard a mortar [*Grimmer, 1984*].

Vegetation and biological attack

The growth of vegetation may provoke significant physical disruption. The development of but shrubs and tree roots can reduce the carrying capacity, while in case of grasses, weeds and small flowers the damages are less dangerous. The presence of creeping plants can disrupt the mortar and inhibit evaporation, moreover hides the wall surface, with problems for inspections and for the detection of incipient defects.

When masonry if damp or wet smaller living organisms, such as bacteria, fungi, algae, mosses and liverworts, may colonise its surfaces, with the risk of a deterioration of bricks, stones and mortar. The presence of biological organisms may result in physical effect, such as osmotic pressure and leverage of roots, or in chemical effects, such as the production of organic acids which can dissolve carbonates. Some kind of bacteria utilise the sulphates present in the groundwater, or even taking them from the atmosphere, to form gypsum and sulphuric acid, which cause decay of masonry. Others kind of bacteria dissolve silica and silicates. In general, deterioration due to biological attack is related to a complex interaction between chemical, physical and biological processes.

Repair with unsympathetic materials

Repair and maintenance works carried out using unsympathetic materials may cause and accelerate the deterioration of masonry. The selection and the use of compatible materials for works on old masonry is of fundamental importance for the repair of deteriorating masonry.

Masonry bridges were typically built with weak hydraulic lime mortar for bedding and pointing. Repointing and repair of old masonry with strong, impermeable cement mortars is a cause of damage and deterioration. In fact, strong mortar reduces the flexibility of old masonry and in case of movements[17] stress concentrations occur. Typically the use of hard mortar leads to damages of masonry units with the loose of their faces and edges.

[17] For instances in response to thermal cycles, changes in moisture content or loading, transfers stresses into the masonry units.

Repair and repointing with impermeable cementitious mortars provoke moisture movement through the masonry units themselves. In fact, it increases the saturation, pushing the moisture into other components or parts of the structure, accelerating their deterioration.

The insertion of ferrous elements, such as pins, clamps, dowels and supports, may provoke spalling damages. The corrosion of ferrous elements creates expansive forces that may damage adjacent masonry.

The use of stones and bricks with very different physical and chemical characteristics respect to the original materials may lead to deterioration. Particular attention has to be paid also to their position in the structure, for instances avoiding to put stones with great porosity, such as sandstone, under material which may release leachates, gypsum or salts, such as limestone, because it will affect the durability of the new units replaced. In general, the use of overly hard masonry units in repairs can damage adjacent original fabric.

Expansion and contraction

Expansion and contraction are due to thermal and wetting/drying cycles. Bricks are subjected to a progressive and permanent expansion on the removal of the kiln and exposure to water vapour, depending by their porosity and the calcitic content. In addition a reversible dilatation occurs in bricks as a result of wetting/ drying cycles. Even if dilatation is very small, recurrent cycles of expansion and contraction, due to wetting/drying and warming/cooling effects, lead to a gradual softening of brick. Similar mechanisms occur in stone: the constituent minerals may show dimensional changes because of moisture.

Differential expansion and contraction may occur due to moisture and temperature gradients in thick sections of bricks and stones masonry, causing differential stress distribution. External skin of bricks exposed usually have different properties because of their service weathering. It implies different dimensional changes respect to the bulk masonry, due to moisture movement and thermal variations. Lime mortars help masonry to absorb movements, giving it the ability of accommodate expansion and contraction cycles without damages [*McKibbins et al., 2006*].

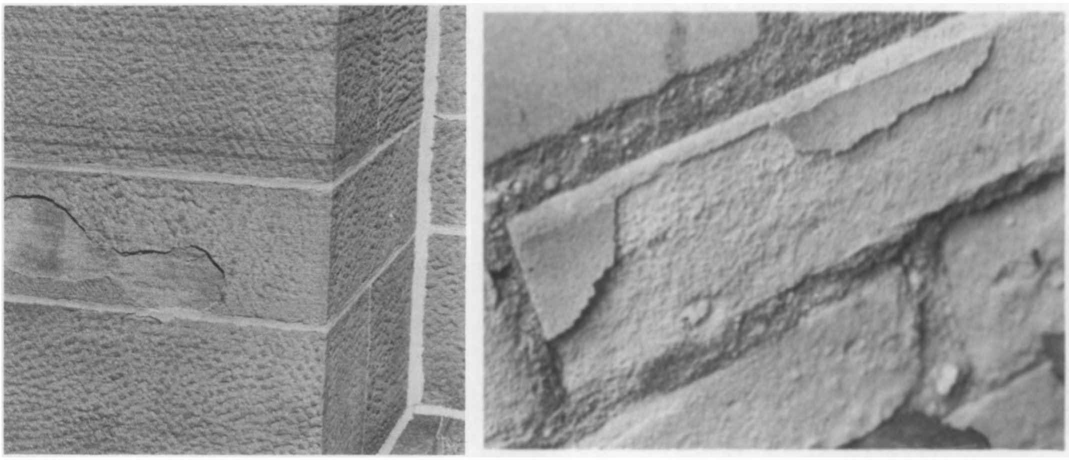


Fig 3.6 - Blistering (left) and spalling (right).

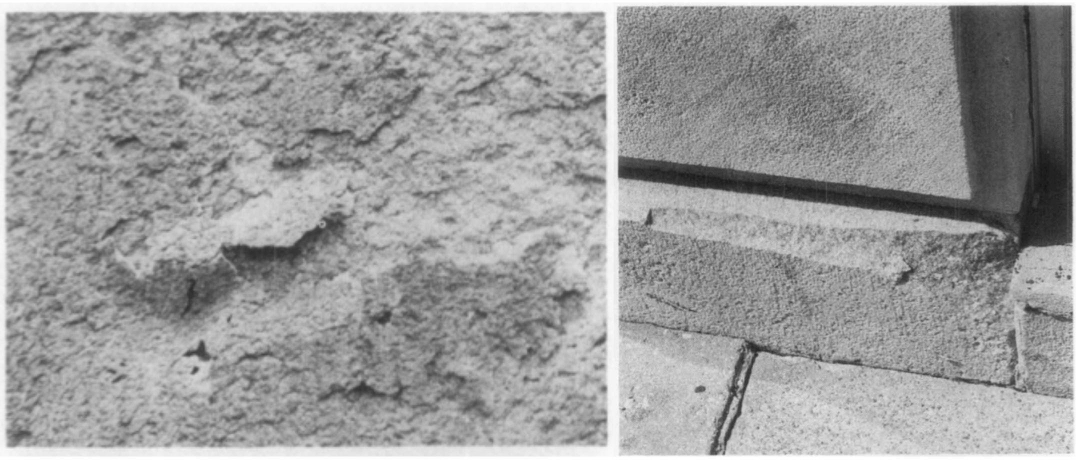


Fig 3.7 - Flaking (left) and chipping (right).

Fig 3.8 - Blistering (left) and leaching (right).

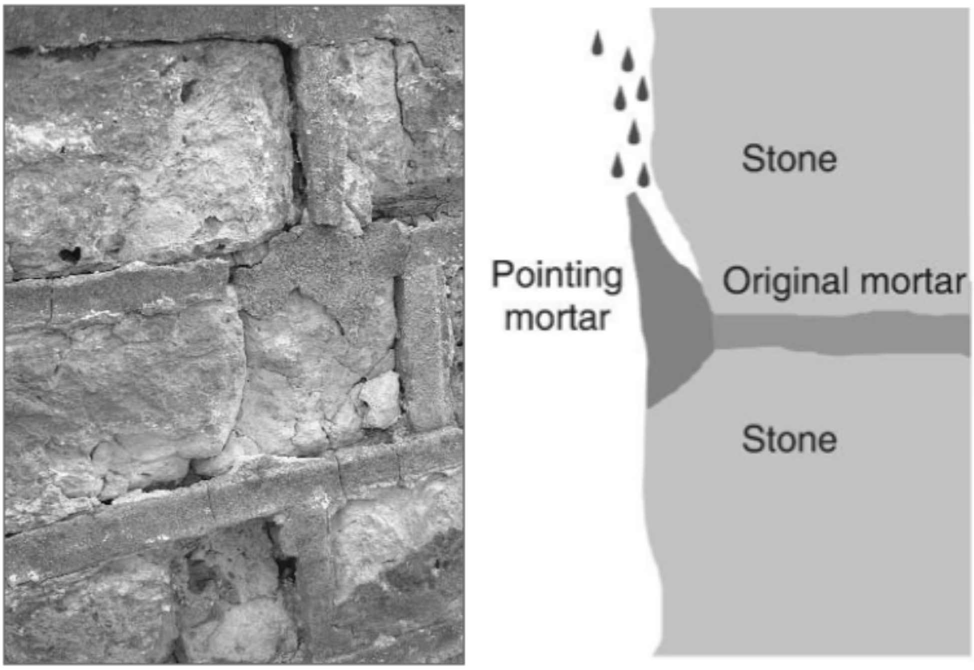


Fig 3.9 - Re-pointing with impermeable mortar (left) wrong application of re-pointing (right).

3.1.4 Catalogue of bridge damages

A very complete report about damages of masonry arch bridge has been drawn up by the UIC[18] workgroup I/03/U/285 to manage the project "*Assessment, reliability and maintenance of masonry arch bridges*", within the larger UIC Project "*Improving Assessment, Optimisation of Maintenance and Development of Data-base for Masonry Arch Bridges*", which involved all the main railway companies collaborated in this catalogue (CD, DB, JBV, MAV, NR, ÖBB, PKP, ADIF, REFER, RFI, RTRI, SBB and SNCF)[19]. The data collected and the direct observation of bridge damages resulted in a very comprehensive catalogue [*Ozaeta and Martin-Caro, 2006*]. Here a selection of illustrations taken from this catalogue is reported. The selection of images regards the most frequent damages in masonry arch railway bridges, with particular attention to the damages affecting the arch barrel. Figures help in understanding the damages and their causes that have been described in the previous paragraph. Illustrations are preceded by the identification of the damage and followed by short descriptive table regarding:

- usual location;
- possible associated damages;
- typical causes;
- structural importance.

[18] UIC, International Union of Railways.

[19] CD (*České Dráhy*, Czech Republic); DB (*Deutsche Bahn*, Germany); JBV (*Jernbaneverket*, Norway); MAV (*Magyar Államvasutak ZRt.*, Hungary); NR (*NetRail AB*, Sweden); ÖBB (*Österreichische Bundesbahnen*, Austria); PKP (*Polskie Koleje Panstwowe*, Poland); ADIF (*Administrador de Infraestructuras Ferroviarias*, Spain) ; REFER (*Rede Ferroviária Nacional, E.P.*, Portugal); RFI (*Rete Ferroviaria Italiana*, Italy); RTRI (*Railway Technical Research Institute*, affiliated to the *Japanese National Railways*, Japan); SBB (*Schweizerische Bundesbahnen*, Switzerland); SNCF (*Société Nationale des Chemins de Fer*, France).

Local erosion of foundation and loss of scour protection

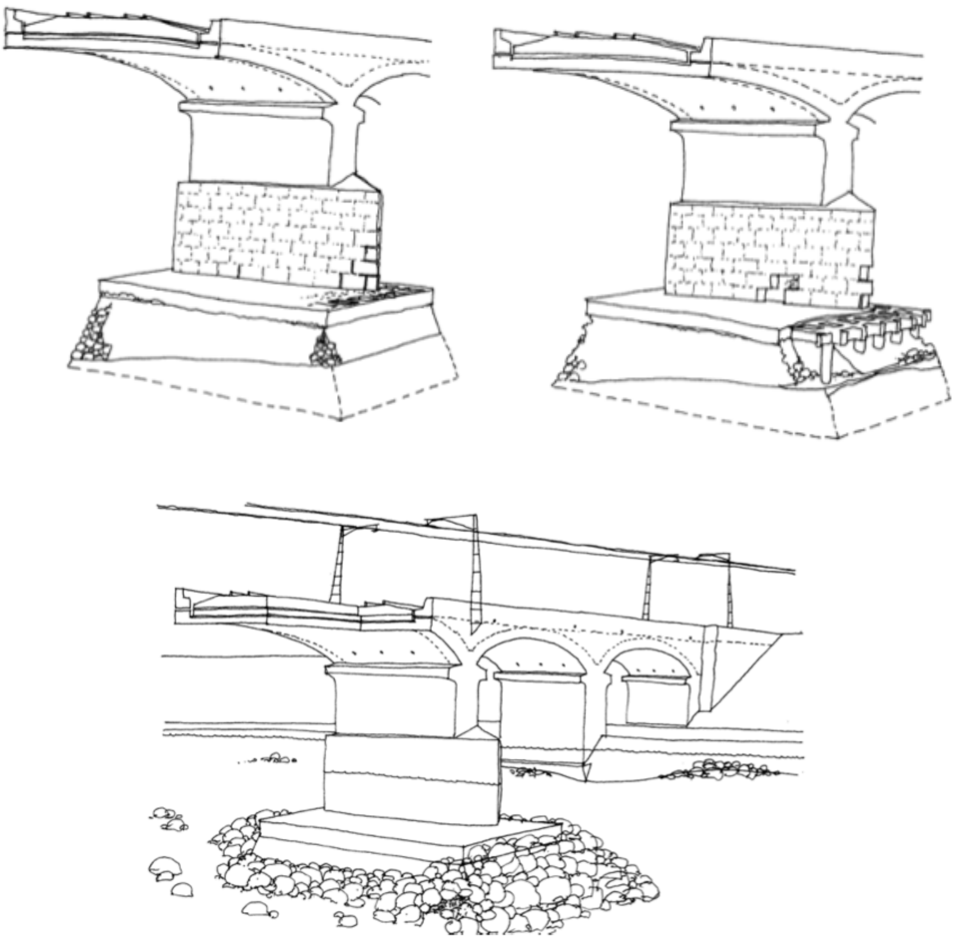


Fig 3.10

Usual location	Pier protection and foundations of both piers and abutments.
Possible associated damages	Abraded or rotten wooden piles; Local undermining of piers and abutments.
Causes	Increase of hydraulic speed due to decrease of river cross section or changes in the longitudinal profile of the river; Loss of scour protection to the bridge worsens the erosion of foundations.
Structural importance	Reduction of durability, with no immediate effects on the structural integrity of the bridge. It is a sign, however, of foundation problems that may result in serious damage or catastrophic collapse.

Table 3.2

General and local undermining of piers and abutments

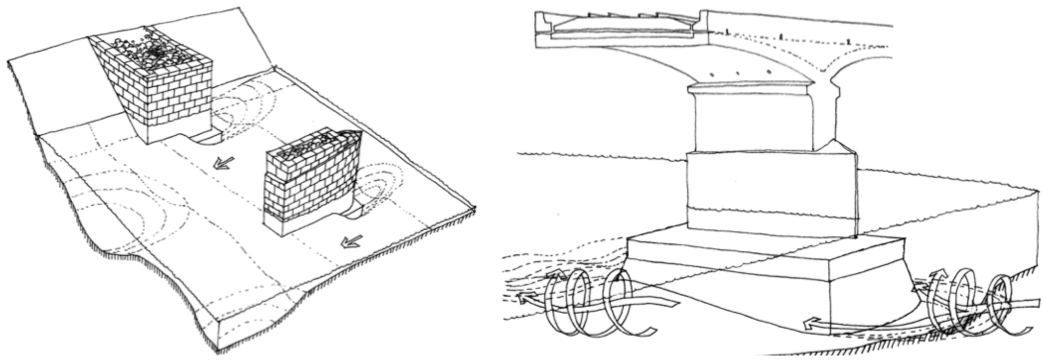


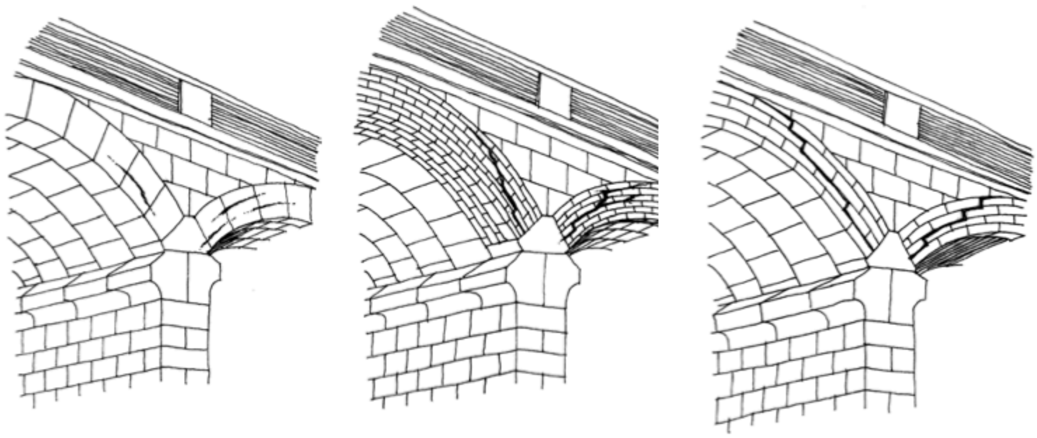
Fig 3.11

Usual location	General undermining occurs in any cross section of the river bed, up and downstream of the structure. Local undermining occurs adjacent to scour hollows immediately upstream of the pier or abutments
Possible associated damages	Undermining provokes loss of scour protection with consequent local erosion of foundations and damages to wooden piles and other foundation elements. Local undermining causes longitudinal, diagonal and transverse cracking in arch, with development of hinges and shear mechanisms and mechanical failure of masonry. In case of piers, multi-arch mechanism are activated. Vertical and/or stepped cracking occur in pier, vertical and/or horizontal cracking occur abutment. Vertical cracking may interest wing and side walls too.
Causes	Increase of hydraulic speed, due to decrease of river cross section or changes on the longitudinal profile. Development of eddy currents upstream of the pier and abutment.
Structural importance	Reduction of the strength behaviour of the bridge. If it occurs simultaneously with non-stabilised cracking on the superstructure or with masonry collapse of piers, the arch or an abutment, there is high risk of structure collapse.

Table 3.3

Mechanical failure of masonry

Fig 3.12



Usual location	Near to the springing and haunches of the arch. In any type of arch when caused by a foundation problem, but it is more usual in very slender and shallow vaults with deteriorated masonry.
Possible associated damages	Transverse cracking of arch barrel associated with kinematic mechanisms.
Causes	Excessive live loads or other actions arising from foundation problems that lead to compression failure or axial bending and shear failure.
Structural importance	Damage that will affect the strength behaviour of the bridge. It is a sign of a high risk of collapse

Table 3.4

Loss or dislocation of arch material

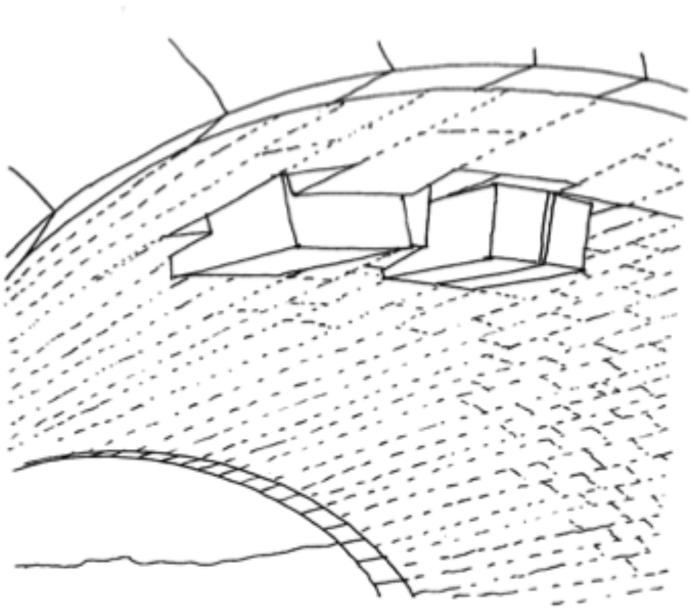


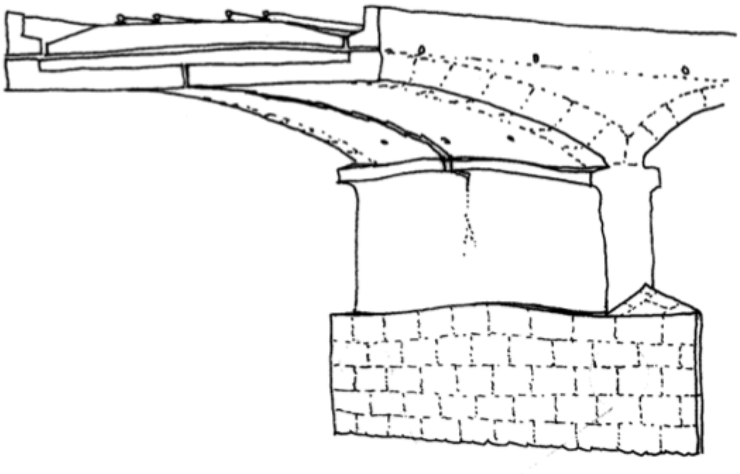
Fig 3.13

Usual location	In the crown of vaults with small depth of fill. Common in arches not very slender and deep. Common on skew bridges with straight bond.
Possible associated damages	General and local undermining of pier and/or abutments; Longitudinal, transverse and diagonal cracking of arch barrel; Three-hinges formation; Stepped cracking of spandrel walls.
Causes	Rotations and settlement of piers and abutments; Development of hinges that cause local compression failure; Impact arising from repetitive loads near to the crown of the arch.
Structural importance	Reduction of the strength behaviour of the bridge. Imminent collapse of the structure.

Table 3.5

Longitudinal crack in the centre of the arch

Fig 3.14

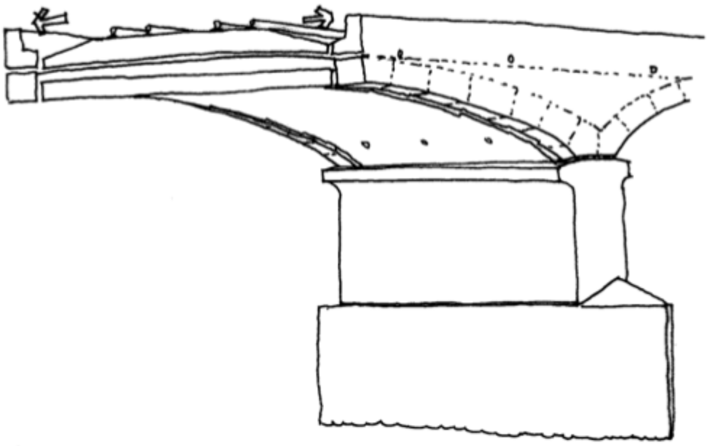


Usual location	In the centre of the arch barrel, in all types of arches. More usual on bridges carrying two tracks with opposite directional flows or side differently loaded, and on bridges with piers in the bed of a river where undermining and other foundation failures may occur.
Possible associated damages	General or local undermining of pier or abutments; Diagonal cracking of arch barrel and loss or displacement of arch material; Mechanical failure of masonry; Vertical or stepped cracking of pier; Vertical and horizontal cracking of abutment.
Causes	Differential settlement of the pier or abutment; Asymmetrical load on the arch due to traffic.
Structural importance	Reduction of the strength behaviour of the bridge. It may affect the bridge structural integrity within a short time if the damage is not stabilised. High risk of failure if other damages due to mechanical failure of masonry occur.

Table 3.6

Longitudinal cracking between voussoirs and arch barrel

Fig 3.15

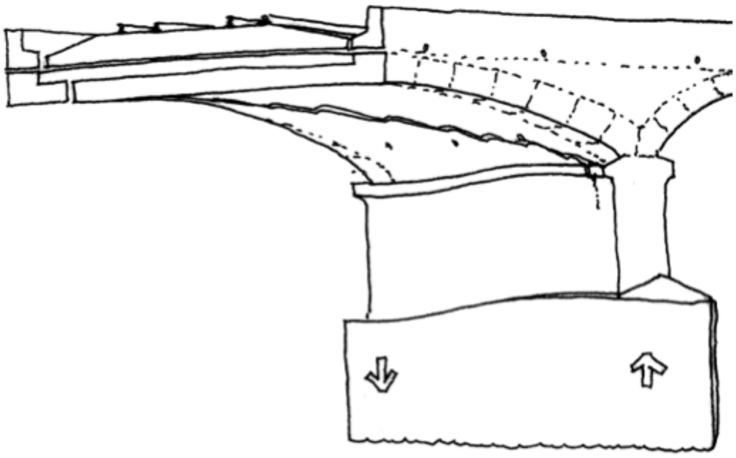


Usual location	Between voussoirs and arch barrel or under the inner face of the spandrel wall. Usually appears on deep bridges with saturated backfill. Furthermore the traffic runs near the spandrel.
Possible associated damages	Bulging of the spandrel Sliding of the spandrel Rotation of the spandrel.
Causes	Horizontal pressure on the spandrels; Pressure due to the water due to defective drainage; Damage of the joint between voussoirs and arch barrel; Heavy traffic loads may increase the horizontal pressure of filling on spandrel walls.
Structural importance	Reduction of the strength behaviour of the bridge. It will affect spandrel structural integrity within a short time unless stabilised.

Table 3.7

Diagonal cracking in arch barrel

Fig 3.16

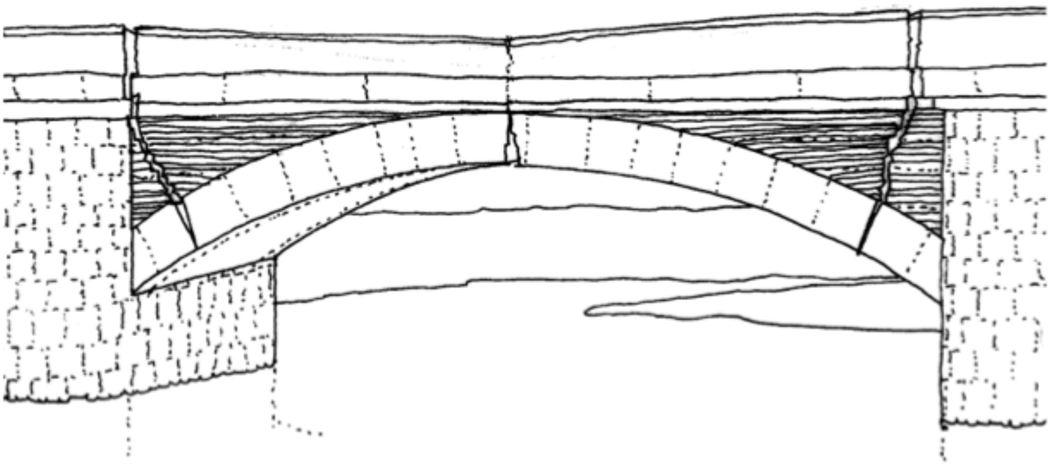


Usual location	Across the central part of the arch barrel between opposite corners, in all types of arches. More usual on skewed bridges with straight bond, and on bridges with piers on the bed of the river where undermining and foundation failures may occur.
Possible associated damages	General and local undermining of piers or abutments; Longitudinal cracking of arch barrel; Mechanical failure of masonry; Vertical or stepped cracking of pier or abutments; Horizontal cracking of abutments; Loss and displacement of arch material.
Causes	Foundation failure of piers and abutments, related with rotation; Inappropriate bond for the skew; Asymmetric load of the arch due to asymmetric traffic;
Structural importance	Reduction of the strength behaviour of the bridge. It will also affect bridge structural integrity within a short time unless stabilised. High risk of failure if other damages due to mechanical failure of masonry occur.

Table 3.8

Transverse cracking of arch barrel

Fig 3.17

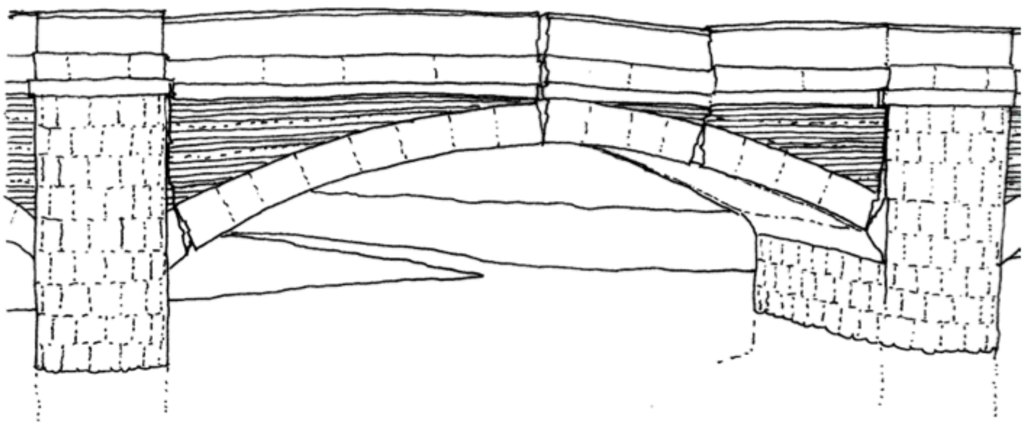


Usual location	Formation of three hinge. Joints and cracks open at the intrados near to the springing and at the extrados near to the crown. In any type of arch. Cracks are wider in case of slender piers and shallow vaults.
Possible associated damages	General undermining and local undermining of pier and/ or abutment. Loss or dislocation of pieces; Mechanical failure of masonry; Stepped cracking of spandrel and/or abutments.
Causes	Failure of the foundations and rotation of piers and abutments due to undermining problems; Premature removal of the centering; Failure and overturn of abutment due to horizontal pressure of the vaults; Mining subsidence.
Structural importance	Reduction of the strength behaviour of the bridge. It will affect bridge structural integrity within a short time unless stabilised.

Table 3.9

Transverse cracking of arch barrel and development of hinge mechanism

Fig 3.18



Usual location	Formation of 4 (a-symmetrical) 5 (symmetrical) or more hinges. Transverse cracking pattern characterised by cracks that open alternatively to intrados and extrados in four or five locations. In any type of arch, but it is more usual on very slender and shallow arch barrel with deteriorated masonry.
Possible associated damages	Mechanical failure of masonry; Stepped cracking of spandrel and/or abutment.
Causes	Insufficient load bearing capacity of the arch to carry the applied loads (or overloading).
Structural importance	Severe reduction of the strength behaviour of the bridge. Imminent collapse of the structure.

Table 3.10

Transverse cracking of arch barrel and development of shear mechanism

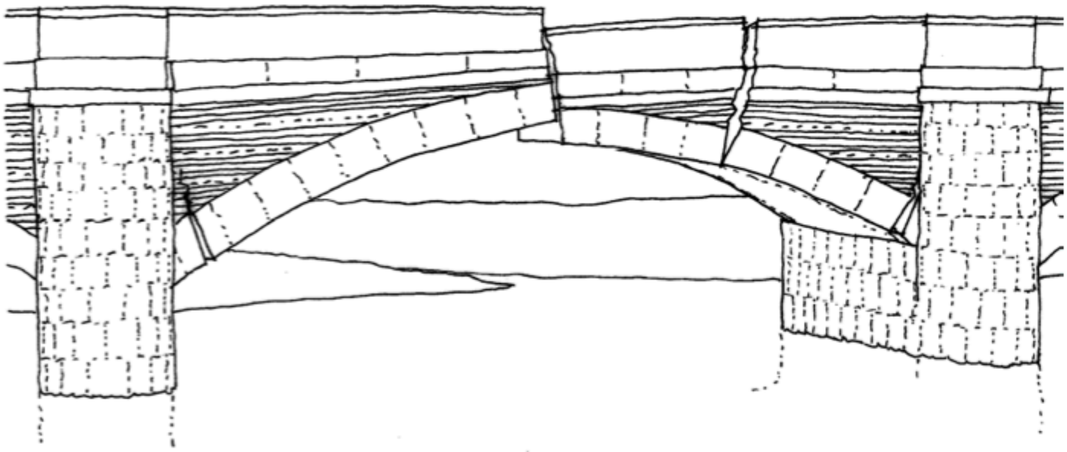


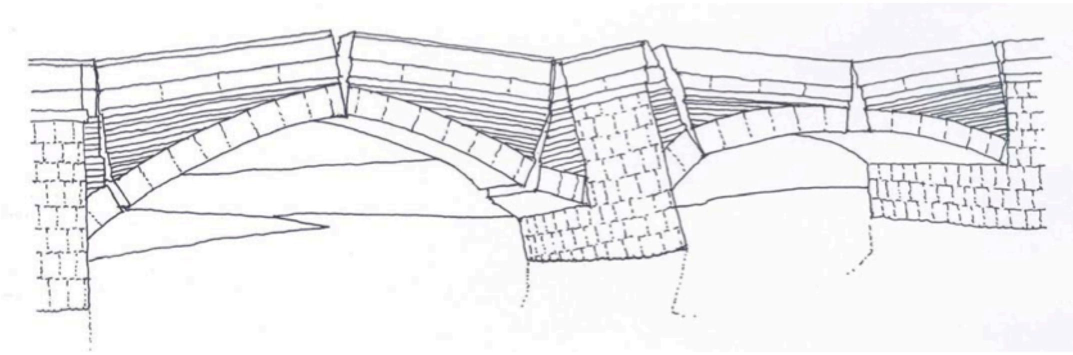
Fig 3.19

Usual location	Transverse cracking pattern, cracks open alternatively at intrados and extrados in three locations. In any type of arch, more frequent in cut down arches with deteriorated masonry and foundations problems.
Possible associated damages	General and local undermining of piers and/or abutments; Loss or displacement of arch material; Mechanical failure of masonry.
Causes	Lack of load bearing capacity of the arch under live loads; Differential settlement of a pier respect to the adjacent pier or abutment; Mining subsidence.
Structural importance	Reduction of the strength behaviour of the bridge. Imminent collapse of the structure.

Table 3.11

Transverse cracking of arch barrel and development of multi-arch mechanism

Fig 3.20

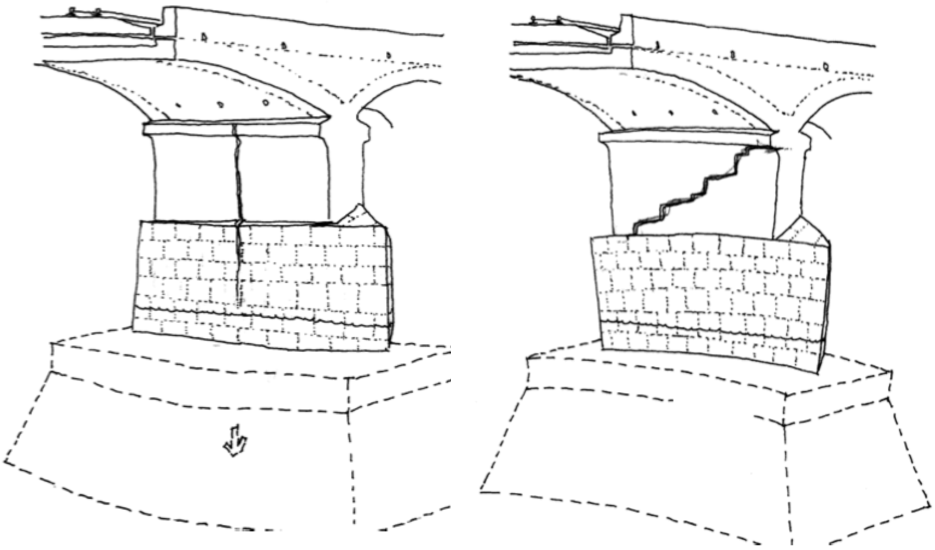


Usual location	Cracking pattern extended to several arches and piers. Transverse cracks with the development of a mechanism failure between adjacent arches, longitudinal slope in the intermediate pier. Typical from bridges with slender piers founded in a river and shallow arch barrels.
Possible associated damages	General and local undermining of pier and/or abutments; Loss or displacement of arch material; Mechanical failure of masonry; Stepped cracking of spandrel.
Causes	Rotation of a pier due to undermining problems or mining subsidence
Structural importance	Reduction of the strength behaviour of the bridge. Imminent collapse of the structure

Table 3.12

Vertical and stepped cracking of piers

Fig 3.21



Usual location	In the centre of the pier. Common in case of undermining problems that produce differential settlements of the pier.
Possible associated damages	General and local undermining of pier and/or abutments; Longitudinal and/or diagonal cracking of arch barrel; Vertical and horizontal cracking of abutment.
Causes	Local failure on the foundation; Differential settlement between the centre of the pier and its extremes.
Structural importance	Reduction of the strength behaviour of the bridge. It will affect bridge structural integrity within a short time unless stabilised.

Table 3.13

Vertical and horizontal cracking of abutments

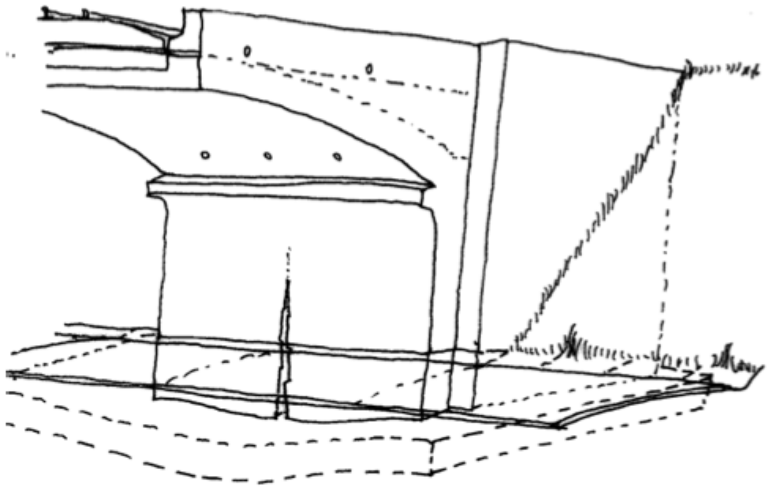


Fig 3.22

Usual location	In the centre of abutments. Common on bridges with undermining problems and differential settlements of the abutments.
Possible associated damages	General and local undermining of piers and/or abutments; Longitudinal and/or diagonal cracking of arch barrel; Vertical and/or stepped cracking of pier; Horizontal cracking of abutment.
Causes	Local failure of the foundation of the abutment; Differential settlement between the centre and the extremes.
Structural importance	Damage that will affect the strength behaviour of the bridge. It will also affect bridge structural integrity within a short time unless stabilised

Table 3.14

Horizontal cracking of abutment

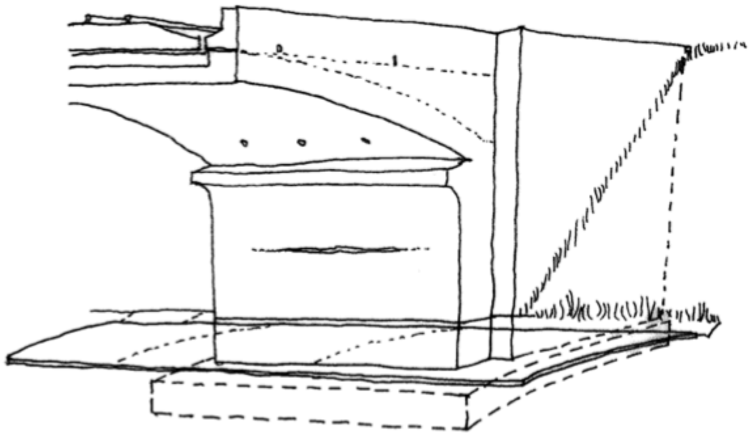


Fig 3.23

Usual location	In the centre part of abutments. Common in shallow arches where abutments are not able to resist the horizontal thrust from the arch barrel.
Possible associated damages	Transverse cracking of arch barrel with development of three-hinges mechanism.
Causes	Inadequate resistance of abutment to the horizontal thrust of the arch; Possible failure of the embankment fill behind the abutment.
Structural importance	Reduction of the strength behaviour of the bridge. It will affect bridge structural integrity within a short time unless stabilised.

Table 3.15

Bulging (or bucking) of spandrel walls

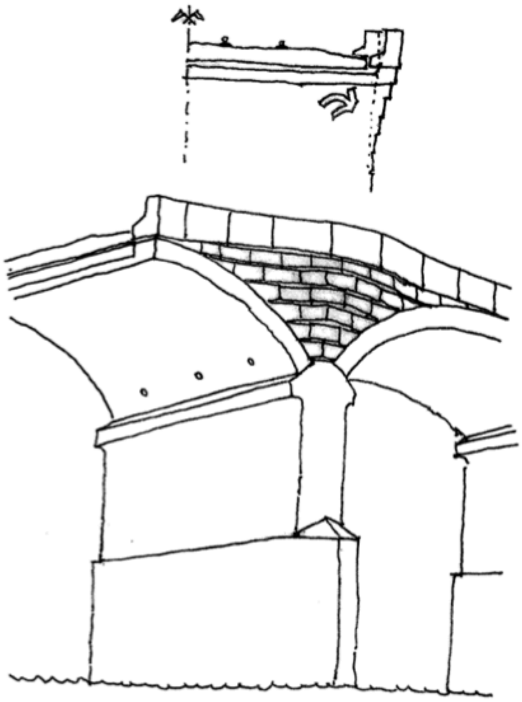


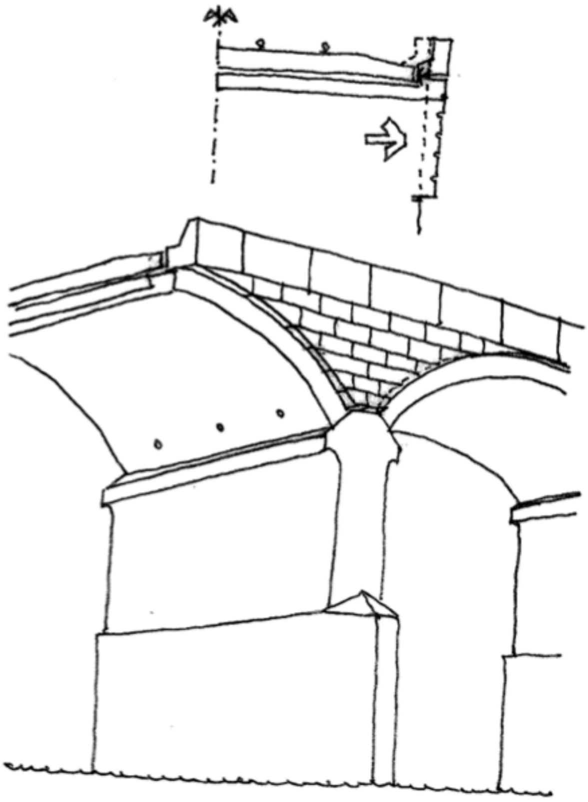
Fig 3.24

Usual location	In spandrel walls over the pier, in correspondence of its greatest height. Common in bridges with deep and not very wide arch barrels.
Possible associated damages	Longitudinal cracking between voussoirs and arch barrel; Sliding of spandrels; Rotation of spandrel.
Causes	Thrust from the backfill to the spandrel; Increasing in the horizontal pressure due to traffic loads; Pressure of water due to defective drainage; Damage of the joint between arch barrel and spandrels.
Structural importance	Reduction of the strength behaviour of the bridge. It affects bridge integrity within a short time if not stabilised. It affects the retention of the fill, with possible consequences to track support (derailment).

Table 3.16

Sliding of spandrel walls

Fig 3.25



Usual location	In spandrel walls over the pier, in correspondence of its greatest height. Common in bridges with deep and not very wide vaults.
Possible associated damages	Longitudinal cracking between ring course and vault; Bulging of spandrels; Overturn of spandrels.
Causes	Thrust from the backfill to the spandrel; Increasing in the horizontal pressure due to traffic loads; Pressure of water dune to defective drainage; Damage of the joint between ring course and spandrels.
Structural importance	Reduction of the strength behaviour of the bridge. It affects bridge integrity within a short time if not stabilised. It affects the retention of the fill, with possible consequences to track support (derailment).

Table 3.17

Spandrel rotation (or overturning of spandrel walls)

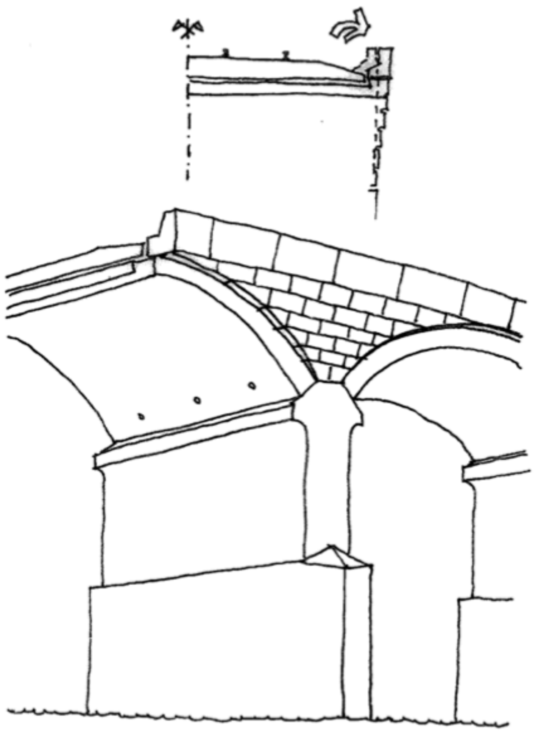


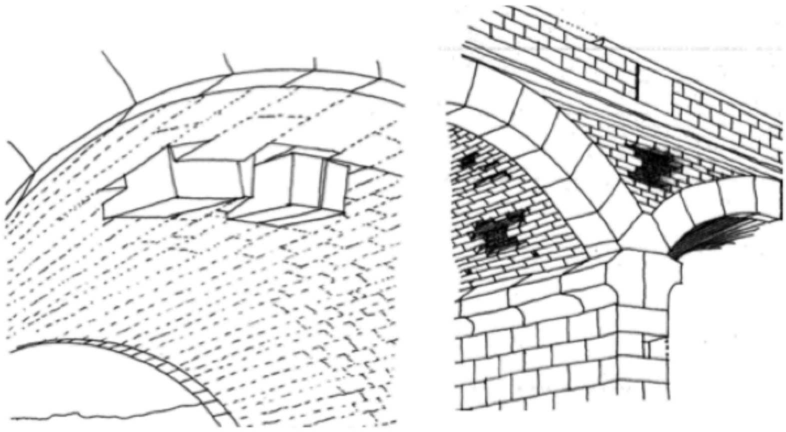
Fig 3.26

Usual location	In spandrel walls over the pier, in correspondence of its greatest height. Common in bridges with deep and not very wide vaults.
Possible associated damages	Longitudinal cracking between ring course and vault; Bulging of spandrels; Sliding of spandrels.
Causes	Thrust from the backfill to the spandrel; Increasing in the horizontal pressure due to traffic loads; Pressure of water dune to defective drainage; Damage of the joint between ring course and spandrels.
Structural importance	Reduction of the strength behaviour of the bridge. It affects bridge integrity within a short time if not stabilised. It affects the retention of the fill, with possible consequences to track support (derailment).

Table 3.18

Loss of material from joints and masonry

Fig 3.27



Usual location	In the arch barrels of structures built in brick, sandstone and limestone subjected to a continuous water flow from the track bed through the backfill and the arch ring to the intrados due to defective drainage.
Possible associated damages	Loss or displacement of arch material; Efflorescence; Honeycombing and Blisters.
Causes	Erosion of the mortar; Loss of pieces of masonry; Combined action of chemical and mechanical deterioration; Occasionally, damages near the crown may be due to the effect of the impact of the live loads on the arch bridge. Result may be falling of masonry pieces from the arch.
Structural importance	Reduction of the strength behaviour of the bridge. It will affect the bridge structural integrity. It should be repaired when damage affects important structural element and its extension is greater than 25%

Table 3.19

Section 3 - Part 2

Repair and strengthening

3.2.1 Works on masonry arch bridges

There are three levels at which work is undertaken on a bridge [*McKibbins et al., 2006*]:

- Routine maintenance, which is often preventative in nature;
- Repair, which is corrective in nature;
- Strengthening, which is intended to provide improvement.

The first two level can be included in the category of maintenance as defined by DIN 31 051[20]: all the actions taken to conserve and restore the normal conditions and to investigate and assess the actual conditions are integral parts of what is called "maintenance." Therefore, maintenance includes all types of inspection, servicing, and repair: damage and failure investigation, undertaking mitigation measures, repairing and mending, replacement and assembly, testing, and clearance. Inspection itself includes all means to investigate and assess the actual conditions of a system: testing, measuring, assessment, and documentation. Servicing includes all actions to keep a system in normal conditions. For example, testing, adjusting, exchanging, supplementing, preserving, and cleaning are parts of servicing. It seems to be meaningful to link inspections and servicing [*Proske and Van Gelder, 2009*] and usually the costs for both are given together [*Curbach et al., 2003b*].

Therefore repair can be considered as a part of the maintenance of structures., that has to be carry out when damages and defects are found intervening to repair or to refurbish it. Instead preventive maintenance has to be carried out in order to prevent the occurring of damages and defects. For this reason, in the thesis, operation of maintenance and repair are tackled separately. On other hand, strengthening have the same purpose of repair, to deal with damages and structural problems, but in

[20] DIN 31 051, 2003: *Fundamental of maintenance* (Grundlagen der Instandhaltung).

addition it aims to provide an improvement of the structural behaviour of the bridge, increasing its strength. Therefore, although sometimes repair and strengthening operations may seem very similar, it is possible to separate similar operations into the two categories on the base of the different target.

Bridges may be interested by several defects which different degrees of severity. Coordination of works is very convenient in order to tackle problems together, so to minimise severe damages and disruption of the bridge's normal service, potentially saving time and money. When choosing the most appropriate approach to works it is necessary to take into account several factors [*Broomhead and Clark, 1995; McKibbins et al., 2006*]:

- The type of fault, or faults, to be repaired;
- Ease of access (the topography of the site, the location of services, the type of obstacle which the bridge overcome -river, valley, road);
- Health and safety (risk assessment, method statement);
- Environmental considerations and constraints (protection of watercourse, pollution including noise and light, presence of protected species, minimisation and safe disposal of waste, reuse of old materials and minimisation of new ones);
- Heritage consideration and constrains (maintenance of original appearance and features, preservation of original structural fabric, listed building and national monument and other designations);
- Performance, long-term durability and maintenance requirements of repairs;
- Purpose of repair and ability to satisfy requirements;
- Available clearances;
- Cost of repair options, expertise required to execute repairs and contractor availability;
- Obstruction of future arch barrel inspections;
- Length of possession times/lane closure requirements.

The type of fault, or faults, determine the needed interventions. Suitable techniques that have to be considered in order to deal with the most common problems have been provided by [*Page, 1993 and 1996*] and provided in [*COST 345, 2006*].

Defect	Technique
Deteriorated pointing	Repoint
Deterioration of arch ring material	Masonry repair Saddle Sprayed concrete to soffit Prefabricated liner to soffit Grout arch ring
Arch ring thickness assessed to be inadequate to carry required traffic loads	Saddle Sprayed concrete to soffit Prefabricated liner to soffit Replaced fill with concrete Steel beam relieving arches Relieving slab Retro-reinforce
Internal deterioration of mortar, such as ring separation	Grout arch ring Stitch
Foundation movement	Mini-piles Grout piers and abutments Underpin
Scour of foundations	Underpin Invert slab Stone pitching Rip rap
Outward movement of spandrel walls	Tie bars Spreader beams Replace fill with concrete Take down and rebuild Grout fill if it is suitable “Stratford method”
Spandrel wall separation	Stitching Tie-bars and patress plates
Weak fill	Replace fill with concrete Grout fill if suitable Reinforced fill

Defect	Technique
Water leakage through arch ring	Make bridge surfacing water resistant High level waterproofing layer Waterproof extrados and improve drainage

Table 3.20 - Suitable techniques suggested to deal with the most common problems[Page, 1993]

The length of possession times and the lane closure requirements depend by the type of work. The primary function of a bridge is to allow pedestrian and vehicular movement across the surface of the bridge as well as any waterway or vehicular traffic beneath the bridge. The traffic across the bridge should be disrupted minimally. Maintenance, repair and strengthening have different impact of works to the rail traffic of the bridge:

- Routine maintenance may be usually carried out without interrupting the traffic, or at least suspending it just for the short time necessary to execute the work in order to ensure the safety of workers;
- Repair operations can lead to an interruption of traffic on the base of their extension: major repair may lead to closure of the bridge, while local repairs may be performed with less discomfort;
- Strengthening operations generally have a bigger impact on the traffic of the bridge respect to maintenance and local repair, however the disruption of the traffic is related to the extension of the intervention and to the area of the bridge interested by works.

A secondary function of bridges is to carry services, such as water or electrical lines, that may run through the bridge. Services may be embedded in the fill or run along the side of a bridge. The locations of these services should be determined before any intervention proceeds.

Performance, long-term durability and maintenance requirements of repairs are key aspects. In fact, when intervening on bridge it is important to remember that bridges are expected to remain in service for a considerable period of time in the future. The potential influence of works on the bridge's long term performances have to be evaluated, it is very important to avoid works which may compromise the durability. Works should offer an economical and effective solution to the immediate problems, but considering that loose or reduction of durability will lead to increasing future costs.

Costs are an other very important factor. However, is not easy to evaluate costs in the conservation of masonry arch bridge, because casts have many variables, such as the location of the bridges, the sizes of the bridges[21], the original materials, and other individual issues for each project. A detailed description of costs is not feasible, however a general comparison of the costs between the different techniques may be very useful for the choice of intervention.

Moreover, many of the historical masonry arch bridges are of considerable age and represent important features of our cultural heritage. They have also an important cultural value beyond their immediate functional purpose, that should be recognised. Their survival to this day owes a great deal to the care of past generations. Its a duty of our generation to ensure their conservation to the next ones. The needs of conservation have to be always taken into account when remedial or strengthening works are found to be necessary. Interventions have to ensure the maintenance of the original appearance and features and preserve the original structural fabric. For this reason, early remediation measures, to restore the carrying capacity, coupled with planned maintenance, to prevent damages, have to be preferred. They extend the life of these structures avoiding, or reducing, the need of urgent reconstruction, which may not be able to avoid the possible loss of the cultural value. Moreover urgent intervention are usually more expensive and usually cause more discomfort, because the bridge may be in severe conditions.

[21] The dimensional characteristics of masonry arch bridges which influence work costs are spans, rises, width, area, and volume.

In case of bridges classified by heritage legislation works should be carried out according to it[22]. However, even where a bridge does not have the benefit of statutory protection, its historical and heritage significance should be taken into account when considering carrying out any works. Beside, many bridges belonging to railway network are usually not protected by heritage regulations. Historical bridges may frequently need to be adapted and adjusted in order to guarantee the modern functional and performance requirements and to respect the actual technical regulations[23]. This is a fundamental aspect to guarantee their conservation, in particular in case of not listed bridges: guarantee their functioning is often the only possibility to ensure their conservation avoiding their replacement. At the same time it is a challenge, because sometimes it is not easy to reach the required standard without create modifications and alterations to the original bridge. Interventions should be carried out respecting the bridge and taking into account conservation and restoration theories. Further information about modern aspects of heritage conservation and new trends in consolidation of historical structures can be found in [*Jokiletho, 1999; Marmo, 2007*].

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[22] Italian legislation about the conservation of architectural and natural heritage (*Codice dei beni culturali e del paesaggio*) has been provided by MIBAC [*MIBAC, 2004*] in and subsequently updated (last update in 2008). Legislation about heritage varies among the different countries. International legislation have been provided by the international organisations for the preservation of the heritage, such as UNESCO and in particular ICOMOS (ICOMOS Charter - *Principle for the analysis, conservation and structural restoration of architectural heritage, 2003* and ISCARSAH Guidelines - *Recommendations about conservation and structural restoration of architectural heritage*). Further discussion of legislation concerning historic masonry arch bridges and guidance on issues associated can be found in [*Tilly, 2002*].

[23] As previously said in the first part of this section, actual technical regulations for the design of new railway bridges has to be respected also by all the existing bridges belonging to the network.

3.2.2 Preventive and planned maintenance

Planned maintenance consists of a series of routine and preventive operations of inspections and servicing that should be taken regularly on a bridge in order to:

- Maintain its performances;
- Prolong its serviceable life;
- Prevent potential damages and problems;
- Reduce the necessity of more significant remedial works over time.

Planned maintenance, which defines and schedules a number of basic maintenance activities, is very useful for ensure a good level of conservation of historical buildings and structures: it is an operation of “preventive conservation” [*Della Torre, 2003; SPRECOMAH guidelines, 2007-2008*]. Regular basic cyclic maintenance should be seen as a routine and very beneficial element of bridge management, with a remarkable saving of costs and resources. Preventive and planned maintenance help to minimise restrictions and to reduce urgent, expensive, disruptive works to the structures. It is strongly suggested especially for railways networks, so to improve conditions and performances of a whole bridge stock.

Planned maintenance includes a combination of routine inspections and activities to keep the bridge in good conditions. The specific activities required for individual bridges vary on the base of their requirements, condition and environment. However typical activities that has to be considered planning routine maintenance should include [*Mckibbins et al., 2006*]:

- Maintaining the bridge drainage, ensuring that is working efficiently by clearing drainage channels, weep-holes, or other systems;
- Management and removal of vegetation from all parts of the structure, including clearance of vegetation from areas immediately adjacent to the structure if these present a hazard or obstruction, or obscure parts of the bridge;
- Repointing of masonry, often following vegetation removal;

Maintaining drainage

Maintaining the bridge drainage, ensuring that is working efficiently, is one of the most important and worthwhile element of a routine maintenance programme, even if frequently is one of the most neglected. A correct management of water and an effective drainage of them is fundamental to guarantee long-term serviceability of bridges. In fact, as seen in the previous part of this section, the presence of water is relevant in most of the processes that lead to a deterioration of the bridge and its materials.

In order to carry out maintenance of the drainage system it is necessary to take into account the specific constructive features of each bridge. Many historical masonry arch bridges did not incorporate any kind of waterproofing system: the water may drain out thanks to the permeability of the structure. the use of lime mortars help the masonry to dry in good weather, avoiding its permanent saturation. In some bridge there are systems to help the drainage, such as weepholes[24] for the drainage of the fill material, or drainage channels, usually included in the parapet walls to allow water drain out from the roadway, instead pooling through the filling.

It is frequent that some waterproofing system has been added subsequently in bridge subjected to restoration or maintenance works. When such systems are present, original or added later, all the drainage paths have to be kept clear and functional. Repair and repointing should be done avoiding the use of impermeable mortars. If the original channels are damaged, in a bad state of conservation or cannot be cleaned, it is necessary to repair them. Drainage maintenance should be planned every year, after the autumn, so to remove fallen leafs.

Management and removal of vegetation

Plants may provoke disrupt and displace of the bridge fabric, obstruct the drainage channels and impede or delay the drying out of wet masonry. For this reason vegetation should be completely removed from all the parts of the structure. Attention has to be paid to roots, remaining roots has to be treated. Even the

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[24] Weep holes (also called weeper holes) are small openings left in the outer wall of masonry construction as an outlet for water inside a building to move outside the wall and evaporate.

vegetation growing up in the immediately adjacent areas to the bridge should be taken under control and cleared away to avoid penetration of roots in the masonry and foundations. Moreover they may hide the structure of the fabric, make inspection difficult. Removal and management of vegetation should be planned every years, possibly in spring.

Repointing

Deterioration of masonry is often concentrated in the mortar, which plays a sacrificial role to preserve masonry units, because mortar is weaker, more porous and permeable than bricks and stones. Water is present in almost all deteriorative processes. The movement of water through the mortar in the joints is the principal cause of its deterioration, which is usually most rapid and severe at the external surface of the joints, that are more exposed to the atmospheric agents.

If the masonry has a correct behaviour, deterioration should be concentrated in the pointing mortar[25] of the masonry, which can be easily repaired by repointing (that is the replacement of the pointing mortar), optimising the durability of the masonry as a whole. Even if is a repair action, selective repointing should be considered as an operation of routine maintenance, that has to be carried out when necessary. In fact, the deterioration of pointing mortar may expose the bridge to other damages.

The frequency of repointing depends by geographical location and weather conditions. It should not be carried out in cold and wet weather, especially if lime mortars are used. The best time is in spring, after the removal of vegetation, so to repair the damages due to plants and winter weather. Moreover repointing mortar need a long period to develop strength before to be exposed to freezing conditions, summer is perfect for it. More information about the techniques of repointing will be provided in the next paragraph, about the repair techniques.

It is very important to do not cover cracking and distortion of the structure with repointing, because it can mask some serious structural problems. If cracks are

[25] The filling and finishing of mortar on the outer part of a joint where the bedding mortar has been raked back from the masonry face or left recessed from it in construction.

longstanding and non-progressive repointing may be considered, but is of fundamental importance to register this operation and to provide detailed report of the defects before and after the intervention [*McKibbins et al., 2006*].

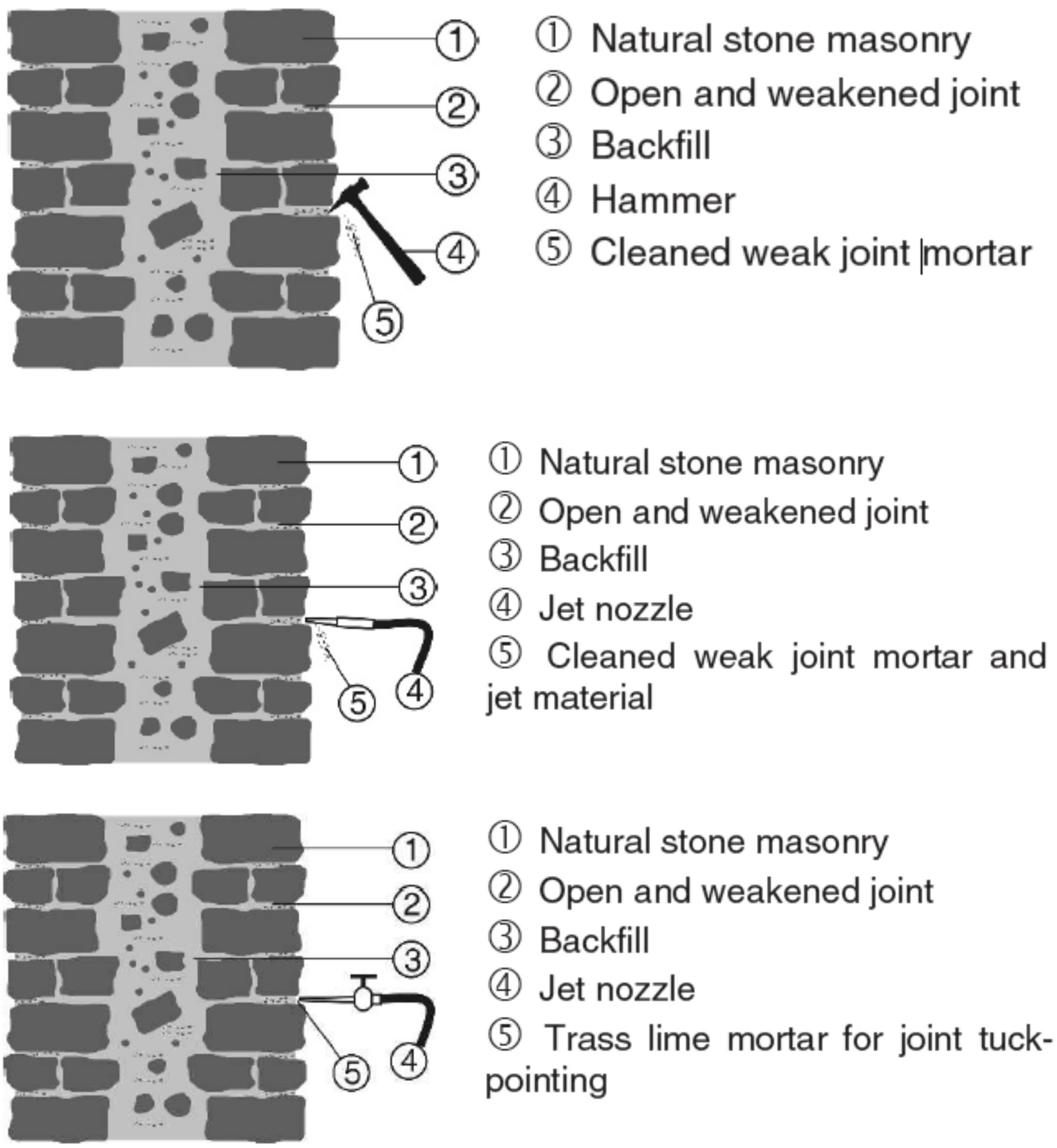


Fig 3.28 - Masonry repointing: sequence of operations of mortar refurbishment using the dry spraying method, according to [Bartuschka, 1995], taken from [Proske and Van Gelder, 2009].

In addition to these classic actions of preventive maintenance it is important to deal with all the other potential hazards which may provoke damages or accelerating the deterioration process. All the potential immediate and long-term hazards to the performance and serviceability of the bridge should be identified, through periodic routine visual inspection. Hazards should be taken under control with routine maintenance activities. For example, in bridges over watercourses the accumulation of twigs and branches in culverts, channels and arches may result in flooding: eventual obstructions has to be removed periodically. All apparent threats to the bridge should be identified and dealt with as necessary, for example evidence of scour or silting-up.

A list of different maintenance actions on arch bridges and their return time has been provided by [*Steele et al., 2006*]:

Maintenance activities	Frequency
Vegetation removal	Every 5 years
Coping stones replacement/realignment	Every 10 years
Brickwork maintenance - repointing/renewal	Every 15 years
Parapet repairs/replacement	Every 15 years
Invert clearance	Every 20 years
Cutwaters replaced	Every 40 years
First refurbishment scheme	Every 120 years
Second refurbishment scheme	Every 200 years

Table 3.21 - Frequency of maintenance actions, according to [Steele et al., 2006].

Beside these routine preventive maintenance operations, planned maintenance is suggested also in case of repair and strengthening, in order to better evaluate and to prolong their efficacy. Moreover it help to discover eventual potential problems that may arise after the intervention. In addition, routine inspections aim to obtain information to plan the maintenance activities, by identifying changes in bridge's conditions and external factors which may affect the bridge and its functioning.

Detailed record of all maintenance work carried out on a bridge have to be registered, with a comparison between the situation before and after the interventions, including photographs, surveys and measurements, when appropriate. The information collected are very useful to budget and programme future maintenance actions and inspections, to assess the bridge capacity and to investigate cause and significance of new defects.

3.2.3 Masonry repair

Repair operation of masonry arch bridges mainly regards the necessity of repairing deteriorated and damaged masonry. Causes and mechanisms of deterioration in masonry have been discussed in the first part of this section. In order to correctly intervene on masonry it is very important to well understand the causes and to properly identify the deterioration mechanism, otherwise remedial works will not be effective and recurrent and new problems may arise. For this reason to reach a complete understanding of the causes, experts advice, assessments, visual inspections, on site testing and laboratory analysis are strongly suggested. Damages of masonry that denote a potential structural cause, such as cracking or distortion, should be carefully investigated prior to carrying out repair actions. A complete list of repair techniques for masonry railway arch bridges and the diffusion of their use in the railway organisations has been provided by [*Orbàn, 2004*].

It has to be point out that frequently deterioration and damages of masonry is associated with the lack of maintenance. Inadequate maintenance may be itself a cause of masonry deterioration. A good solution to masonry deterioration in bridge is the combination of local masonry repairs and replacement combined with improved maintenance in future. The main remedial treatments for masonry arch bridges are outlined in [*McKibbins et al., 2006*]:

- Repointing;
- Deep pointing and filling of joints;
- Pressure injection of grout within the structure;
- Superficial crack repairs;
- Patch repairs;
- Application of consolidants and sealants;
- Cleaning.

Repointing

Repointing has been already introduced in the previous paragraph as an operation of routine maintenance. However, when the diffusion of loss of mortar in joints is wide or when repointing is a part of other remedial treatments, it can be considered as a repair work. Although the need for repointing can vary considerably depending on the structure, its materials, design, location and exposure, masonry typically requires extensive repointing at intervals of between 25 and 50 years, or even longer. When deterioration and loss of mortar occur there is the risk of loos of masonry units, which may present a hazard to traffic and members of the public using the area below the bridge. Loss of mortar from joints also reduces the ability of the masonry to transmit and evenly distribute forces, focusing stresses in localised areas and potentially leading to cracking and distortion.

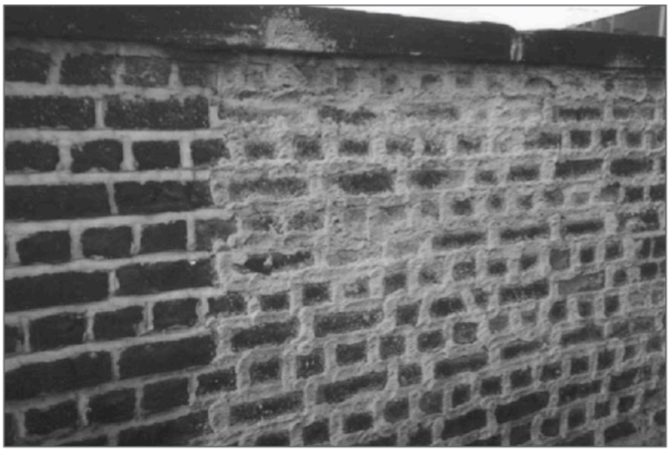
In repointing it is very important to pay attention to appearance and performances characteristics of the original mortar in order to ensure the maximum compatibility between the new and the existing mortar and avoid possible future problems. The choice and application of suitable mortars for conservation and restoration requires a careful approach: the incompatibility of materials may seriously compromise the conservation. Testing are usually performed in order to determine the existing mortar constituent and approximate mix proportions. The strength of the two mortar should be the closest possible and new mortar must be always weaker than masonry units. New mortar need to have an adequate permeability, so to allow moisture evaporation through the joints rather than through the masonry units. In general the use of lime mortar has to be preferred to cementitious mortars, in particular in case of buildings with a special historical value natural hydraulic lime mortar should be used. More information about the correct choice of mortar can be found in [*Ashurst, 1990*].

In order to carry out a correct repointing it is necessary to clean the joints out to a depth of at least twice the width of the joint, or to a minimum depth of 15 mm from the finished face of the joint. It is very important to take care to avoid damaging the arises of the brick/stone. For this reason is preferable to use hand tools[26].

[26] Quirks and long necked jointing chisels with parallel faces.

However, hand tools are adequate in case of weak and soft mortar, while in case of thin joints and dense mortar the cleaning of joints may be very difficult and the risk of damage masonry units is high. Particular caution should be exercised where aesthetic or historic value is a concern. In order to remove hard mortar it may be necessary to use hand-grinder fitted with a thin diamond blade to score the centre of a joint, then remove the rest with a hand chisel. The use of small pneumatic chisels, although it can work very well for mortar removal, can provoke chipping[27] to the edges of masonry units if it is not done carefully. Therefore, these techniques require the utmost care and skill and can be carried out only by specialised masons.

Once removed the deteriorated mortar, joints have to be clean from dust and loose material, brushing and then flushing out with water. Any joints which have dried out since cleaning should be re-wetted. The new mortar should be plastic and workable but stiff as possible and it should be pushed into the back of the joints in layers and finished flush with the surrounding brick/masonry, avoiding recessed joints should be avoided. Literature about repointing is wide, repointing can be observed in many historical buildings. Specific information and indication of good practice in the selection and application of mortars for repointing can be find in [*Mack and Speweick, 1998*]. Wrong application of pointing mortar may lead to unaesthetic results or even increase the exposition of masonry to deteriorating agents instead of protect it [*Bartuschka, 1995*].



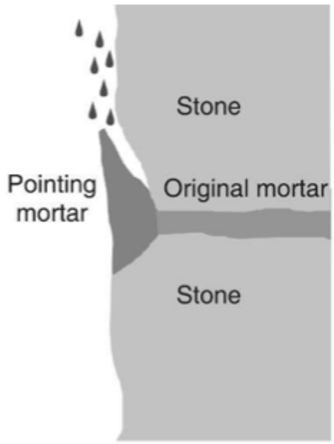


Fig 3.29 - Unaesthetic results of repointing (left)
and example of wrong application of repointing [Bartuschka, 1995] (right).

[27] See masonry damages and deterioration in the previous part of this section.

Deep pointing and filling of joints

In case of extensive deterioration of mortar joints resulting in voids and friable mortar inside the joints for a depth greater than 50 mm, normal repointing is not sufficient to repair the damage. In this case it is necessary to intervene with pressurised mechanical pointing using a dry-mix process[28]. Deteriorated mortar has to be removed, using hand tools or high pressure water jetting, in order to reach the more solid material. Usually it is necessary to excavate the joint up to a100 mm in masonry made of bricks and even more in case of stones. It is necessary to pay attention to the facing course of brickwork, because the excavation of joints may provoke units loosening. It is necessary to block the units, putting inside pinning stones to fill wide and deep joints in the same style as the original build. Then mortar is injected to fill the joints whit a spray pointing equipment. In case of bridges having an important aesthetic appearance, particular attention has to be paid to the mortar surface, doing the necessary finishing. This techniques have been frequently used starting from the 60's of the last century [S*owden, 1990*].

Pressure injection of grout within the structure

In case of voids inside the structure it may be necessary to intervene with grout injections. This technique can be used to repair such types of problems, but is more frequently applied in order to strengthen the bridge. Therefore it will be discussed in the next paragraph.

Superficial crack repairs

As previously said, before to repair cracking it is absolutely necessary to have a complete understanding of the causes and, when possible, to deal with them. In fact cracks repairing if effective only if the causes that provoked them, forces and/or movements, do not recur in the future. In any case superficial cracks repair do not

[28] In the dry-mix process the mortar compound is mixed with water shortly before use; in case of pressured mechanical pointing the mix between mortar compound and water is made directly during the injection.

restore the structural connections between the masonry, however it prevent the ingress of moisture that can deteriorate the other materials.

In case of old and inactive cracking repair should be carried out using plastic and quite soft mortars, which are able to accommodate small movements and so to prevent the recurrence of cracking. When further movements are already foreseen in the future, cracks have to be sealed with more flexible material that is able to accommodate the expected movements.

When cracks interest only the mortar joints is possible to intervene with repointing. Instead, when cracks are passing through the masonry units patch repairs may be required. A different technique consists of the widening and undercutting of the cracks, which subsequently has to be cleaned from dust and filled with stiff mortar. Instead, when cracks are very fine, widening is not necessary and sealant material or fine mortar may be directly injected inside.

Patch repairs

Patch repair has to be carried out when cracking affect also masonry units. It consists in a local intervention of replacement of the damaged bricks or stones with new or recycled units. In case of brickwork usually all the external bricks belonging to the area surrounding the damage are replaced, while in case of masonry made of stones the replacement is limited only to the damaged stones. When the damage is limited, the cracked part of the stone are cut out and substitute with a new piece pinned in in place with dowels. While, when damage is more extensive may be necessary to complete substitute one or more blocks. In order to preserve the appearance it needs to use the same material of the original stone or one that match with it.

Usually patch repairs are used to restore the appearance of the bridge and to protect the underlying materials, but sometimes it may be carried out also to provide a structural repair. In this case the choice of material should consider even the physical and mechanical properties and replaced should be correctly bonded to the adjacent ones with grouting or pinning.

Fig 3.30 - Operation of patch repair of arch (above) and spandrel wall (below).

Application of consolidants and sealants

Masonry consolidants are chemical compounds, both organic[29] or inorganic[30], used to consolidate stones and masonry with the purpose of fortifies weathered stone and prevent further deterioration. There are a myriad of procedures[31] to applied to or to introduced into masonry consolidants, the choice of the method depend by specificities of each project. Prior to application it is necessary to perform extensive testing in order to determine the characteristic of masonry or stone - porosity, density and water absorption, modulus of rupture, compressive strength - and the effects produced by the consolidant application - mainly the abrasion resistance.

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[29] Organic consolidants encompasses a vast assortment of polymers, ranging from ethyl silicates, silicones (silanes, siloxenes, siliconates), and resins (epoxy resins, methacrylic resins, unsaturated polyester resins, and polyurethane resins). Further information can be found in [*Horie, 1987; Allen et al., 1992*]

[30] Inorganic consolidants include lime-water, puzzolan-cements, waterglasses, fluates, and baryte water. Further information can be found in [*Ashurst and Ashurst, 1988*].

[31] Most prevalent techniques of masonry consolidant are: brushing and spraying, pocket methods and bulk procedures. Further information can be found in [*Ashurst and Dimes, 1990*].

There is much debate about the use of consolidant and sealant to preserve masonry. Although it appears that they can be beneficial to the structure, incautious use can have the reverse effect and their long-term effects are less well known; therefore great care should be exercised in considering their use, particularly in old and historic structures [*McKibbins et al., 2006*]. An exhaustive annotated bibliography with a review of products has been provided by Collins[32].

Consolidants can be either applied to the surface of masonry or can be injected inside the masonry through a network of holes. They are low-viscosity liquids that fill the pore-spaces in permeable masonry in order to reduce the porosity and improve cohesiveness and strength. However their effects may not be always beneficial. When applied to the surface, consolidant penetrates for a limited depth, creating a hard superficial skin, which is less porous of the underlying masonry and that responds differently to thermal difference and moisture movements. Sometime it may accelerate deterioration processes, such as de-lamination or blistering. When injected, if the masonry is weak the drilling of the holes and the pressure of injection may damage it. Moreover, when the pressure of injection has to be limited due to conservation needs, penetrations may not be uniform, resulting incomplete.

Sealants and water repellents applications are widely diffused. They may be very effective in order to reduce the penetration of moisture through the sealed surface, for instances in case of driving rainfall. At the same time they may reduce the surface evaporation of moisture coming from other sources, such as rising damp, increasing the masonry saturation and creating moisture concentration, which may lead to deterioration processes. Their application on historical masonry has to be evaluated, because it can change the masonry appearance.

Cleaning

When the surface of masonry becomes dirty and discoloured, due to waterborne or airborne deposits or biological agents, it may be desirable to restore

[32] Masonry Consolidants - Annotated Bibliography; provided by Allison Collins in the introduction to Architectural Conservation course at the Roger Williams University, Bristol, Rhode Island.

the attractive original appearance of brick or stone masonry by cleaning. Several cleaning techniques and products are available. There are three main groups of masonry cleaning methods [*Mack and Grimmer, 2000*]:

- Water methods; soften the dirt or soiling material and rinse the deposits from the masonry surface;
- Chemical cleaners; react with dirt, soiling material or paint, allowing it to be rinsed off the masonry surface with water;
- Abrasive methods; mechanically remove the dirt, soiling material or paint (and, usually, some of the masonry surface) and may also be followed with a water rinse.

Particular attention has to be paid to the possible damages and undesirable effects of cleaning. It is better to use the gentlest method possible and to carry out trials in hidden parts of the structure. Literature about masonry cleaning is wide. In case of masonry arch railway bridges, cleaning is usually not so diffused such as in case of monumental buildings or bridges. Further information can be found in [*Ashurst, 1994*].

Other masonry repair treatments

In addition to the above, there are a variety of specialist treatments for preserving and repairing masonry, depending on its type and the cause of the damage, which would not normally be routinely considered for application to bridges. Although not suitable for routine use, specialist treatments might be worth considering in certain circumstances, for instance to treat limited areas of bridges with substantial historic value [*McKibbins et al., 2006*].

The restoration and strengthening of masonry material and structures is a very wide topic. A complete review of restoration and conservation of masonry materials and masonry structures can be found in [*Carbonara; Musso and Torsello, 2003*], new trends of consolidation can be found in [*Marmo, 2007*].

3.2.4 Strengthening of masonry arch bridges

Nowadays, consolidation techniques of masonry structures aim mainly at two targets [*Baruchello and Assenza, 1995; AA.VV. Manuale delle murature storiche, 2001*]:

- The improvement of mechanical characteristics, with particular reference to shear and compressive strength:
- An increment in masonry structure arrangement in order to achieve an almost rigid body response for masonry wall.

The first target is obtained by improving joint mechanical properties in terms of stiffness and strength and by substitution of inadequate blocks. This objective can also be reached by adding stiffer materials like composites into the joints, or by repointing technique. As far as the second target is regarded, different techniques can be adopted. Percentage of voids can be reduced in the masonry wall by injections that act both to decrease dimensions and amount of void spaces, while wall texture can be improved by inserting transversal reinforcements[33] through its thickness [*Lemme et al., 2002; Brignola et al., 2006*], as recently provided also by regulation[34]. The topic is huge, the thesis focus on the strengthening of masonry arch bridges.

Strengthening of masonry arch bridges include a number of different types of intervention on the bridge that aim on one hand to repair damages and problems that affect the structural behaviour, on the other hand to improve its structural behaviour, increasing its load-bearing capacity and the strength of its elements or improving its seismic behaviour.

Prior to carry out strengthening of bridges it is necessary:

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[33] “Diatoni”, bond stones, natural or artificial tie elements.

[34] Direttiva del Presidente del Consiglio dei Ministri, 9 febbraio 2011, “*Valutazione e riduzione del rischio sismico del patrimonio culturale con riferimento alle Norme Tecniche per le costruzioni di cui al decreto del Ministero delle infrastrutture e dei trasporti del 14 gennaio 2008*”, Supplemento Ordinario n. 54 alla GURI n. 47 del 26 febbraio 2011.

- To have a complete understanding of the structural behaviour;
- To detect damages and structural problems and to identify the causes;
- To evaluate the effects of the intervention to the structural behaviour of bridge.

The choice of the more appropriate intervention on bridges depends mainly by the problem that has to be solve. The comprehension of the structural behaviour of the bridge and the identification of the causes that have led to damages and problems are fundamental to avoid unnecessary or over-dimensioned intervention, which may be a risk rather then an opportunity for the conservation of the bridge. The effect of consolidation have to be carefully estimated. A peculiarity of masonry arch bridges is their capacity to adapt themselves to the changes that may occur in loads or supports through movements and cracking: their strength is due to their ability to articulate [*Mckibbins et al., 2006*]. Eventual modification of this aspect has to be evaluated in advance during the design of intervention of consolidation. Sometimes changes made in the original behaviour to deal with a failure may result in the occurring of new failure modes. Therefore, modelling and analysis have to be performed in order to study the behaviour of the bridge both before and after the intervention.

Repairs must be sympathetic to the structure, strengthening should not alter its working mode and materials have to be compatible with those existing. Interventions may use a variety of materials: concrete, steel, epoxy resins, soils, mortars, stones, and bricks. When designing interventions that insert new materials in historic structures their compatibility, whether chemical or physical, between each other and with the older materials, has to be evaluated both for the immediate future and for years to come, in order to ensure the purpose of stability and appearance. In fact incompatibility may lead to unexpected mechanical problems, such as local stresses, alteration of load paths, or over stiffening, or to chemical and physical alteration of materials.

Moreover not only the material itself must be durable, but the way in which new material and elements are applied in the structure must be durable too. Attention has to be paid to the design of the connections between materials and to the protection of the new materials and elements against environmental and accidental factors. The stresses in service conditions has to be limited, in order to guarantee admissible stress states for all of the materials involved. Strengthening and repair interventions should be most durable to the environment, cyclic loading and fatigue, in order to avoid, or at least reduce to the minimum, further interventions. This aspect has to be considered in the design of the intervention. Sometimes repair and strengthening options may be limited or their execution complicated due to the presence of previous works on the bridge. However, the response of the bridge to past works and their success is very useful to the evaluation of the potential effects and their chances of success of the new interventions.

The literature regarding intervention of repair and strengthening of masonry arch bridges if very wide: there are several different techniques that have been adopted. Several authors outlined the strengthening and repair techniques commonly used to deal with the frequent damages and structural problems of masonry arch bridges [*Page, 1993 and 1996; Bartuschka, 1995; Tilly, 2002; McKibbins et al., 2006; Proske and Van Gelder, 2009*] and guidances have been provided by Sustainable Bridges Reports[35] and by [*COST 345, 2006*] Cost. It is possible to divide the most common operations of reinforcement according to their purpose and to the elements of the bridge on which they act. In fact there are some main principles of strengthening masonry arch bridges. The main structural element of a masonry arch bridge is the arch, many strengthening methods aims to stabilise it and and improve its performance. The principles used to design strengthening interventions on masonry arch reflect the geometric considerations at the base of its behaviour[36]: the geometric shape should make the structure subjected predominantly to compressive forces and allows an appropriate path for the line of thrust[37], while deformations and tensile stress should be limited.

To make the arch behave as required is possible to act in different way, intervening directly to the arch or the other bridge elements:

[35] Sustainable Bridge Report, *Repair and strengthening of railway bridges - Guidelines,* Chapter 4.5.

[36] The behaviour of masonry arch has been described in the second part of the second two of the thesis.

[37] The line of thrust due to the applied loads has to be completely inside the arch profile in any cross-section.

- Increasing the arch thickness. When the line of thrust becomes tangent to the perimeter of the arch ring, hinges form in those points and the arch develop a kinematic mechanism that may lead to the collapse. The original stability of the arch may be restored increasing the thickness of the ring so that the line of thrust may remain completely inside the geometry of the arch. Moreover, increasing the arch ring thickness may also allows the load-bearing capacity of the bridge, because the enlarged cross section may be able to contain line of thrust of loads bigger than before.

- Increasing the effective thickness, applying tensile resistant materials at the intrados or at the extrados of the arch. In in this way it is possible to obtain an "increasing" of the thickness of the arch providing a better eccentricity of the thrust line. The line of thrust may go out from the arch geometry avoiding the formation of hinges and without provoking any kinematic mechanism, because the tensile forces that develop in the arch are transferred to the tensile resistant material, typically FRP[38] strips.

- Increasing the weight of the abutments, through intervention on the backfill. The weight increase provide an increase of the vertical force on the abutment that modify the path of the line of thrust, which will be more central at the base of the abutment, improving the stability.

- Increasing the dead loads over the arch, providing more compression in the arch cross-section and enlarging the horizontal thrust. This may have a beneficial effect to the capacity of the bridge to carry live loads, because it

[38] Fiber reinforced Polymer, it is a composite material made of a polymer matrix reinforced with fibres. The fibres are usually glass, carbon, or aramid, although other fibres such as paper or wood or asbestos have been sometimes used. The polymer is usually an epoxy, vinylester or polyester thermosetting plastic, and phenol formaldehyde resins are still in use. In strengthening of masonry structures FRP strips are glued on masonry surface in opportune points of the structure to provide tensile strength. Typical application are strengthening of arch and seismic retrofitting. Further information may be found in [*Focacci, 2008*].

increases the eccentricity of the line of thrust. However, it is necessary to assess the amount of load that can be added on the bridge and its foundations.

- Applying a uniformly distributed load on the arch adding a stiff continuous material, such as concrete, symmetrically above the arch to increase the capacity under live loads. In this way the line of thrust will have a parabolic shape, which can be contained in a easier way inside the arch respect to an irregular curve. A stronger uniform backfill can better distribute forces in the arch, reducing the local effects of concentrated loads. Moreover it increases compressive forces in the arch reducing the tensile forces, improving the arch stability.

- Changing the path of live loads. Typically it can be done applying relieving slab above the deck bridge in order to transmit live load to specific points: the abutments, obtaining an effect similar to one due to the increasing of their weight, or the crown, when an increase of horizontal thrust is needed. This principle is interesting because the intervention increase the strength of the bridge as the live loads increase.

- Repairing and restoring the arch which has suffered of damages, distortion or other defects, in order to re-establish the original strength and load-bearing capacity or to improve its mechanical properties, with reference to compressive and shear strength.

- Repairing and restoring other structural elements, such as spandrel walls., and repairing the deteriorated masonry, to restore the original integrity of the bridge.

There are several specific methods to strengthen masonry arch bridges according to the principles mentioned above. The most common strengthening and repair techniques are reported in the next paragraph, with a brief description of their purpose and execution and some consideration about the conservation needs and the impact to the service disruption. Attention is focused to repair and strengthening of the superstructure of bridge: arch, backfill and spandrels. Instead intervention on foundations are not reported in the thesis. Common interventions are:

- Interventions on the arch:
  - Arch distortion and deterioration remedial works, which includes four methods:
    - Steel ribs;
    - Support truss;
    - Sprayed concrete lining;
    - Pre-fabricated liners.
  - Arch injection and grouting;
  - Retro-reinforcement of arch barrel;
  - Stitching;
  - Reinforcement with composite materials.

- Interventions on the backfill:
  - Backfill replacement or reinforcement:
  - Concrete saddle;
  - Over slabbing.

- Interventions on spandrel:
  - Spandrel tie-bars and patress plates;
  - Stratford method (spandrel wall strengthening).

Several factors influence the choice of the appropriate intervention between the different strengthening or repairing methods, other than just the type of deterioration the bridge has experienced. A complete list of the recommended requirements that has be considered when selecting and designing the intervention has been provided by [*Garrity, 2001*][39]:

- Increase the load-carrying capacity;
- Improve the in-service performance;
- Improve the robustness and ductility of the existing construction;
- Accommodate a variety of existing defects and the highly variable nature of the existing masonry;
- Offer versatility in design to accommodate additional defects identified during intervention works;
- Create a safe working environment (some strengthening intervention may provoke a temporary instability of the bridge);
- Minimise disruption to traffic and services;
- Avoid over-strengthening or over-stiffening;
- Avoid significant increases of the self weight;
- Avoid changing the arch profile;
- Avoid creating localised highly stressed region that could lead to future damages;
- Avoid creating maintenance liabilities.
- Finally, intervention should be cost-effective within all the constrains identified above.

In any case it is important to point out that, first of all, interventions must respect the authenticity of the bridge: both the materials used and the appearance should remain as similar to the original bridge as possible. The preservation of the historical and cultural value has to be coupled to all the other structural and serviceability requirements.

[39] The list of the recommended requirements is wide and provide more details about each recommendation. Only some of the main points has been briefly reported here. For the complete list see the cited paper.

3.2.4.1 Interventions on the arch

Arch distortion and deterioration remedial works

When an arch suffers of distortion, misalignment or tilting of the regular shape, it is necessary to improve the arch integrity. The intrados of the arch barrel is lined it in order to stop any further movements and improving its load bearing capacity. There are four main techniques to deal with arch distortion which different on the base of the elements used to line it the arch intrados [*McKibbins et al., 2006*]:

- Steel ribs;
- Supportive truss;
- Sprayed concrete lining;
- Prefabricated liners.

The first two techniques are repair techniques, used just to repair distorted or deteriorated arches, while the second ones have also the purpose of increase the load bearing capacity of the bridge. The last one is a real strengthening measure, meaning that it increases the strength of the existing structure, while the third is usually designed to substitute the existing structure.

Such repairs have usually a great visual impact, which may be a problem in case of bridges subjected to statutory protection. Moreover the lining it of the intrados may reduce the clearance under the bridge. The durability may vary considerably: it depends if the lining it is designed as a temporary or permanent measure.

Steel ribs

The arch receive support from steel ribs, which are opportunely manufactured on the base of the arch shape, installed under the barrel. Concrete foundations have to be built on the ground level to support the ribs, paying attention to the existing ones. Ribs are positioned at a short distance from the barrel and connected with the insertion of timber wedges supports. This technique may be performed closing the bridge only for the short time necessary to insert the timber wedges between the steel ribs and the arch barrel.

Supportive truss

A series of truss are placed under the arch to support it. Truss are usually made in timber but sometimes they may be realised with other materials such as steel. Also in this case timber wedges are inserted between the truss and the barrel. This technique is suitable only for arches with enough vertical clearance, because it occupy a great space under the arch.

Prefabricated liners

When the structure exhibit significant signs of distortion or deterioration or when it has an inadequate structural capacity, new corrugated steel or precast concrete liners are installed beneath the existing arch structure to provide a secondary support mechanism. Any resultant gap between the liners and the arch has to be filled whit grout in order to provide a continuous support to the arch. Grouting may be necessary also after the intervention, in response to shrinkage or settlement which could occur. More rarely liners are made with GRP[40] panels or steel section ribs profiled on the base of the arch intrados shape or inserted in specific groove in the brickwork to reduce the impact on clearance.

Usually liners are designed as a “substitution” of the existing structure: they have to carry all the dead and live loads and the existing arch it is considered as a load itself. However sometimes, when the condition of the existing structure are not too severe, it is dimensioned as a strengthening measure to increase the load bearing capacity of the arch. Existing foundations have to be checked because subjected to increased load and may require interventions.In general the access to the bridge may be maintained during the realisation of works, because all the yard is under the structure.

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[40] Glass Reinforced Plastic is a fiber reinforced polymer made of a plastic matrix reinforced by fine fibers of glass.

The appearance of the bridge may be completely compromised by the liners, however the intervention may be carefully designed so that only the beneath part of the structure is altered while the vertical elevation remain equal to the original. Therefore it may be acceptable in the bridges in which the intrados of the arch is not usually visible. The durability of this kind of intervention is very high, however attention has to be paid to the possible future corrosion of the corrugated steel liners.

Fig 3.31 - Prefabricated liners.

Sprayed concrete lining

Structural sprayed concrete is applied to the arch barrel intrados to repair and strengthen arches. This techniques may be used also in case of arch suffering from deteriorated masonry and severe cracking. The layer of concrete gives an additional strength to the original arch and can be reinforced through connectors previously inserted in the arch intrados. The reinforcement is linked to the dowels prior to spray the concrete. In case of fiber reinforced concrete, traditional mesh reinforcement are not necessary. The lining can be extended between the springing of the arch or even to cover also the piers, including the foundations. Instead it is important to do not continue the lining up to the side elevation, because it may increase the possibility that water go inside between the concrete lining and the masonry, accelerating the deterioration processes. This techniques do not require the construction of complicated and expensive curved formwork. The concrete lining is usually designed to carry the live loads, anyway its thickness may be enlarged if is necessary to carry even the dead loads[41]. As in the previous interventions of lining, the appearance of the bridge is altered, even if the vertical elevation may remain almost equal to the original one. However this technique is not removable, therefore its application has to be carefully evaluated.

Fig 3.32 - Sprayed concrete lining, taken from [Page, 1996].

Arch injections and grouting

Arch grouting is performed to fill voids present within the arch barrel and consolidate the masonry. When coupled with pinning bars it may be used to re-establish the connections between the arch rings. Voids are usually due to water percolation, therefore this intervention has to be coupled with repair and maintenance of the waterproof system. Injections can be use also to repair cracking in the arch barrel in addition to cross stitching.

There are different types of grouting, the most common is pressure grouting. A grid of holes is drilled in the structure to insert pipes for injections. Holes have to be realised avoiding vibrations and damages to the barrel. The grid pattern is

[41] Becoming a "substitutive" intervention, such as the insertions of prefabricated liners.

designed to reach the voids and on the base of the brickwork bond and of the thickness of the underlying layers. The depth of the holes should not damage the waterproof membrane. When the holes are ready and clean grout is injected in the arch starting from the lowest point and going up progressively. Grout have to be injected until the hole is full, then it has to be temporary sealed. In case of large voids more holes may be interested, so adjacent holes have to be provisionally closed to avoid the spillage of grout. When injection have been completed holes have to be drilled again to insert pinning bars.

To repair intrados cracking cracks are cross stitched with steel bars prior to injection. Cracks have to be properly clean and sealed with mortar or other sealant, waiting that reach enough strength to resist to pressure of grouting. When ready, cracks should completely filled with grout.

Such repairs are not visible, pinning and bars are inserted in the brickwork and covered with mortar, the visual impact is minimised. However, the grout used for the injections should have characteristics as similar as possible to the existing mortar. The compatibility of the grout with the original materials is of fundamental importance to ensure the conservation. Mortar properties must be evaluated in terms of physical and chemical compatibility with existing masonry structure, with particular attention to water percentage and cement type. Saturation and penetration efficiency, and then the application success, are deeply influenced by these choices. In order to be effective grouting should completely fill the voids, but attention has to be paid to the pressure of injection to avoid subsequently damages to spandrel walls. Pinning and bars must be of the minimum size possible to avoid over-stiffening that may modify the original structural behaviour. However, injections and grouting include a great uncertainty subject to the effectiveness and any long-term effects.

The traffic has to be avoid during and even after the intervention. Premature passages of vehicles may lead to cracking the repair, which need time to reach the required strength.

Fig 3.33 - Operation of arch grouting (taken from McKibbins et al., 2006).

Retro-reinforcement of arch barrel

Retro reinforcement techniques consists of the installation of additional structural reinforcement to the arch barrel in order to increase the structural capacity of the bridge, providing a resistance to the formation of hinges mechanisms or cracking. The technique allow to provide an increase of the structural capacity without reducing the clearance or altering the appearance. There are two types of retro-reinforcement:

- Internal reinforcement, also called Archtec technique, which has been applied many times in UK, US and Australia since 1998 [*Brookes and Mullet 2004; Mullet et al., 2006*]. Stainless steel reinforcements bars are inserted into holes cored through the arch barrel from the crown to the springing and inserted from above the arch or from below the arch, usually in case of multi-span bridge. Steel bars are placed in an approximate tangent position in the area in which usually hinges develop, such as at quarter of span. Reinforcement reduces intrados strains, preventing from the loosening of masonry units under live loads. In addition the bars positioned across transverse cracks hinder to the opening and closing of cracks, reducing damages due to cycling loads and improving the service life.

- Surface reinforcement, also called near-surface reinforcement, is a quite new technique, originally developed for the strengthening of masonry buildings. Longitudinal and transversal stainless steel reinforcing bars are grouted into pre-drilled holes or pre-sawn grooves made in the surface of the arch in the areas subjected to tensile stresses due to the external loads or settlement. Usually surface reinforcement is made with a grid of bars with small diameter. Once reinforcement have been inserted holes and grooves are grouted. Surface reinforcement helps improve lateral load distribution and increases the transverse flexural strength of the arch. Examples of surface reinforcement can be found in [*Woodward, 1997*].

There are a series of proprietary systems of retro-reinforcement techniques, mainly based on the two main type of reinforcements mentioned above. Cost of these methods are usually less expensive and less disruptive than others interventions with the same purposes, such as saddling, sprayed concrete or pre-fabricated liners. Also the execution of works is easier. Moreover generally these techniques may be carried out without completely closing the bridge, depending by the wide of the bridge. Under bridge traffic may be subjected to minor disruption. In case of internal reinforcement attention has to be paid to the eventual presence of services inside the bridge, which may be damaged. In case of surface reinforcement services are not affected, but attention has to be paid the the finishing, in order to maintain the bridge appearance. There are not yet enough information and experimental data about the durability of this intervention. Compatibility of the grout material with the existing ones has to be verified.

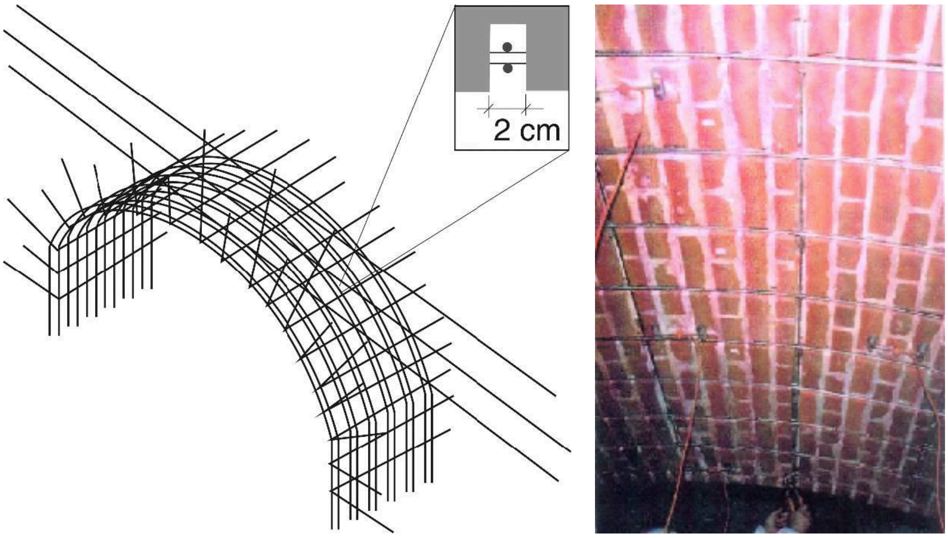
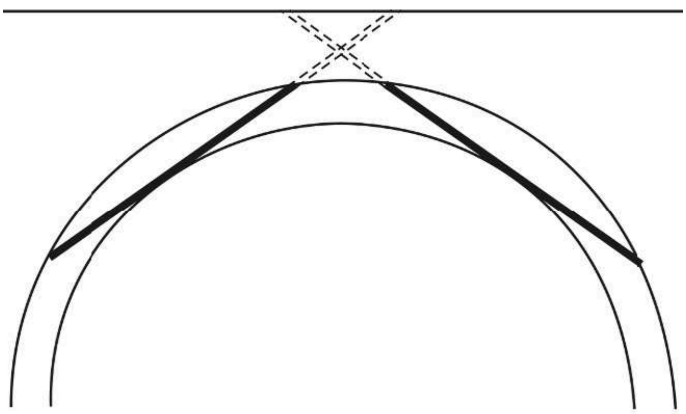


Fig 3.34 - Surface reinforcement concept according to [Woodward, 1997] (letf) and example of application of the reinforcement (right).



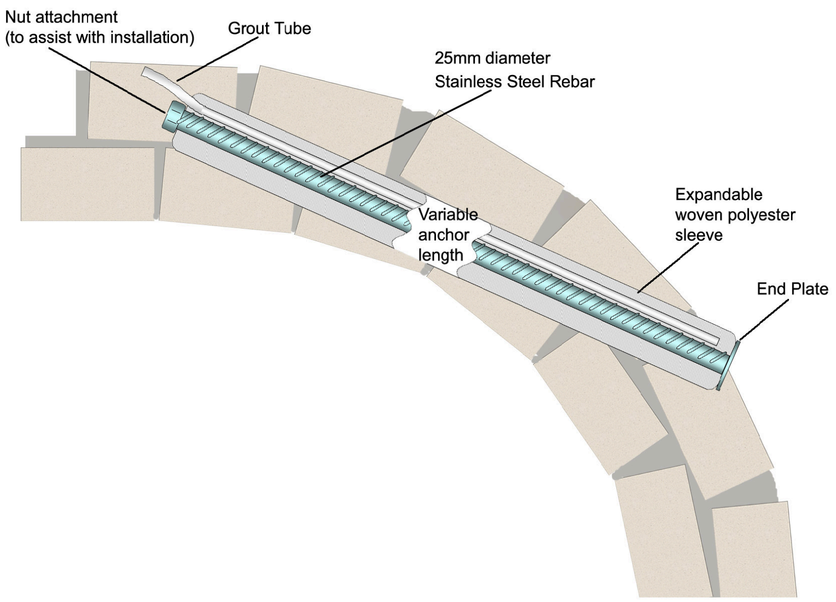


Fig 3.35 - Typical position of bars in internal reinforcement (above) and the Cintec Anchor, taken from [Mullet et al., 2006] (below).

Stiching

Stitching, also called anchoring, is an intervention that aims to re-establish the mechanical connections inside the arch in order to restore the structural integrity of the bridge. This type of intervention restores or increases the shear transfer and to the transversal and longitudinal continuity. There are two main techniques, related to the different damages that have to be repaired or prevented:

- Transversal anchoring, also called tie bars, is used in case of longitudinal cracking of the arch barrel and in case of separation between arch and spandrel walls or other spandrel damages. The installation of transversal reinforcement elements into the arch has been proposed by [*Olivera and Lourenco, 2004*] and above the extrados by [*Falconer, 1999*]. Oversized holes are drilled through the full width of the bridge and profile bars are inserted inside and then grouted under low pressure. As any intervention on historical masonry that implies the addition of grout attention has to be paid to compatibility. Sealant may be used to prevent problems due to corrosion. Finally the tie rods are secured to steel anchorage plates at each side of the arch. To preserve the bridge appearance steel plates may be set in the hole and then covered with grout or stone.

- Through ring stitching, also called radial pinning, in case of ring separation. This techniques aims to re-establish the mechanical connections between arch rings and to prevent from further separations through the insertion of stainless steel bars or galvanised dowels into the arch barrel intrados. The modality of execution is similar to the transversal anchoring. In this case rods are inserted in a grid of holes drilled with predetermined angles from the intrados through the ring for a pre-defined depth into backfill and then filled with grout. Rods are finally secured to steel anchorage plates on the visible end at intrados.

Transversal anchoring and radial pinning improve the stiffness and the elastic properties of the arch barrel and limit movement. The radial pins restore the full thickness of the ring in the case of a multiple layer ring. The transversal anchoring restore the full width of the bridge in case of longitudinal cracking. Stitching restores the integrity of the bridge and improve the transversal behaviour. When damages affect seriously the spandrels in can be coupled with other specific treatments[42]. Disruption of traffic on the bridge is usually not necessary, however vibration due to live loads should be avoided. Traffic under the bridge may be subjected to slight disruption. Bridge appearance may be maintained if steel plates covered with mortar or stones and hidden. Costs are usually lower respect to other interventions, however the drilling of the full width of the arch can be difficult.

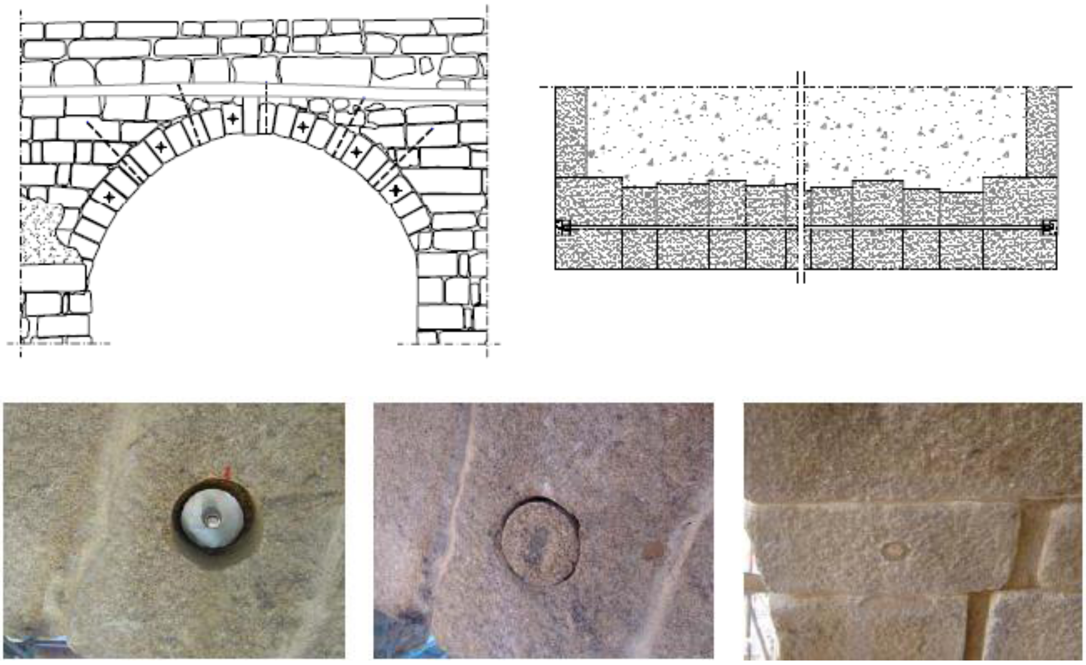


Fig 3.36 - Anchoring and underpinning strengthening, scheme (left) and cross section (right) and final appearance of plugged anchoring covered by stone cap (below), taken from [Oliveira and Lourenco, 2004]

Reinforcement with composite materials

Besides the application of classical steel reinforcement, recently the use of composite materials had widespread in conservation filed. The use of strips of FRP [43] and/or CFRP[44] has many advantages respect to other reinforcement techniques:

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[42] Spandrel tie bars/patress plates and/or spandrel strengthening (Stratford method)., which will be described in detail in the following paragraphs.

[43] Fiber Reinforced Polymers (see note 19).

[44] Carbon-Fiber Reinforced Polymer, an FRP material made with carbon or aramid fibers.

- Exclusion of corrosion;
- Do not add mass to the structure;
- Do not reduce the available space;
- Do not alter the structural behaviour;
- Are removable, although their removal may damage the surface on which they are applied.

Such techniques have been first applied to classical steel reinforced concrete elements. More recently its application has been introduced in the conservation of masonry buildings in the form of surface reinforcement for wall panels, bell towers, arches and vaults [*Di Tommaso and Focacci, 2001*]. Exhaustive information about the strengthening of masonry structures can be found in [*Focacci, 2008*]. Studies and examples of their use in the strengthening of masonry arches bridges have been provided by many authors [*Modena et al., 2004; Melbourne and Tomor, 2004; Hodgson, 2003; De Lorenzis and Nanni, 2004; Foraboschi, 2004; Borri et al., 2002; Drosopoulos et al., 2007*][45].

The typical use of FRP or CFRP in the strengthening of masonry arches consists of the application of continuous sheets across the surface of the arch at the intrados and/or at the extrados. The FRP reinforcement transfers the tension force across the crack, avoiding the opening of cracks and the formation of plastic hinges. In this way the kinematic mechanisms of collapses are avoided or will activate in case of bigger loading. In fact the tensile resistant material applied on the arch surface allow bigger eccentricity the line of thrust. The application at the intrados or at the extrados depends by the mechanism that has to be prevented: strips should be applied at the opposite side respect to the one in which hinges occur. When FRP is applied to the extrados, the line of thrust can go outside the lower hedge of the arch profile, while when it is applied to the intrados it can be go outside the upper hedge of the arch profile. However their application at the extrados may be difficult, because implies the complete removal of the backfill, with a long disruption of the

[45] The literature concerning the use of FRP and CFRP in the strengthening of masonry arch and their application to the strengthening of masonry arch bridges is wide, here only few references have been mentioned.

traffic, therefore reinforcement are more frequently applied on the intrados. The maximum load-bearing capacity of the arch is reached when the reinforcement prevents from fourth or five hinges mechanisms and failures occur due to crushing of masonry, sliding or debonding between FRP and masonry surface or in case of FRP rupture. These failure modes dependent by strength of the constituent materials and by their interaction at the local level [*Valuzzi et al., 2001*].

The application of composite materials surface reinforcement can significantly increase the load-bearing capacity of masonry arch bridges, preventing from kinematic collapse mechanisms and reducing the horizontal thrusts. Moreover the ultimate bridge behaviour after intervention is more predictable. Attention as to be paid to the bond between FRP or CFRP and the masonry surface. Their application at the arch intrados is easy and fast, does not require particular equipment and the bridge may remain open, as well as in case of its eventual removal. Instead the application on the extrados require intensive works and the complete removal of the backfill.

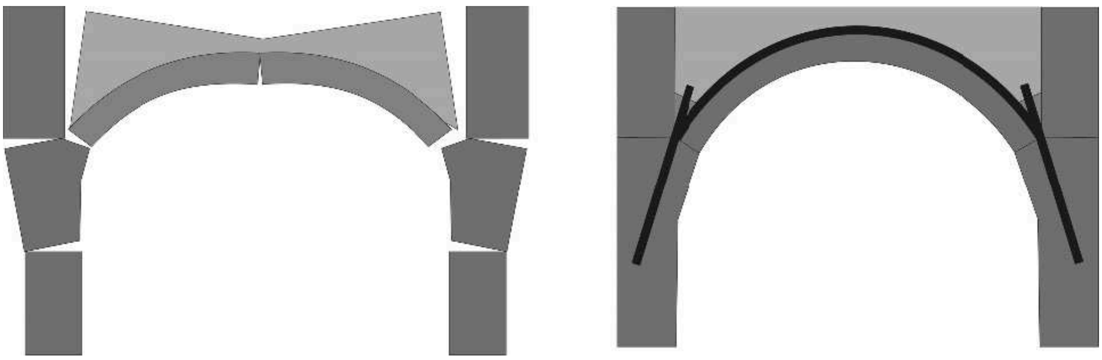

Fig 3.37 - Strengthening of masonry arch bridge with FRP according to [Borri et al., 2001]

3.2.4.2 Interventions on the backfill

Backfill replacement or reinforcement

Backfilling over the arch help the global structural behaviour of the bridge: it better distribute live loads and provide a stabilising dead load. Reinforcement interventions on backfill are carried out to improve its capacity to distribute load, and its strength. Instead replacement are usually carried out when there is the necessity to reduce dead loads, in particular in case of shallow arches. In this case filling is replaced with lower density materials.

When performing this intervention particular attention have to be paid to the removal of filling, which may completely modify the structural behaviour of the bridge. Any intervention on backfill should include the application of a waterproof membrane.

Concrete saddle

The technique consists in the replacement of existing backfill material with a reinforced concrete saddle above the arch. The purpose is increase the stability of the ridge creating a composite structure made of the existing masonry arch and the new concrete saddle. This intervention may be also carried out in order to create or upgrade the waterproof system. It is a common techniques that may be found in many bridges that have showed any signs of distress. It is often coupled with spandrel walls repair or strengthening.

During the construction of the concrete saddle, the structural fill between the spandrel walls and the arch barrel has to be completely excavated. The excavation should be done in a symmetric way in order to minimise the risk of movements or collapse. The void obtained is filled with reinforced concrete on the arch barrel extrados. The saddle may have an uniform thickness above the arch or a variable one, up to completely replace the backfill. Sometimes spandrel walls and extrados are anchored with structural ties, such as stainless steel bars, to ensure a proper transmission of forces between the existing structure and the saddle. In some case spandrel walls may need to be dismantled and rebuilt to prevent damage or collapse.

The arch may need to be supported by centering during the excavating and the construction of the saddle. Attention has to be paid to the arch stability when it is un-loaded during the fill removal. Drainage system are usually designed and installed in the saddle.

The new arch formed by the saddle should be designed to work together with the existing structure as an increase of arch thickness, which better distribute the loads too. In this case concrete saddle is connected to the existing arch and has a weaker strength and a smaller reinforcement and. Instead the new arch may be designed to replace the old one. In this case it is not connected and the old arch ring loose its structural function, with negative effect due to the lack of stress that may lead to the loos of masonry units. Moreover the new arch has to be connected to the foundations, through abutment and/or piers, in order to transfer the loads. If the damages of arch were due to support movements this intervention may increase the problem. It may be necessary to strengthen abutments and foundation too in order to carry the increased load due to saddle. Attention has to be paid to transversal tensile in the saddle, in particular in case of width bridges.

On one hand saddling provide an intervention that may stabilise many damages at once. On other hand it require extensive works and need a complete and prolonged closure of the bridge, with disruption of both traffic and services. Costs are usually very high, especially in large bridges. The intervention do not modify the appearance of the bridge, however when saddle is designed to replace the existing arch it completely change the structural behaviour and affect the authenticity of the bridge. Moreover its removal is almost impossible.

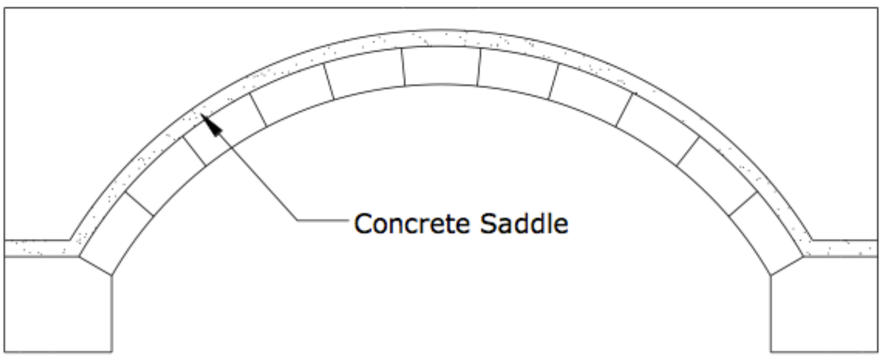

Fig 3.38 - Uniform thickness concrete saddle

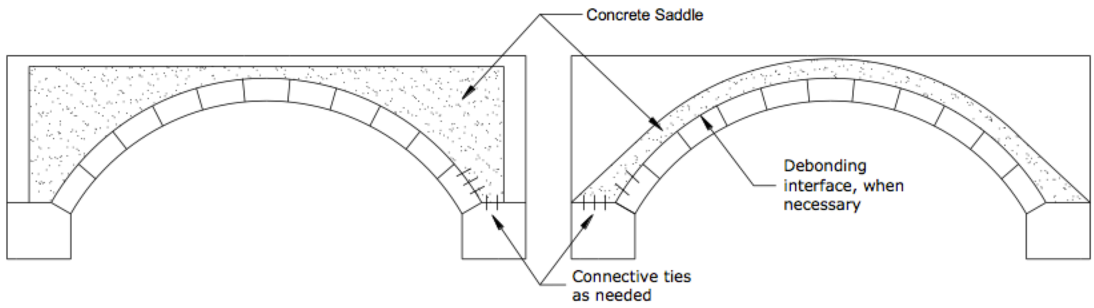

Fig 3.39 - Varying cross section concrete saddle

Fig 3.40 - Construction of concrete saddle.

Over slabbing

Intervention of over slabbing, also called relieving slabs, consists of the installation of an horizontal reinforced concrete slab over the filling and extended on the abutments. The purpose is to give a better distribution of loads on the arch and to modify the path of the line of thrust in order to improve the transfer of loads to the abutments. Relieving slabs reduce the lateral pressure on spandrel walls and provide a waterproof system. A partial excavation of the filling, realised symmetrically respect to the crown, has to be made to accommodate the slabs. The existing structure continues carrying the dead loads - the filing between arch extrados and slab intrados - while the slab transmit its own weight and live loads to the abutments. The forces transferred by the slabs to the abutment are just vertical, without horizontal components. The partial removal of filling and the construction of the slab implies the closure of the traffic on the bridge.

When relieving slabs are built inside the bridge profile the appearance of the bridge is maintained. In some case slabbing may be used to enlarge the width of the bridge, with lateral span. This type of intervention completely modify the bridge appearance. The applicability of this system to the widening of historical bridges and possible alternative solutions have been discussed by [*Troyano, 2006*].

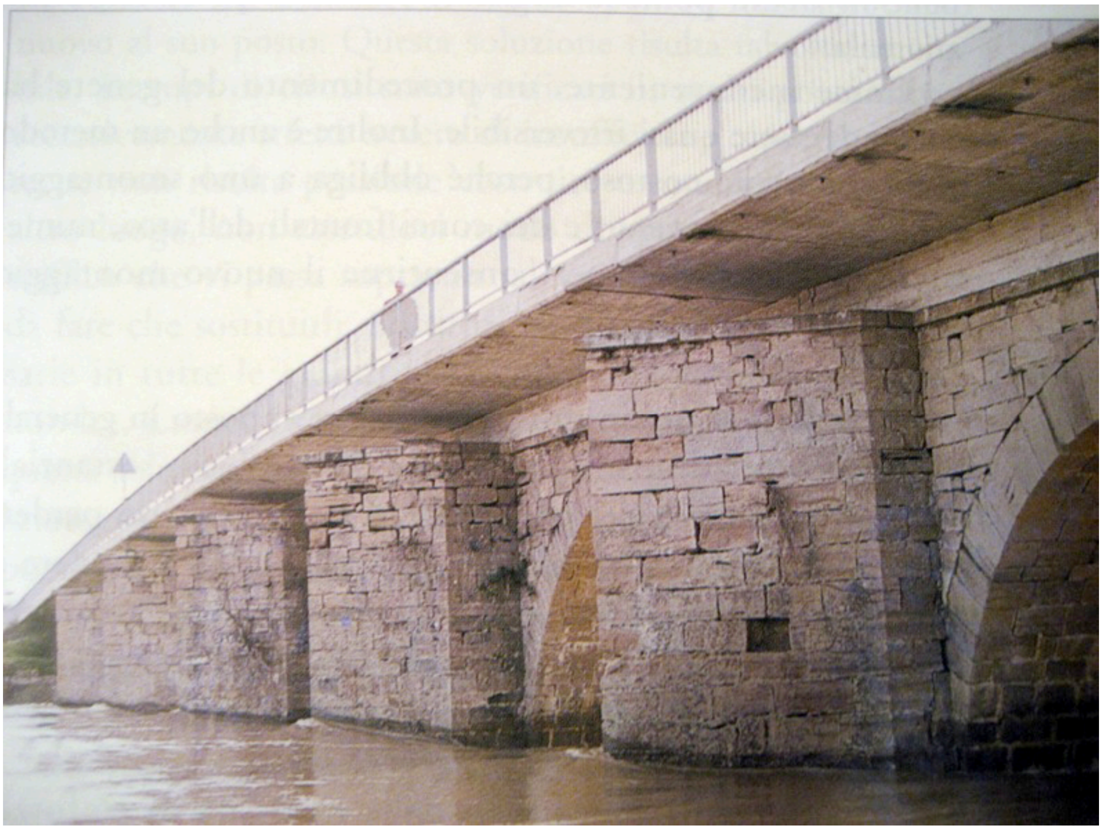

fig 3.41 - Widening of the bridge with over slabbing and lateral span, taken from (Troyano, 2006).

3.2.4.3 Interventions on spandrel

Spandrel tie-bars and pattress plates

The insertion of tie bars to connect the spandrel walls aims to avoid further outward movements and to reduce the excessive lateral forces due to the pressure of the filling as a consequence of traffic, obtaining an improvement of the lateral stability. It is an intervention of repairing the typical damages affecting the spandrel: bulging, tilting and sliding. The intervention is similar to the transversal anchoring of arches, but in this case tie bars are inserted above the arch and pass through the filling to restrain the spandrel walls. Bored holes are drilled between and perpendicular to the spandrel walls and across the barrel. Reinforcing tie bars are inserted into the holes and fixed with pattress plates and bolts at each end of the bars. Tightening the bolts the bars reach the prescribed tension.

Stratford method

The "Stratford method" is a repair technique for spandrel walls suffering of tilting, sliding or bulging. It has the same purpose of the tie bars/pattress plates insertion, but this method involves the insertion of reinforced concrete elements. A trench is excavated directly behind and parallels to the spandrel and extended downwards to the arch. The trench is filled with reinforced concrete and placed on the arch intrados in the point which it is connected, through structural ties, with the spandrel. The idea is to create a composite spandrel having an high mass positioned backward respect to the spandrel face, in order to avoid or reduce the outward movements and improve the lateral stability. It may be coupled with tie bars and pattress plates to improve the transversal connections.

The tie bars/pattress plates method is less expensive and disruptive then the stratford method. However some problems may occur in the drilling of the bridge, in case of encountering hard concrete infill or steel obstruction. Attention has to be paid to the presence of services. The pattress plates on the bridge elevation may alter the bridge appearance, but design of details may reduce their visual impact. On the other hand Stratfor method is usually not visible, but pattress paltes may be neede too. Depending on the width of the bridge, slight disruption of traffic may occur.

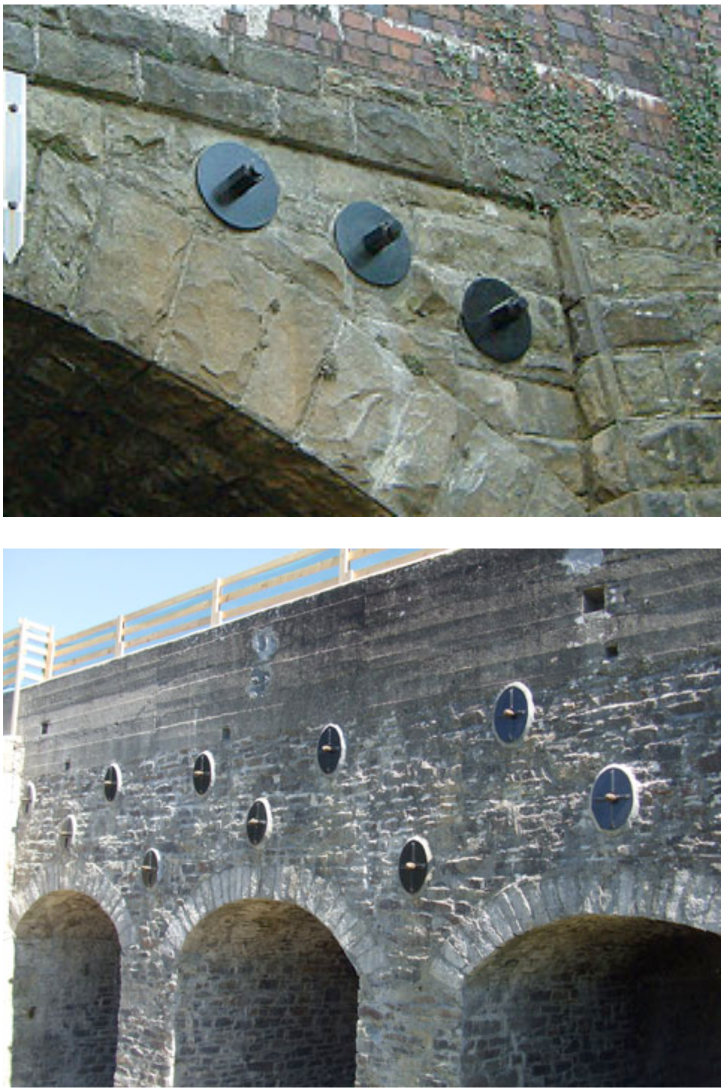

Fig 3.43 - Tie bars and pattress plates inserted in spandrel walls.

Conclusion of the third section

Some considerations about the different approaches to the strengthening of masonry arch bridges are here provided. Attention is focused to the impact of strengthening to the conservation of the bridge: if on one hand conservation is assured by the keeping of structural performance, on the other it should guarantee, or reduce as max as possible, the loss of architectural and cultural value. In this view, the respect of the original structural function can be considered as an important requirement.

Moreover, with reference to the second section of the thesis regarding modelling and analysis, some further consideration about the capability of the different models and methods to deal with the deterioration and the strengthening of masonry arch bridge are provided. In fact the possibility of analyse the effects of damages and strengthening is of fundamental importance to evaluate the improvement provided by the interventions.

The capability of model to represent damages and strengthening is reported in the next table:

Capability of model to take into account defect and strengthening		
Model	Damages	Strengthening
MEXE and other modern rules for load-bearing assessment	• Can take into account only few typologies of damages	• No specific consideration; • Does not take into account backfill and spandrel
Castigliano's method and its evolutions	• Can take into account only few typologies of damages	• No specific consideration; • Does not take into account backfill and spandrel

Capability of model to take into account defect and strengthening		
Methods based on limit analysis	• Can take into account only damages regarding the cinematic mechanisms of collapse	• No specific consideration; • May take into account backfill, but doe not take into account spandrel
Finite Elements method and Discrete Elements method	• Can take into account any kind of damages or deterioration	• Can take into account the effects of strengthening

Table 3.22

The main advantages and weak point are summarised in the next tables. Moreover the impact to the conservation of the different techniques are provided, indicating the possible problems.

Interventions on the arch			
Technique	Advantages	Weak points	Conservation
Steel ribs or supportive truss	• Ease of installation • May increase the live loads capacity, however they are mainly repair measures	• Potential bond problems • Appearance and clearance	• Alter the aesthetic appearance
Prefabricated liners	• If hidden, does not change to appearance • Increase the live loads capacity	• Traffic disruption during construction • Relative cost • Appearance and clearance	• Alter the aesthetic appearance of the intrados, and if is not hidden also the lateral view • Modifies the original structural form

Interventions on the arch			
Sprayed concrete lining	• Increase the live loads capacity • Does not require the construction of expensive curved form-works	• Appearance and clearance • If water goes inside between the concrete liner and the arch barrel may increase the deterioration of masonry	• Alter the aesthetic appearance • Not removable • Modifies the original structural form
Arch injection	• Established method • Simple	• Traffic disruption during construction • Needs further inspection/ testing to confirm repair effectiveness	• Not removable • Compatibility of materials
Retro reinforcement of arch barrel	• Usually less costly and disruptive than other methods • Ease of installation • Repair is hidden	• There are not yet enough information and experimental data about the durability of this intervention	• Not removable
Stitching	• Established method • Simple • Hidden if cover with mortar	• Traffic disruption during construction • Drilling may be difficult	• Not removable

Interventions on the arch			
Reinforcement with composite materials	• Exclusion of corrosion; • Does not add mass to the structure; • Does not reduce the available space; • Does not alter the structural behaviour;	• The application at the extrados may be difficult • Does not prevent form crushing of masonry • Bond problem	• May alter the aesthetic appearance of the intrados • Their removal may damage the surface on which they are applied

Table 3.23

Interventions on the backfill			
Technique	Advantages	Weak points	Conservation
Replacement and reinforcement	• No change in appearance • Better distribution of loads • Relative cost	• Traffic disruption during construction • Does not increase the structure life expectancy	• Not removable • The original filling material may have archeological significance
Concrete saddle	• No change in appearance • Upgrade waterproof system	• Traffic disruption during construction • Relative costs	• Not removable • The original filling material may have archeological significance • Modifies the original structural form

Interventions on the backfill			
Over slabbing	• Better distribution of loads • Upgrade waterproof system	• Traffic disruption during construction • Relative costs	• Alter the appearance, however its visual impact may be minimised if slab are built inside the bridge profile • ....

Table 3.24

Interventions on the spandrel			
Technique	Advantages	Weak points	Conservation
Tie bars and platters plates	• Simple • Minimise traffic disruption • Relative cost	• Localised high stresses to spandrel at plates • Possible water paths into bridge • Negligible effect of service behaviour	• May alter the aesthetic appearance
Stratford method	• Ease of installation • Does not need specialists • can accommodate parapet strengthening	• Traffic disruption during construction • Relative cost • Not widely used • The change in structural behaviour of the spandrel wall should be investigated	• May alter the aesthetic appearance • Not removable

Table 3.25

Concluding, some remarks may be pointed out. The correct identification of damages and deterioration is of fundamental importance. Repair interventions, when necessary, should be carried out as soon as possible. Preventive and planned maintenance has to be carried out constantly to avoid the occurrence of problems that may lead to severe damages or to very invasive repair and strengthening, reducing the interruption of service. The length of works is a relevant parameter in the choice of strengthening: when possible interventions that do not imply a closing of the bridge have to chosen.

Intervention that not modify the original structural function of the bridge should be preferred, as well as interventions that do not alter the aesthetic appearance. The aesthetic impact of the intervention depends by the location of the bridge, therefore it has to be evaluated in each case. Durability of the intervention and of the material utilised has to be considered, as well as the compatibility of new and old materials. As in all the restoration works, the possibility of removability and reversibility of the intervention should be assured. However, in the case of masonry arch rail bridge the reaching of the needed performance is essential for the conservation, which can be guaranteed only by the possibility to remain in service. Some compromises have to be accepted.

In this view interventions aimed to an increase of the mechanical properties of the backfill material may be an excellent solution. Such type of intervention does not alter aesthetically and conceptually the bridge. In fact, even if not removable or reversible, the increasing of the stiffness of the original filling material does not induce significant changes in the structural and material configuration of the bridge. At the same time the effect to the global behaviour is positive, both under service and ultimate loads. Strengthening aimed to increase the stiffness of the backfill material may be an innovative solution to provide an increase of the load bearing capacity of the bridge. An increasing of the backfill stiffness may be realised through injections of consolidants directly in the backfill material from holes drilled in the arch barrel and in the spandrel walls. This technique may be be realised without interrupt the service, or interrupting for a short period, the train traffic. However some difficulties may be found in the realisation of this intervention. The effective distribution of consolidants in the backfill material has to be verified and the drilling may be difficult. Moreover the effectiveness of the intervention respect to the increasing of the mechanical properties of the backfill material has to be verified. Modelling for the simulation of the interventions may be realised with F.E. Models, however their reliability should be assessed by tests.

Case Study

Multi-scale analysis of the Venice Trans-Lagoon Bridge

In the case of study a procedure for structural analysis based on a multi-scale approach is proposed. Different types of models and with different levels of detail are analysed in order to evaluate their applicability and reliability. The target is to define guidelines for the structural analysis of masonry arch bridges. The methodology is applied on a case study: the Venice Trans-Lagoon masonry arch railway bridge.

Most common approach to model and analysis of masonry arch bridge are the Finite Element Method (FEM) analysis and the limit analysis. A wide literature about methods of model of masonry arch bridge is available, the issue has been discussed in the second section of the thesis. Main advantages and problems of the two different methods are summed up in the following table:

Method of modelling		Opportunities	Weakness
FEM	Common	Could give exhaustive results	Difficult to characterise masonry
	2D	Low computational efforts	Do not take into account transversal behaviour
	3D	Longitudinal and transversal behaviour	High computational efforts
Limit analysis		Reliable to find trust line	Not exhaustive results

Table 4.2 - Dimensions of the Venice Trans-Lagoon Bridge

The procedure proposed consists of analyses performed with the two methods on a series of models made with different details. FEM analysis are performed on models representing the whole bridge, including structural and non-structural elements, to evaluate the global behaviour of the bridge under service loads. The analysis have been performed through a commercial software, Straus7. Limit analysis are performed on beam elements, which take into account also the height and the mechanical properties of the backfill, to evaluate the mechanism of collapse and the load multiplier. The analysis have been performed using a commercial program, Limit State Ring, specific for the limit analysis of masonry arch bridge.

Models represent the bridge at different scales with the aim to analyse both the local and the global behaviour. This means that the bridge had been subdivided in its components: single arch and sequences of arches.

Static analysis and natural frequencies are carried on FE models to find the worst combination of loads and the modes of vibration. The purpose is to identify the areas of the bridge subjected to maximum values of stress and displacement. That area could be damaged: future intervention of strengthening have to be designed in order to increase the resistance of these areas. Loads are the ones proposed by Italian technical regulation[1], coinciding with the ones proposed by European regulations. FE models are both two-dimensional and three-dimensional. FE two-dimensional models have been used to try the different combinations of loads, restrains and material properties. FE Three-dimensional models have been used to investigate both longitudinal behaviour, applying the loads on different spans, and the transversal behaviour, applying the loads on the different sides of the bridge. They have been used also to evaluate the modes of vibration.

Limit analyses are carried on beam elements. Limit analysis is very useful to identify the load failure factor and the line of thrust due to loads applied. Mechanisms of collapses happen when the line of thrust is not included inside the arch. Hinges appears in the contact points between the line of thrust and the upper or lower arch boundary. The arch begins to behave like a cinematic chain of rigid blocks. Mechanisms are usually of two type: five-hinges mechanism or four-hinges mechanism. The identification of cinematic mechanisms and corresponding positions of hinges are very useful for the design of strengthening.

The role of backfilling has been studied. In particular, the effects due to modifications of the mechanical properties of the filling material respect to the global behaviour of the bridge have been evaluated. The idea is to provide a strengthening of the backfill material in order to increase the behaviour of the bridge in service

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[1] Italian Railway Code, n° I/SC/PS-OM/2298, provided in 1995.

condition without altering the structural configuration and the external appearance of the bridge. A parametric analysis has been performed on a two-dimensional model varying the mechanical properties of the backfill: the single arch model used in the multi-scale analysis and a model in which the backfill is modelled by means of spring elements.

Beside the multi-scale analysis, performed to evaluate the behaviour of the Venice Trans-Lagoon Bridge in service conditions, some furthermore analyses have been performed with the purpose of evaluate improvement of this procedure.

The capability of the models to represent the real structural configuration of the bridge is evaluated. In fact FEM may provide exhaustive results but a weak point of this type of modelling is the difficulty in properly characterising the masonry material. As previously discussed, masonry is an heterogeneous material obtained by the juxtaposition of natural or artificial blocks which interposition or not of mortar joints. The arrangements of blocks plays a fundamental role in the behaviour of a masonry structure. For this reason it is important to evaluate the sensitivity of the model respect to the texture of the masonry. An homogenisation procedure has been adopted to take into account the micro-structure of the masonry in order to obtain an equivalent continuum [Cecchi et al., 2005]. At micro-scale level the masonry texture of the barrel vault has been modelled in order to obtain the corresponding equivalent orthotropic continua. Then the mechanical properties obtained have been applied at the meso-scale on a full three-dimensional Finite Element model of the bridge.

Moreover a study has been carried out about the influence of the presence of external stone arch rings on the global behaviour of the bridge. In fact, during the nineteenth century, the masonry arch bridge were built with typical structural form [Torre, 2003]: sometimes the whole arch barrel was completely realised in brick masonry, such as in the case of the Venice Trans-Lagoon Bridge, however in many cases the barrel vault was usually made of brick masonry while the external arch rings were made by stone voussoirs. The role plaid by the external arch rings has been evaluated. Here an homogenisation procedure has been adopted to model an arch made of blocks of Istrian stone, the typical stone utilised in the historical Venetian architecture and even in some part of the Venice Trans-Lagoon Bridge, such as the piles. A comparative study is performed. Models represent the two possibilities: the real configuration of the bridge, in which the barrel vault is completely made of brick masonry, and a configuration in which the external arch rings are made of stone voussoirs.

Therefore, the case of study consists in two parts:

The first part provide a complete study regarding the Venice Trans-Lagoon Bridge: the history, the constructive characteristics and the materials, the original design and the actual configuration, the state of condition. It has to be remarked that the complete knowledge of a bridge is first fundamental step in order to perform analysis and assess the structural behaviour so to design correct strengthening.

The second part provides the analyses carried out on the bridge. It consists of two parts:

- The multi scale analysis regarding the behaviour of the bridge in service condition. Moreover an evaluation of the influence of backfill to the global behaviour of the bridge is presented. The purpose is to evaluate the influence of strengthening interventions applied to the backfill.

- A more fine characterisation of the masonry material by means of an homogenisation procedure. Analysis are performed on a three-dimensional single arch model. Moreover a study about the effect of the presence of external stone arch rings to the global behaviour of masonry arch bridges is presented.

Part 1

The Venice Trans-Lagoon Bridge

The Venice trans-Lagoon rail bridge was built in 1846, has a total length of 3603 meters and consists of 220 masonry arches of 10 metres of span each. It was the result of the work of three engineers that elaborated three different projects, based on a common idea. The bridge was subjected to several interventions during its life and currently consists of three bridges coupled. The bridge object of this research is the first one built, the historical masonry arch railway bridge.

A brief history of the bridge and its geometrical description, its technology and the structural materials are given. The study of historical and material characteristics of the building and the modification which have been subjected to during its life is the first fundamental step in the modelling and analysis of historical structures [*Siviero et al., 1997*]. In fact only through a complete knowledge of the structure is possible to represent the structure with appropriate models.

Fig. 4.1 Venice and the Trans-Lagoon Bridge.

The construction of the bridge

The first railroad was built in England in 1825[2]. Starting from this date, the railway had been a great development firstly in Europe[3] and then in the other continents[4]. Initially railway net was composed of short railway sections, then the railroad net spreads rapidly during the nineteenth century and it was sufficiently developed about 1870, at international level, connecting the industrial centres to mine and port centres. The first Italian railway was the Naples - Portici, built in 1939[5] under the Borbone's Kingdom.

The railway Milan - Venice was built in the 40's of the nineteenth century, under the Austrian government, by the Ferdinandea Society[6], an Italian public company. First engineering inspector of the company was Giovanni Milani, which designed the railroad in 1940. The railroad connects Milan to Venice crossing the main cities of the region, Brescia, Verona, Vicenza and Padua, and finally over-crossing the Venice Lagoon. The connection between Milan and Venice has been proposed since from the eighteenth century, in order to resolve the economics decline of Venice[7]. But at the same time the connection between Venice and its mainland has been considered a problem and generated a big debate on the opportunity to do it. In fact the main problem of Venice was its insularity that at the same time was also its main defence against enemies. The bridge to connect Venice to mainland was considered a great opportunity but also a potential danger for the town. The end of

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[2] The first railway net arose in Unietd Kingdom on 27th September 1825, the Stockton and Darlington Railway.

[3] The railway net developed in many European countries: in France in 1832, Belgium in 1835, Germany in 1835 and Austria in 1838.

[4] United States in 1831, Canada in 1836, Australia in 1854, Egypt and Argentina in 1857, South Africa in 1860 and Japan in 1872.

[5] The first railway section, which connected Napoli to Portici, was built on 13th October 1839, under the Kingdom of Ferdinando II di Bordone. The railway section was constructed by a French company named Bayard, and its length was about 7,25 km.

[6] The "Privilegiata Strada Ferdinandea Lombardo – Veneta", established in 1837 under the authorisation of the Austrian Emperor.

[7] A road connection between Venice and the mainland was firstly proposed by the Doge Marco Foscarini in 1763.

the Venice Republic in 1797 brought the town close to the ruin, so finally the idea of a trans-lagoon bridge was accepted [*Bernardello, 1996*].

The bridge had to guarantee two aspects: the safety of Venice from enemies attacks and the safeguard of the environmental equilibrium of the lagoon. Starting from 1830 several projects have been proposed, with different path and technical solutions[8]. The final bridge realisation was the result of the contribution of three engineers: Tommaso Meduna, Giovanni Milani and Andrea Noale [*Facchinelli, 1987*].

Tommaso Meduna make the first proposal in 1836. The project consisted of a masonry arch bridge, made of 234 arches divided in six modulus of 39 arches and large 8 meters, in order to carry one line track and two platform. Subsequently Giovanni Milani was asked to plan the railway Venice - Milan, therefore he made a new project based on the Meduna's one. His project provided a movable wooden bridge, that could be closed in case of war, and a tunnel to bring the aqueduct. His project was hardly criticised because considered too similar to the Meduna's one and more expensive. Because of the disagreement with the Ferdinandea society Milani left the task and Meduna was asked to revised the project. But the construction of the bridge was assigned to Andrea Noale. He started from the previous project but he changed it, keeping only the foundations. The final bridge is 3602 meters long, made of 222 masonry arches divided in 6 modulus, and large 9 meters it carries two rail lines. It is completely made of bricks and stones. The foundations are based on wooden piles fixed in the lagoon bed. The final project was accepted in 1842, the construction lasted 4 years: the bridge was inaugurated on 11th January 1846.

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[8] The first project was presented in 1823 by Luigi Casarini: an embankment to connect Venice to Campalto. The embankment would have to be long about 4852 metres and high more than the lagoon higher tide and would have allowed the transit of both wagons and pedestrians. In some points arches were allocated to permit the boats crossing. Subsequently several projects have been proposed, based on Casarini's project.
The first railway project had been proposed by the engineer Baccanello together with the contractor Biondetti – Crovato in 1830. This project provided that the railway had to arrive at San Giorgio island in front of Saint Mark square, passing along the Giudecca island. The bridge was designed in stone and timber, but the solution was inadequate to carry a railway and too invasive for the town.
The final proposal for a railway bridge connecting Venice to mainland was carried out on 26th May 1836. Five different railway routes have been proposed. The path that has been adopted starts from the south side of Marghera's blockhouse (Forte Marghera) and arrive at Saint Lucia.

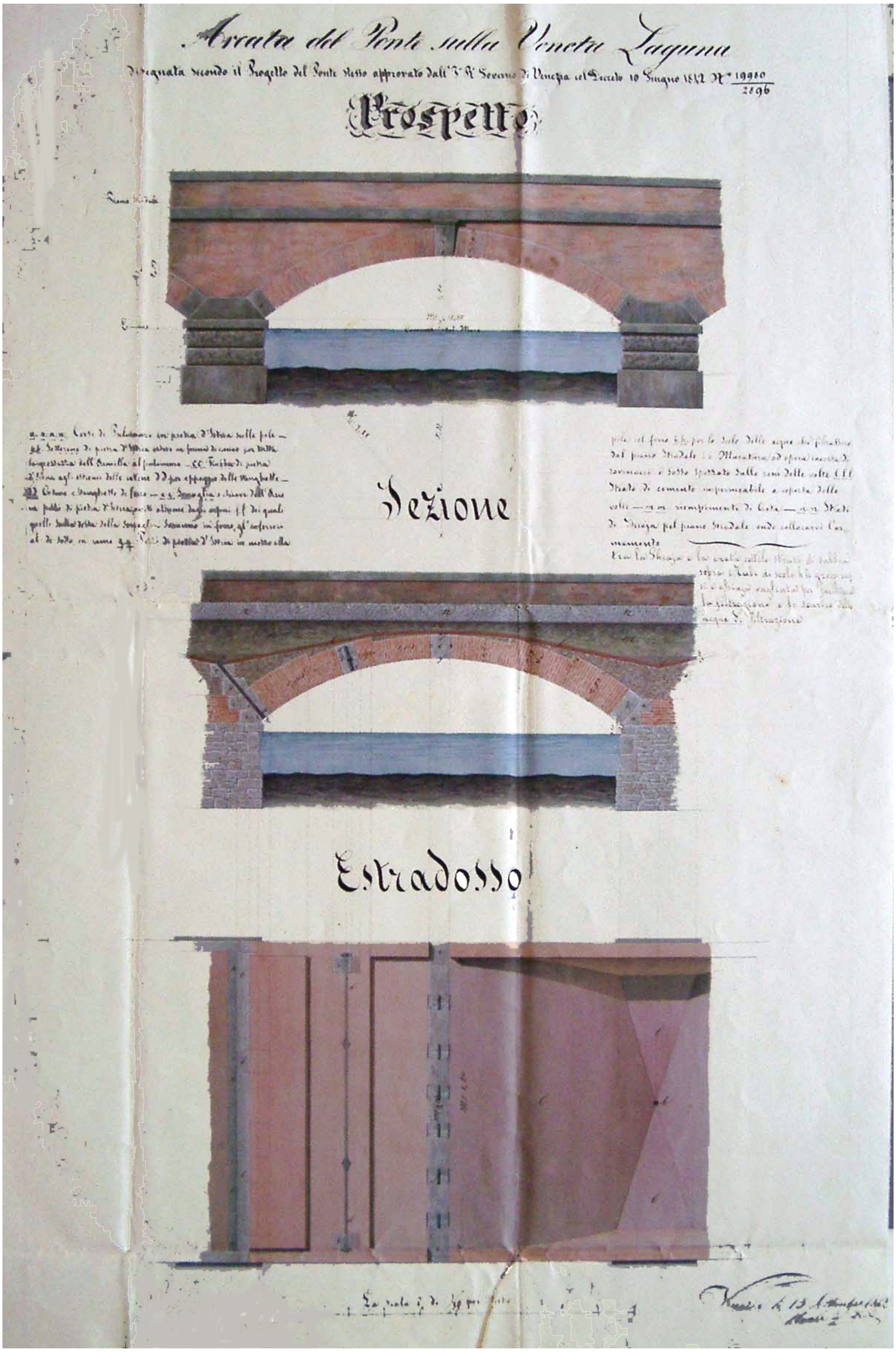

Fig. 4.2 A drawing of the original project:
side elevation of an arch, on top;
longitudinal section, in the middle, showing the arch thickness and the original waterproof system;
a view of the arch extrados with (right) and without backfill (left), down.

Fig. 4.3 The historical bridge in the XIX century.

Interventions and modifications during life

During its life the trans-lagoon bridge was subjected to many interventions. During the revolution of 1848 a part of the bridge was destructed to cut the link between Venice and the mainland and to built a defence line on the main artificial island in the middle of the bridge. This possibility was anticipated during its design: in fact the mines used to destroy the bridge were already installed during its construction. In 1849, after a complete survey and monitoring, the destroyed part of the bridge was rebuilt by Tommaso Meduna. To remember this fact two cannons have been placed in the main island.

In 1933 a roadway bridge, designed by Eugenio Miozzi, was built on the south side of the old bridge. The new bridge, initially named "*Littorio*" and after the 2nd World War renamed "*Della Libertà*", is similar to the old one, but only the arches are made of bricks and the abutments of stones, while the foundations are made of reinforced concrete. Thanks to the innovative techniques adopted its construction lasted only 21 months.

During the first half of the twentieth century it was necessary to increase the number of platforms of the Venice rail station, “Santa Lucia”. The final modulus of the bridge was enlarged. The new part of the structure is made such as the old one, but is more wide, reaching a maximum width of 20 meters.

The rail traffic between Venice and Mestre became too intensive to be carried by the old bridge. Moreover the old bridge needed some intervention of adjustment, in order to allow the greater velocity of trains, and restoration. Thus in 1973 a new rail bridge was built on the north side of the old bridge. The new bridge is made of pre-stressed reinforced concrete beams and was the first rail bridge built in Italy with this method.

Thanks to the construction of the new rail bridge it was possible to restore the old bridge. In fact, even if the bridge was still working, the signs of time began to appear. A first project of restoration was designed in 1959, but the works began only in 1980, with a new project. The main problems were in the arches: water infiltrations, cracking and blocks deteriorated; the piers instead were in a good state of conservation. The first 153 arches were close to traffic so that the extrados was removed and a new waterproof system was realised. Moreover concrete slabs have been placed in order to better distribute the loads. The sub-ballast, the ballast and the rails have been substituted with new ones. At the intrados of the arches have been realised mortar injections to strengthen the arches and the deteriorated blocks have been substituted. Instead the last 69 arches were restored only from the intrados, because of the impossibility to close the train station: the water proof system was increased through resin injections. Restoration works lasted 5 years, in 1985 the bridge was reopened: its capacity of traffic was quadruplicated.

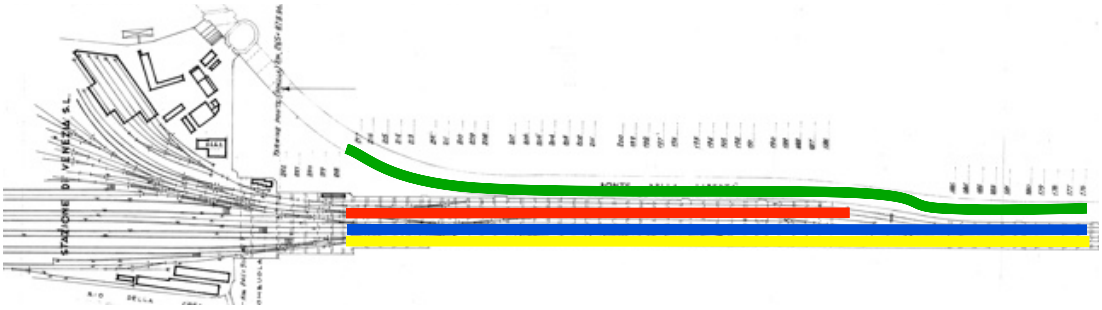

Fig 4.4 The four bridges, marked with different colours:
Blu: the old masonry arch railway bridge (1846);
Red: the widening of the bridge close to Santa Lucia train station (1930);
Green: the road bridge (1933);
Yellow: new railway bridge (1973).

The actual configuration

At the moment the old bridge is hidden by the new ones: the only visible part of the old bridge is the last one, where the road bridge turns to Piazzale Roma. Anyway this part of the bridge is not the original one: during the first half of the XX century it was necessary to increase the number of platforms of the Venice rail station, "Santa Lucia". The final modulus of the bridge was enlarged. To describe the bridge we refer to historical drawings[9] and historical documents, and to surveys and inspections [*Barbieri et al., 2004*]. However, during the construction the project has been changed many times, in fact there are several different drawings.

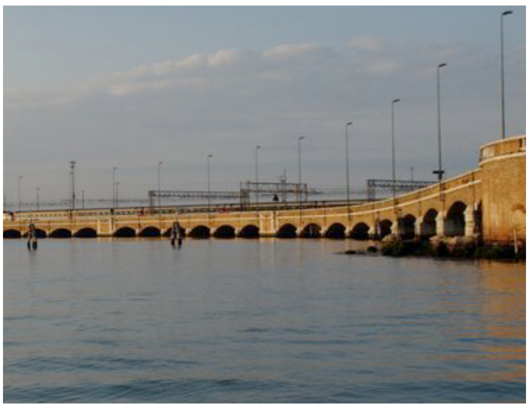

Fig. 4.5 - a) The road bridge (left); b) the new rail bridge (right).

[9] Drawing from the original project are reported here thanks to the courtesy of Arch. Valentina Chiaradia, which graduated at IUAV with a thesis about the Venice Trans-Lagoon Bridge, mentor Prof. A. Di Tommaso.

Fig. 4.6 - The only visible part of the old bridge, on the right, is the widening of the rail bridge close to Santa Lucia train station. The picture is taken when the road bridge, on the left, turns.

The bridge consists of 222 arches whit a span of 10 metres, divided in 6 modulus of 37 arches, named "*stadii*", separated from the close ones by artificial islands. Each modulus is divided in 7 sequences of 5 arches, except the central one of 7 arches: between each sequence there is a big pier, which can be considered as an abutment ("*pila-spalla*") in order to prevent from a global collapse in case of the fall down of a single arch. For this reason the bridge could be considered as a sum of minor bridges. The next figure represents a stadium, indicating the sequences of arches and the difference between the artificial islands, the big piers and the normal piers.

Fig. 4.7 - Scheme of one "stadio".
***P**: artificial islands;*
***A**: the piers-abutments between sequences of 5 arches;*
***B**: the piers-abutments of the central sequence of 7 arches;*
Red numbers: piers; Green numbers: spans.

Technology and materials of the bridge

Each arch has a span of 10 m and a rise of 1,73 m, so with a ration S/R equal to 1:1,58. The vault has a curvature radius of 8,80 m at intrados and a transversal length of 9 m. The thickness changes along the span, increasing at abutment and decreasing at crown. The dimensions of each arch are summarised in the next table:

Elements and geometrical parameters	Dimensions (meters)
Span	10
Rise	1.73
Ratio Span/Rise (a-dimensional)	5.78
Radius of curvature	8.08
Thickness at springing	0.94
Thickness at quarter os span	0.80
Thickness at crown	0.65
Width	9

Table 4.2 - Dimensions of the Venice Trans-Lagoon Bridge

Fig. 4.8 - Historical drawing of on arch, 1844.

The arches are completely made of bricks, even if the original drawings indicate a crown made of Istrian stones and the presence of some transversal metallic chains at haunches. As previously said, the project changed many times, and different modification happened also during the construction, which lasted five years: Noale changed many times the projects, in fact there are several different drawings. The realised structure is an unique masonry barrel vault made of bricks. The bricks utilised are the typical venetian bricks. Unfortunately there is no information about the realisation of the bricks utilised during the construction of the bridge. The characteristics of the bricks have been provided by a study regarding the traditional materials of the venetian architecture [*Zago and Riva, 1981*].

The dimension of bricks is (25 x 12 x 5) cm, the thickness of mortar joints is about 1 cm. The texture of masonry changes along the span on the base of the thickness of the arch, but in the lateral view the thickness of arches is constant. The transversal section of the arch has a constant thickness. As previously said the original bridge is completely hidden by the new railroad bridge and by the road bridge, the only visible part is the enlargement, which has been built almost a century later. Moreover many intervention of substitution of blocks have been carried out during the history of the bridge, therefore the real texture of the masonry of the arch may only be assume don the base of drawings and looking at the enlargement. More information about the texture of masonry will be provided afterward, in the paragraph regarding homogenisation procedure.

Fig. 4.9 - Particular of the masonry of one arch of the enlargement, it is possible to notice the substitution of blocks.

Over the vaults the backfill is made of sand, stones and bricks. It was covered by a layer of lime and bricks to create the waterproof system, that has been substituted with a concrete layer during restoration of 70's and 80's of the twentieth century. Over the backfill there is another layer of filling, made of stones, called sub-ballast, necessary to reach the correct height of the rails. The rails lay on the ballast. The spandrel walls are made of bricks, the same used for the arch. The texture of masonry in spandrel is an english bond, which guarantee good mechanical properties. The railing are made in Istrian stones.

Fig 4.10 - Lateral view of the 221st arch, in the enlargement,
it is possible to notice the masonry of spandrel walls and the railings in Istrian stone.

The abutments are made of Istrian stones, the block is placed perpendicular to the line of trust. The piers have different dimension. Usually their cross section is variable. The normal piers are made of Istrian stones, with regular blocks and thin mortar joints on top and mixed stones and concrete in the deeper part, close to foundations. The big ones present a similar external layer but are filled with incoherent material. There are not drawings or documents about the artificial islands, but probably they must have a configuration similar to the big piers with more filling. There are some difference in the original drawings: in some case the Istrian stone of the piers has the same height of the springing of the arch, however in other cases the Istrian stones reach the level of the backfill. Unfortunately it is not possible to know which one is the real case.

The foundation have been realised with the typical technology of historical venetian buildings: wooden larch piles fixed in the lagoon bed [*Zuccolo, 1975*]. This is due to the peculiar characteristics of the venetian lagoon soil: the piles contribute to strengthen the ground and at the same time reach the deep layer of soil, called *Caranto*, that has better mechanical characteristics. The piers base on a double wooden plank, placed in the two orthogonal directions, with a thickness of 6 cm, that connect each other the piles. During the construction of the road bridge the foundations have been surveyed by Miozzi: he observed that the state of conservation was good, but that it decades rapidly when the wood is put in contact with the air.

Hence, summing up, the structural materials utilised in the bridge are:

- Istrian stone, for the piers and for decorative elements;
- Venetian bricks, used for arches and vaults and for the spandrel walls; bricks have been used also in the filling, mixed with other materials;
- Larch wood, poles are used to reinforce the ground under the foundations, which are realised with the traditional venetian technique;
- Stones, sand, and other incoherent heterogeneous materials used for backfill and filling.

Their mechanical characteristics have been evaluated on the base of specific studies carried on typical historical masonries of Venice. [*Zago and Riva, 1981*].

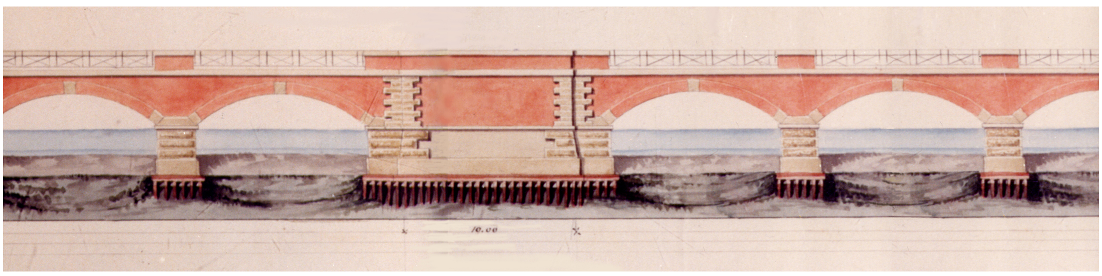

Fig 4.11 - Lateral view with indication of foundation larch piles, taken from Noale's project, 1844.

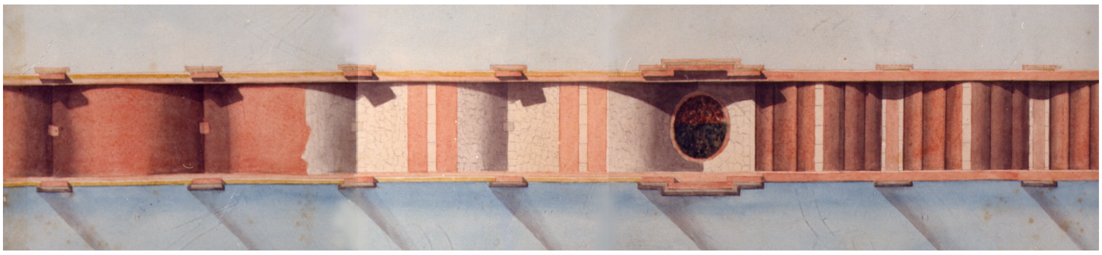

Fig. 4.12 - Extrados of arches with (left) and without (right) backfill, taken from Noale's project, 1844.

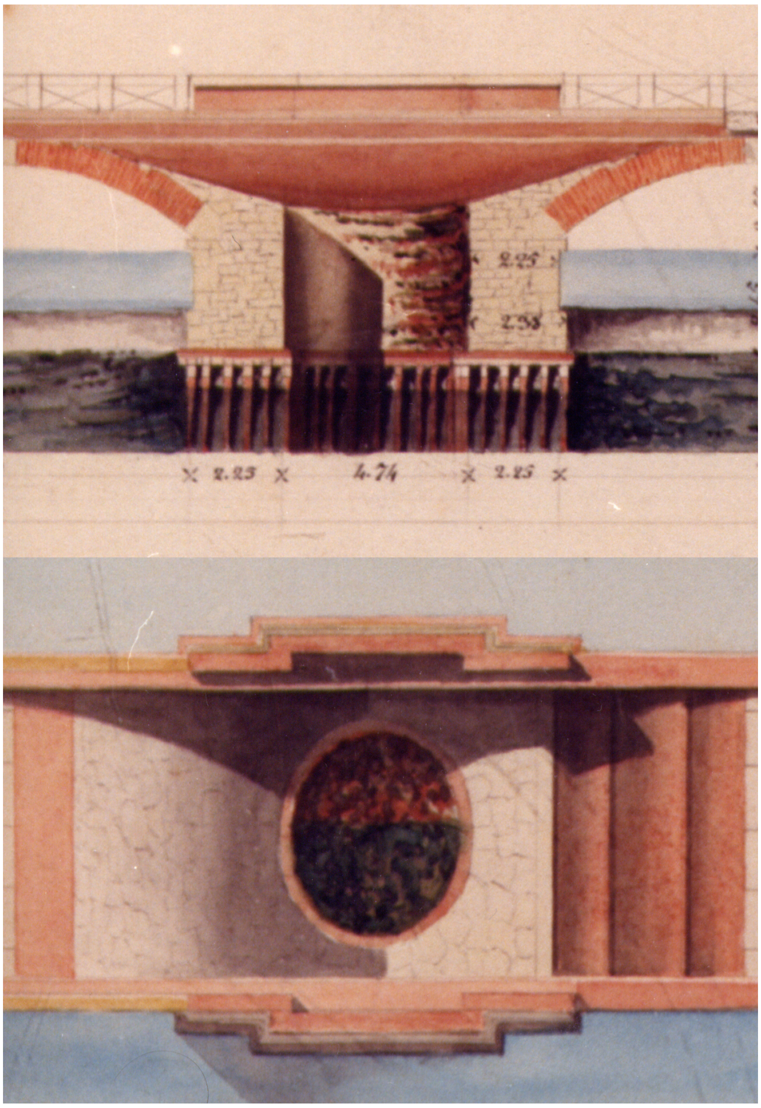

Fig. 4.13 - Longitudinal section and plan without the filling of the pier-abutment. The core of the pier-abutment is filled with incoherent material.

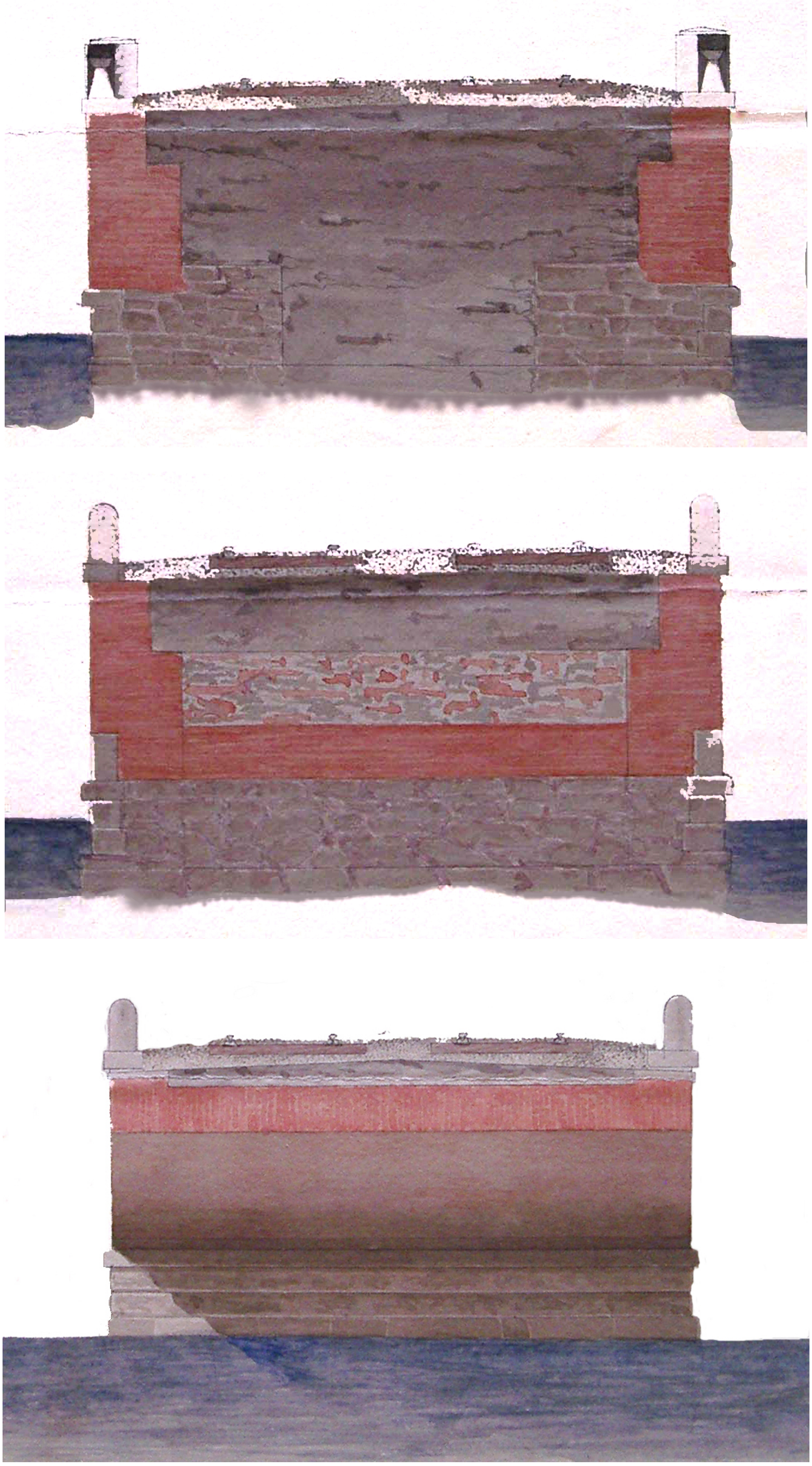

Fig 4.14 - Cross sections of:
a) the pier-abutment (on top);
b) the pier (in the middle);
c) the arch at crown (down).

The original structural design

At the time of its construction the Venice Trans-Lagoon Bridge, such as all the masonry arch bridges built at that period, has been designed on the base of geometrical empirical rules. The masonry arch stability before the eighteenth century was based on geometrical rules derived from the experience and the observation of other existing constructions of such type of structure [*Benvenuto, 1991*]. Such rules were provided by the many treatises written by the most famous bridge builders and engineering of the french school[10] [*Rondelet; Perronet*].

A comparison between the dimension of the arches of the Venice Trans-Lagoon bridge and the geometrical rules provided by the methods provided by treatises in the first half of the nineteenth century has been given by [*Barbieri et al., 2004*]. Some of the structural elements of the bridge - the arch thickness at springing, at quarter of span and at crown, and the thickness of piers - have been evaluated using the formula reported by treatises considered by authors[11], assuming a masonry arch with a span equal to ones of the Trans-Lagoon bridge, which was imposed by military prescriptions. The dimensions established using the Scheffler's method [*Scheffler, 1864*] look to be the best fit to the real bridge dimensions. Moreover, Milani has been working in Austria and Germany before to design the bridge, and at that time Venice was under the influence of the Austrian Empire. Therefore it may be possible that Milani knew the Scheffler's theory and used it in the design of the Venice Trans-Lagoon bridge. In fact arches seem to be right designed and their structural dimensions are comparable with the ones suggested by Scheffler.

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[10] The Ècole Nationale des Ponts and Chaussées of Paris, the first modern school of engineering, in which the most famous bridge engineers of the eighteenth century have studied.

[11] Perronet, Rondelet, Cantalupi, Scheffler, Breyman.

The state of conservation

The state of conservation of the Venice Trans-Lagoon Bridge is the result of the peculiar environmental conditions of the Venice Lagoon. In particular a series of factors provoke an increasing of the natural ageing of the bridge:

- The very high concentration of salts in the water of the Lagoon;
- The presence of many biological agents;
- The pollution of the water and the presence of chemical agents;
- The daily tides;
- The waves due to the traffic boat.

As previously said, the bridge has been subjected to restoration in the 70's and 80's of the last century: effects of the restoration are still visible, such as signs of the interventions carried out. However, even if restoration occurred, some kind of deterioration of masonry and stones, and some structural damages have been surveyed[12] some years ago in the 219th, 220th and 221st arches. Some interventions of substitution of blocks have been realised not in the correct way. Also the waterproof system seems to do not work perfectly. The defects which have been observed mainly regards the deterioration of materials, masonry and Istrian stone:

- Delamination;
- Loss of material;
- Salt attacks;
- Biological attacks and vegetation;
- Leaching;

Moreover, some cracking in spandrel walls have been observed. Some pictures, which have been taken during this survey, about the defects that may be found in the Venice Trans-Lagoon Bridge, are here reported.

[12] Chiaradia V. "Il ponte ferroviario in muratura sulla Laguna Veneta: indagini storiche e in situ", degree thesis, Iuav University of Venice, supervisor Prof. A. Di Tommaso

Fig 4.15 - Detail of springing of one arch, it is possible to notice:
delamination of blocks, substitution of blocks and repointing carried out with heavy mortar;
the Istrian stone shows is stained at the height of tides.

Fig 4.16 - Detail of the vault form below, it is possible to notice:
un-correct intervention of substitution of block, use of cementitious mortar,
biological and vegetation attacks in the pier

Fig 4.17 - View of the vault form below, it is possible to notice:
the pipe of the waterproof system, lecheing in the masonry of the barrel vault.

Fig 4.18 - Detail of the vault, calcareous crusts

Fig 4.19 - View of the bridge from below, it is possible to notice the three bridge: the enlargement, the old rail-bridge, the new rail-bridge.

Fig 4.20 - Detail of the connection between the enlargement and the old rail-bridge, it is possible to notice diffused salt attack, leaching and dislocation of material.

Fig 4.21 - Detail of the dislocation of material.

Fig 4.22 - Detail of the masonry of the vault, it is possible to notice one of the holes drilled for the injection carried out during the restoration and repointing..

Fig 4.23 - The 219th pier, it is possible to notice vegetation and biological attacks and stains.

Fig 4.24 - The 220th pier, .it is possible to notice stain in the Istrian stone,
substitution of blocks and repointing in the spandrel.

Fig 4.24 - Cracking in spandrel walls, vegetation in the railing.

Part 2

CS.I Multi-scale analysis

Models represent bridge at different scales corresponding to its component: one single arch and sequences of five and seven arches. Models of the entire bridge and of a whole *stadio* are not meaningful, because of the artificial islands between *stadii* and piers-abutment between the sequences of 5 or 7 arches. The bridge could be considered as a sum of minor bridges, consisting of sequences of 5 and 7 arches. Model of a single arch is more detailed: it is fit for study local behaviour of the arch. Instead models representing sequences of five and seven arches have a lower level of details: they are used to investigate the global behaviour of the bridge. Different combinations of load are applied in several positions to evaluate the response of the bridge in order to find the worst combination of loads and the worst position. Analyses performed are: linear static and natural frequencies on FE models and limit analysis on beam models.

Two-dimensional models represent a longitudinal section of the bridge, taken in correspondence of a rail road. By a computational point of view the two-dimensional model is very manageable, therefore it is possible to perform many analyses with different restraints, material properties and combinations of loads. Two-dimensional models are made with finite elements and beam elements. Finite element models are utilised to carry on linear static analyses and natural frequencies. Beam elements models, which take into account the effect of backfill, are used to carry on limit analysis[13].

Three-dimensional models are made with finite elements and utilised to carry on linear static analysis and natural frequencies. Three-dimensional models have a high level of detail. The attention has been paid to global behaviour, with particular care to transversal effects. They have been used to carry on natural frequencies analysis too.

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[13] Finite Elements analysis have been performed with Strauss7 program, which has been already used to carry analysis of masonry arch bridge [Ford et al., 2003].
Limit analysis have been performed using Ring 3.0 (Limit State Ltd 2008) a specific software for the analysis of masonry arch bridge.

FEM elements and material properties

Elements used in two-dimensional F.E. models are 2D plane strain elements with 4 nodes. Three-dimensional F.E. models are made by brick elements with 4 nodes. Both horizontal and vertical displacement are constricted at the base of piers, instead only horizontal displacements are constricted at external abutments. The elements of the models belong to different categories corresponding to different materials and structural functions. Elements of models of 5 and 7 arches belong to three categories:

- Piers;
- Arches;
- Backfill.

In single arch model, which is more detailed, the backfill is divided in two further different sub-categories:

- Lower filling;
- Upper filling.

Bi-dimensional models represent a longitudinal section of the bridge. The three-dimensional models include also spandrel walls and railings. Materials are described by 3 mechanical parameters:

- Young's modulus E (MPa);
- Poisson's coefficient v;
- Density ρ (Kg/m3).

Values of parameters for each structural element are defined on the base of the materials and texture [*Cecchi et al., 2010*].

Piles are made of Istrian stone, a limestone used by Venetians for foundations, blocks are squared and regular, the strength of material is high and the quality of masonry is good:

E = 6000 (MPa)	$\nu = 0.2$	$\rho = 2700$ (kg/m^3)

Table 4.3 - Mechanical properties of piers

Masonry arch is made of solid bricks, the geometrical disposition and texture of bricks guarantee a good meshing of blocks:

E = 3000 (MPa)	$\nu = 0.2$	$\rho = 1800$ (kg/m^3)

Table 4.4 - Mechanical properties of the arch barrel

Backfilling is made incoherent materials, a mix of with bricks, stone and sand, an average value between upper and lower backfill has been used:

E = 1000 (MPa)	$\nu = 0.2$	$\rho = 1900$ (kg/m^3)

Table 4.5 - Mechanical properties of backfill

The lower backfilling, just above the arch, is made of bricks and stone and gives a partial contribution to the structural behaviour of the bridge. It has a larger strength respect to the upper filling, made of stone and sand, which duty is to create a plan surface on the bridge to place the railway:

E = 1200 (MPa)	$\nu = 0.2$	$\rho = 1800$ (kg/m^3)

Table 4.6 - Mechanical properties of lower backfill

E = 800 (MPa)	$\nu = 0.2$	$\rho = 2000$ (kg/m^3)

Table 4.7 - Mechanical properties of upper backfill

Spandrel walls are made of well organised masonry of solid bricks. They contribute to structural behaviour of bridge:

E = 2000 (MPa)	$\nu = 0.2$	ρ = 1800 (kg/m^3)

Table 4.8 - Mechanical properties of spandrel walls

Railings are made of Istrian stone and bricks:

E = 800 (MPa)	$\nu = 0.2$	ρ = 2300 (kg/m^3)

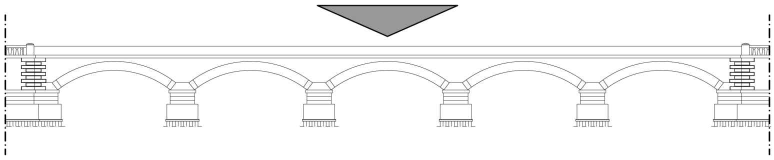
Table 4.9 - Mechanical properties of railings

Loads

Loads and combinations are provided by specific technical railway regulation[14]. Three loads schematise the train traffic: *LM71*, *SW0* and *SW2*. All schemes have been multiplied for the coefficient φ that increase their value in order to represent the dynamic effect due to train. This coefficient is evaluated through a parameter, $L\varphi$, based on the structural configuration of the bridge, but there is not a specific value for masonry structure. The value utilised is the one proposed for principle beams on a series of arches with filling, that seemed the most appropriate. The value of φ obtained utilising is 1.15.

In F.E. models trains have been positioned on the central arch, in order to obtain a symmetrical behaviour, and on a lateral arch, in order to obtain an anti-symmetrical behaviour. Moreover trains are positioned on the middle and at quarter of span. Those positions provoke the more common mechanisms of collapse. Load on the middle of arch could provoke the five-hinges mechanism, instead load at quarter of span could cause the four-hinges mechanism.

[14] Italian Railway Code, n° I/SC/PS-OM/2298, provided in 1995.



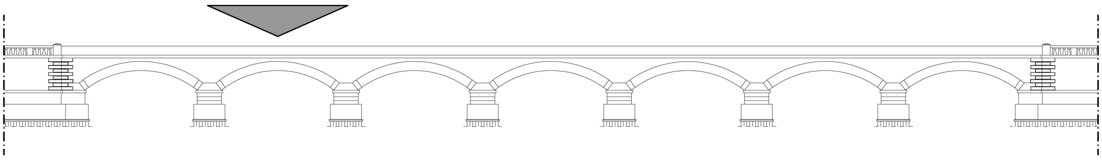


Fig 4.25 - Positions of load in multi span models:
a) central span loaded (above); b) lateral span loaded (below).
c) load at the middle of span; d).

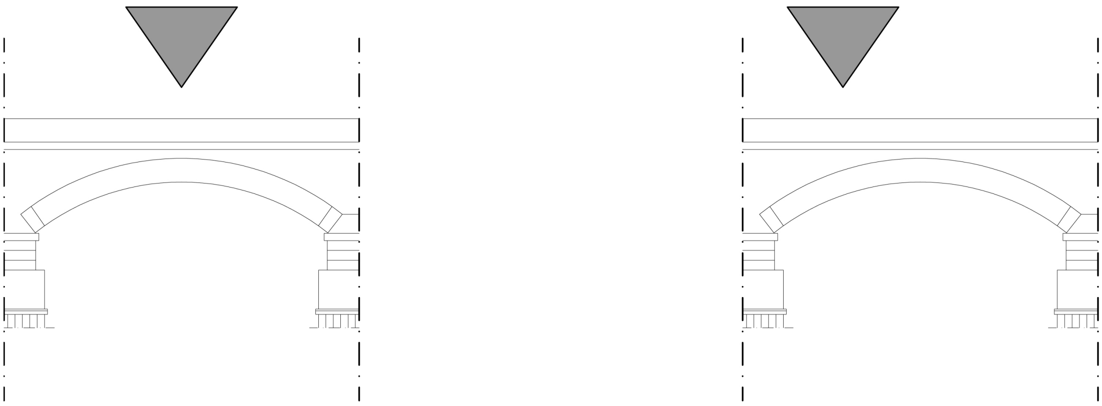


Fig 4.26 - Positions of load in single arch loaded:
a) load at the middle of span (left); b) load at quarter of span (right).

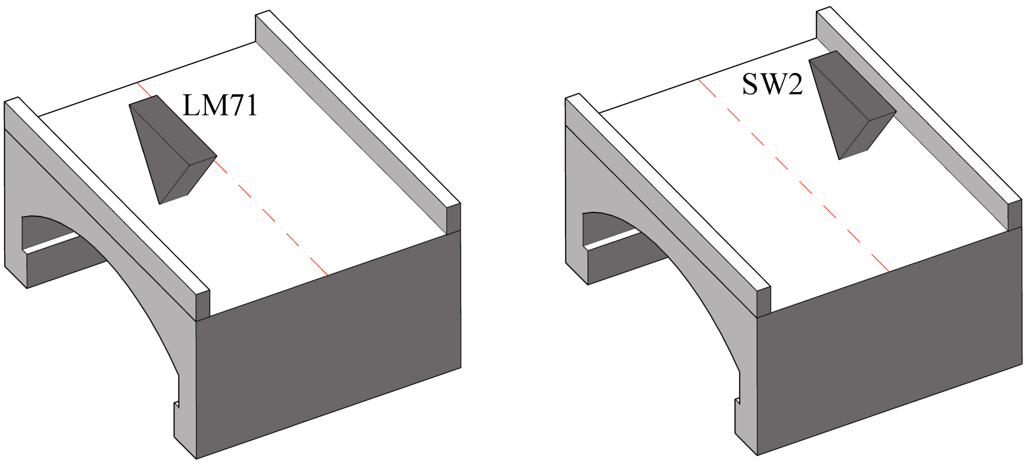


Fig 4.26 - Positions of load in three-dimensional models:
left side and right side loaded.

In the limit analysis train have been moved on all the bridge length in order to define the critical load. The bridge carry two railroads. Simultaneous loads are the ones provided by regulations. Loads are applied in all the possible positions of trains.

Single arch models

Static analysis of bi-dimensional and three-dimensional F.E. models of a single arch have been performed. A three-dimensional reference system is assumed:

- X is in longitudinal direction;
- Y is in vertical direction;
- Z is in thickness direction of bridge.

Bi-dimensional model consists of 864 nodes and 764 elements. Trains have been placed both on the middle of span, loading the whole arch, or sideways, loading half arch. Worst combination of load is given by LM71 on middle of span. Max values of stress in longitudinal direction are:

- σ_{xx} = +0.8893 MPa in intrados of middle of span;
- σ_{xx} = -2.7707 MPa at abutment.

Three-dimensional model consists of 15600 nodes and 12672 elements. Trains have been placed alternatively on both side of bridge and together. Analysis show distribution of stress at arch intrados and extrados in the cross section taken at middle of span due to different loads.

Limit analysis is performed on bi-dimensional macro element model. Model is realised using Limit State Ring 2.0, a specific software for limit analysis of masonry arch bridges. LM71 has been applied in all the possible positions simulating its movement on the bridge. Critical load case is obtained when train is positioned in the middle, exactly at 7,2 m from the left abutment. The result is congruent with the one obtained with bi-dimensional F.E. model. Failure load factor is equal to: 23.045.

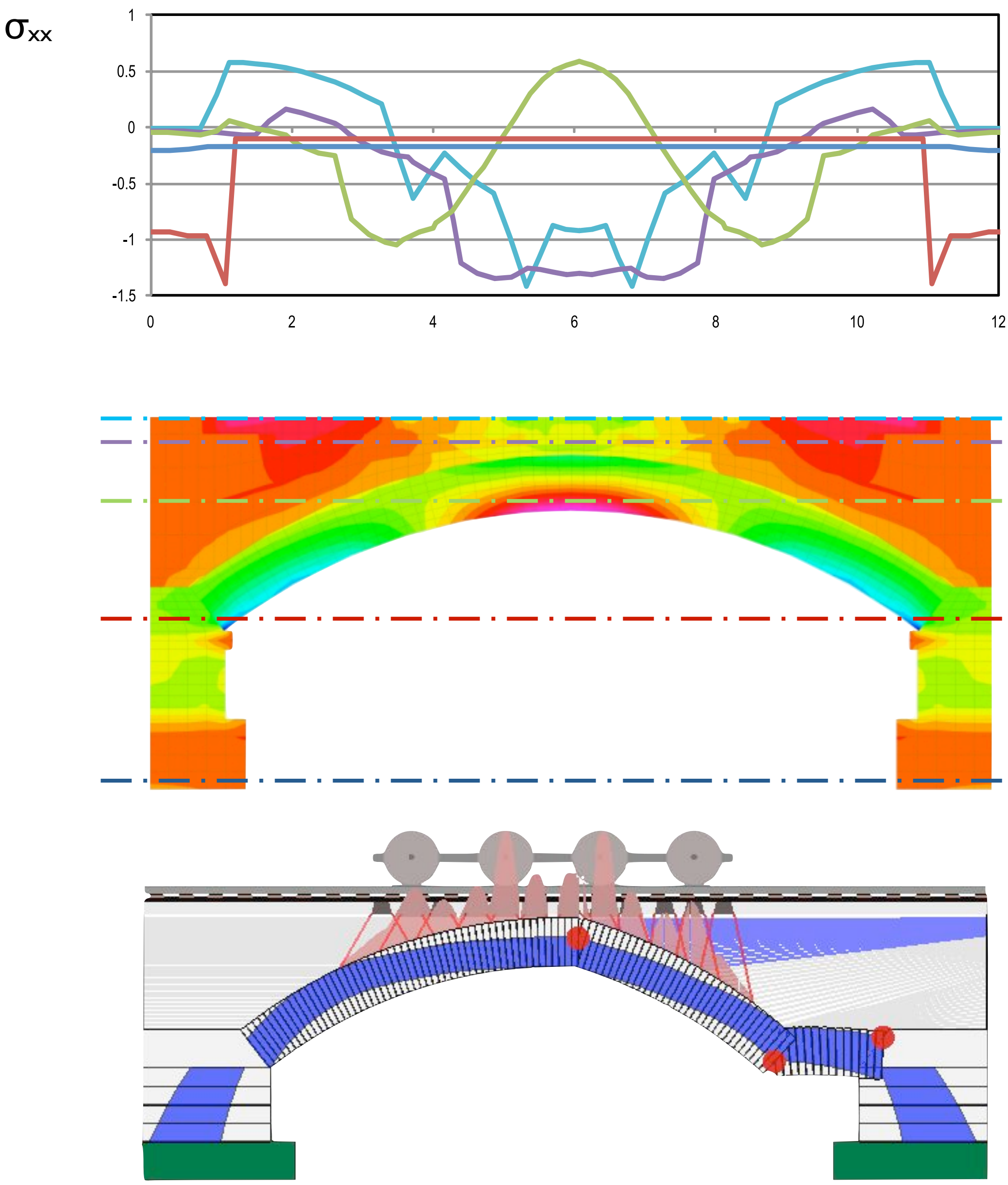


Fig. 4.28 - Two-dimensional single arch models:
a) Two-dimensional F.E model arch and distribution of stresses (on top);
b) Limit state analysis and mechanism of collapse (down).

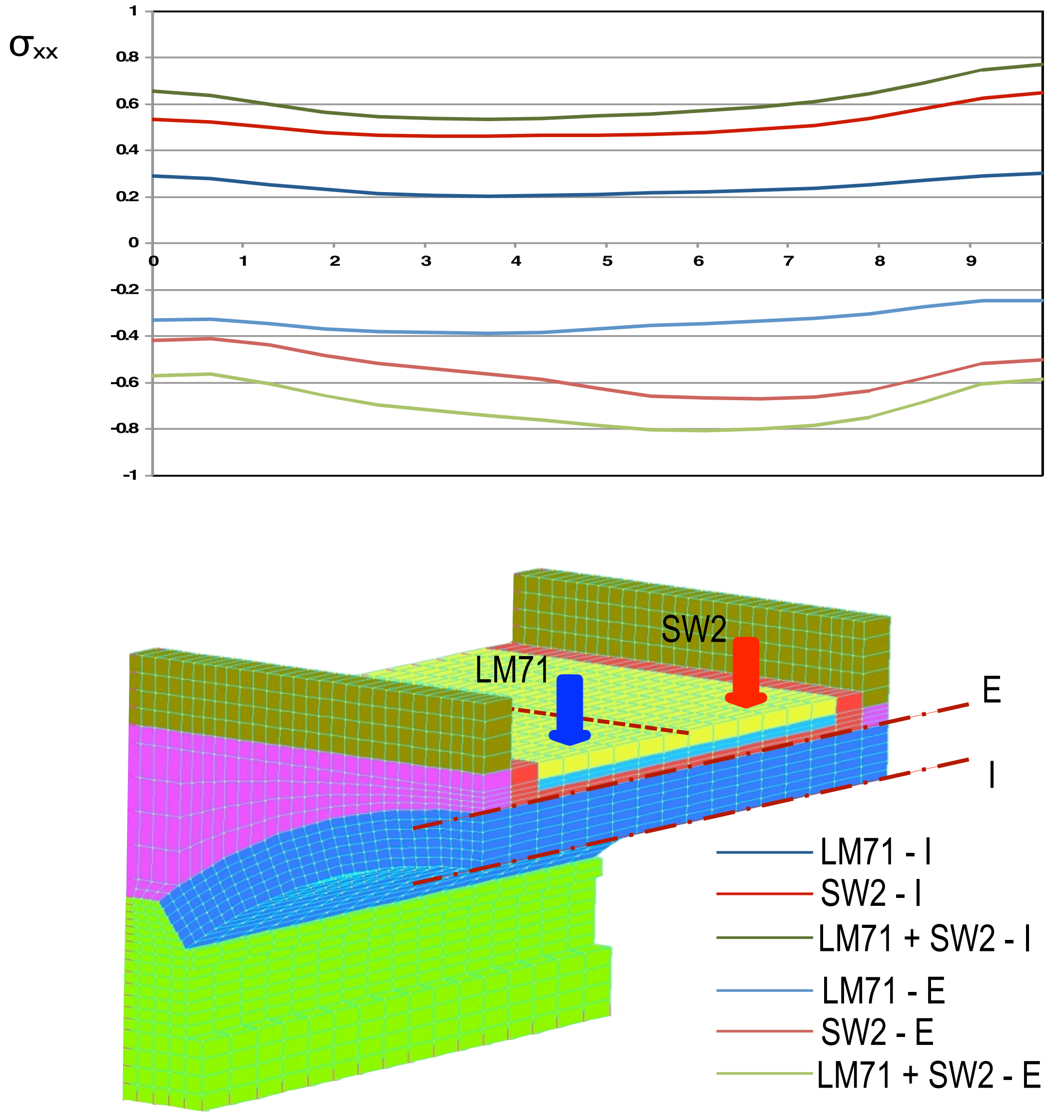


Fig. 4.29 - Three-dimensional single arch model:
transversal distribution of stresses at arch intrados and at extrados.

Five and seven arches bi-dimensional models

Five arches bi-dimensional F.E. model consists of 1249 nodes and 1040 elements. The central arch is loaded. Worst combination of loads is given by LM71 at middle of span. Max values of stress in longitudinal directions are:

- σ_{xx} = +1.3118 MPa at intrados in middle of span of central arch;
- σ_{xx} = -2.9415 MPa at abutment of central arch.

Seven arches bi-dimensional F.E. model consists of 1743 nodes and 1456 elements. A lateral arch is loaded. Worst combination of loads is given by LM71 at middle of span. Max values of stress in longitudinal directions are:

- σ_{xx} = +1.1869 MPa at intrados in middle of span of the loaded arch;
- σ_{xx} = -3.6024 MPa at abutment of loaded arch.

Pick values are obtained with LM71, but SW2 show distributed consistent values of stress.

Limit analysis is performed on five arches model applying train in all the possible positions, simulating its movement on the bridge. The critical load is obtained when train is positioned on the last arch. Anti-symmetrical behaviour could be very dangerous for this kind of structure, but at the same time big piles between sequences of arches prevent this mechanisms. Failure load factor is equal to: 11.951.

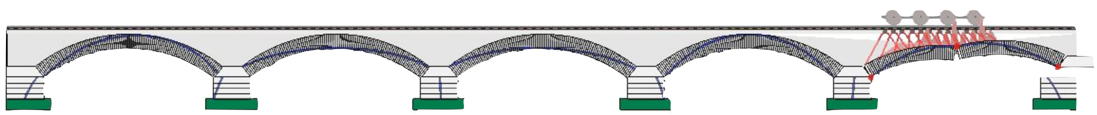


Fig. 4.30 - Two-dimensional five arches model:
limit state analysis and mode of collapse.

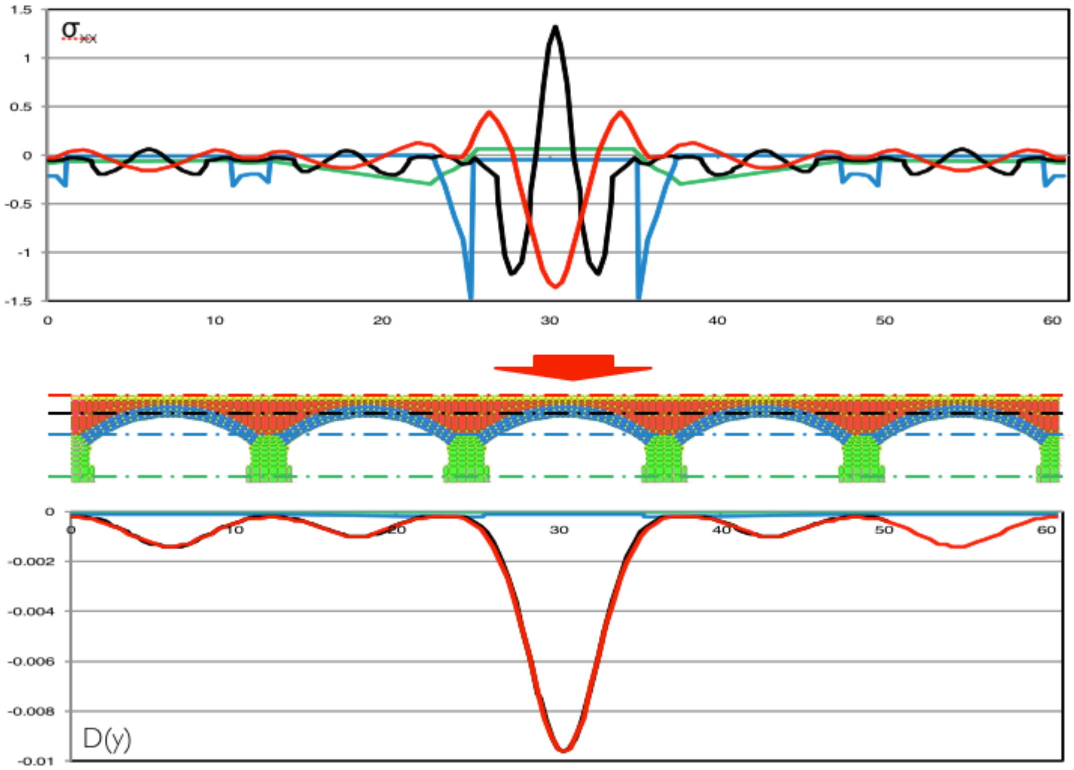


Fig. 4.31 - Two-dimensional five arches model:
stress in longitudinal direction (on top) and displacements in vertical direction (below)
due to LM71 applied on central arch at middle of span.

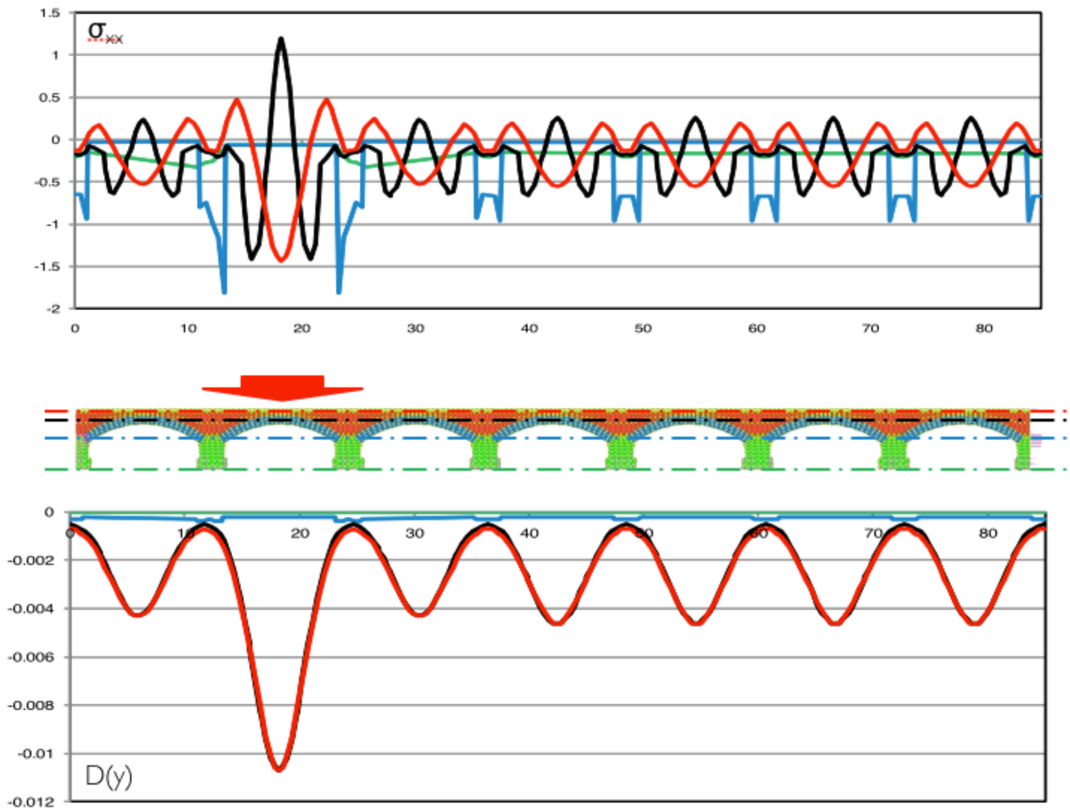


Fig. 4.32 - Two-dimensional seven arches model:
stress in longitudinal direction (on top) and displacements in vertical direction (below)
due to LM71 applied on lateral arch at middle of span.

Seven arch three-dimensional F.E. model

3D model is utilised to investigate the global behaviour of the bridge with attention to the effects due to anti-symmetrical position of trains respect to transversal cross section. Combination simulate the effects of only one side or both sides loaded in the different position. Worst combination of loads is given by LM71 + SW2 on lateral arch. Max value of stress in longitudinal direction are:

- σ_{xx} = +0.7593 MPa in correspondence of pile number 4;
- σ_{xx} = -1.558 MPa in correspondence of pile number 3.

The model has been cut in longitudinal sections in order to evaluate differences in stresses. Sections are taken in correspondence of rails and spandrel walls in longitudinal direction, and in correspondence of backfilling in transversal direction. The attention is paid to the distribution of stresses and to the function of spandrel walls.

3D model has also been used for evaluate natural frequencies of the bridge. The four modes with highest participation of modal mass are showed in the figure 4.34.

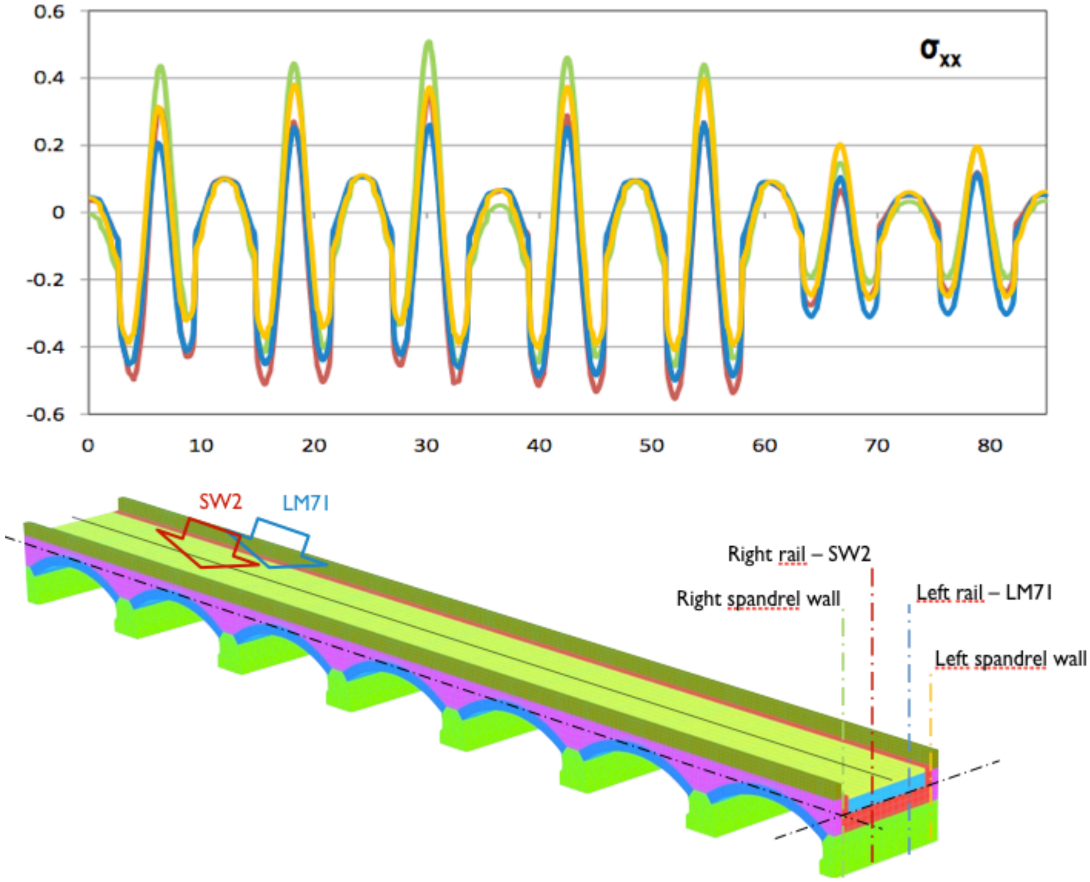


Fig 4.33 - Three-dimensional seven arches model:
stress in longitudinal direction in correspondence of different longitudinal sections

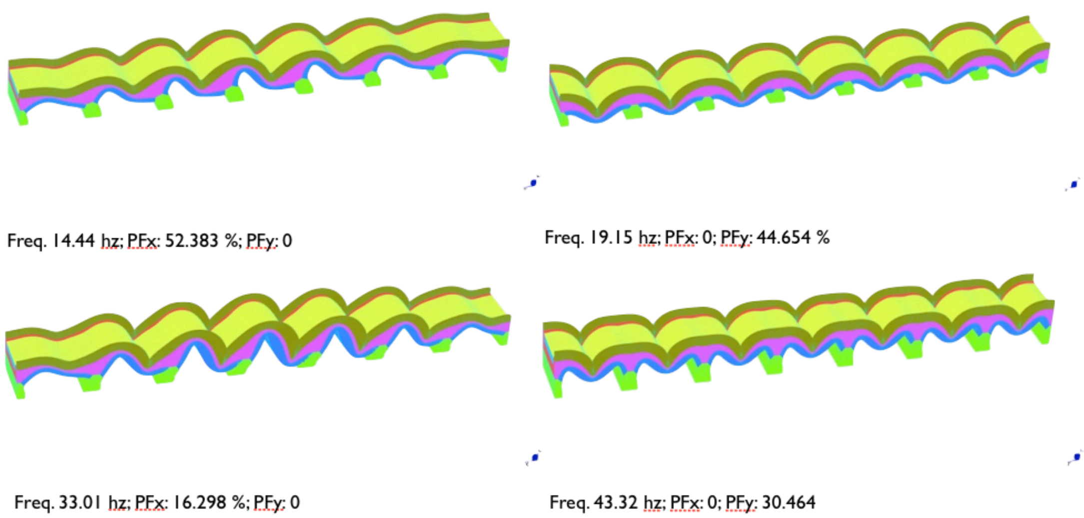


Fig. 4.34 - Three-dimensional seven arches model:
modes of vibration with highest participation of modal mass

CS.I.2 The effect of backfill

A parametric analysis has been performed in order to evaluate the effect of modification of the mechanical properties of the backfill. As outlined in the second section of the thesis, the backfill plays a key role in the global behaviour of a masonry arch bridge. Strengthening interventions aimed to increase the strength of backfill material may provide improvement in the structural behaviour of masonry arch bridge without modifying the original structural form and without altering the aesthetic appearance of the bridge, as outlined in the conclusions of the third section of the thesis.

Two models of a single arch have been used to evaluate the behaviour of the bridge at the varying of the mechanical properties of the backfill. One model has been prepared modelling the backfill by means of spring elements. The strength of backfill is modified varying the spring stiffness. This model has been compared with the two-dimensional single arch model used for the multi-scale analysis. In this case the strength of backfill has been modified varying the elastic modulus of plates elements. A parametric analysis has been carried out to evaluate the effects of the modification of backfill properties to the global behaviour of the bridge.

Linear static analysis has been performed analysing the behaviour of the bridge. The mechanical parameters used for the different element of the bridge are the same previously utilised in the multi-scale analysis. Three values of spring stiffness and young modulus backfill has been used, in order to simulate strengthening of the backfill material. Two limit case and an intermediate one has been simulated:

- Backfill 1 (marked in blu in the diagrams), which has good mechanical properties, close to the ones of the arch barrel;
- Backfill 2 (marked with red in the diagrams), which has mechanical properties with medium values between backfill 1 and 3;
- Backfill 3 (marked in green in the diagrams), which has poor mechanical properties, as if it were only sub-ballast.

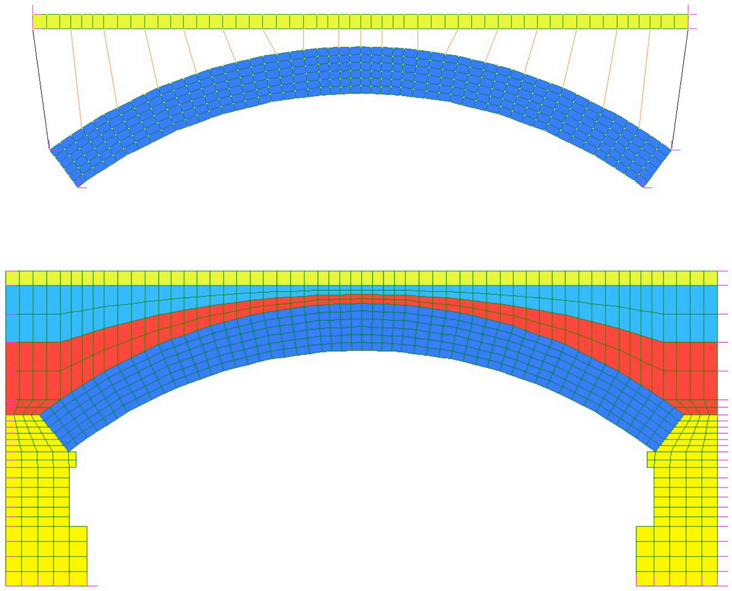


Fig 4.35 - Two-dimensional single arch models:
backfill modelled with spring elements (above) and
backfill modelled with plates elements - red backfill, heavenly sub-ballast - (below).

The values of vertical displacements and stress XX of the arch are reported in the following diagrams. Vertical displacements are taken in the axis of the arch. Stresses are taken in vertical sections of the arch at crown, at haunches and at springing.

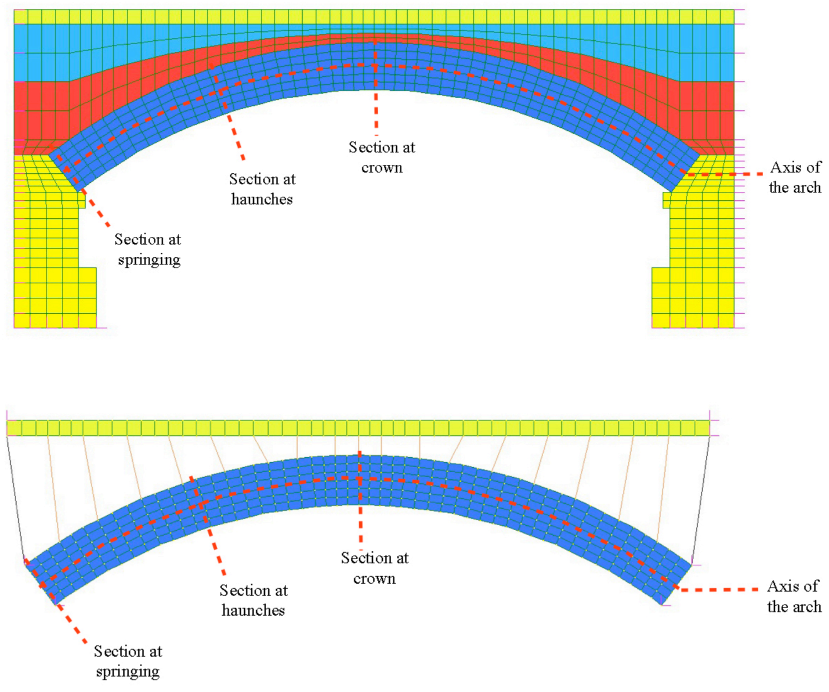


Fig 4.36 - The sections used for the diagrams.

Spring model

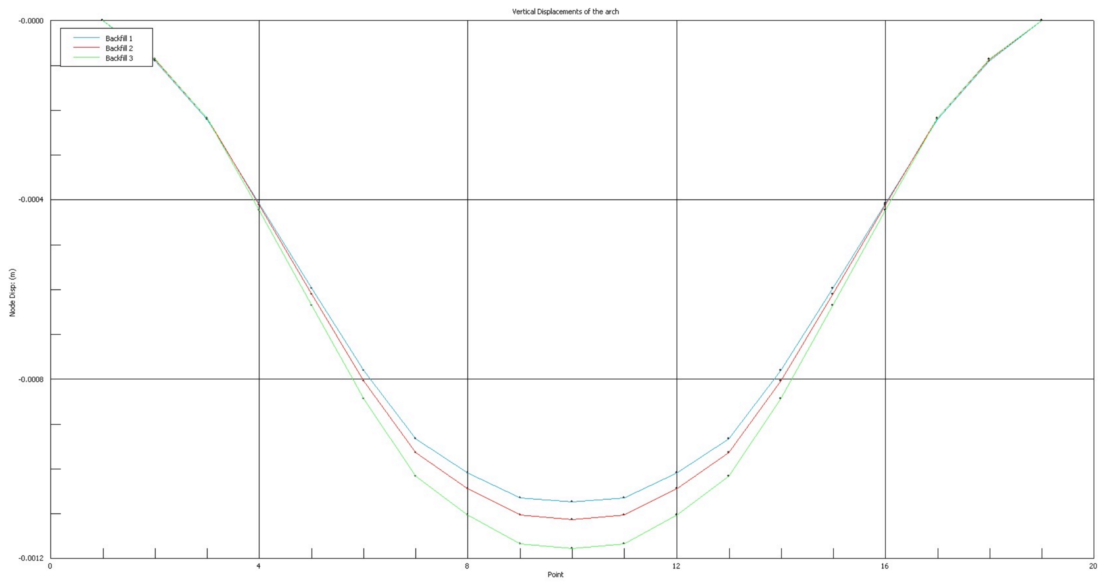


Fig 4.37 - Spring model, vertical displacements of the arch;
backfill 1 in blue, backfill 2 in red, backfill 3 in green.

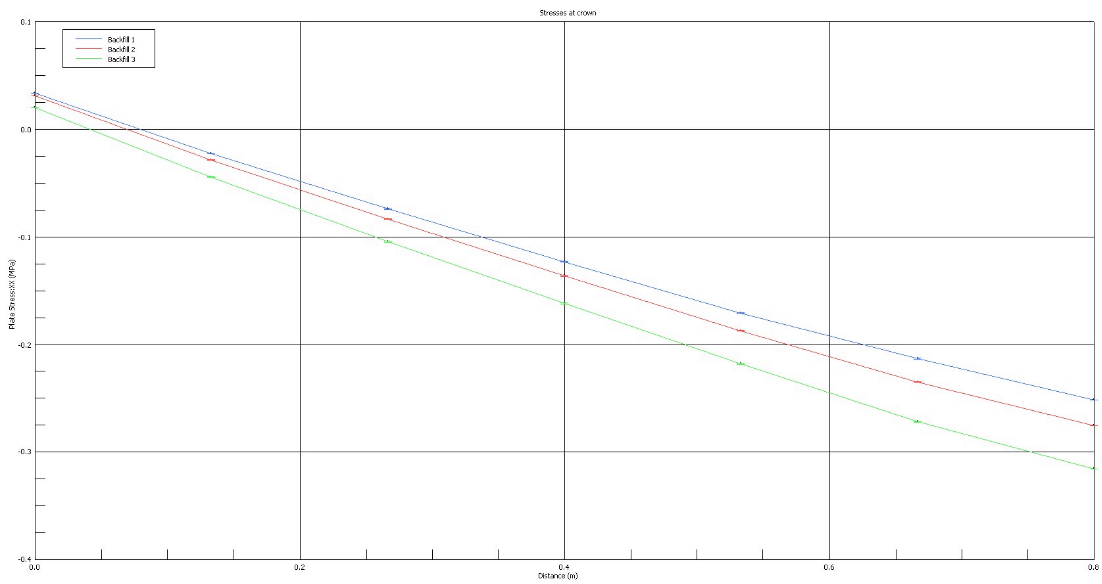


Fig 4.38 - Spring model, stress XX at crown;
backfill 1 in blue, backfill 2 in red, backfill 3 in green.

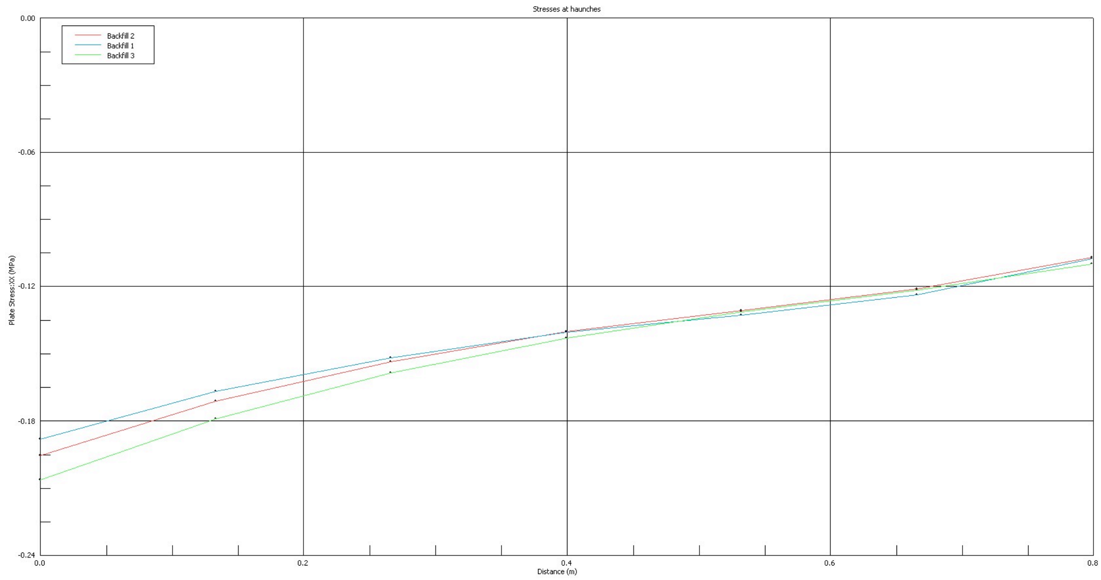


Fig 4.39 - Spring model, stress XX at haunches;
backfill 1 in blue, backfill 2 in red, backfill 3 in green.

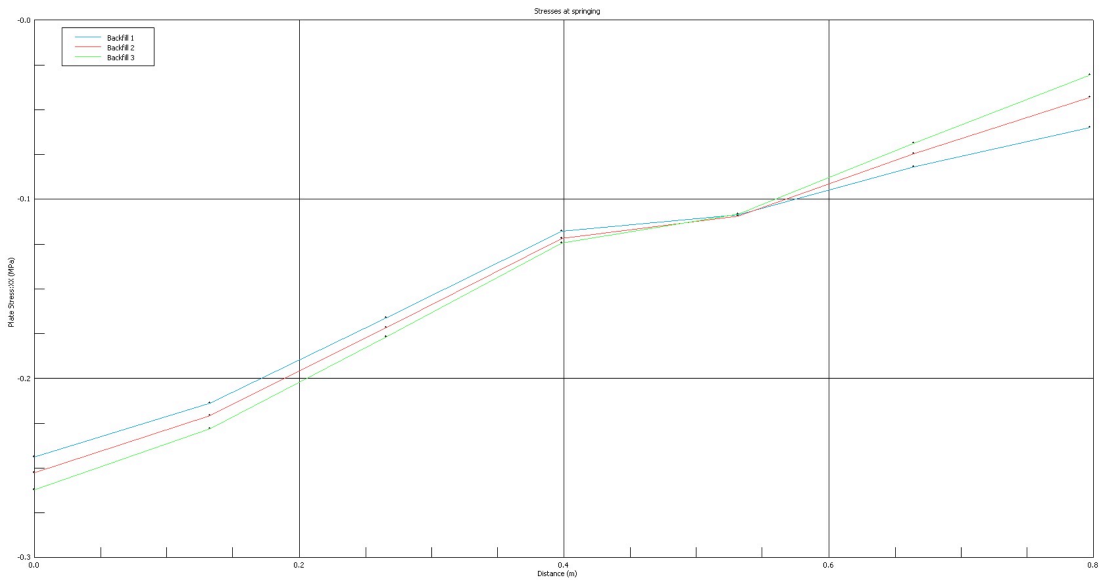


Fig 4.40 - Spring model, stress XX at springing;
backfill 1 in blue, backfill 2 in red, backfill 3 in green.

Plate model

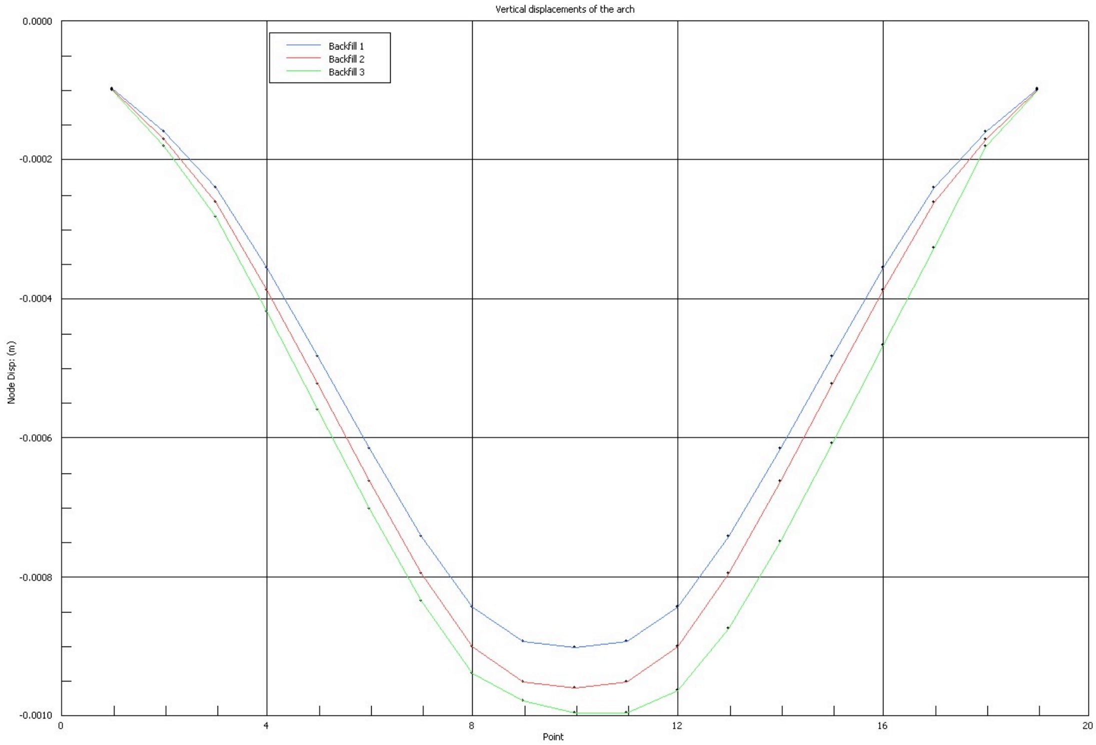


Fig 4.41 - Plate model, vertical displacements of the arch; backfill 1 in blue, backfill 2 in red, backfill 3 in green.

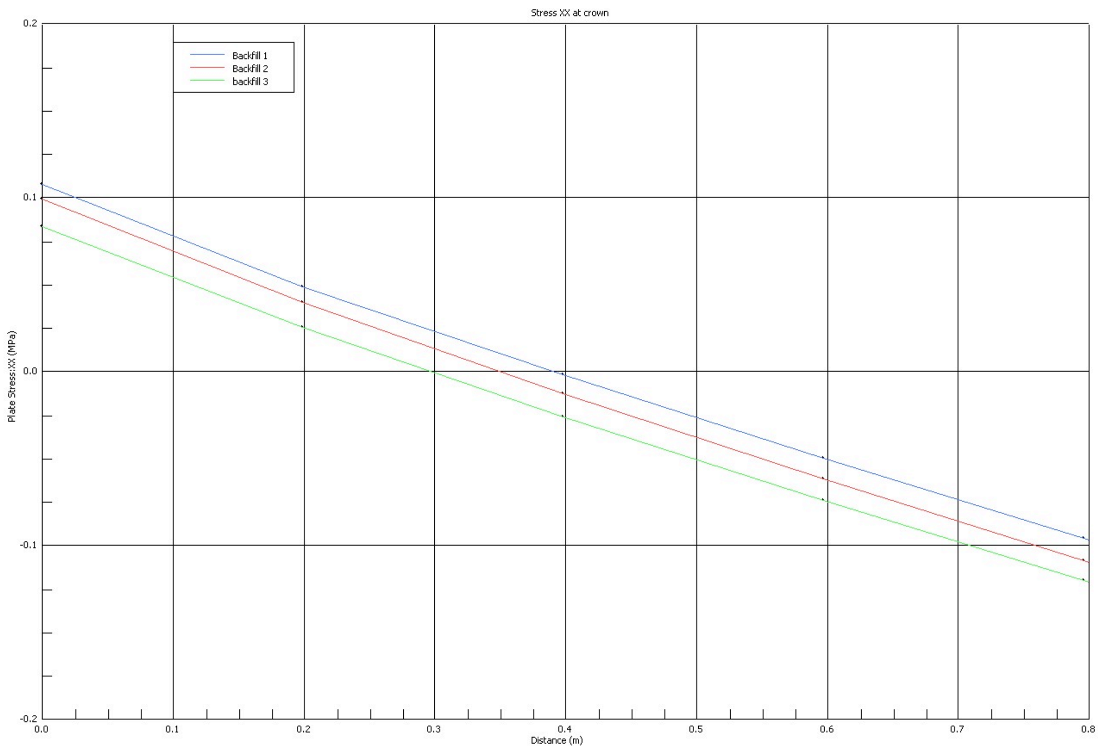


Fig 4.42 - Plate model, stress XX at crown; backfill 1 in blue, backfill 2 in red, backfill 3 in green.

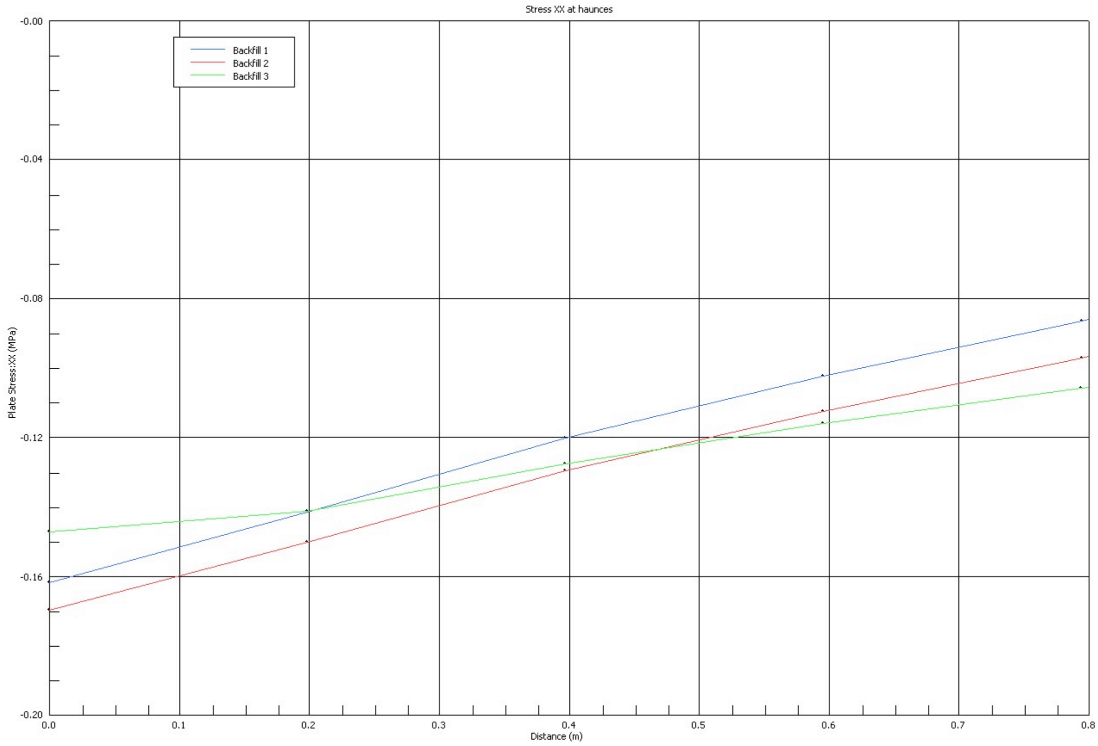


*Fig 4.43 - Plate model, stress XX at haunches;
backfill 1 in blue, backfill 2 in red, backfill 3 in green.*

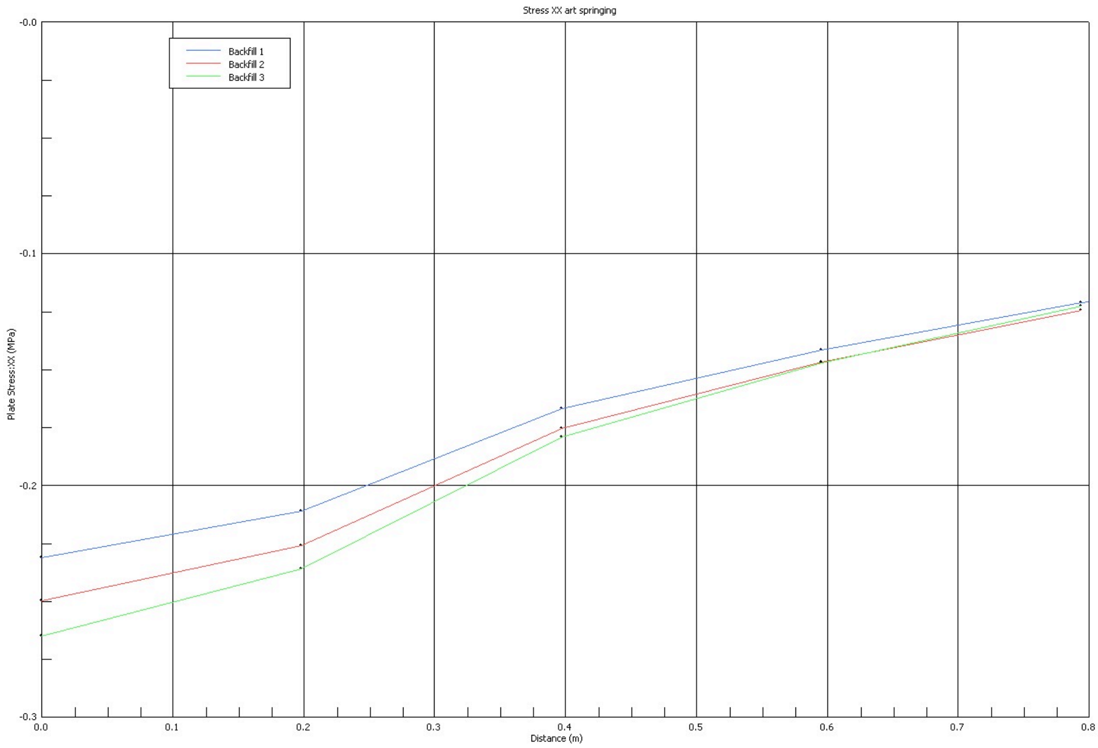


*Fig 4.44 - Plate model, stress XX at springing;
backfill 1 in blue, backfill 2 in red, backfill 3 in green.*

The parametric analysis shows that an increasing of the stiffness of backfill reduces both the vertical displacements of the bridge and the stresses in the arch. Intervention of strengthening that increase the stiffness of the backfill are therefore suggested to improve the service behaviour of the bridge. Multi-scale analysis performed on simple F.E. Model is fit to evaluate the effects of the backfill strengthening on the service behaviour of the bridge.

Conclusions

Multi-scale analysis could be a good instrument to investigate the structural behaviour under service loads of historical masonry arch bridges. Models having different levels of detail allow to choose every time which is the more fit on the base of results purposed and computational efforts.

Results obtained with different analyses - stresses, modes of vibration and mechanisms of collapse - give an exhaustive response in order to evaluate on one hand the safety respect to collapse and on the other the behaviour and the level of stress under service loads of the bridge. Each step of analysis proposed is fast and the comparison between result obtained give a reliable procedure.

On the base of the results obtained, if problems are found, it is possible to evaluate the possibility of further investigation. When necessary, the proposed procedure can be easily improved through the implementation of more detailed methods of analysis, such as non-linear analysis, or more sophisticated models, such as Discrete Element models, which may represent the non linear and heterogeneous character of masonry.

Multi-scale analysis of masonry arch bridge may be coupled with a more precise characterisation of the masonry behaviour through techniques of homogenisation. The procedure may help evaluating the effect, both local and global, of eventual movements of support, which may lead to mechanisms of damage or even collapse of masonry arch bridge.

Moreover the procedure can be used to analyse the behaviour of the bridge both after and before strengthening, in order to evaluate its efficacy and adequacy. It will be necessary to model the interventions of strengthening by the modify of both material properties and the model itself. In particular the procedure may be used to perform parametric analysis aimed to the evaluation of the effect of strengthening of backfill. Increasing of the backfill stiffness provide improvement of the global behaviour of the bridge without altering its aesthetic appearance and without modifying the original structural form. Moreover, intervention made through injection of consolidants in the backfill may be realised without interrupting, or minimising the interruption, of the rail traffic.

Part 2

FEM analysis with homogenisation procedure

FEM is a method which may provide exhaustive results. However it is difficult to properly characterise the masonry material. In fact masonry is an heterogeneous material obtained by the juxtaposition of blocks, which may be made of different materials, natural or artificial, with the interposition or not of mortar joints. The characteristics of masonry material and the possible methods of modelling and analysis have been discussed in the first part of the second section of the thesis. Briefly it is important to remark that the arrangement of blocks, the alignment and the thickness of joints and more in general the geometric texture of the masonry is of fundamental importance in the global behaviour of a masonry structure.

Here a more fine characterisation of masonry material has been performed through a procedure of homogenisation. The homogenisation procedure allows to take into account the micro-structure of the masonry in order to obtain an equivalent continuum. Therefore at the micro-scale level the masonry texture of the barrel vault has been modelled in order to obtain the mechanical properties corresponding to an equivalent orthotropic continuum. Then the mechanical properties obtained have been applied at the meso-scale on a Finite Element model of the bridge. In particular in this work the model represents one single arch: the model is full three-dimensional and very detailed, representing both structural and non-structural components.

Moreover, the homogenisation procedure has been used also to simulate other different possible structural configurations of masonry arch bridges. In particular a very large number of masonry arch railway bridges built in the nineteenth century are made with the external arch rings made of stone voussoirs while the barrel vault is made of bricks. Hence, it was assumed the presence of external stone arch rings made in voussoirs of Istrian stone, the typical stone utilised in the historic venetian buildings. A model of the bridge made with the external stone arch rings and the internal barrel vault made in brick masonry has been realised. The equivalent orthotropic continuum representing the stone arches has been obtained through the homogenisation procedure. An evaluation of the influence of the external arch rings to the global behaviour is here presented. In particular attention has be paid to the transversal behaviour through an anti-symmetric condition obtained by a model of the bridge with only one arch ring.

CS.II.1 The homogenisation procedure

An exhaustive description of homogenisation theory can be found in [*Sanchez-Palencia, 1992; Cecchi and Sab, 2002a and 2002b; Cecchi and Sab, 2004*] and a validation of the proposed procedure can be found in [*Cecchi et al., 2005*]. Here the homogenisation procedure is briefly summarised. The homogenisation procedure for the masonry material may be easier described referring to a simplified two-dimensional model[15]. Masonry is modelled as an heterogenous material obtained by the regular repetition of blocks among which there are mortar joints, on the base of the real texture of blocks. The masonry can be usually defined as a periodic composite continuum. This assumption is true when inside a portion of a masonry wall it is possible to define an elementary cell which provides all the geometrical and mechanical characteristics needed to completely describe the whole masonry wall and which can reproduce the whole masonry wall by its repetition. The elementary cell, named REV, has to be chosen in order to guarantee that the mechanical properties remain constant if the cell is translated.

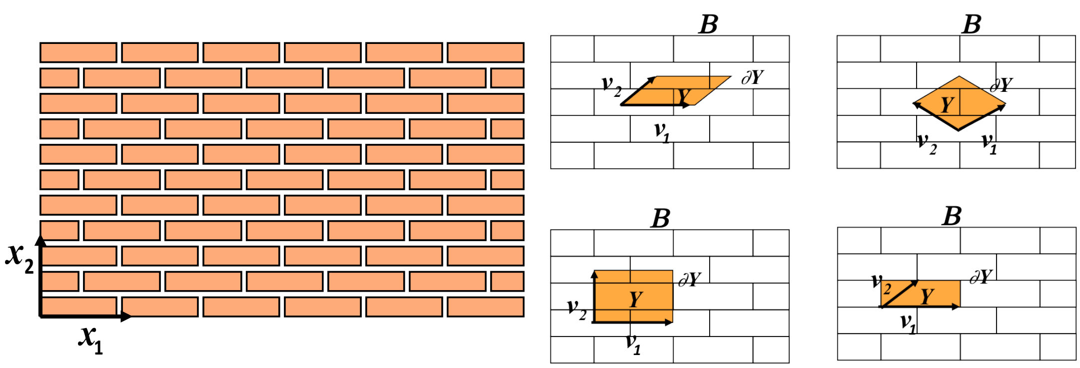


Fig. 4.45 - Identification of the REV:
A portion of masonry walls (left)
Possible different elementary cells (right).

[15] The adopted procedure is three-dimensional, as reported in the following paragraphs.

The condition of periodicity allows to formulate the elastic problem on the REV applying an uniform strain field. The average value of tension obtained by the resolution of the elastic problem applied to the REV provides the values that have to be used for the elastic modulus of a continuum which is equivalent to the original heterogeneous material. In this way it is possible to model the masonry material as an homogeneous continuum but taking into account its characteristics: dimension and quality of blocks, thickness of joints, texture and arrangement of blocks. Once identified the REV and defined the internal composition law of periodicity of the REV, it is possible to define the equivalent constitutive function through the resolution of the elasto-static problem[16].

Thanks to the symmetry of the REV, periodic boundary conditions, which obtained by the sum of two periodic conditions, one symmetric and the other anti-metric, are applied to it. From the average values of the tension obtained by the solution of the elastic problem for the different boundary conditions it is possible to deduce the mechanical properties of an orthotropic continuum [*Lekhnitskii, 1963*] which is equivalent to the original heterogeneous material. Finally the mechanical properties obtained are used in the model at the meso-scale.

Therefore, briefly, the homogenisation procedure consists of four steps:

- Identification of the REV;
- Definition of a law of internal composition;
- Solution of the field problem applied to the REV in order to define the equivalent constitutive function;
- Application of the mechanical properties to the equivalent continuum.

[16] The elasto-static problem consists in the determination of the solution, in terms of stress, strain and displacements of a body with an assigned geometry and subjected to loads and deformations. The solution must be in accordance with the equilibrium relations between external loads and internal tensions, the relation of congruence between kinematic displacements and strains and the constitutive function.

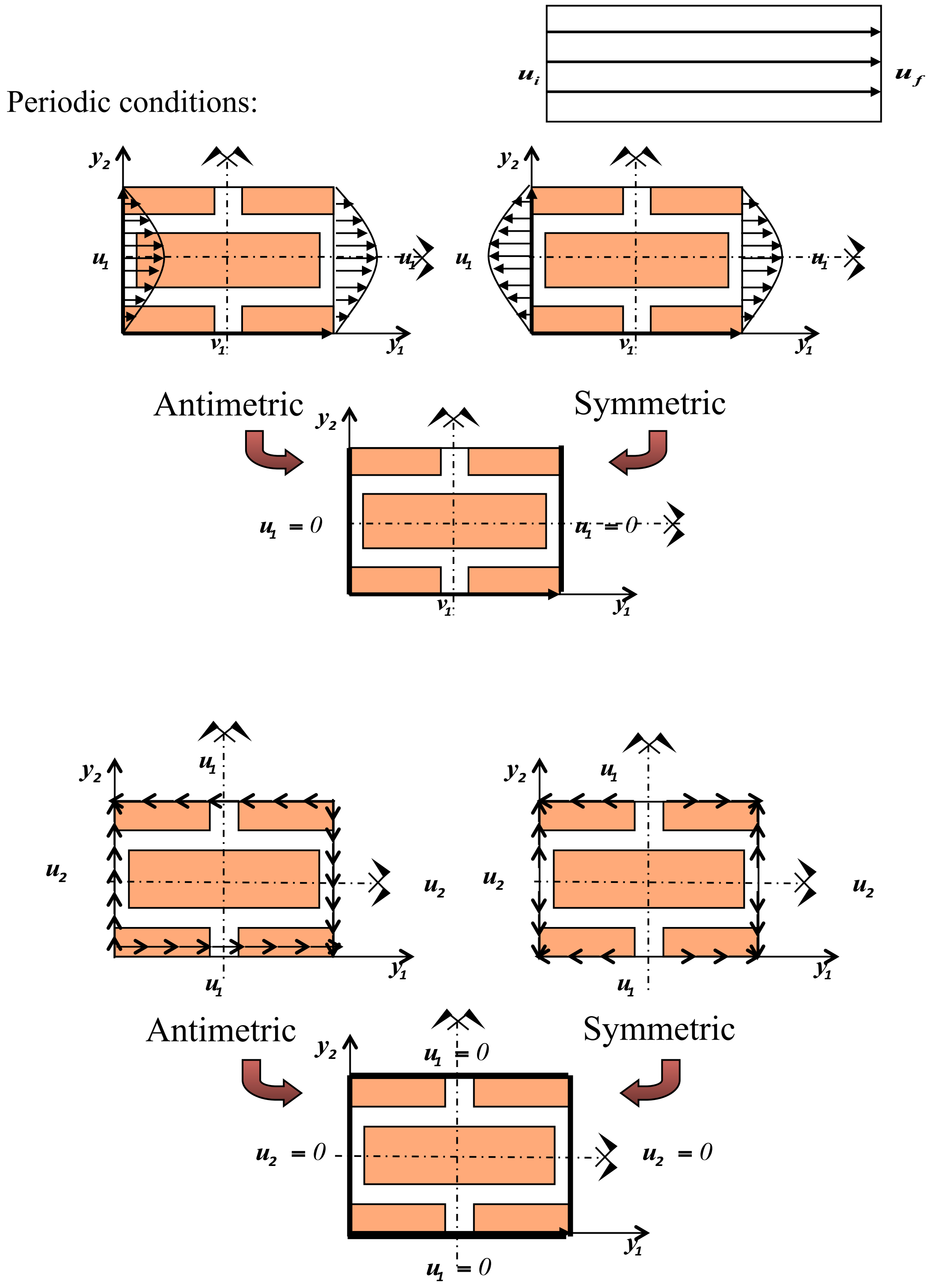


Fig. 4.46 - Periodic boundary condition are applied to the a two-dimensional REV:
a) strain field applied in direction y_1 (on top);
b) strain field applied in directions y1y2 (below).

The homogenisation of the masonry barrel vault

The homogenisation procedure has been applied to obtain an orthotropic continuum equivalent to the original masonry texture of the barrel vault of the Venice Trans-Lagoon Bridge. The elementary cell used for the homogenisation of the bridge has been identified on the base of bricks arrangement. However some hypothesis have been formulated on the base of the visible bricks pattern and on the base of drawings of the original project due to the impossibility of a direct view of the internal bricks. Considering the arrangement of blocks along the arch ring two rows of bricks have been identified.

The barrel vault has been considered with a constant thickness of 80 cm, however in the reality it has a lower thickness at crown and an higher thickness at abutments. The bricks have the typical dimensions of historical venetian bricks, equal to (25 x 12 x 5) cm. The thickness of mortar joints is equal to 1 cm. Both bricks and mortar have been modelled as isotropic brick elements. The mechanical properties adopted for the model of the elementary cell are:

E = 5000 (MPa)	$\nu = 0.2$	$\rho = 1800\ (kg/m^3)$

Table 4.11 - Mechanical properties of bricks

E = 1000 (MPa)	$\nu = 0.2$	$\rho = 1800\ (kg/m^3)$

Table 4.12 - Mechanical properties of mortar

The texture of the masonry barrel vault and the elementary periodic cell identified are reported in the following figures. Considering the small dimensions of the elementary cell respect to the whole barrel vault the problem has been linearised. A global Cylindrical Coordinates System (Z, R, θ) with origin in the centre of the the masonry barrel vault is assigned. Hence, the elementary vault element shown in the following figure is considered. Let η_1, and η_2 be a local Curvilinear Coordinates Surface System with origin in a vertex of the element and y_3 be an axis orthogonal to the surface. If the geometrical dimensions of the element are very small in comparison with the curvature of the vault, a local Orthonormal Coordinate System may be fixed: y_1 and y_2 are the tangent axes to η_1 and η_2 axes, respectively, while y_3 is the axis in the normal direction coincident with the y_3 axis orthogonal to the surface.

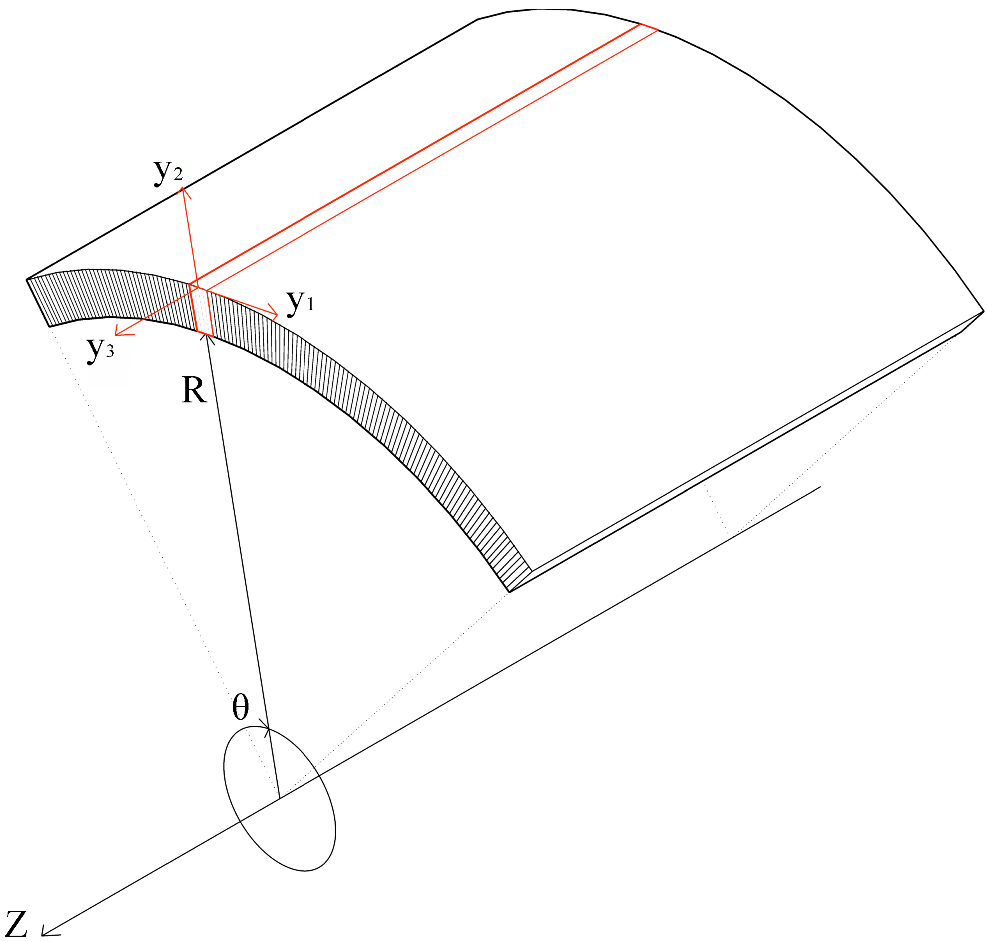


Fig. 4.47 - Linearisation of the problem:
the Cylindrical Coordinates System (Z, R, θ) and the Orthonormal Coordinate System (y_1, y_2, y_3)

Fig. 4.48 - Particular of the 97th arch: the texture of the original masonry

Fig. 4.49 - Arrangement of the bricks along the arch ring

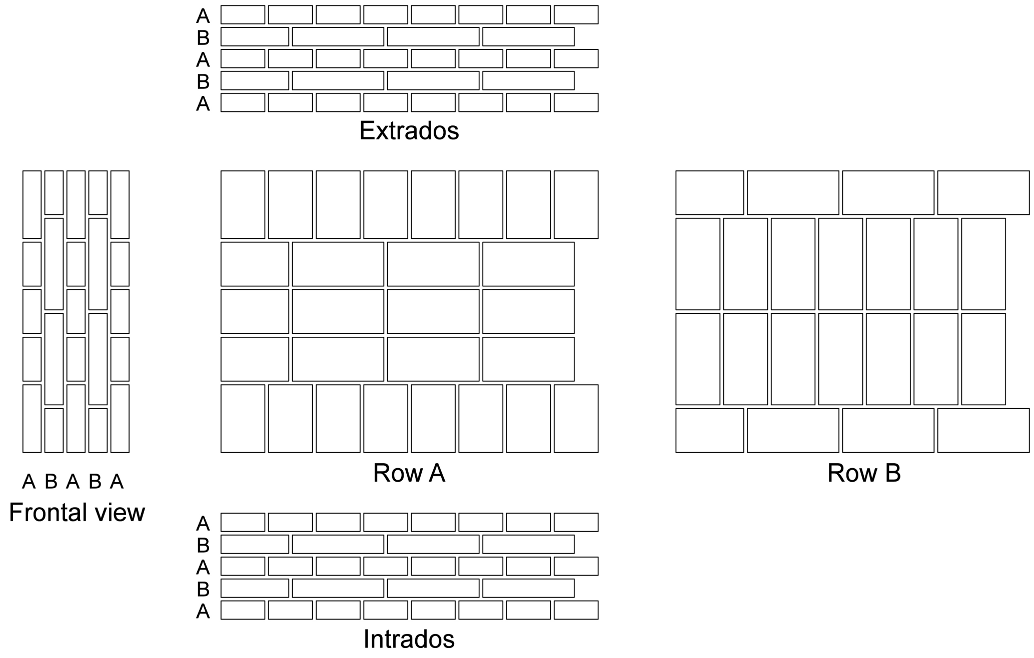


Fig. 4.50 - Hypothetic arrangement of bricks, the two rows of bricks

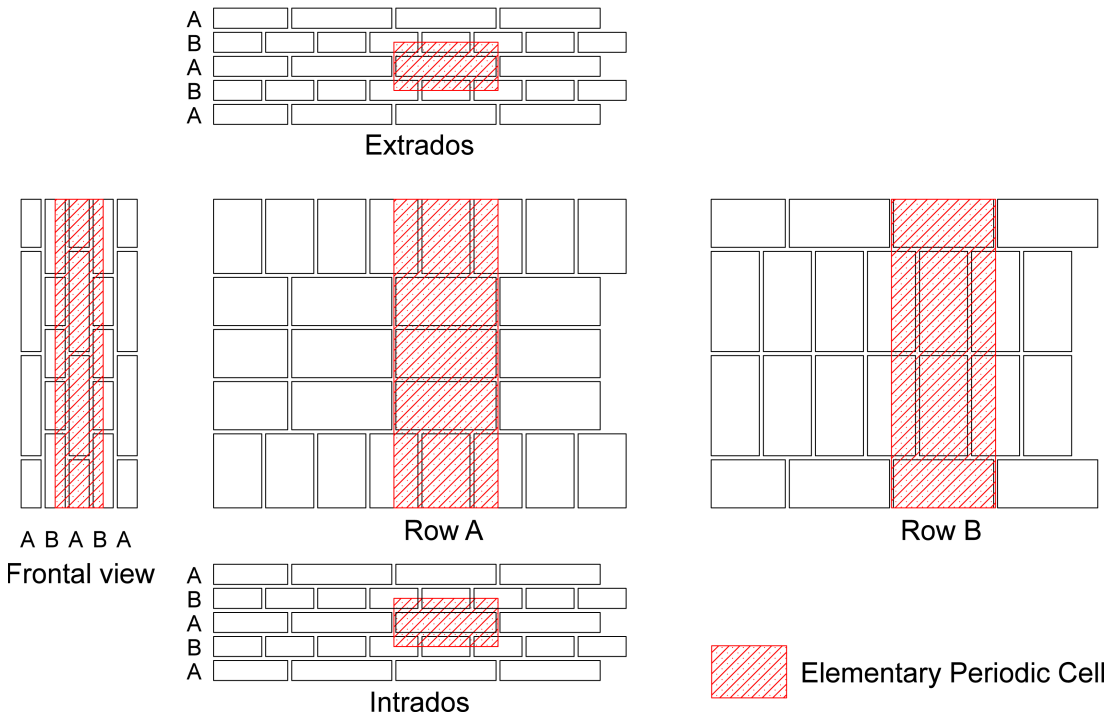


Fig. 4.51 - Identification of the REV

The elementary cell is periodic in the directions y_1 and y_2. There are three planes of symmetry, hence it is possible to model only 1/8 of the REV. Model consists of 24948 brick elements. The model of the cell and the boundary conditions are reported in the following figures.

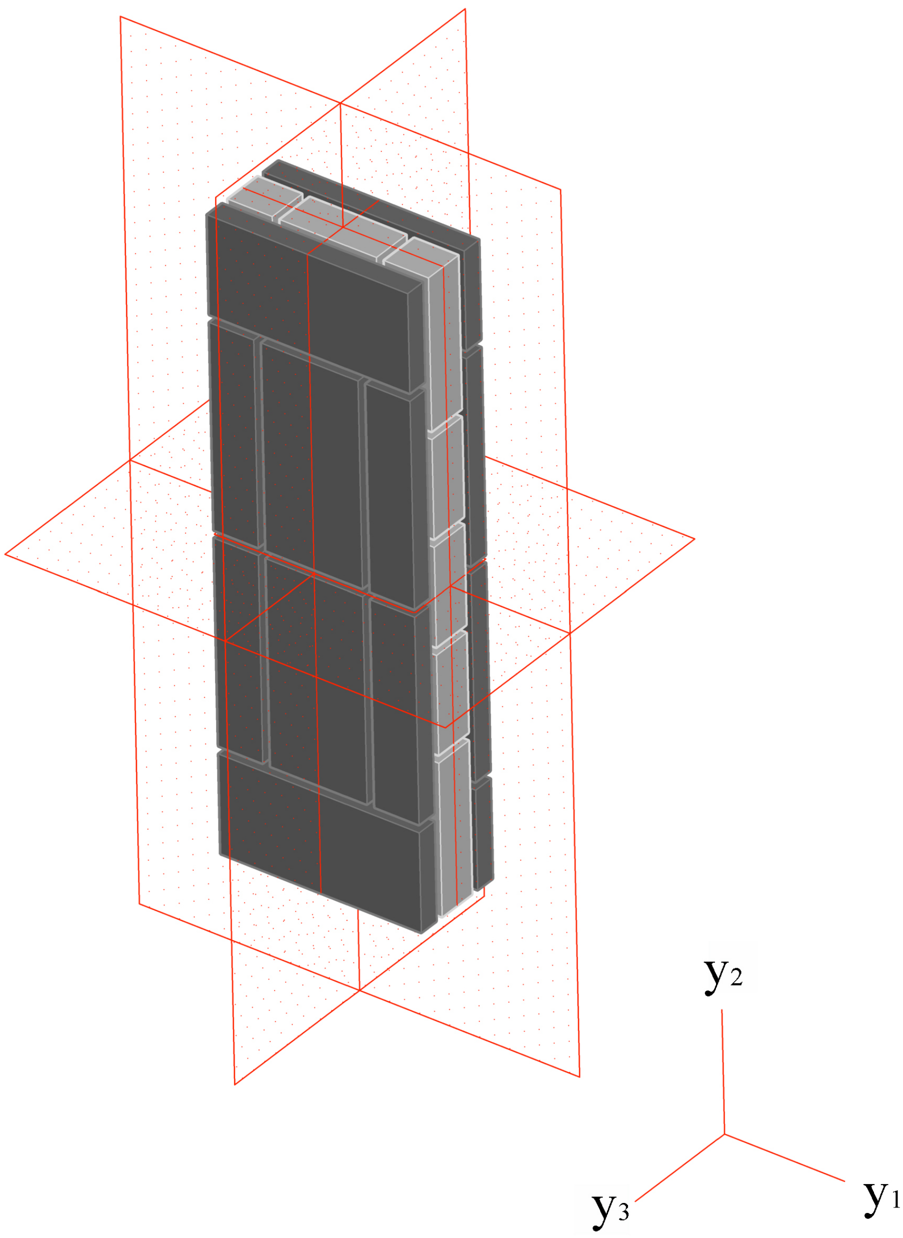


Fig. 4.52 - Three dimensional representation of the elementary cell and the three plane of symmetry.

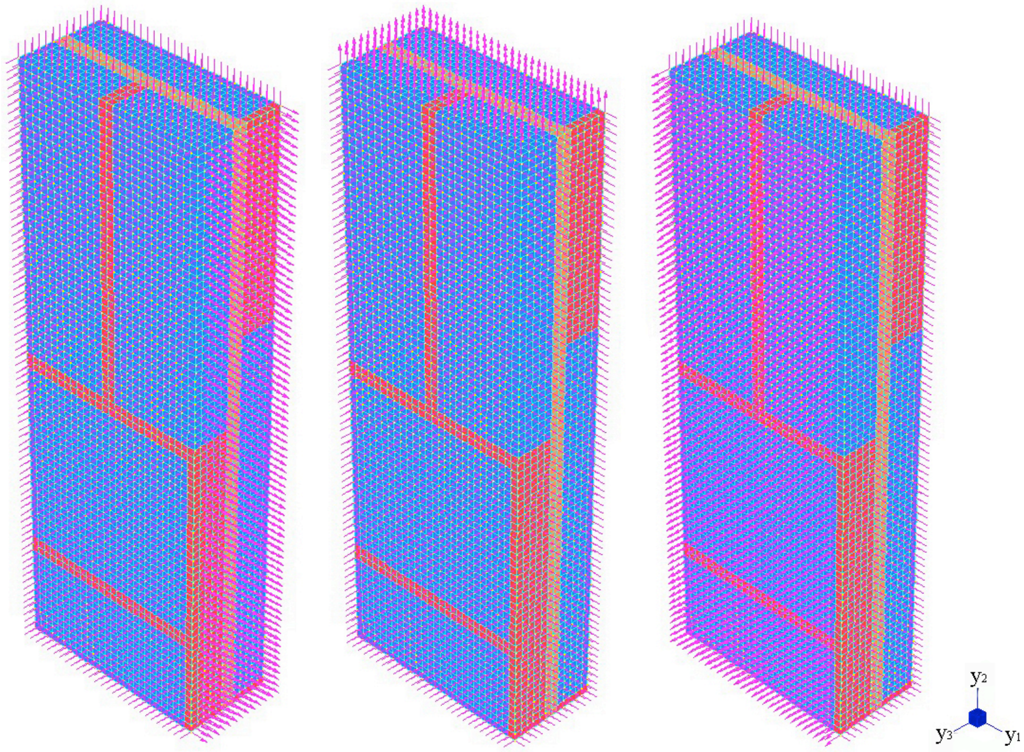


Fig 4.53 - Model of the cell, bricks in blu and mortar joints in red and orange, imposed displacements boundary conditions:
a) direction y_1 (left); b) direction y_2 (middle); c) direction y_3 (right).

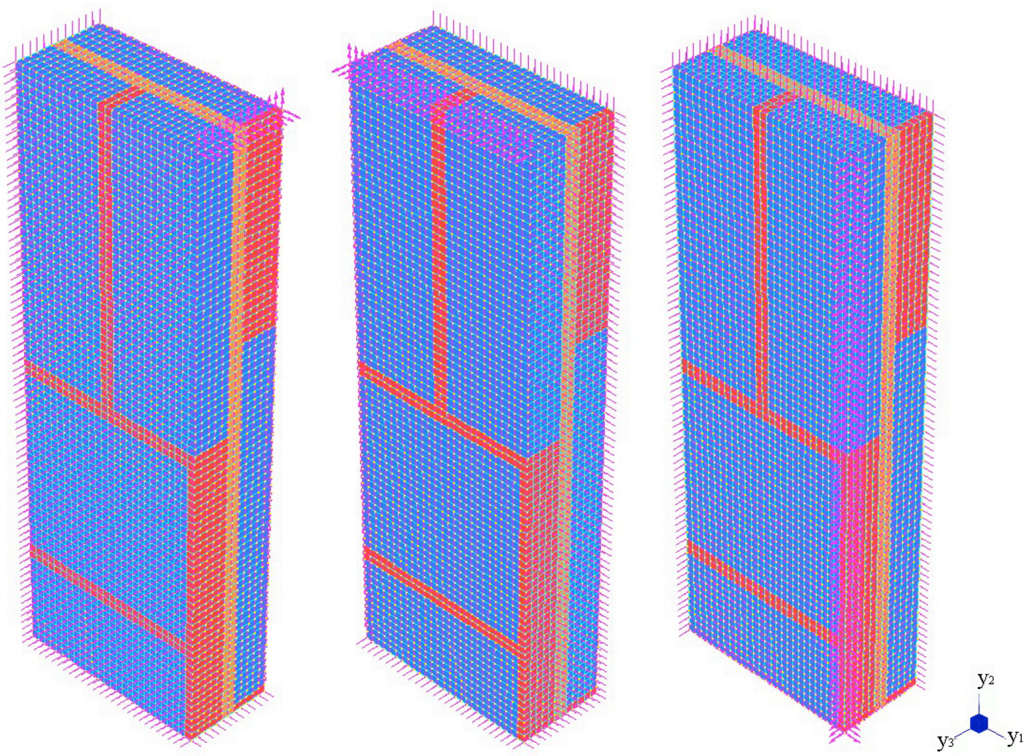


Fig 4.54 - Model of the cell, bricks in blu and mortar joints in red and orange, imposed displacements boundary conditions:
a) direction y_1-y_2 (left); b) direction y_2-y_3 (middle); c) direction y_1-y_3 (right).

The values obtained for the barrel vault through the homogenisation procedure for the equivalent orthotropic continuum are reported in the table below:

Young's Modules (MPa)		
E11	E22	E33
3480	3592	2861

Tangential Modules (MPa)		
G12	G23	G31
1340	1100	1120

Poisson's Coefficients		
v12	v23	v31
0.18	0.194	0.158

Table 4.13 - Values obtained through the homogenisation procedure for the barrel vault

The mechanical properties obtained have been used to model the barrel vault. In this way the equivalent continuum takes into account the real texture of the masonry used to build the Venice Trans-Lagoon Bridge.

CS.II.2 The effect of the presence of external stone arch rings

In general, the typical masonry arch railway bridges built almost entirely between the second half of the XIX Century and the first half of the XX Century have been constructed with a typical form [*Torre, 2003*]: the barrel vault is usually made of brick masonry while the arch rings are made by stone voussoirs. In this case, sometimes, the external arch rings may have a greater thickness respect to the masonry barrel vault. However several bridges are completely made of brick masonry, with the same thickness along the width. The Venice Trans-Lagoon Bridge is of this typology. Instead only rarely this typology of bridges were realised completely made of stones, which was typical in the monumental historical bridges. Considering the elevated number of masonry arch bridges that are still in service [*UIC, 2005*], an evaluation of their behaviour under service load related to their structural configuration may provide interesting information in the assessment of their load bearing capacity in order to ensure their conservation or to improve their structural performances.

Here an evaluation of the sensitivity to the presence or not of the external stone arch rings respect to the global behaviour of the bridge is presented. In order to achieve this purpose a multi-scale analysis is carried out on models of an arch bridge representing the different configurations:

- The barrel vault completely made of brick masonry;
- The external arch rings made of stone voussoirs and the barrel vault made of bricks.

In the second case a further investigation is proposed: an un-symmetrical condition due to the presence of external arch rings made in stone voussoirs only on one side of the bridge. The skew condition may have consequences to the global behaviour of the bridge respect to the traffic load.

The traffic loads are provided by Italian Railway regulations. They are applied in different positions in order to simulate symmetrical or un-symmetrical conditions. In particular loads are applied on the whole arch or only on one side, both in longitudinal and in transversal direction.

The same homogenisation procedure which has been adopted in the previous paragraph has been used to take into account the micro-structure of the stone arch rings in order to obtain an equivalent continuum. Here at the micro-scale level both the masonry texture of the barrel vault and the stone texture of the arch rings have been modelled in order to obtain the corresponding equivalent orthotropic continua. Then the mechanical properties obtained have been applied at the meso-scale on a full three-dimensional Finite Element model of the bridge. The models represent the three configuration previously described.

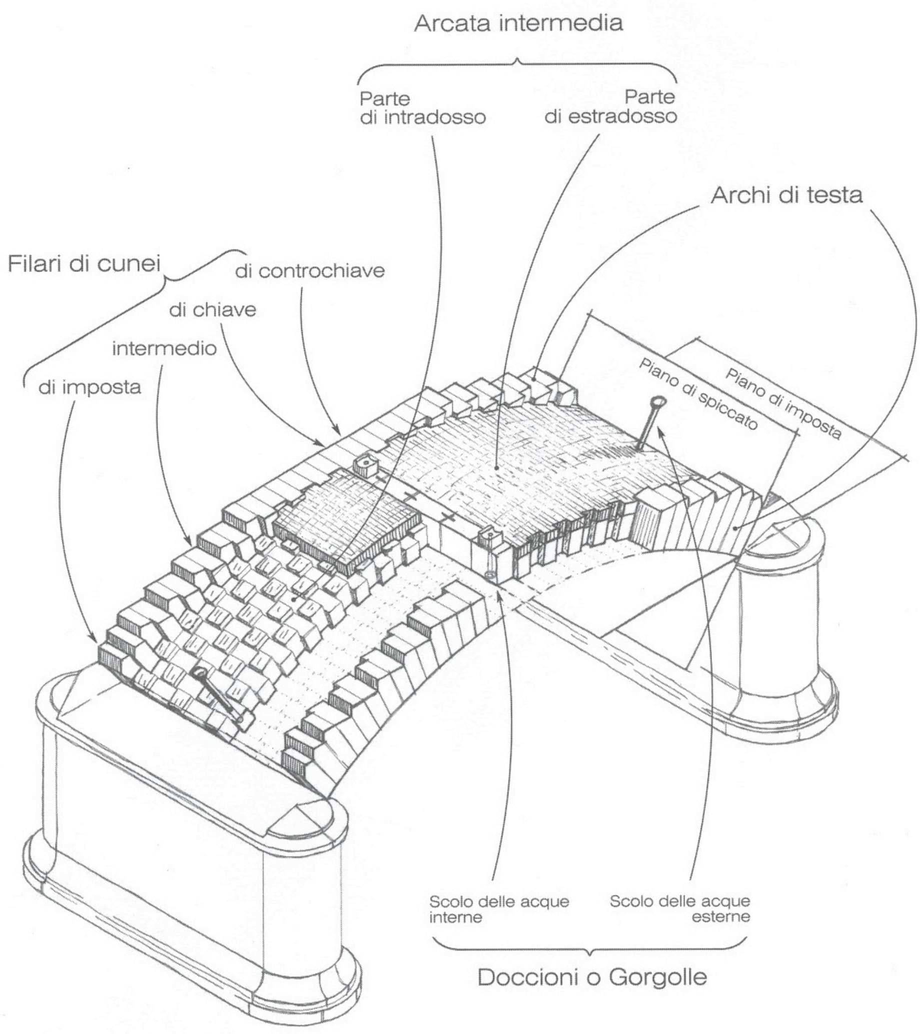


Fig. 4.55 - Typical configuration of a masonry arch bridge built during the XIX Century: it is possible to notice the different texture used for the barrel vault and the arh rings, taken from [Torre, 2003].

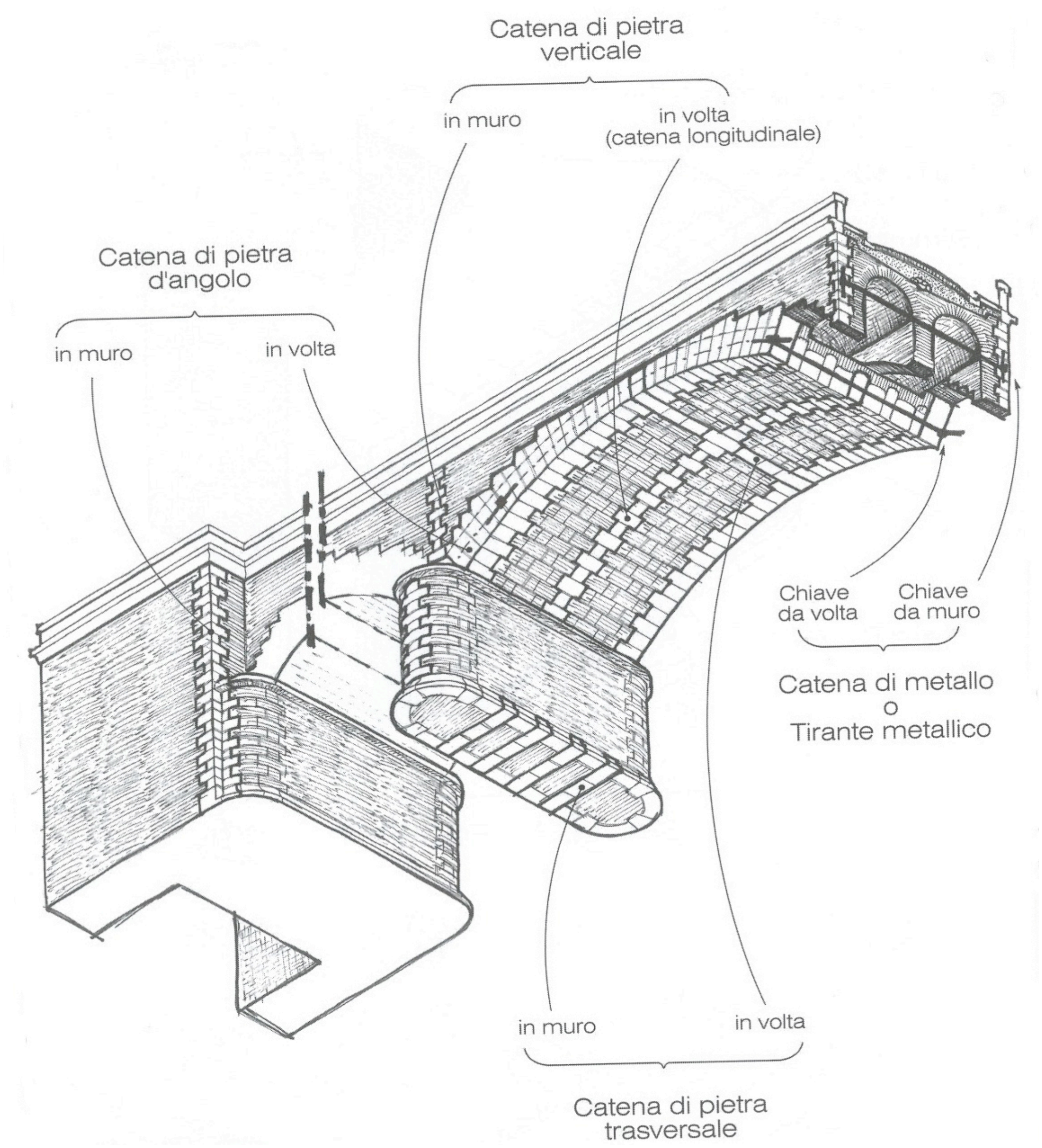


Fig. 4.56 - Typical configuration of a masonry arch bridge built during the XIX Century: it is possible to notice the presence of transversal and longitudinal stone "chains", taken from [Torre, 2003].

The homogenisation of the stone arch rings

The structural form of the Venice Trans-Lagoon Bridge is the one in which the barrel vault is completely made by brick masonry. The real arrangement of blocks has been adopted in the homogenisation of the barrel vault, as described in the previous paragraph. Instead the stone voussoirs arch rings have been modelled assuming that they are made with blocks of Istria's Stone, the typical stone used in the historical venetian architecture, which is present in other parts of the Venice Trans-Lagoon Bridges, such as piers and foundation. The homogenisation procedure has been applied to obtain an orthotropic continuum equivalent to an hypothetic texture of a stone arch.

The thickness of the arch rings has been assumed constant and equal to the thickness of the barrel vault, therefore equal to 80 cm. The arch rings are assumed to be made of blocks having the reported measures: (80 x 40 x 30) cm. The thickness of mortar joints is equal to 0.5 cm. Both bricks and mortar have been modelled as isotropic brick elements. The mechanical properties adopted for the model of the elementary cell are:

$E = 50000$ *(MPa)*	$\nu = 0.2$	$\rho = 2700$ (kg/m^3)

Table 4.11 - Mechanical properties of stone voussoirs

$E = 1000$ *(MPa)*	$\nu = 0.2$	$\rho = 1800$ (kg/m^3)

Table 4.12 - Mechanical properties of mortar

The texture of the hypothetic stone arch ring and the elementary periodic cell identified are reported in the following figures. Also in this case, as previously described, considering the small dimensions of the elementary cell respect to the whole arch ring the problem has been linearised. A global Cylindrical Coordinates System (Z, R, θ) with origin in the centre of the stone arch is assigned. Hence, the elementary vault element shown in the following figure is considered. Let η_1, and η_2 be a local Curvilinear Coordinates Surface System with origin in a vertex of the element and y_3 be an axis orthogonal to the surface. If the geometrical dimensions of the element are very small in comparison with the curvature of the arch, a local Orthonormal Coordinate System may be fixed: y_1 and y_2 are the tangent axes to η_1 and η_2 axes, respectively, while y_3 is the axis in the normal direction coincident with the y_3 axis orthogonal to the surface.

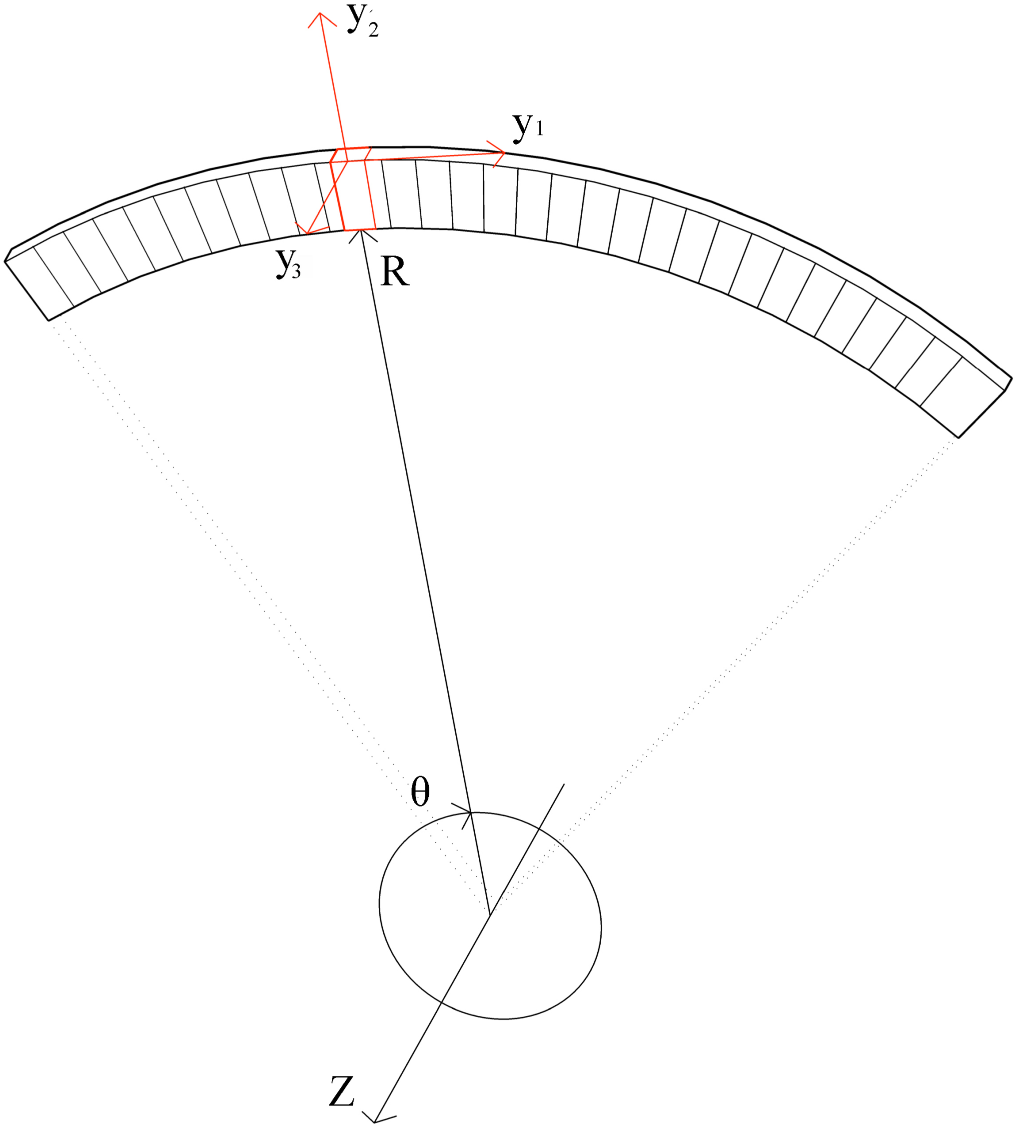


Fig. 4.57 - Linearisation of the problem:
the Cylindrical Coordinates System (Z, R, θ) and the Orthonormal Coordinate System (y_1, y_2, y_3)

The elementary cell is periodic in the direction y_2. There are three planes of symmetry, hence it is possible to model only 1/8 of the REV. Model consists of 6300 brick elements. The model of the cell and the boundary conditions are reported in the following figures.

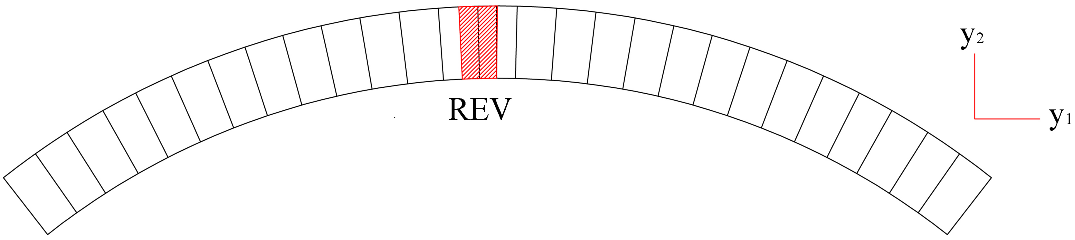


Fig. 4.58 - Arrangement of stone voussoirs and identification of the REV

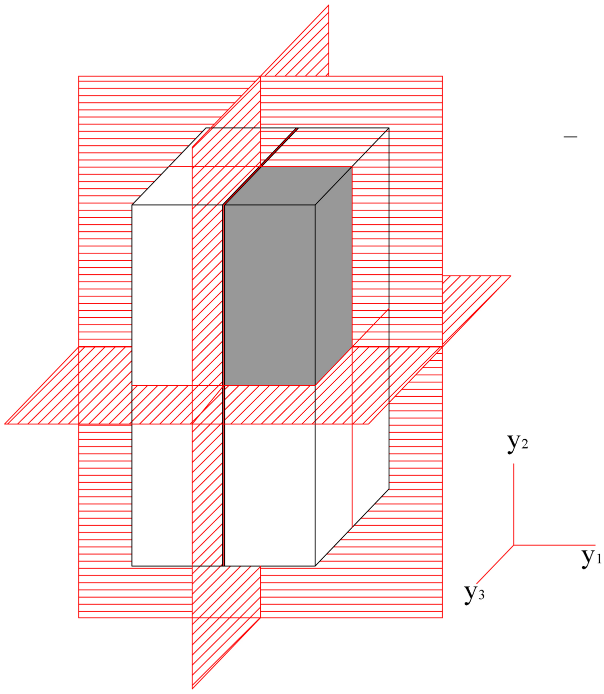


Fig. 4.59 - The elementary cell and the three plane of symmetry.

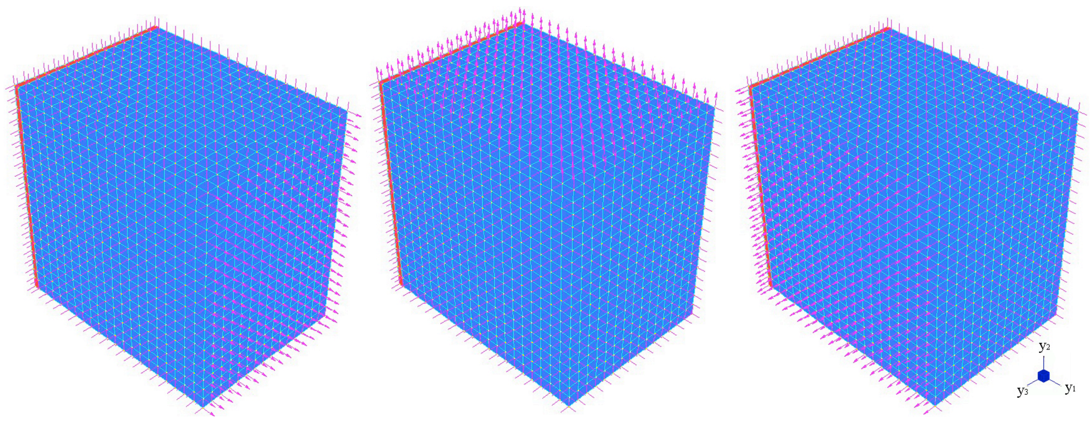


Fig 4.60 - Model of the cell, stone block in blu and mortar joint in red, imposed displacements boundary conditions:
a) direction y_1 (left); b) direction y_2 (middle); c) direction y_3 (right).

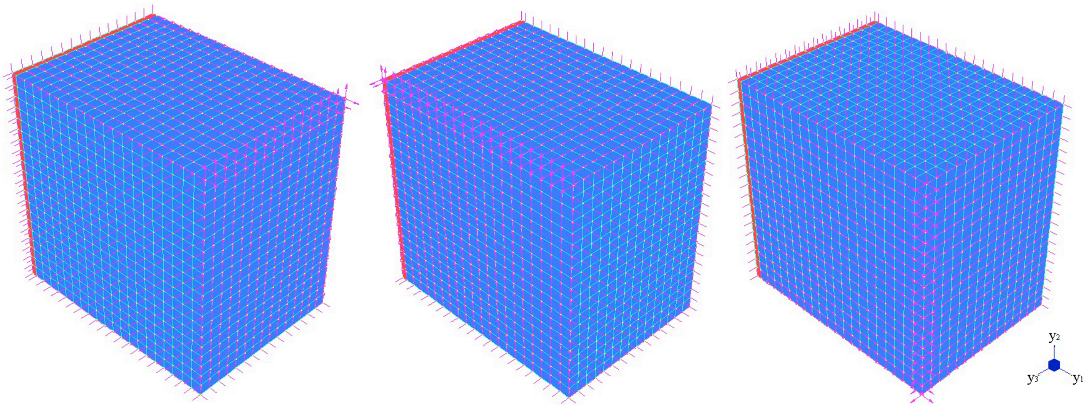


Fig 4.61 - Model of the cell, stone block in blu and mortar joint in red, imposed displacements boundary conditions:
a) direction y_1-y_2 (left); b) direction y_2-y_3 (middle); c) direction y_1-y_3 (right).

The values obtained for the stone arch rings through the homogenisation procedure for the equivalent orthotropic continuum are reported in the table below:

Young's Modules (MPa)		
E11	E22	E33
24160	46670	46670
Tangential Modules (MPa)		
G12	G23	G31
9490	9490	20300
Poisson's Coefficients		
v12	v23	v31
0.076	0.41	0.148

Table - Values obtained through the homogenisation procedure for the barrel vault

Starting from the observation that, especially in the case of historical masonry, the blocks are generally much stiffer than the mortar and mortar joints show a very small thickness if compared with the size of the blocks, a simplified solution in an “analytical” form may be found. In particular, in the modelling of the REV of the stone arch ring the ratio between the stiffness of blocks and the one of mortar is very high, equal to 50:1, such as the ratio between the thickness of block and the one of the mortar joint, equal to 80:1. Moreover the typology of texture of the stone arch can be considered very similar to the so called multi-layers masonry. Here a comparison with an analytical model [*Cecchi and Sab, 2002a*] has been performed to validate the homogenisation procedure. Elastic block and cohesive joint have been assumed. The results obtained by the numerical homogenisation procedure are congruent with the ones provided analytically.

Analysis

The mechanical properties obtained for the masonry barrel vault and for the stone arch rings have been used to model on single arch of the bridge. In this way the barrel vault and the stone arch rings have been modelled as orthotropic continua. The mechanical properties used for all the other structural elements of the bridge are the same previously used in the multi-scale analyses. Other elements are therefore modelled as isotropic bricks. Models represent one single arch of the Venice Trans-lagoon Bridge. The Model consist of 36672 bricks elements. The three different structural forms are considered:

- The real case, in which the barrel vault is completely made of brick masonry;
- An hypothetic case, in which the external arch rings are made of stone voussoirs;
- An un-symmetrical condition, in which only one external arch is made of stone voussoirs.

On the models have been applied the loads provided by technical Italian regulation. In particular, the bridge has been loaded by the two train provided by regulation, LM71 and SW2. The loads have been applied in order to evaluate the longitudinal and the transversal behaviour of the bridge. The combination of loads used are the following:

- The self weight plus the dead loads;
- Longitudinal loads:
    - Whole span loaded by LM71;
    - Half span loaded by SW2
- Transversal loads:
    - One side loaded by LM71;
    - One side loaded by SW2.

A parametric linear static analysis has been performed to evaluate the influence of the presence of the stone arch rings respect to the global behaviour of the bridge in service condition. Diagrams of results are reported in the following figures. In particular are reported:

- Stresses in X direction.
- Vertical displacements.

The values of stresses and displacements are taken in:

- Transversal section at the intrados of crown;
- Longitudinal section at the extrados in the middle of the deck.

The loads and the transversal sections in which values are taken are reported in the next figure.

In the diagrams blu lines represent the barrel vault completely made of bricks, green lines represent the bridge with two external stone arch rings and the barrel vault in brick masonry while the red lines represent the un-symmetrical condition in which only one external stone arch ring is present, on the left side of the bridge.

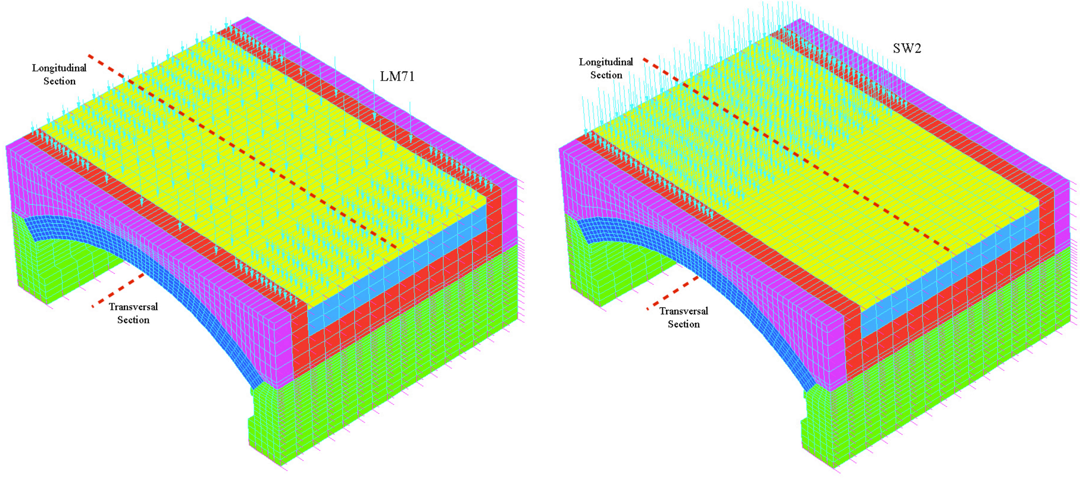


Fig. 4.62 - Longitudinal loads and indication of the sections:
a) LM71 on the whole span (left); b) SW2 on half span (right).

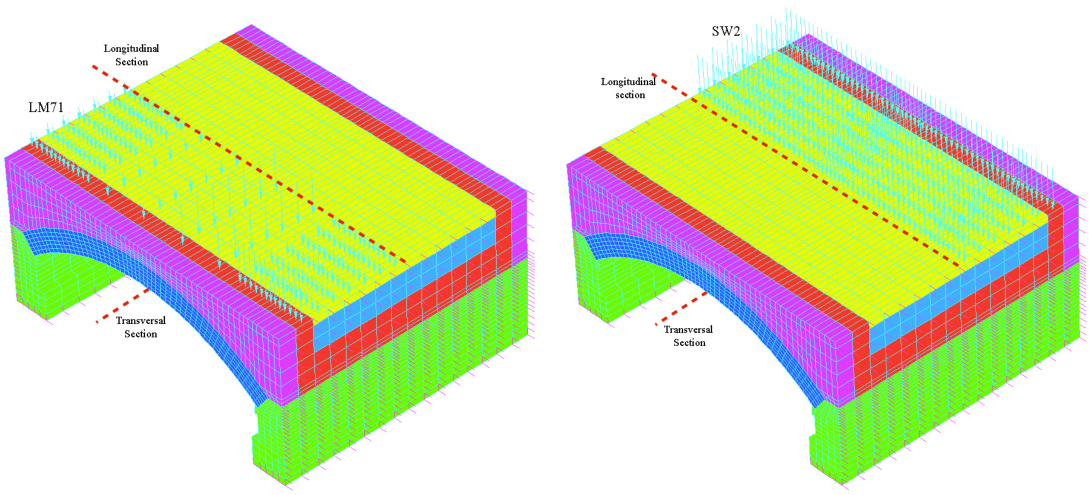


Fig. 4.63 - Longitudinal loads and indication of the sections:
a) LM71 on the left raili (left); b) SW2 on the right rail (right).

Transversal behaviour

Vertical displacements at intrados of crown

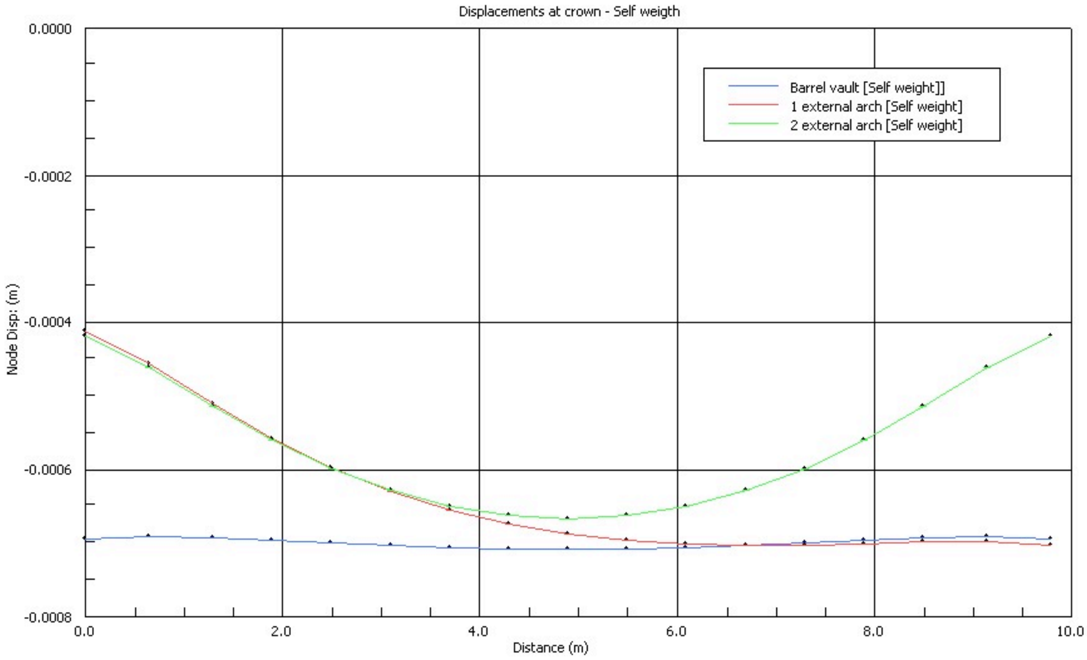


Fig. 4.64 - Vertical displacement at the intrados of crown, self weight and dead load

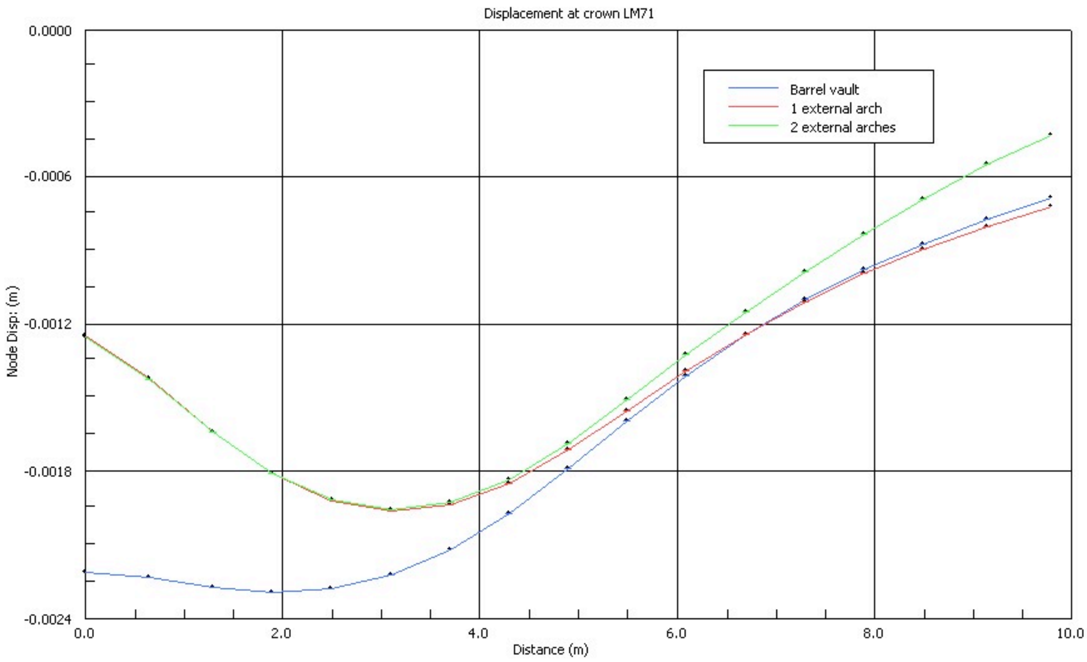


Fig. 4.65 - Vertical displacement at the intrados of crown, LM71 applied to the left side

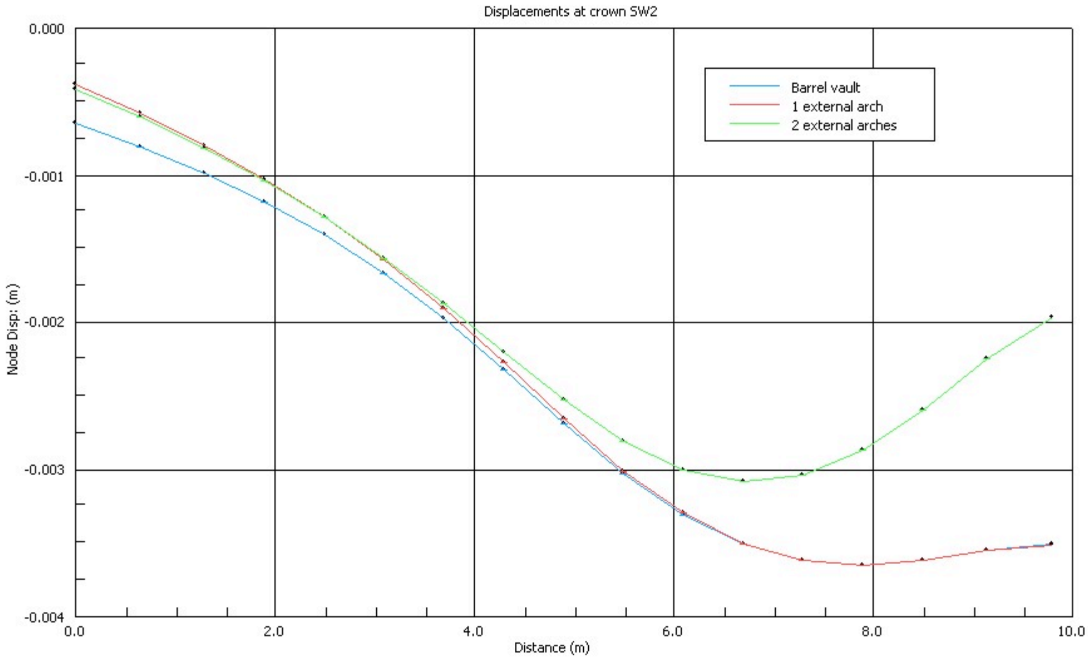


Fig. 4.66 - Vertical displacement at the intrados of crown, SW2 applied on the right side

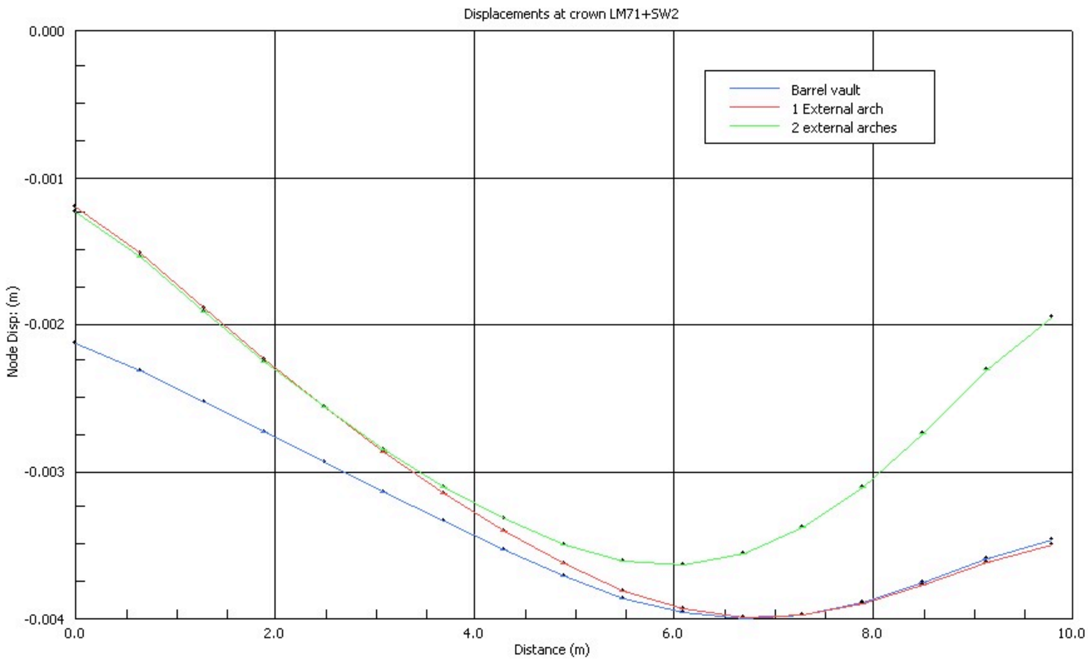


Fig. 4.67 - Vertical displacement at the intrados of crown, LM71 + SW2

Transversal behaviour

Stress XX at intrados of crown

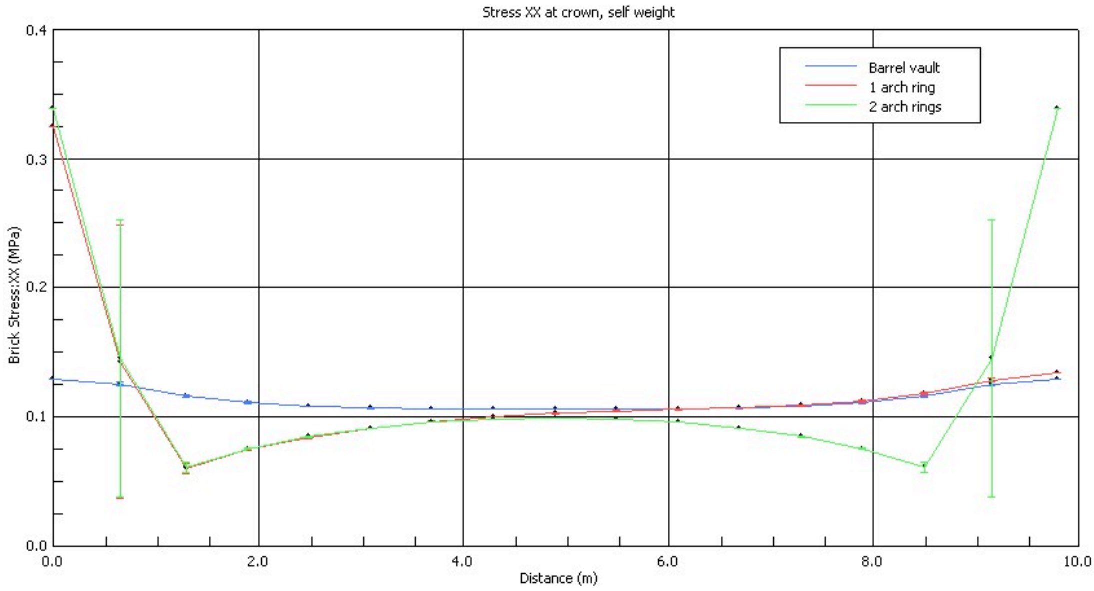


Fig. 4.68 - Stress XX at the intrados of crown, self weight and dead load

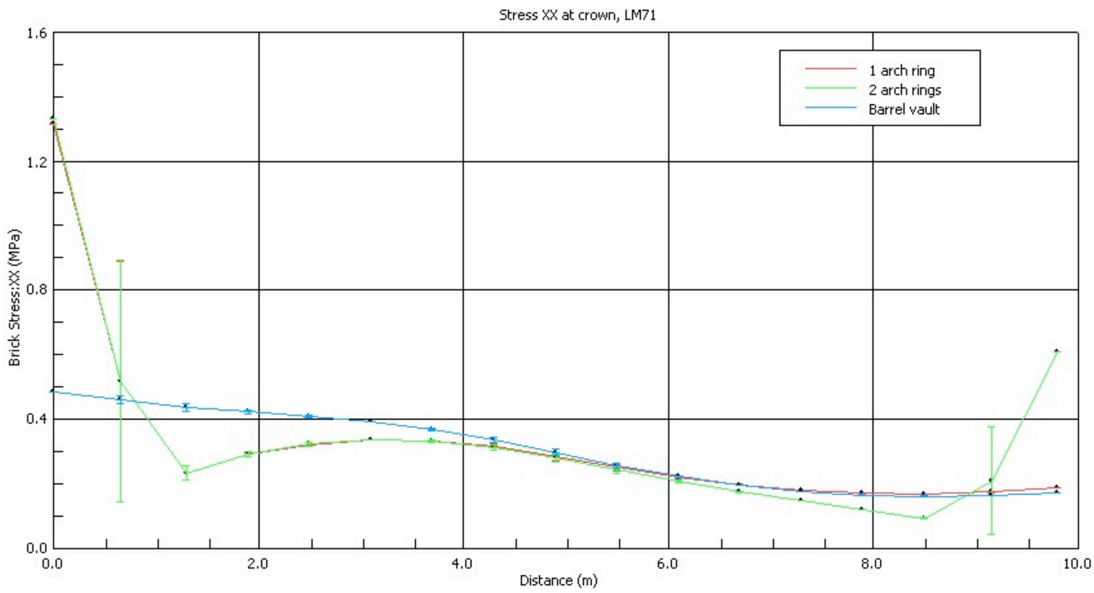


Fig. 4.69 - Stress XX at the intrados of crown, LM71 applied to the left side

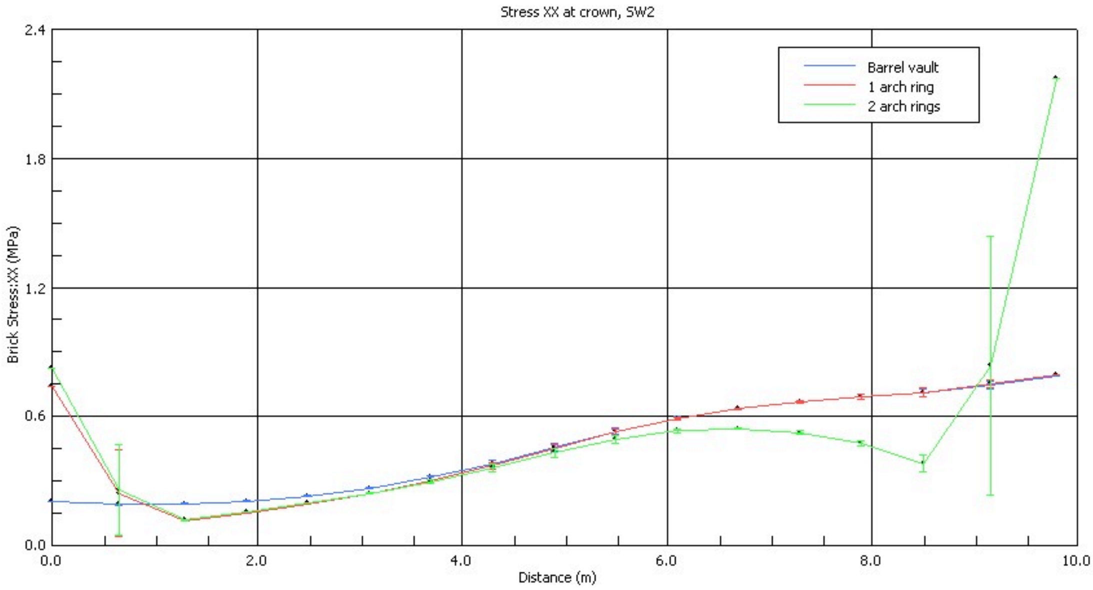


Fig. 4.70 - Stress XX at the intrados of crown, SW2 applied on the right side

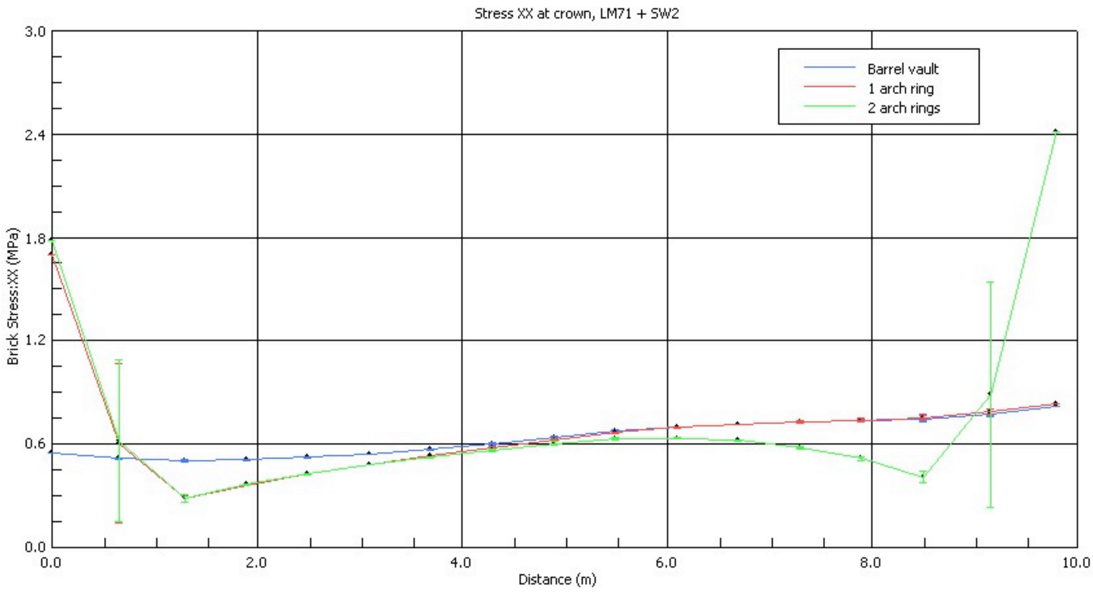


Fig. 4.71 - Stress XX at the intrados of crown, LM71 + SW2

Longitudinal behaviour

Vertical displacements at extrados in the middle of the deck

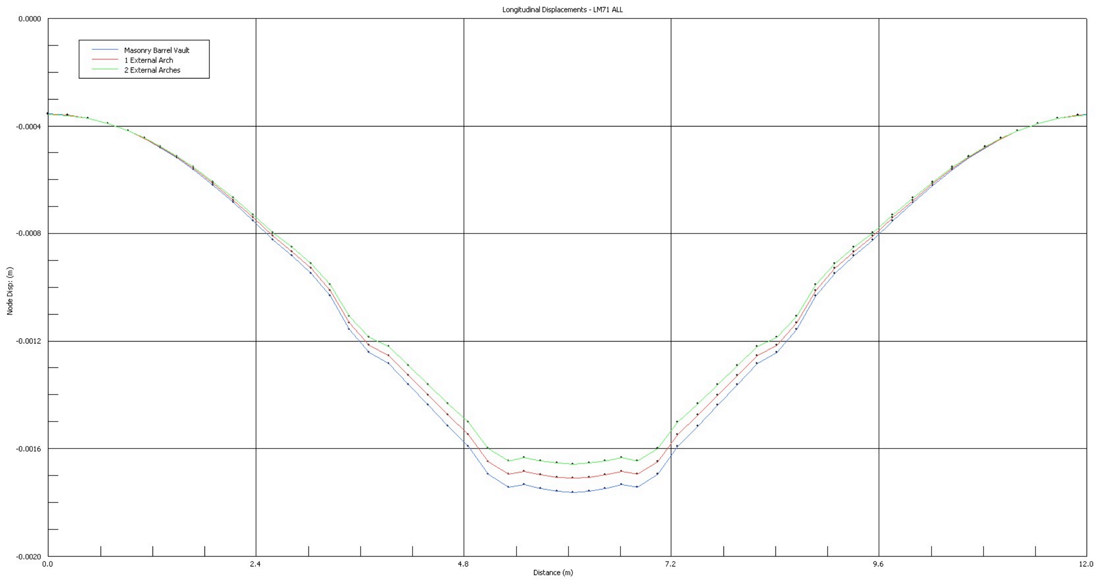


Fig. 4.72 - Vertical displacements at extrados in the middle of the deck, LM71 on the whole span

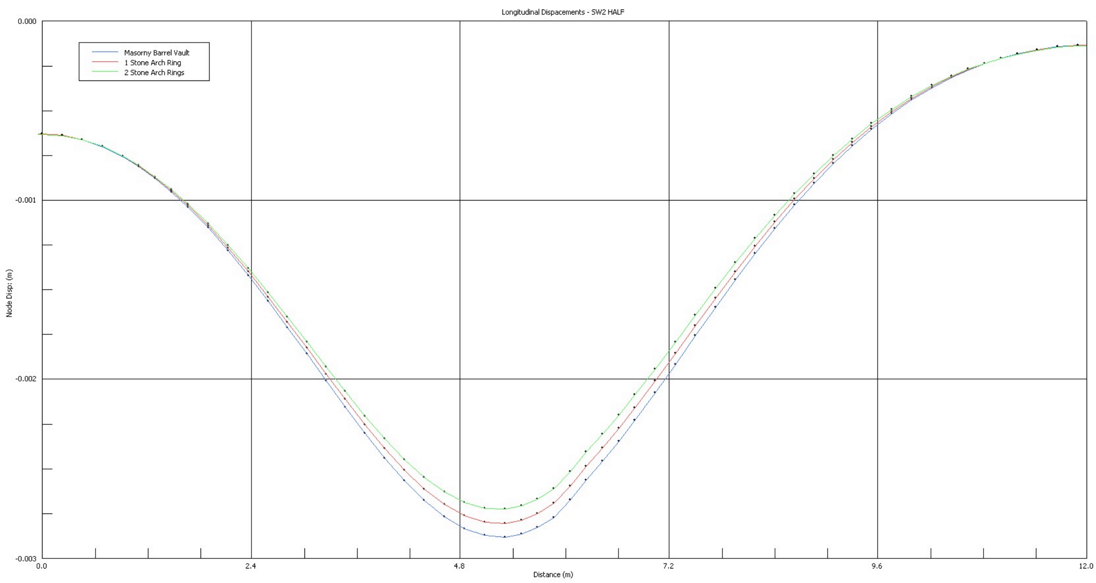


Fig. 4.73 - Vertical displacements at extrados in the middle of the deck, SW2 on half span

Longitudinal behaviour

Stresses in X direction at extrados in the middle of the deck

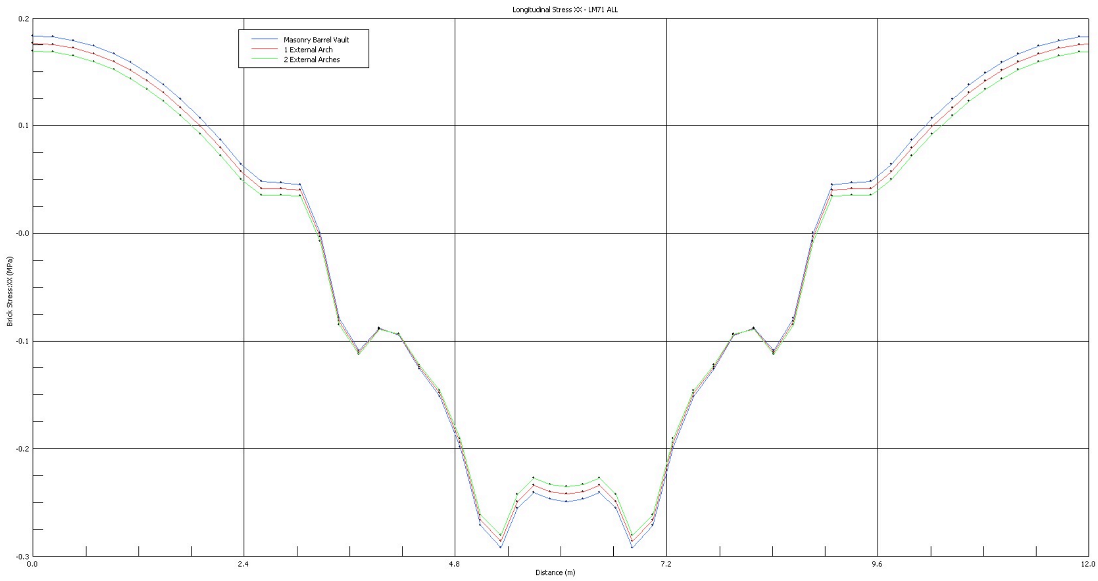


Fig. 4.74 - Stresses XX at extrados in the middle of the deck, LM71 on the whole span

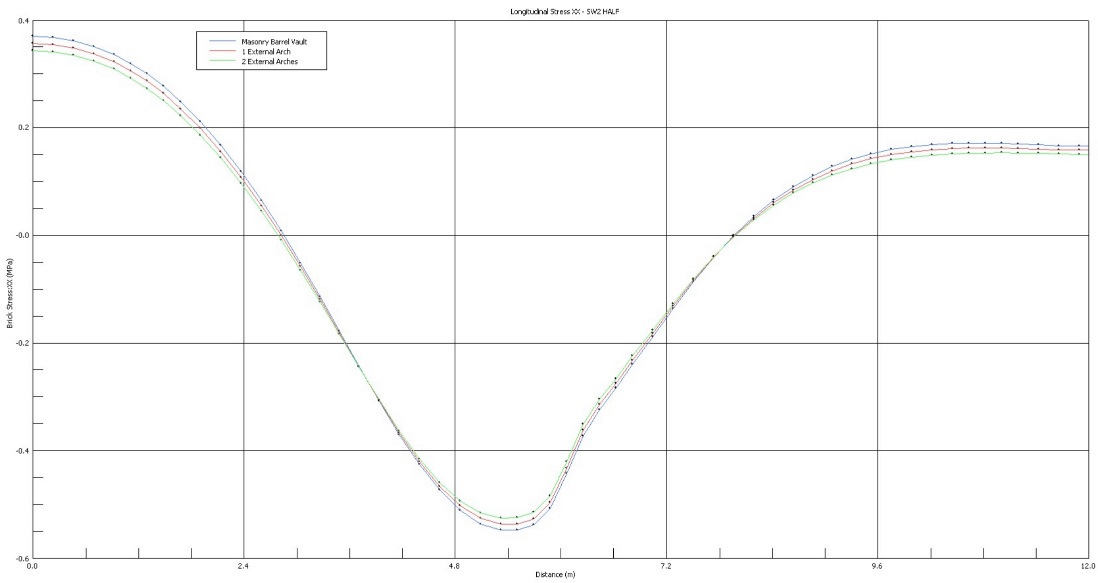


Fig. 4.75 - Stresses XX at extrados in the middle of the deck, SW2 on half span

Conclusions

Considering the values obtained by the parametric analysis performed some interesting remarks have to be point out.

The homogenisation procedure allows to model the masonry material as an homogeneous continuum that is able to take into account the real structural configuration of the original material. As previously said, the texture plays a fundamental role in the global behaviour of a masonry structure but it is difficult to take it into account in the FE models. Homogenisation make possible to analyse the behaviour of masonry with the advantages of the continuum models, hence with the use of field equation and without dimensional limits, and avoiding at the same time the typical defect of this type of modelling when applied to masonry, therefore taking into account shape, dimension, arrangement and quality of the constituent elements.

The use of homogenisation procedure is therefore strongly suggested in the study of masonry building. This techniques is also fit to study the masonry arch bridges, evaluating the effects of the real structural form to the global behaviour of the bridge. Its use, coupled with multi-scales analysis, allows to reach a complete and deep knowledge of the bridge, which is of fundamental importance in the design of future strengthening intervention.

The presence of the external stone arch rings has a strong influence in the global behaviour of masonry arch bridge. In fact, external stone arch rings increase the structural performances of the bridge under service load. Looking at the results obtained it is possible to notice that the stone arch rings are subjected to an higher level of stress respect to the barrel vault, which instead shows a lower level of stresses respect to the structural configuration of the bridge in which the barrel vault is completely made of bricks masonry. At the same time the vertical displacements are lower. The un-symmetrical configuration make the result more evident.

There is an effect of transmission of loads from the barrel vault to the external arches that reduces the stresses in the barrel vault while increases the stresses in the stone arch rings, which are more resistant respect to the barrel vault. This aspect was well known by the bridge builder of the past. In fact they used to build a very high number of bridges with this structural form. Moreover, they usually realised the stone arch rings with a greater thickness respect to the barrel vault, just because they knew this aspect.

Close to the middle of the barrel vault both displacements and stresses of the three different configurations have similar values. This is due to the fact that the bridge has a square shape, in fact the width is almost the same of the span. The effect of external stone arch rings should be more relevant in bridge having the dimension of the span bigger than the width. In fact many masonry arch bridges show a structural form whit transversal and longitudinal stone “chains”. In the figure 4.?? it is possible to notice the presence of a longitudinal stone arch ring in the middle of the barrel vault. It is interesting to study the different possible structural forms of masonry arch bridges.

Conclusions

The thesis has provided an overview of the issues regarding the conservation of many arch bridges, with particular attention to the structural modelling and analysis. Some of the considerations that have been outlined during the thesis are here summarised. Some general remarks are reported below:

I. Considering the huge number of masonry arch bridges belonging to the different European railways networks that are still in service their conservation is of fundamental relevance. The essential requirement that guarantee their conservation is the capability of the existing masonry arch bridge to carry the current train traffic, which has growth considerably in the last fifty years. Hence, in many case, it may be necessary to realise intervention of strengthening and/or adjustment. In order to design the correct interventions it is necessary to assess the structural behaviour of the masonry arch bridge. Considering the elevated number of bridges, the procedure of analysis must be fast and reliable.

II. The correct interpretation of the behaviour of masonry arch bridges is of fundamental importance. The global behaviour of masonry arch bridge is strongly related to the influence of each single element, structural (piers and arch) and non-structural (backfill and spandrel). Therefore, models should be able to take into account all the elements of a masonry arch bridge. Hence, F.E.Models are particularly fit to represent the real structure of the bridge. However the characterisation of the mechanical properties of masonry material may be difficult: homogenisation procedures are suggested to overcome this weakness. D.E.Model may be a very powerful method for the study of masonry arch bridge, especially if combined with FEM. However, its practical application is still difficult. Limit analysis is a consistent method for the assessment of the safety of the bridge, however does not provide many information about the service behaviour of the bridge. Considering the availability of different effective methods a combined use of them is suggested, on the base of the needs. In this view multi-scale analysis seems to be very suitable to establish a procedure of analysis.

III. The correct identification of damages and deterioration is of fundamental importance. Repair interventions, when necessary, should be carried out as soon as possible. Preventive and planned maintenance has to be carried out constantly to avoid the occurrence of problems that may lead to severe damages or to very invasive repair and strengthening, reducing the interruption of service. The length of works is a relevant parameter in the choice of strengthening. Intervention that not modify the original structural function of the bridge should be preferred, as well as interventions that do not alter the aesthetic appearance. The removability and/or reversibility of the interventions should be assured. However, in the case of masonry arch rail-bridge the reaching of the needed performance is essential for the conservation: some compromises have to be accepted. Strengthening aimed to increase the stiffness of the backfill material may be an innovative solution to provide an increase of the load bearing capacity of the bridge. Such type of intervention does not alter aesthetically the bridge and does not modify the structural form. The effect on the global behaviour of the bridge is positive. Modelling for the simulation of the interventions may be realised with F.E. Models, however their reliability should be assessed by tests.

On the base of the results obtained in the development of the case of study it is possible to outline some specific consideration about the proposed procedure:

1. Multi-scale analysis could be a good instrument to investigate the structural behaviour under service loads of historical masonry arch bridges. Models having different levels of detail allow to choose every time which is the more fit on the base of results purposed and computational efforts. The results obtained with different analyses provide an exhaustive evaluation of the behaviour of the bridge. Each step of multi-scale analysis proposed is fast and the comparison

between result obtained give a reliable procedure. The procedure may be easily implemented with more sophisticated models and methods of analysis when necessary.

2. The procedure can be used to analyse the behaviour of the bridge after and before strengthening, in order to evaluate its efficacy and adequacy. It will be necessary to model the interventions of strengthening by the modify of both material properties and the model itself. In particular the procedure may be used to perform parametric analysis aimed to the evaluation of the effect of strengthening of backfill.

3. Multi-scale analysis of masonry arch bridge may be coupled with a more precise characterisation of the masonry behaviour through techniques of homogenisation. The homogenisation procedure allows to model the masonry material as an homogeneous continuum that is able to take into account the real structural configuration of the original material. Hence homogenisation make possible to analyse the behaviour of masonry with the advantages of the continuum models but at the same time taking into account shape, dimension, arrangement and quality of the constituent elements. The use of homogenisation procedure is strongly suggested in the study of masonry masonry arch bridges in order to evaluate the effects of the real structural form to the global behaviour of the bridge. Its use, coupled with multi-scales analysis, allows to reach a complete and deep knowledge of the bridge.

4. The presence of the external stone arch rings has a strong influence in the global behaviour of masonry arch bridge. In fact, external stone arch rings increase the structural performances of the bridge under service load. There is an effect of transmission of loads from the barrel vault to the external arches that reduces the stresses in the barrel vault while increases the stresses in the stone arch rings, which are more resistant respect to the barrel vault. This aspect was well known by the bridge builder of the past, which very often used to build bridges with this structural form.

Selected annotated bibliography

This annotated bibliography lists all the sources that have been useful for the preparation of the thesis. Sources are divided in 2 sections:

- The fundamental references: this section concerns the main references that have been fundamental for the author to the study of the subject and have been cited several times in the thesis;

- The specific references: this section concerns all the sources that have been cited for each topic that has been discussed in the thesis; it includes texts, scientific papers, standards, regulations, PhD thesis on the same subject, online citations and every other type of material that has been cited in the thesis. Specific references are organised in sub-sections regarding each specific topic[1].

[1] Some reference (both fundamental and specific) may be reported in more then one sub-section, because it has been cited in different parts of the thesis.

Fundamental references

Giuffrè, A. (1991) *Letture sulla meccanica delle murature storiche*, Roma, Kappa Edizioni.

This book concern the different structural behaviour of historical masonry on the base of their constructive features. It has been fundamental in the study of the behaviour of historical masonry.

Heymann, J. (1982) *The masonry arch*, Chichester, Hellis Horwood

Heymann studied the behaviour of masonry arch, from the elasticity to the plasticity. It has been fundamental in the study of the behaviour of masonry arch.

McKibbins, L; Melbourne, C; Sawar, N; Sicilia, C (2006) "Masonry arch bridges: condition appraisal and remedial treatment" London, Ciria, C656

This book provides all the information regarding the assessment of masonry arch bridges. It has been fundamental in the study of the behaviour, modelling and analysis of masonry arch bridges.

Ozaeta, R.G. and Martín-Caro, J.A. (2006) "Catalogue of Damages for Masonry Arch Bridges" UIC Report

A very complete UIC report about damages of masonry arch bridges, which involved all the main railway companies. It has been fundamental to study typical damages of masonry arch bridges

Proske, D. and Van Gelder, P. (2009) "Safety of historical stone arch bridge" Springer, 2009

This book provides all the information regarding the safety of masonry arch bridges. It has been fundamental in the study about the intervention on masonry arch bridges: maintenance operations, techniques of repair and strengthening.

Tassios, T. P. (1988) "Meccanica delle murature" Napoli, Liguori Editore

This book provided exhaustive information about the mechanical properties of masonry and its constituent material. It has been fundamental in the discussion of mechanical properties of masonry.

Torre, C. (2003) “Ponti in muratura. Dizionario storico-tecnologico”, Firenze, Alinea.

The dictionary provided exhaustive information about historical and technological characteristics of masonry arch bridges, with specific descriptions about the constituent elements and the materials. It has been fundamental to study the morphology of masonry arch bridge.

Troyano, C.F. (2006) “Terra sull’acqua. Atlante unversale dei ponti”, Palermo, Dario Flaccovio Editore.

This Universal Atlas of bridges provided all the information about the history and the diverse typologies of bridges. It has been fundamental to outline the historical and typological evolution of masonry arch bridges.

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