ADVANCED MASTERS IN STRUCTURAL ANALYSIS
OF MONUMENTS AND HISTORICAL CONSTRUCTIONS

Master's Thesis

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Structural analysis of Palladio's timber bridge in Bassano del Grappa

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Year: 2016

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I hereby declare that the MSc Consortium responsible for the Advanced Masters in Structural Analysis of Monuments and Historical Constructions is allowed to store and make available electronically the present MSc Dissertation.

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…and finally I would like to say Thank You to my granny, who was the first one to tell me "go…!"

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ABSTRACT

The Bassano's bridge is a covered timber bridge 65 m long and 8 m wide located in the Veneto Region in Italy. During its history it was destroyed several times and then reconstructed following the idea of Palladio's original project, which dates back to 1570. Reconstructions and refurbishments provoked some changes in the structural behaviour, especially in the foundation system; further study is required in order to understand the current structural behaviour of the foundations. Moreover, the structural degradation and the heavy weight led to significant settlements that required urgent consolidation.

This thesis is a continuation of the previous studies of the bridge and it aims at analysing the stress levels of the foundation elements with respect to different loads configurations and at understanding the role of the piers' geometry in the out of plane behaviour.

The FEM model of the structure was built and static and dynamic analyses were performed. The floods damaged many times the bridge structure provoking the collapse in the 1966; therefore static analyses were performed at the daily and at the floods conditions of the bridge. Italy is a seismic country and hence, it is essential to evaluate the risks connected with this type of natural disaster, especially in the case of historical structures designed in the absence of seismic principals. In this view, the linear dynamic analysis of the earthquake state was performed applying the response spectra proposed by the Italian codes.

The foundation structure shows unsafe conditions at the daily, earthquake and flood states. It is significant how the daily configuration, performed by applying the quasi-permanent load combination, presented levels of stresses already higher than the minimum levels requested by the Italian code. Based on the analyses carried out, the flood condition is the load configuration causing the highest levels of stresses in the foundation elements, showing how urgent and necessary is a retrofitting plan.

In terms of out-of-plane behaviour of the structure, the geometry of the piers proved to play a significant role. Two models representing different geometries of the piers were analysed. The trapezoidal-shaped structure has an important role in the out of plane stiffness but its role is negligible in the case of pure vertical loads.

Keywords: Timber structure, bridges, dynamic behaviour, historical building, FEM

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RIASSUNTO

Title: Analisi strutturale del Ponte degli Alpini di Bassano del Grappa

Il ponte degli Alpini di Bassano del Grappa è una struttura in legno lunga 65 metri e larga 8 metri. Durante gli anni il ponte è andato distrutto diverse volte e successivamente ricostruito fedelmente al progetto di Andrea Palladio, risalente al 1570. Le ricostruzioni e gli interventi di mantenimento hanno provocato diversi cambiamenti nel comportamento strutturale del ponte e, in particolare, nel sistema di fondazione; per questo motivo studi approfonditi delle fondazioni risultano essere necessari. Il degrado della struttura e i pesi ingenti hanno dato luogo ad evidenti deformazioni e cedimenti, sono urgenti alcuni interventi di consolidamento.

Il progetto di tesi è a continuazione di precedenti studi svolti sul Ponte di Bassano ed è mirato principalmente all'analisi dei livelli di stress degli elementi di fondazione sotto diverse condizioni di carico ed allo studio del ruolo della geometria delle pile, in particolare nel comportamento fuori dal piano.

E' stato sviluppato il modello FEM della struttura e sono svolte analisi statiche e dinamiche. Le piene del fiume Brenta hanno danneggiato numerose volte la struttura provocando il collasso nel 1966; per questo motivo l'analisi statica è stata eseguita con i carichi corrispondenti alla piena del fiume e alle condizioni giornaliere. Poiché il territorio italiano presenta alta sismicità, la valutazione dei rischi connessi agli eventi sismici è imprescindibile; in particolare nel caso di edifici storici progettati in assenza di presidi antisismici. In quest'ottica, è stata svolta l'analisi dinamica lineare della struttura definendo l'azione sismica con lo spettro fornito dalla normativa italiana.

Gli elementi di fondazione risultano non in sicurezza alle condizioni giornaliere di carico, alle condizioni di piena del fiume e sotto l'azione sismica. E' significativo come le condizioni giornaliere, analizzate tramite la combinazione quasi permanente dei carichi, presentino già livelli di stress più alti rispetto ai massimi consentiti secondo il codice. Sulla base delle analisi svolte, la piena del fiume presenta le condizioni di carico più gravose per gli elementi di fondazione, risulta quindi evidente la necessità di consolidare la struttura per metterla in sicurezza.

Il comportamento fuori dal piano del ponte è fortemente influenzato dalla geometria delle pile ed in particolare dalla presenza dei Rostri. Al fine di analizzare il ruolo di questi elementi, sono stati sviluppati ed analizzati due diversi modelli rappresentanti due geometrie delle pile. La forma trapezoidale delle suddette ha una forte influenza nella rigidezza fuori dal piano della struttura ma è trascurabile in caso di carichi puramente verticali.

Key words: legno, ponte, comportamento dinamico, edifici storici, FEM, modellazione numerica

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1. INTRODUCTION

The timber bridge of Bassano del Grappa, is also called in Italian "Ponte degli Alpini" due to the contribution of this specifc army in the bridge reconstruction after the WW2 . The structure represents a piece of Italian history and it is a masterpiece of one of the greatest architects of the country, Andrea Palladio. Naturally, through the course of time, this historic bridge became a beloved symbol of the town. Due to this affection, in spite of all the damaged and destruction it suffered, the bridge was rebuilt every time through the effort of the community. The structure we appreciate nowadays is mainly from 1948, but the original Palladio's project is from the second half of the 16th century.

The Bridge passes over the Brenta River, connecting the two towns of Bassano and Angarano; this river flows from the Alps and is fast and turbulent when it reaches Bassano. For this reason, the force of the water flow has, in several cases, caused severe damages in the structure of the bridge, such as the flooding of 1966.

In all the European countries the will to conserve built heritage is very strong and a certain level of experience is nowadays achieved; nevertheless each historic construction represents a unique case with all the problems that this entails. The Bridge of Bassano has no equals in the world; its structure has its peculiarities and its own vulnerabilities. The assessment of the structure is, therefore, focused on understanding these characteristics because only a deep knowledge of the building allows defining the proper conservation approach.

This thesis is a continuation of the previous studies of the bridge undertaken by the University of Padova. The geometric and damage survey, proceeded last year by the University of Padova together with SM engineering and Foppoli Moretta Associati with the support of the Municipality of Bassano del Grappa, are at the base of this work and presents fundamental information.

Nowadays, the bridge presents a damaged state: high deformations are visible on the deck level and the foundations are heavily decayed; an intervention seems to be necessary. The purpose of the project is to assess the safety of the bridge, focusing on the foundation system. This part of the structure was modified many times during the years and the morphology of the riverbed has undergone natural changes causing changes in the structural behaviour. Using a FEM model, built with Straus 7, the stress level on the foundation elements is assessed considering different load configurations and conditions.

The second key aspect of the study is the understanding of the role of the piers structure and geometry, especially in the out of plane behaviour. The unique shape of the piers, composed by eight

vertical poles under the deck and additional poles forming a trapezoid-shaped pier, make their structural behaviour hard to understand. During the past major floods events, the piers were the most damaged elements; for this reason, two different FEM models are analysed, with and without those additional triangular parts, called *Rostri* in Italian. The stress levels at the foundation and the displacement at the control point is studied for both the cases.

Due to the risks connected to the force of the river, a static analysis is also performed considering the flood state; this is represented by a river force equal to the 1966 flood event which is slightly higher than the flood calculated for a return period of 200 years required by codes. The foundation elements are assessed in the flood conditions and the results are compared to the daily state in order to have an overall understanding. Moreover, the daily analysis helps to understand the actual state in current damaged conditions.

Large areas in Europe and in Italy are characterized by a high level of seismic hazard and the vulnerability of the ancient structure is often relevant. The seismic assessment is, therefore, fundamental in the study of an historic monument, such as the Bridge of Bassano. Following the static approach, the spectral analysis, based on the Italian code's seismic requirements, is performed in order to estimate the seismic stress levels of the foundation elements.

Thanks to the results of the three described analysis, the out of plane behaviour and foundations conditions may be evaluated.

1.1 Location

Figure 1 – Location of the Bridge of Bassano del Grappa

The timber bridge called "*Ponte degli Alpini*" is located in *Bassano del Grappa*, a town it the Vicenza province, in the Veneto region, in the northern Italy. The first news regarding the existence of this mediaeval city dates from 998, several families succeeded in the government of the town, such as the Ezzelini in the 13th century, for a long period it was part of the Republic of Venice who had an important role in the bridge reconstructions.

The Bridge is located on the river Brenta, it runs from Trentino to the Adriatic Sea just south of the Venetian Lagoon. The river in the area of Bassano is still stormy and floods are frequent. Two hydraulic structures are present in the portion of the river inside the town: 400m north there is the Arcon weir where a secondary channel starts, instead 250 m south there is the derivation of the Medoaco channel.

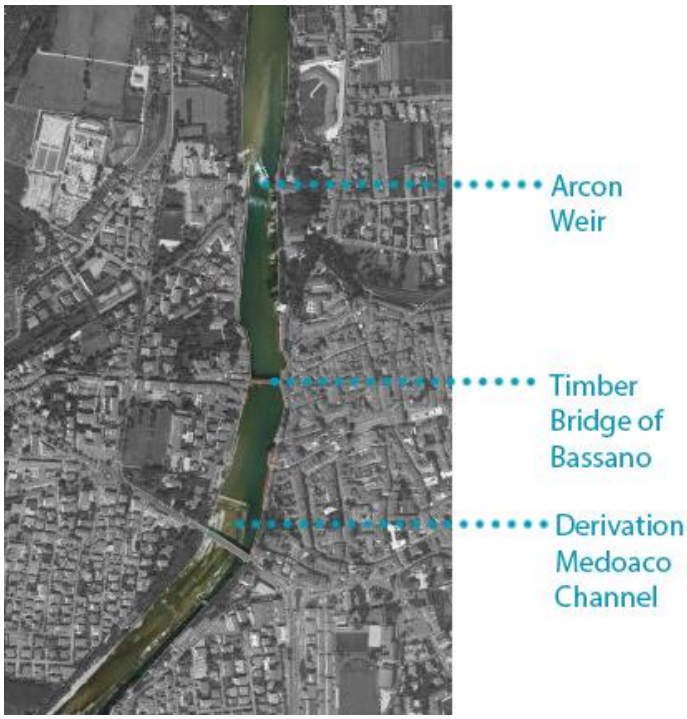


Figure 2 - Brenta River

2. MORPHOLOGICAL DESCRIPTION

It is important to describe the complex structure of the timber bridge before the historical overview of the main phases. The timber bridge of Bassano is a unique example in Italy, its structure and style is not similar to any existing case. The actual structure is a result of a long history of almost one thousand years: several reconstructions and repairs were necessary and the river force has been a problem through all these years because of its energy and the frequent floods. The main project is from Palladio, who construct the bridge in the 1567 (paragraph 3.1).
The bridge present five spans of 13.2 m; the structure is loading on the two abutments and on four central piers.

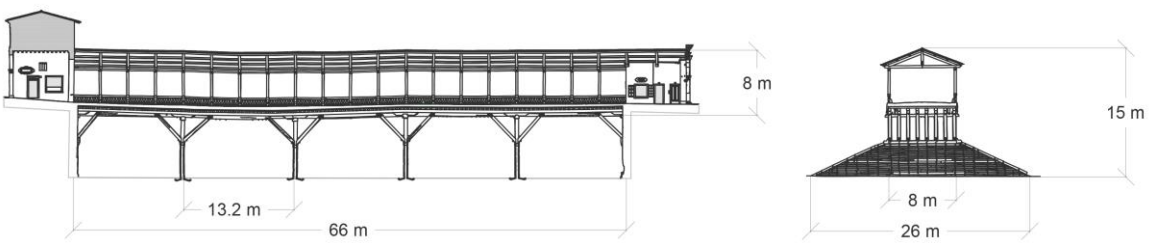


Figure 3 - Longitudinal and transversal sections of the bridge

In the Four Books of Architecture, written by Andrea Palladio, there is the description of the bridge and the drawings defining the different parts (Figure 4). The Palladio's drawings present five spaces between the columns for each span, instead the actual bridge just four.

+ is the water level

A is the longitudinal view of the bridge

B are the piles inserted in the river

C are the heads of the transversal beams

D are the beams posted in the longitudinal direction (supporting beam)

E are the oblique elements (strut)

F are the columns

G is the transversal section of the bridge

H is the plan

I is the referring measure of ten feet

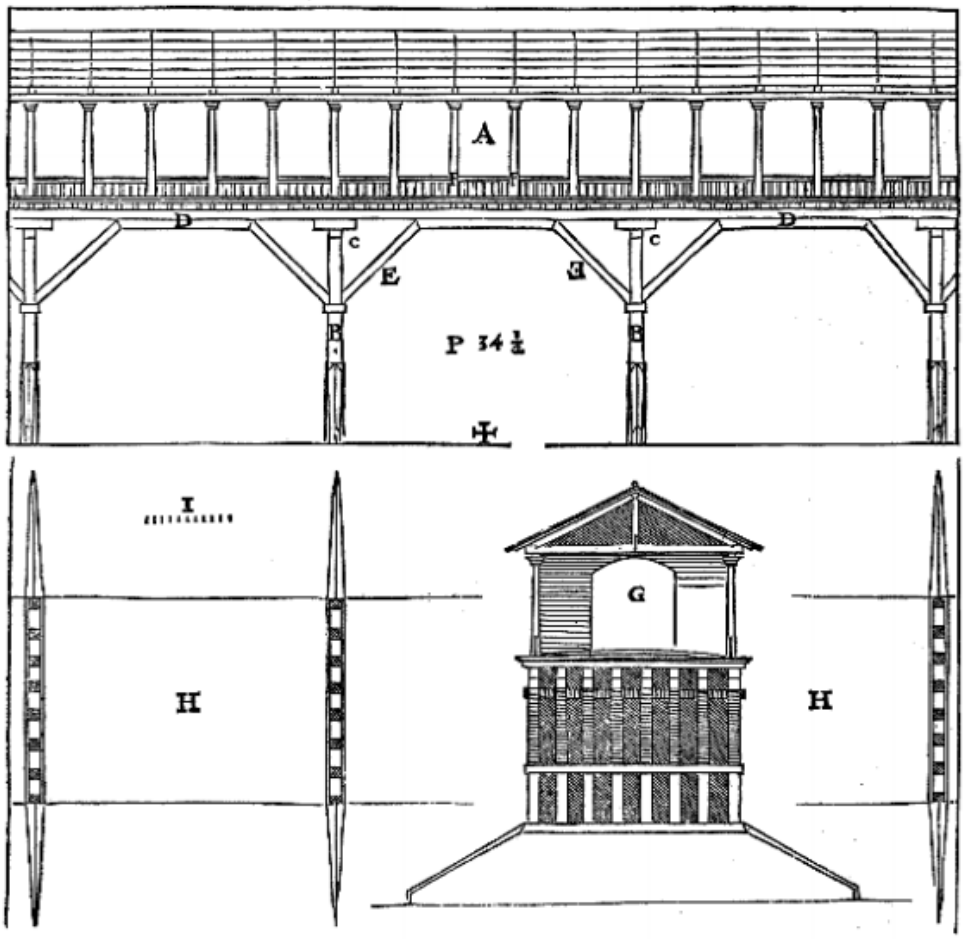


Figure 4 – A. Palladio, The Four Books of Architecture

The structure consist on five spans resting on four piers in the riverbed. Each pier has eight piles of the same length (1.1m) supporting the structure above, as well as 6 more columns of decreasing length on both sides, protecting the bearing columns; these triangular lateral elements are called *Rostri* in Italian. Two transversal beams connect the columns together on the top and at the base. The columns are covered by a planking structure with wooden elements (10x17cm) with a spacing of 25 cm. The connection between columns and beams is done by threaded bars of 16mm diameter; instead, the planks are not connected horizontally and their main aim is to protect the internal columns. In the 1991 a survey was done by the engineer Ugo Bonato, his drawings are a great source of information. Each span presents the same type of structure: longitudinal beams loading on the cantilever beams which is supported by transversal continuous beams. The latter distribute the loads on the eight pile constituting each of the eight pier.

Figure 5 - Timber Bridge of Bassano del Grappa (2016)

The oblique elements (struts), taking load from the longitudinal beams, are connected by the longitudinal supporting beams. The connection between those two elements is done by metallic plates 8mm thick and 9 bolts of 16 mm diameter. The other connections are made with bolts of 25 or 30 mm diameter with the use of Appel connectors.

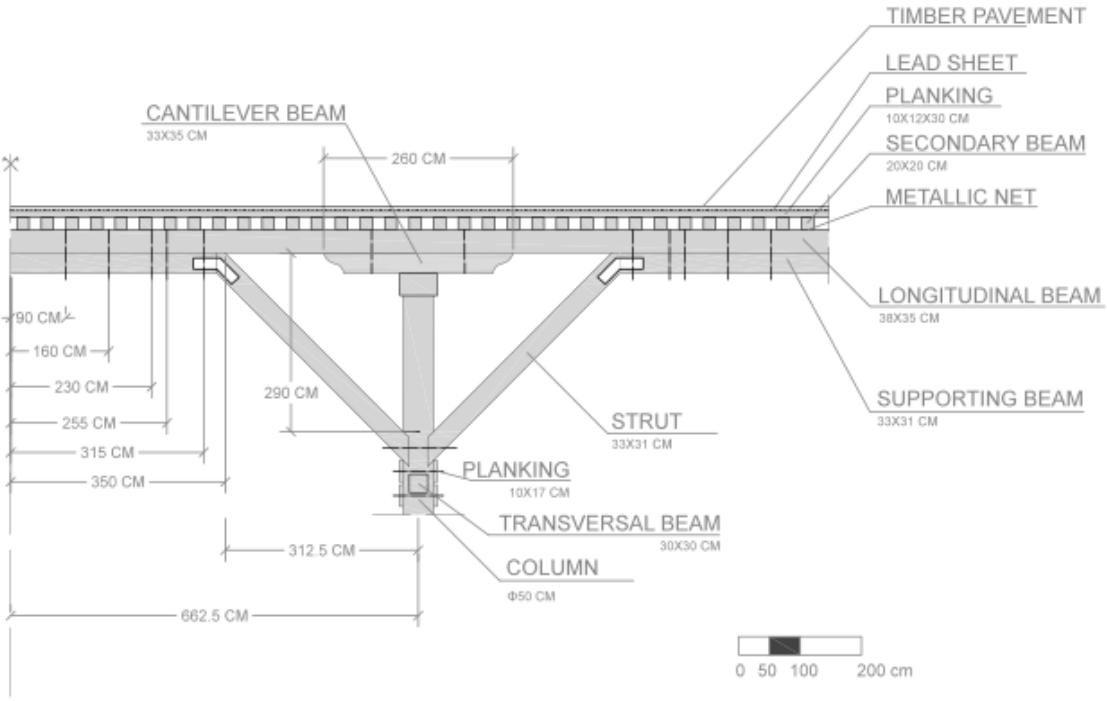


Figure 6 - Structural detail (based on the drawings of U.Bonato)

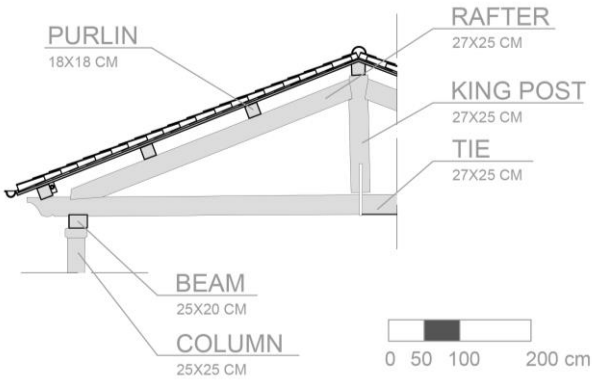


Figure 7 - Detail of the roof system

The bridge is covered by a simple roof structure. The roofing system is composed by timber trusses with a distance between each other of 3.3 m. The covering is made with timber boards and terracotta tiles, as typical for the region.

Figure 8 - Timber Bridge of Bassano del Grappa (2016)

3. HISTORY OF THE CONSTRUCTION

The timber bridge in *Bassano del Grappa* was built between 1124 and 1209 over the Brenta river, it was designed in order to connect Bassano to Angarano, whose nowadays are part of the same municipality. The bridge was constructed for economical, social, political and military reasons, it was the connection between the town and the road direct to Vicenza; because of its position, it was fundamental in the goods transportation. Due to its strategic importance, two towers were built in order to protect it, even dough it was destroyed many times during its history. The bridge felt down five times and it was destroyed by men other three.

The first information regarding a bridge in this position is from the XII century, when the city wall was built. The Ezzelini family wanted the construction for economical reasons, the connection of Bassano with Vicenza was important due to their alliances. The bridge was built in accordance with the city wall and two defensive towers were located on the two rivers (Figure 9). Other than due to the severe and frequent floods, the bridge was damaged by the materials, such as wood, transported by the water flow. The timber trunks, coming from Trentino, were used as a building material and as combustible and they were transported to the port of Brenta, southern than Bassano; this type of commerce represented a source of richness for the main families of the town.

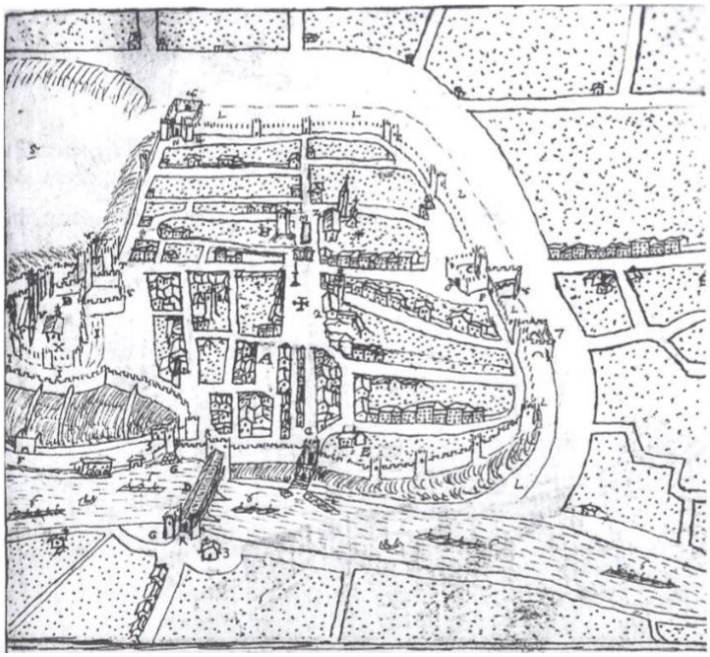


Figure 9 - Representation of Bassano by Francesco Chiuppani (1730)

In the *De Pontatico Pontis Brentae* (1259), it is stated that in order to cross the bridge people had to pay a tax; moreover, in the *De lignis Pontis Brentae*, it was said that whoever steal wood from the bridge have to pay 40 coins, these documents testify the importance of the bridge at the time.

In the Middle Ages the bridge was protected by the two over mentioned towers, even thou adjustments and reconstruction succeeded over the years. After the 14th century, the town of Bassano was under the domination of Venice, the "Serenissima". In the 1406 the bridge was destroyed by floods and in the following year the reconstruction took place, mainly founded by Venice and other close towns. However, in the 1450 the bridge collapse again and the doge Francesco Foscari ordered the city of Bassano the re-construction. In the 1452 Jacopo Barbano was in charge for the reconstruction, the following year the bridge was opened again. Due to a flood, in the 1493 the bridge was severely damaged and it was necessary another reconstruction. It took a few year to decide which was the most suitable material to be used; the idea of a stone bridge was rejected and the bridge was built as it was, in timber. In the 1506 an inspection regarding the safety and the conditions of the bridge was done as requested by the doge of Venice; at the time the bridge was well built and the roof was already present. The bridge was already a symbol in the landscape of Bassano.

After a war between the *Lega dei Cambrai* and France, in the 1509, the powerful French army conquered Bassano. One year later, the latter burned the bridge to stop Bassano troops' advance. In the 1522 the bridge was rebuilt, and again in the 1524 but using stones token from Priara. The new bridge was composed by two arches with one pile in the center, it was wide 28 feets and high 27. The masonry bridge lasted only a couple of years, in the 1526 there was a collapse due to a flood of the Brenta river. As it happens before, when the bridge was under construction a ferry was used to move the goods and the people. Six years later the Potestà Alvise Grimani ordered to destroy the bridge and rebuild it in timber with the original shape, in the meantime the occidental access was restored. The power of the river strike the bridge again in the 1567.

3.1 The bridge from Andrea Palladio

The following year the Council of Bassano ordered a new bridge, and in the 1570 the project was awarded to Andrea Palladio. The architect proposed a three-arches bridge based on the classic roman model; this project is close to the one designed for Rimini and present in the third of the four books about architecture written by Palladio (Figure 10).

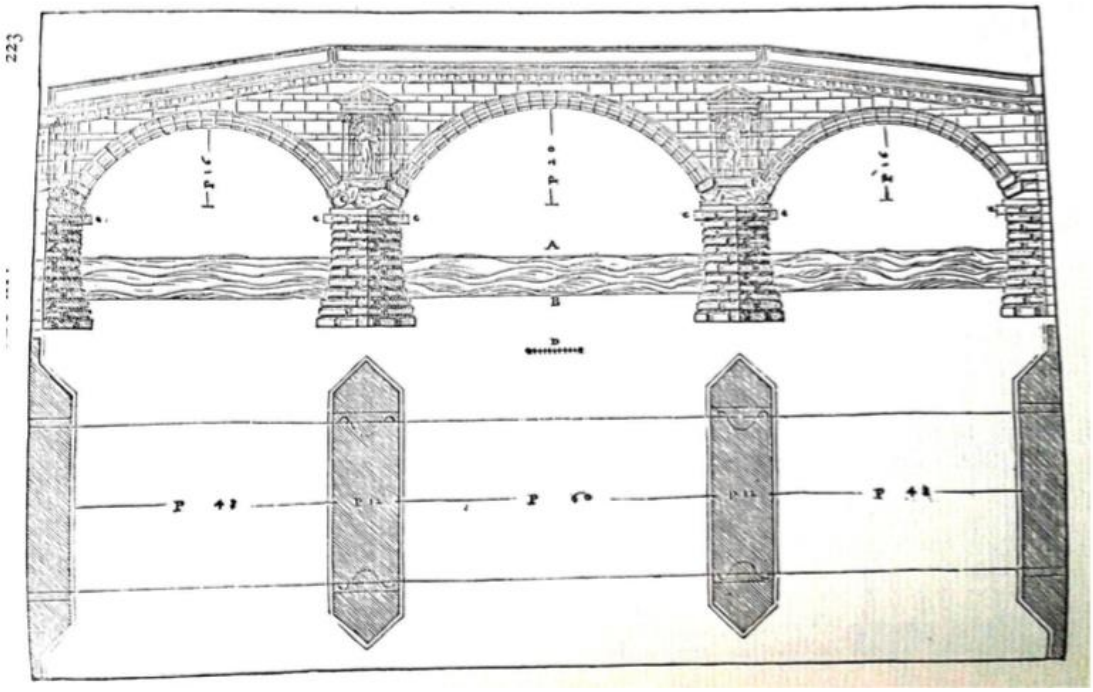


Figure 10 - The four books of architecture: 3rd book, 14th chapter

However, the population of Bassano strongly request a timber bridge, it was a symbol of the city from years and a stone one would not be appreciated. The city council of Bassano ask Palladio to design the new bridge as the previous but with all the necessary modifications:

"esso ponte sia rifatto e costrutto nel modo et forma che era il precedente menato via dalla Brenta, con quelle aggiunte che parerà alli proti che lo costruiranno".

The city of Venice allows Bassano to cut of 200 oak trees for the new project, before the project approval the material had already been set up.

It is hard to say how free Palladio was in the project, he was asked to build a timber bridge but it is not clear if he entirely designed it o if he could just make some adjustments. According to Francesco Memmo, the result is a bridge similar to the previous one, there was probably not so much space for the architect choices. In addition, Guglielmo Marchesi, not Palladio, ,supervised the operations since July 1569. In the *Four Books of Architecture* (Venice, 1570) Palladio describe and present the project of the bridge for Bassano, he affirm he was asked to realize a timber bridge, but he never say that he also designed it. Palladio, anyway, applied his concepts about architectural orders and proportions to the bridge that was previously an historical element of the city with a more "spontaneous" design.

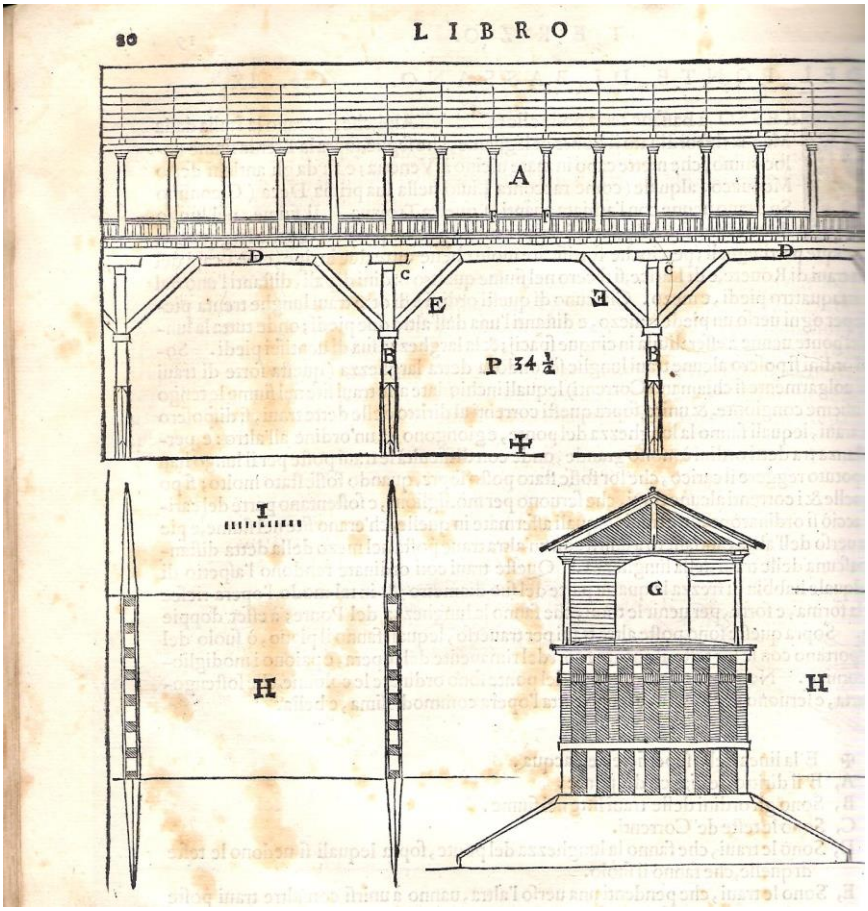


Figure 11 - Palladio, The Four Books of Architecture: 4th chapter, 3rd book

Palladio built the scaled model and the building phase was assigned to Marchesi and the bridge was ready in the 1570. After a survey done by Giovanni Piccoli in the 1593, the dimensions of the columns result different from the Palladio's drawings; the poles should have been on one piece but it was hard to find the trees of this dimensions (30 feets long and 1.5 wide), for this reason they built them in two pieces. A few repairs and substitutions were needed but the bridge lasted until the 1748, when a flood destroyed it.

The following year, the rebuilt was done by Bartolomeo Ferracina exactly as the previous one. The project of Ferracina was cheaper than the other proposed by Giovanni Miazzi and Tommaso Temanza. Ferracina used a machinery of his invention in the building phase, it was an innovative machine to beat the poles using the water energy and it was supporting the building at the same time. Nevertheless, some modifications has been done: a new balcony is present at the middle of the bridge and the distance between the columns is different, from 5 to 4 column-span each pile-span; however the balcony was already present before Palladio's design. The result was even more similar to the original bridge than the one of Palladio.

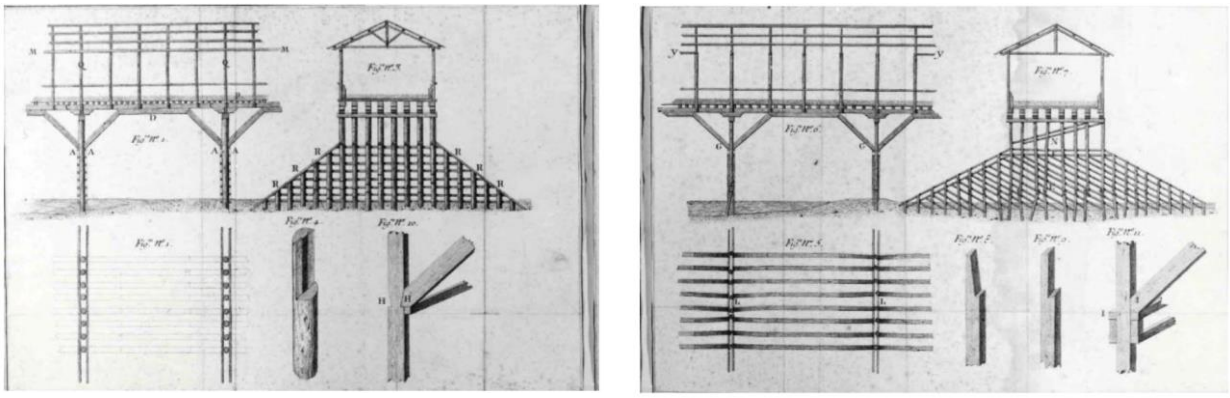


Figure 12 - Comparison between the projects of Palladio (left) and Ferracina (right), Francesco Maria Preti 1750, Civic Museum of Bassano del Grappa

Miazzi built a temporary structure in the 1749 to link the two riverside during time of the bridge construction, this structure was destroyed by another flood a few time later. The 30 of September 1751 the bridge designed by Ferracina was opened.

Figure 13 - Roberto Roberti "il ponte di Bassano", 1807, Civic Museum of Bassano del Grappa

The new bridge was resistant, in the records there are just a few repairs in the following years. However, between the 1796 and the 1813 it was damaged by military campaigns. From the 1805 Bassano is part of the *Regno d'Italia* but during the Napoleonic time it was damaged by many battles such as the one of May 1811 (cannon ball holes are still visible on the façade of the distillery building next to the bridge). In the 1813, during the war between Austria and France, the French troops burn the bridge. Between the 1819 and 1821 the bridge has been rebuilt by Angelo Casarotti da Schio. The new architect made some modifications respect to the project of Ferracina: he modify the supporting

structure adding a new longitudinal beam in the foundations in order to have a better distribution of the forces, in addition he took off the additional beams and the balconies in order to reproduce the Palladio's project. A drawing from the 1852 describe a intervention to straight the columns by Sebastiano Mocellin but it is hard to say if it was realized or not.

3.2 The Bassano Bridge in the 20th century

At the beginning of the 20th century the traders wanted a wider bridge for the movement of goods but this plan was interrupted by the First World War. During the conflict the bridge was largely used as the main supply route for alpine troops, the vehicular and pedestrian traffic was intense; the bridge was damaged by the bombing and repaired in the 1923.
At the end of the Second World War, in the 1945, after the destruction of the new Bassano bridge the *Partigiani* blew up the bridge in order to stop the German invasion, but that was not too effective and it was still possible to pass. A few months later the German army destroy the occidental part the bridge as a revenge (Figure 14).

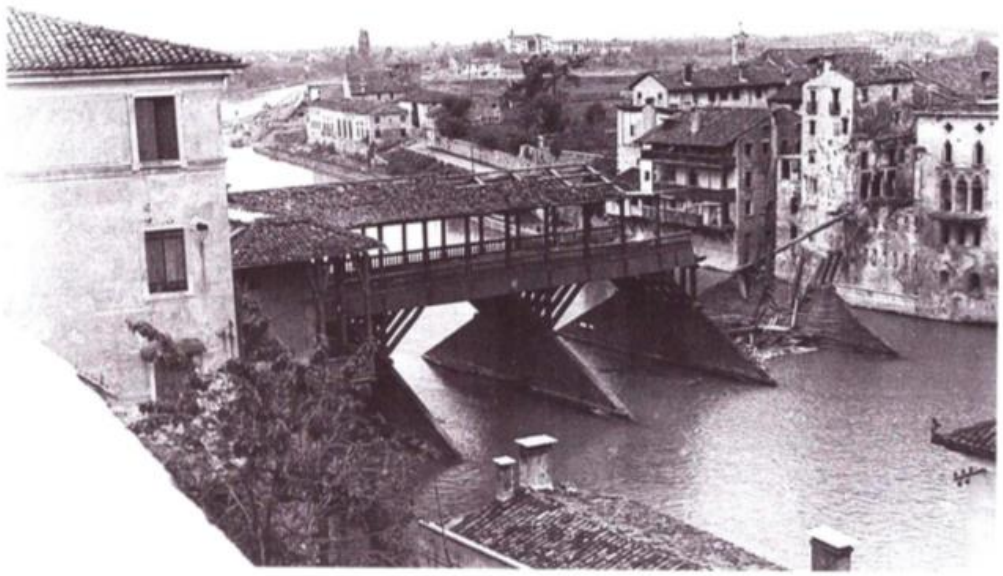


Figure 14 -The bridge after the German attack of April 1945

In the 1948 the president of the Italian government, Alcide the Gasperi inaugurated the restored bridge. The expensive works were partially founded by the *National Association of the Alpine* for this reason the bridge is nowadays known as *"Ponte degli Alpini".* The repair was done by the local construction company Giulio Tessarolo e Figli, during the works six overflows caused damages. The wood used was of larch, oak, chestnut and locust tree, over 400 cubic meters. The image of the bridge

we have nowadays is almost the one of the 50s, even if in the 1966 there was another flood and the pavement was changed in the restoration. During this event the central part start to bend in the south direction. In 1968-69 there was an intervention to restore the bridge, some parts were substituted, in the 90s new works were necessary.

Table 1 - Main construction phases

Year(s)	Architect/constructor	Main Changes
1568-1748	Palladio	New structure based on the previous bridge.
1751-1813	Ferracina	Total reconstruction. New lateral balcony, change the columns span.
1821-1945	Casarotti	Total reconstruction. Removal of the balconies and modification of the supporting structure.
1948	Giulio Tessarolo and sons.	Reconstruction of the 4rd pier, the roof and the last two spans of the bridge.

Intervention of the 1948

The intervention of the 1948 was mainly focused on the fourth pier, it was reconstructed together with the foundation system. Looking at the picture of the time (Figure 15) it's clear that the last two spans (near Angarano) were destroyed, during this intervention they were rebuilt. It was necessary even to repair and reconstruct the roof and the entire part over the footing level (columns, balustrade, balconies); the pavement was done with a layer of rubble stones of 5 cm thickness. Moreover, the stone gate was damaged during the war and the reconstruction was necessary.

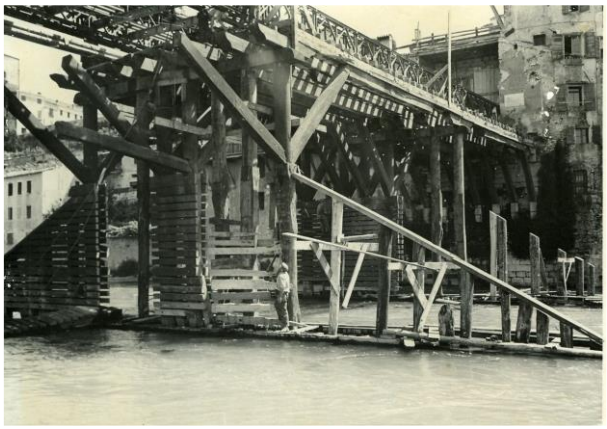


Figure 15 - Restoration of the 1948; third pier. Civic Museum of Bassano

Other secondary interventions were done, such as the repair of some damaged timber elements, the substitution of metallic elements in the connections and in addition, the entire bridge was painted with line oil mixed with red pigment.

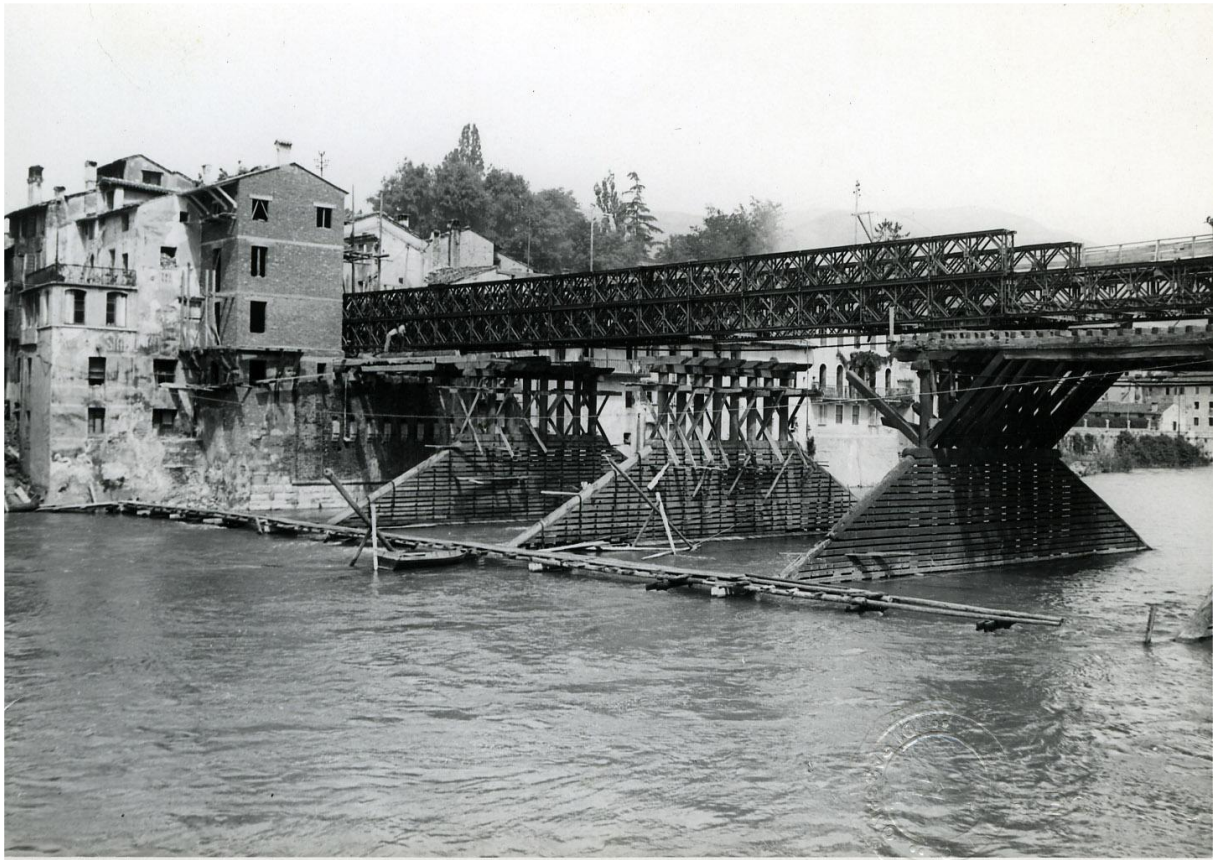


Figure 16 - reconstruction of the Bridge, 1948 (Civic Museum of Bassano)

Intervention of the 1965

There are not fully information about this intervention, however it is known that the second span (from Angarano) was restored. The cantilever beam on the east side was substituted together with the first and second longitudinal beam on the north side.

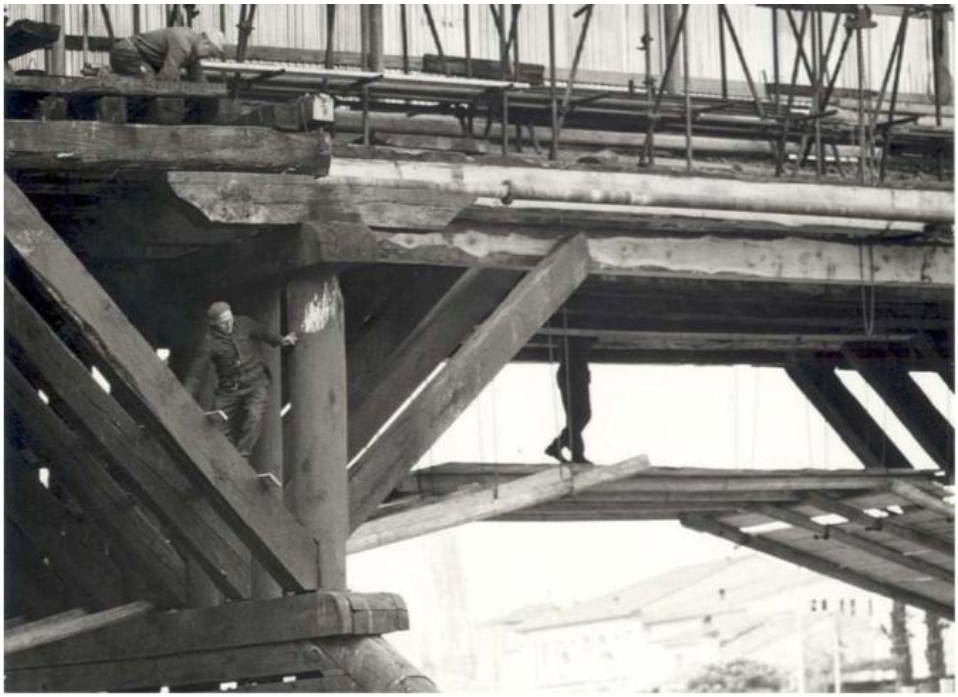


Figure 17 - Repair of the damaged elements (1965)

Intervention of the 1968/69

After the overflow of the 1966, due to the deformations and the damages it was necessary a new intervention on the bridge. The engineer Benetti was the responsible of the works. During the excavation it was possible to assess the soil stratification: on the east side there was a layer of 4.5 m of conglomerate, on the other side only 3.5m; below there was a gravel and clay alternatively. The procedure consisted on the dismantle of the structure in order to create a more resisting foundation and on the substitution of the elements carried away by the latest overflowing of the river. The fourth pier was strengthened and stabilized again, as well as the foundations piles.

A new foundation structure was designed, moving from one central line of piles to several not aligned piles connected by a new beam called *"Cavezzale".* Four new *cavezzali* for each pier were added in order to support the foundation structure. These supporting beams are located in different positions along the threshold; they have two piles each one as a support, the piles are in timber encapsulated in a concrete sheet and they stand for 7 m depth from the end of the concrete layer. The addition of these piles, other than increasing the stability, strengthened the soil foundation.

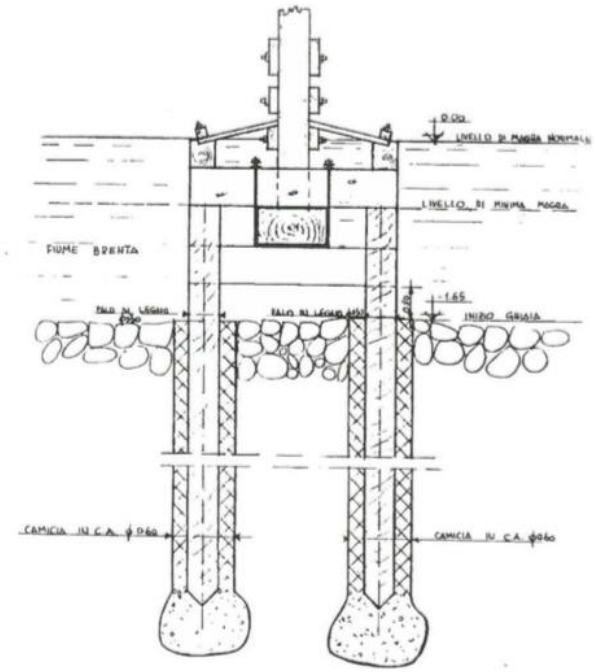


Figure 18 - Scheme of the new foundation system, Benetti 1968

Due to the damages of the overflow, the Genio Civile of Vicenza asked to improve the safety with a specific intervention: the addition of four Benoto pile placed on the north side; these elements had to hold a system of two ties that works at the two sides of each pier.
The entire intervention started from south and after moved to north in order not to have stability issues. Together with these main interventions, sever timber elements were substituted such as the *Rostri* or the planking and the connection were improved using bulldog plates and connector Appel. The timber element were treated with antiseptic products, lined oil, with lead an iron red oxide.

Intervention of the 1983

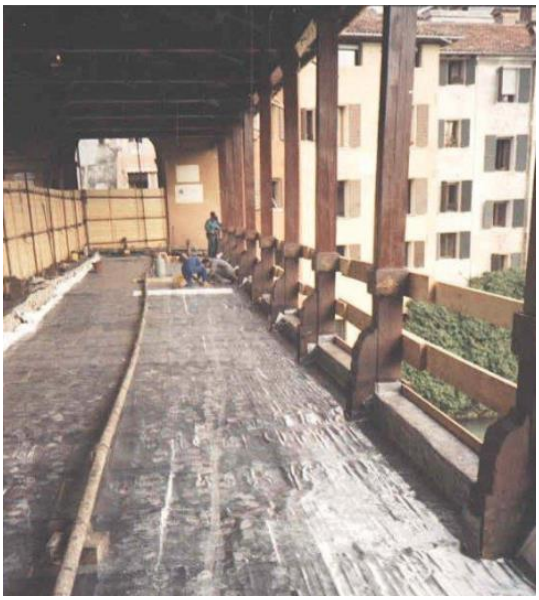
After the intervention of the '68/69 there was not necessity of other restoration for 22 years. However, in the 1983 a survey was done in order to check the safety level of the structure. The paint used in the previous intervention was used again. At the time the bridge was largely decayed, once again the Angarano side was the most damaged, this is because of the different thickness of the concrete layer and because of the direction of the flooding. The foundations of the piles supporting the threshold designed by Casarotti had to be repaired. A metal plank was placed under this timber element.
The fourth pile was restored in the '68; the same intervention (addition of the cavezzali and the new piles) was extended to the other three. The Benoto piles at north were reduced in dimension, the new one are 80/100 cm.

Intervention of the 1990/91

A few years later, another intervention took place; it started on the fourth pile and after it was extended to the entire structure. Reinforced concrete piles covered with stainless steel and cathodic protection were added, for a total of 24 new piles. The piles supporting the *cavezzali* were damaged by the water stream during the years, as result just one of the four was actually supporting the longitudinal beam (threshold). Therefore the foundation at the time was not sufficient, a new foundation system was designed in order to help the original system in supporting piers, the interventions was made with 8 piles (4 each side) supporting the transversal beams that were supporting the threshold, long 9 meters. The new piles were made in reinforced concrete with ribbed bars in stainless steel (AISI 316), a cladding in steel sheet at the concrete lever (2 m) continuing with tubes in Fe360 for 6 meters down. It is important to notice how the new structure was designed to help the original foundation system and not to substitute it.

Additional intervention of maintenance were necessary, such as the substitution of many timber elements. New connection between the longitudinal elements and the supporting beam were designed using bulldog plates and connector appel in order to obtain rigid connections. The intervention of *scuci&cuci* was applied on the masonry abutments.

The car passing on the bridge caused vibrations and, consequently, damages; for this reason the pavement was substituted and the bridge became pedestrian. A lead planking was placed to insulate the timber beams from the water. A new "massicciata" was realized on the base of the previous pavements; the stones used are verdello, chiarofonte and a hard limestone from Asiago.



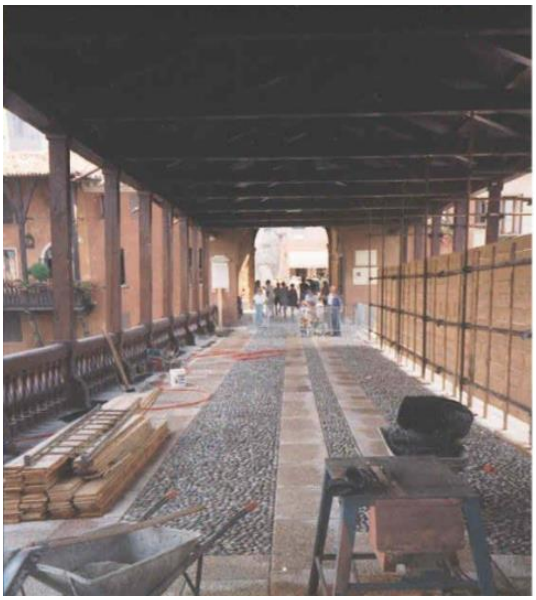


Figure 19 -Realization of the new pavement, 1991

Figure 20 - The new foundation system, the cavezzali and the piles in RC

Figure 21 - The repairing of a column

Intervention of the 2005/2006

In the 2005 it was again clear that a new intervention was necessary in order to maintain the safety level of the structure. The bridge was painted with protective oil and iron oxide. Many timber elements resulted to be damaged such as part of the columns, of the balustrade. The *Rostri*, as many times before, were again damaged and therefore they were substituted.

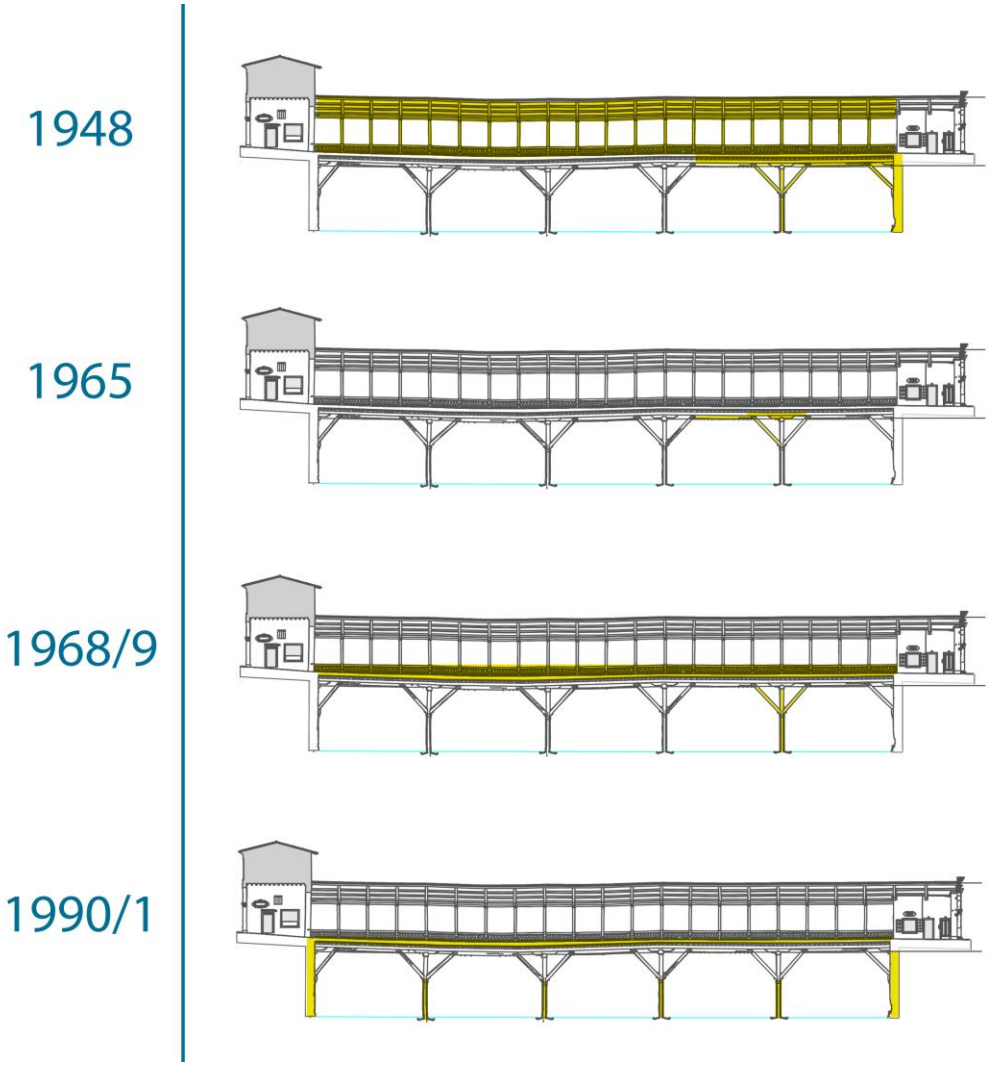


Figure 22 - Location of the main interventions of the 20th century

3.3 Foundation System

The timber bridge of Bassano presents a very unique structure and reconstructions and refurbishments provoked some changes in the structural behaviour, especially in the foundation system. The foundations changed many time during years manly because of the damages caused by the water force: the rivers is still stormy in this portion of the river. The Angarano side of the bridge has always been the most damaged, therefore many intervention started from this part.

It is possible to define to sum up the foundation system history in 4 main phases, explained in the following table.

Table 2 - Main construction phases of the foundation system

Before 1821	The original foundation system consist on timber piles on a line for each pier.
1821	Casarotti introduce a longitudinal beam where the 8 principal columns load, this beam is supported by 15 new piles (3/4 meter deep and 25-30 cm of diameter). This transversal element was used to redistribute the load from the upper part to the soil in a larger number of elements. At the time of the design project this element was not under flexure because it was directly supported by the soil; during the years soil erosion took place and in the 1968 the soil level was lower than the beam itself, therefore the beam result to be under bending.
1968	Benetti introduce four new elements in the piers, trestle-elements support the threshold beam; now the structure load not on one central line of piles but on several not aligned piles
1983	The *Cavezzali* and the corresponding piles are added even on the other three piles. The Benoto piles are added in order to sustain the structure. The intervention suffer strongly of erosion, after ten years most of the piles were no more working, the one still active presented a reduced cross section.
1990	Addition of 8 new piles in reinforced concrete (in order to avoid the problem of the erosion). New timber *cavezzali* were added under the threshold beam.

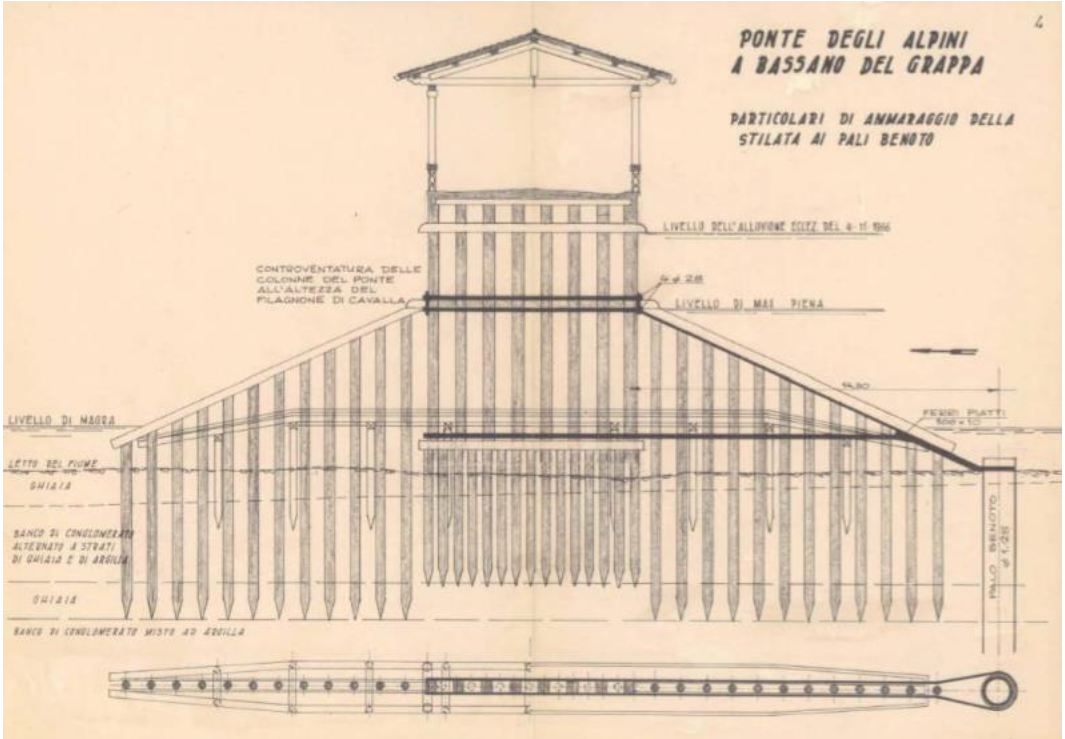


Figure 23 - Section of the bridge (1968). Ing.Roberto Benetti

4. HYDRAULIC ASPECTS

The purely hydraulic aspects are not studied in this work, however it is essential to make some considerations. The energy of the water in this part of the river is still high, the Brenta here is turbulent, and therefore, the natural erosion of the riverbed is an important aspect to take in account. Above all, the foundation beams designed by Casarotti suffer of the bed river changes: at the time of the construction it was supported by the ground and it was not under bending but after some years the soil level decrease and the beam started to be under bending.

A second aspect to consider concerns the phases of construction and repair. In order to access the bridge lower part, especially the foundations, it is necessary to redirect the water flow in one half of the bridge and then on the other half. In the definition of the working phases this is something to take in account.

Finally, in order to proceed the structural analysis the flow rate has to be defined. The impact between the water and the bridge causes stress on the elements, therefore an equivalent force it was calculated in order to represent the maximum flow rate of the Brenta River (ES.R.IDR.01, October 2015). The data were obtained by the ARPAV (Agenzia Regionale per la Prevenzione e Protezione Ambientale del Veneto). It is necessary to evaluate the water action with at least 200 years of return period (NTC 2008, par. 5.1.2.4). Considering the ARPAV data, corresponding to 200 years, the flow rate is equal to 2643 mc/s; however it is chose to take in account the flow of the event of the 1966 to be on the safe side. The overflow of the '66 presented a flow of 2800 mc/s and a velocity equal to 4 m/s on the Bassano side and 5.4 m/s on the side of Angarano.

Table 3 - Data of the river flow

Maximum flow rate	Q= 2800 m^3/s
Maximum river velocity	V= 5.4 m/s (Figure 24)
Equivalent force	$F=1/2\ \rho V^2 C_p$= 19.4 kN/m^2
Water density, ρ	1000 kg/mc
Drag coefficient, C_p	1.33

The calculation of the force is referred the drag coefficient is referred to the shape of the piers, in this case the shape is elongated and rounded, therefore the coefficient is chosen equal to 1.33 (L.Da Deppo, 2013).

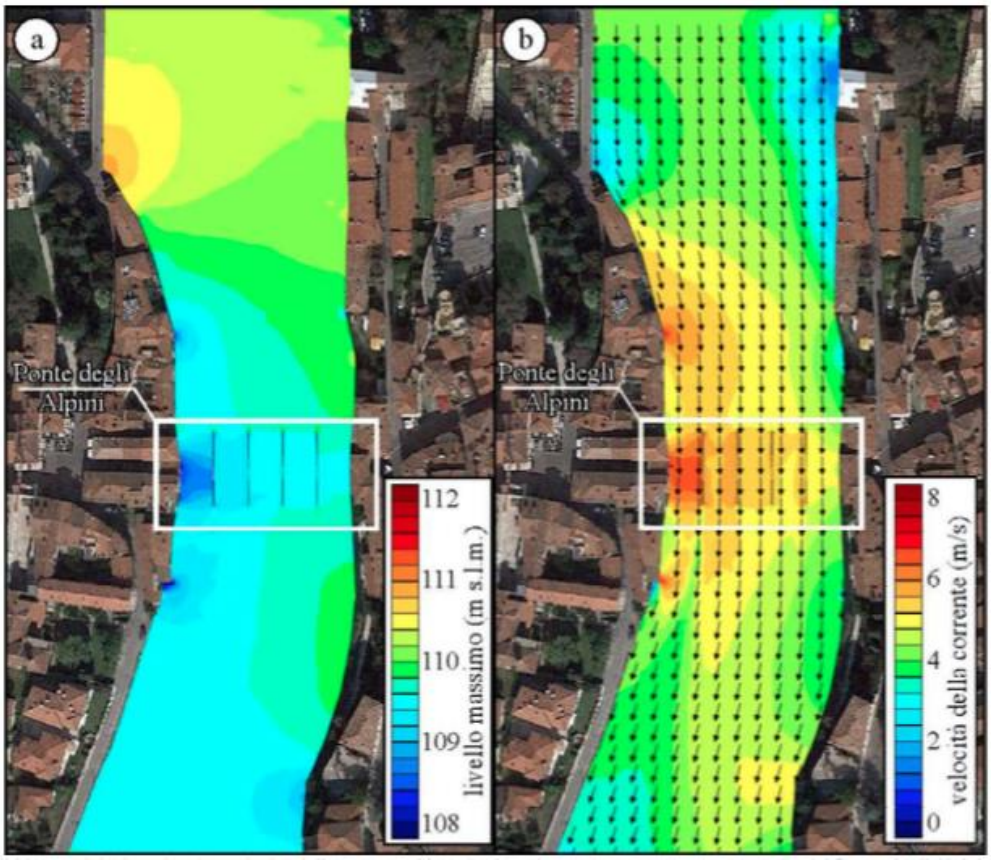


Figure 24 - a) water level, b) flow velocity (ES.R.IDR.01, October 2015)

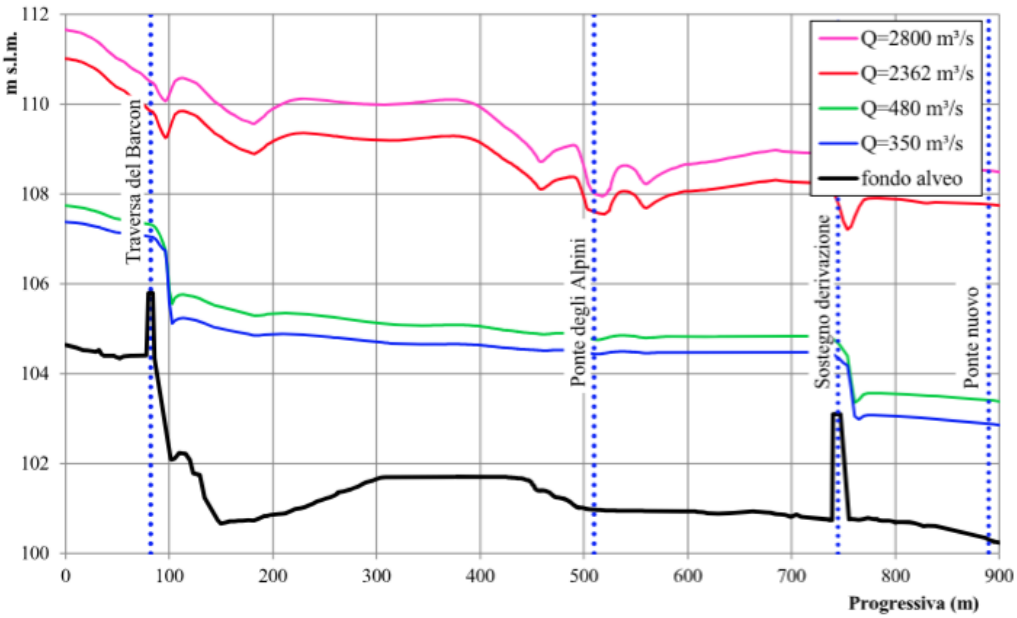


Figure 25 - Longitudinal profile of the Brenta River (ES.R.IDR.01, October 2015)

In the Figure 25 the longitudinal profile of the river is showed, in black the riverbed level (101 m on the sea level) and in pink the water level during the event of the 1966 (108 m). The water high can be assumed as 7 m (it does not reach the deck).

The hydraulic information and analysis were proceeded by the Department of Civil Environmental and Architectural Engineering of the University of Padua (report of October 2015), the ARPAV data from the 1984 to the 2012 were analysed.

5. STRUCTURAL ASSESSMENT

The aim of the project is to achieve a high level of knowledge of the timber bridge of Bassano in terms of structural behaviour and specifically on the safety of the foundation system. The understanding of out of plane behaviour is one of the main issues, for this reason two different bridge configurations are analysed: the actual state of the structure and a second configuration without the *Rostri* elements. The second configuration is analysed because historically those elements are the most damaged by floods and, therefore, vulnerable. The safety level of the structure is analysed with three main load states representing the conditions the bridge may suffer.

The Floods condition, this configuration is the one that caused the worst damages in the structure provoking the collapse many times, this is the case concerning the municipality and the community the most. A static analysis is proceeded taking in account the river load corresponding to the river flow of the 1966.

The daily state, this load configuration represent the daily life of the structure, a static analysis is proceeded. The daily loading is a good instrument to relate the effect of the exceptional actions to the every day stress level of the bridge.

The earthquake condition, in this case the seismic dynamic analysis is proceeded to evaluate the risks connected with this type of natural disaster. From the 2008, all the Italian regions result to have a seismicity hazard, therefore it is necessary to evaluate the structure response to this action. In this project the response spectrum analysis is proceeded following the Italian code.

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5.1 Background of the analysis approach

The use of timber in the historical constructions is part of the traditions of many countries, the properties and the characteristics of this material strongly change among species, therefore it is fundamental to have a deep knowledge of the material used and to refer to the local standards. The building techniques and the types of connections can change due to the local traditions, for example in the regions on the coast it is common to notice roofs made with the technique used for the keel of the boats.

The hardwood is not an engineered material but it is natural, therefore it is inhomogeneous and irregular. The presence of the fibres in the wood internal structure gives a strongly orthotropic behaviour; its properties differ in the two perpendicular directions (along the fibres and perpendicular to the fibres). In order to proceed the assessment of a timber structure it is necessary to evaluate the condition of the timber elements, their level of degradation and the humidity percentage influence the mechanical properties.

In the literature, just a few examples of assessment of historical timber bridges are present. One of those is the Barrackville Bridge in West Virginia (Spyrakos 1999), during this assessment, a geometrical survey and non-destructive test were proceeded and once all the data were collected, linear static analysis and seismic analysis were done. According to the AASHTO, a detailed seismic analysis is not required for simple span bridges; however, because of the historical significance of the bridge a multimode spectral analysis was applied to obtain the bridge response to four different spectra. In this case, a finite element analysis was employed to study the response of the structure. The Barrackville Bridge was modelled with three-dimensional beam elements; end-moment releases were specified at diagonal and vertical elements to simulate pinned joints.

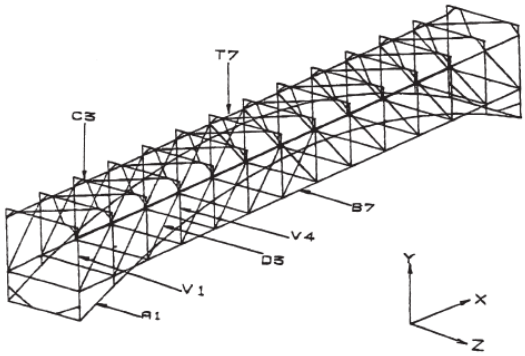


Figure 26 - Fem model of the Barrackville Bridge
(Spyrakos 1999)

The Wenxing Lounge Bridge in China is another example of structural analysis of an historic bridge made in timber (Chun, 2015). In the 2011 the bridge presented various types of damages such as asymmetric deformation, inclination and torsion. An FEM model was defined within two configurations, before and after the damages the result showed that some arch-element passed from a compression state to a tension one after the deformations; the structural safety and integrity was compromise, the stress level increased of the 50%. According to the FEM analysis, the main reason for the deformation was the strong asymmetric load. A combination method of structure dismantlement, member strengthening, and structure reintegration was chosen to be the adaptive repair method.

The bridge of Schaffhausen (Ceraldi 2003), a Swiss cover bridge designed the eighteen century, was analysed through a structural scheme constituted by one-dimensional elements with joint reproducing the connections. The analysis has been led in linear elastic range; in this case, the most important loads are the dead ones, especially because of the presence of structure with an extremely closely-traced texture. The stress level and the deformations were evaluated and the results show that the structure is far from the local collapse but the maximum inflection is little above the admissible values of 1/200. The most interesting aspect of the studied structure is due to its spatial behaviour, the combination of all the elements define a spatial entity with a box-like behaviour manly due to the presence of the a truss supporting the covering system which is rigidly connected to the side surfaces.

This behaviour has been verified confronting the structural analysis results of bridge spatial schemes with and without timber coverings on the sides and on the top.

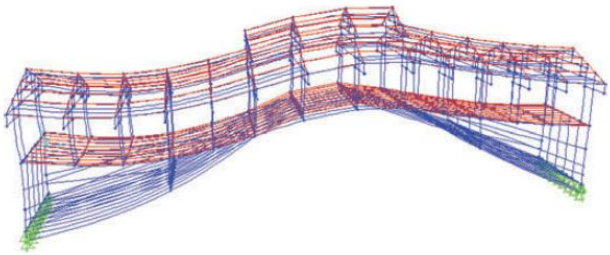


Figure 27 - Wenxing Lounge Bridge, FEM model (Chun 2015)

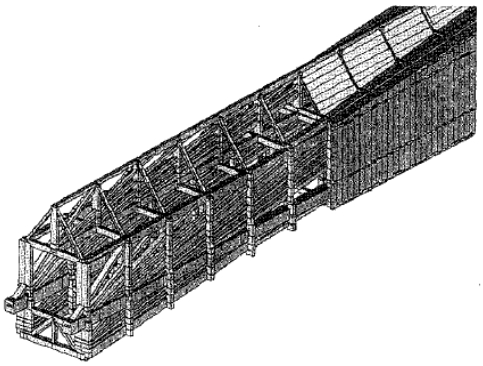


Figure 28 - Schaffhausen Bridge, rendering of the structural scheme (Ceraldi 2003)

5.2 Damage Evaluation

Timber structures are particularly vulnerable to the natural decay. The main source of damage for this type of structure is the humidity, both for the decrease of the mechanical properties and for the easy attack by insects. The nature itself of a bridge made in timber is therefore vulnerable and the structure requires constant interventions of maintenance, the durability was the reason why Palladio at the time suggested to use stones. However, thanks to the continuous repairs and sometimes reconstructions, nowadays we still have the chance to appreciate this masterpiece of the Italian architecture.
The University of Padua together with the Municipality of Bassano del Grappa analysed the damaged state of the bridge (*Report Bassano Bridge Assessment*, ES.R.RIL.04, 2015); in order to have a better understanding of the building a brief sum up of the conditions is presented.

The actual state of the roofing system is good and the structure is said to be stable, however big deformations are visible: the columns are tilting and the covering system is having a global movement. Some of the columns present torsion but the state of the timber is still good as it is verified by the penetrometer test. At the time of the survey the floor was made by 35 cm of sand and rubble stones giving a high weight to the structure, nowadays this layer has been substituted by a light timber floor.

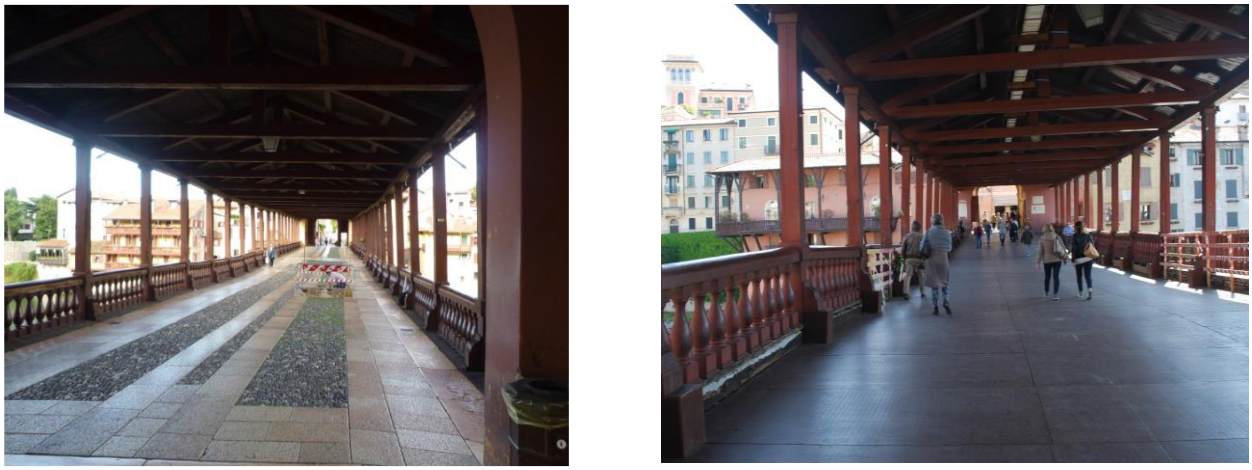


Figure 29 - The deck of the bridge before and after the last changes

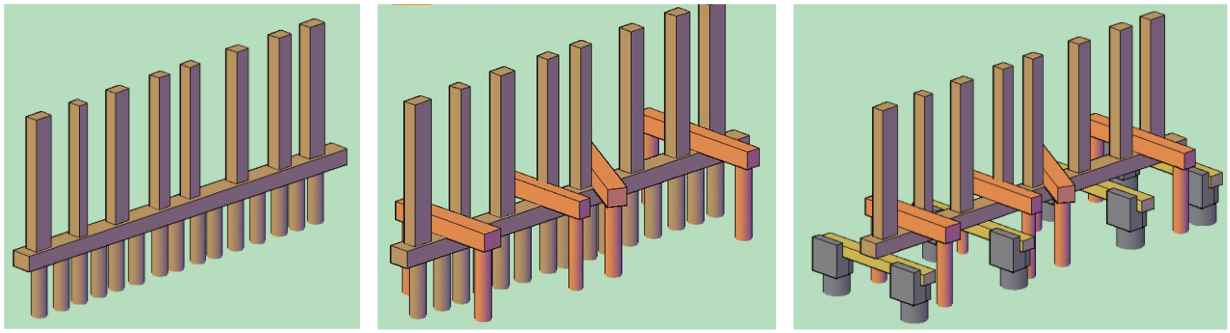
The deck shows high levels of deformation in the horizontal direction. The supporting and longitudinal beams show high levels of degradation and the consistence of the timber is really weak.

Figure 30 - Degradation of the timber beams

The piers are strongly deformed, horizontal and vertical displacements are easily visible. These damages are mainly due to the weak conditions of the foundations system. The poles constituting the piers structure were repaired in the 1990 with resins injections but this caused a faster degradation especially at the contact between the two materials. In the 2005/6 a new intervention was done an in some cases new elements added. The problem of the resins is still unsolved. In addition, the steel elements added in the columns caused the propagation of cracks and increased the water attack.

The foundations are the elements presenting the highest level of degradation, the forth pier is the in the worst conditions because the water force is higher in this side. As already mentioned, several interventions were done in this part of the structure; in the 1968 and 1983 new trestle elements were added in order to support the transversal beam. The reason of this intervention was the level of degradation and the deformation due to the settlement. Just in the 1990, a new refurbishment was necessary and new elements below the transversal beam were added. This time concrete vertical

elements are supporting the wooden beams. From the assessment of 2014 it comes out that the 90's structure is nowadays the only supporting, the wooden piles are suffering from erosion and they are no more effective.



Original foundation | Intervention of the 1968-83 | Intervention of the 1990

Figure 31 - Fonudation interventions (ES.R.RIL.04, 2015)

5.3 FEM model

The purpose of the structural analysis of the bridge is to understand the behaviour of the structure; it describes the overall structural system and it determines the stress levels within the structure. In case of historic buildings, the structural analysis is a difficult process with many uncertainties because it is not always possible to estimate the strength and stiffness of structural members and joints, the load history and the boundary conditions. The structural analysis of the bridge is carried out with the structural software Straus 7, which is a finite element analysis system developed by the University of Sydney and the University of New South Wales. It is adopted a linear element model due to the beam structure.

The structural behaviour of the Bassano Bridge is analysed with a FEM model, the geometrical drawing was done with the software Autodesk-Autocad and then, imported in Straus7. The CAD model is based on the drawings done after the years 2014/2015 surveys, when the 3D laser scanner was used and the analysis of the damages has been performed.
It is proposed a numerical model with "beam 2" elements in accordance with the geometrical survey. In order to use beam elements the dimensions perpendicular to the axis of the bar have to be small in relation to the bar's length. Beam elements may have axial deformation, shear deformation, curvature torsion; therefore they can describe axial force, shear force and moment.

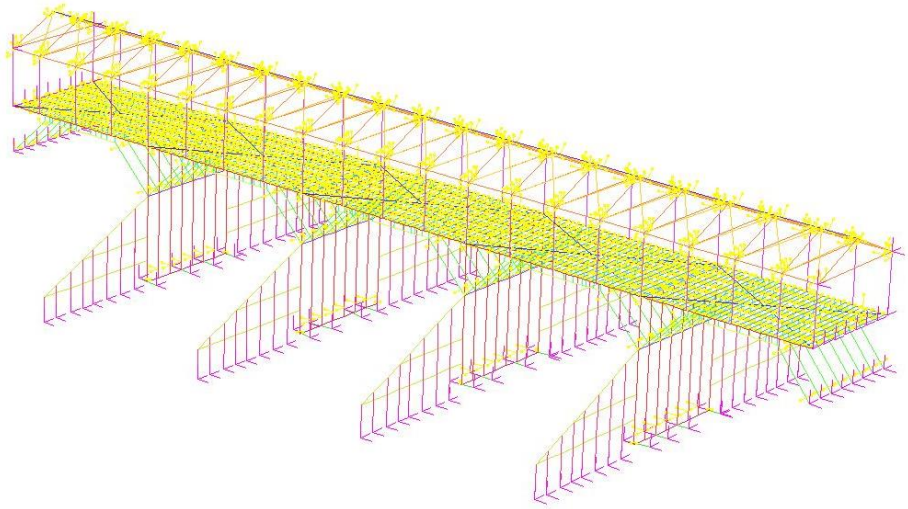


Figure 32 - FEM model of the bridge, linear elements

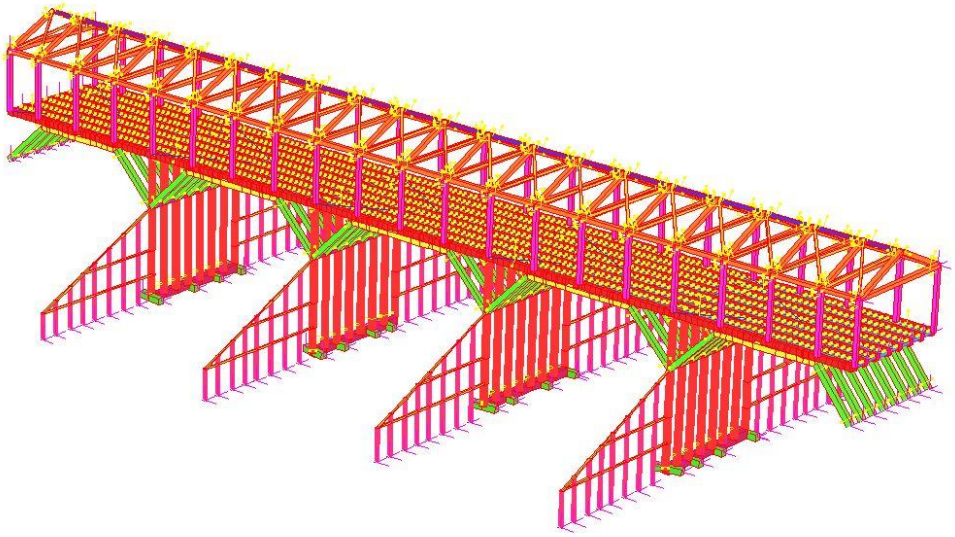


Figure 33 - FEM model of the bridge, solid view of the linear elements

Analysing a timber structure, it is fundamental a good understanding of the joints and of the connections between the timber elements (Spyrakos 1999). End-moment releases were specified at diagonal, vertical and horizontal elements to simulate pinned joints. In the historic timber structure is hard to find a fix joint but it is most probable to have hinges between the elements. As an example, at the connection between the primary and secondary beams and at the one between the supporting beams and the struts the two main moments are released (Figure 34 and Figure 37).

Figure 34 - Connection between longitudinal beams and secondary ones.

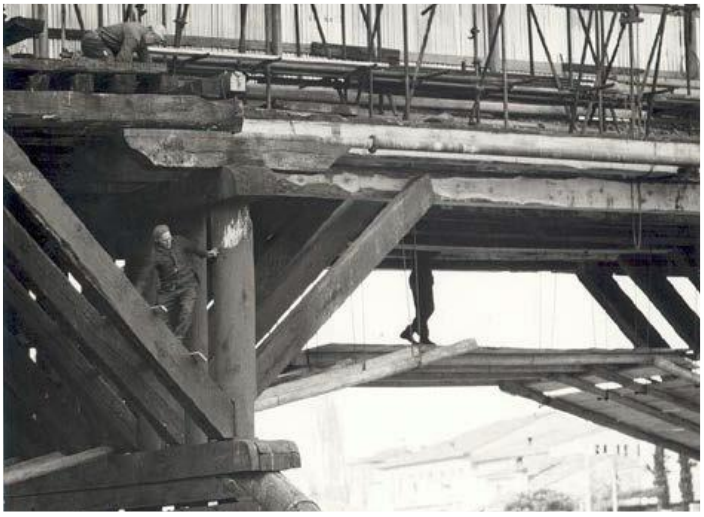


Figure 35 - Refurbishment of the 1965.

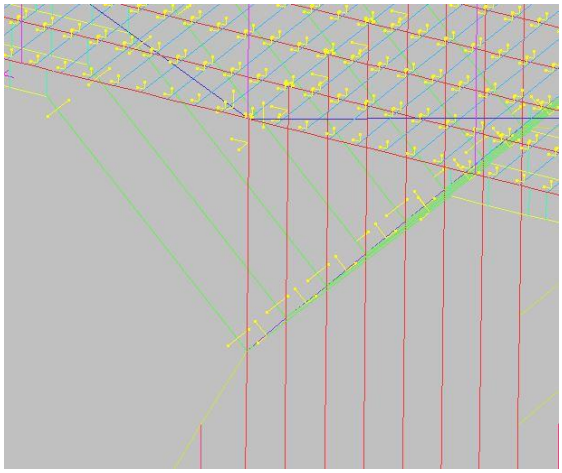


Figure 36 - FEM model, End Releases, pier 2

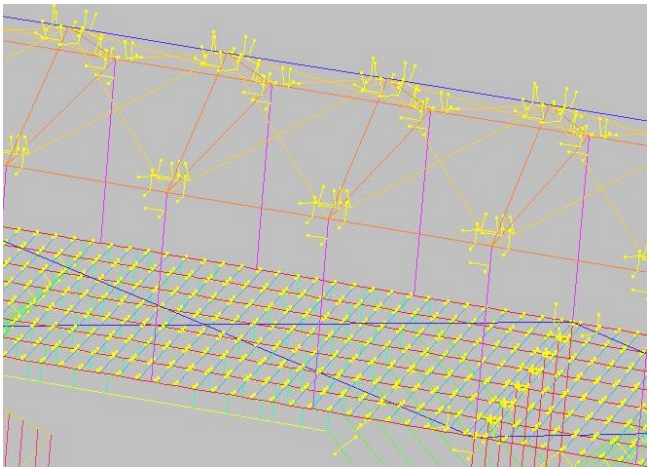


Figure 37 - FEM model, End Releases, second span

The damage level is considered in the FEM model mainly in the mechanical parameters. However, concerning the foundation system, the "*cavezzali*" are modelled but not the original poles under the longitudinal beams, this choice is due to the high level of degradation: many of those poles are no more effective.

Figure 38 - Damage of the foundation, the elements "*Cavezzali*" are visibily new

Figure 39 - Picture of the foundations in 2014, some elements are no more working.

Truss elements are used in the piers, to connect vertical elements and to simulate the planking action.

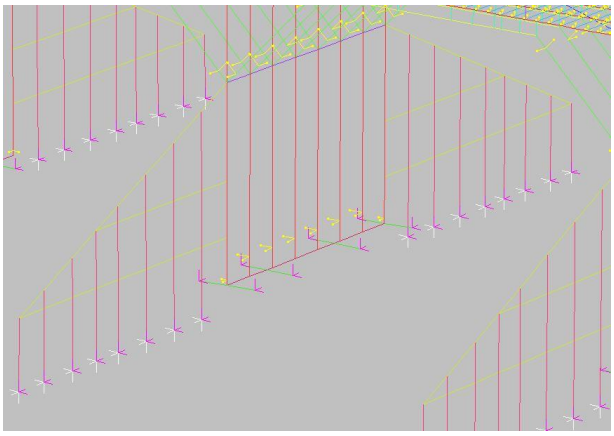


Figure 40 - Truss elements (in yellow)

5.4 Structural elements and material properties

The 3D model represents the structural elements and their relative connections, the entire structure is made with line elements in order to better represent the structural behaviour of the timber structure made with timber beams. Regarding the mechanical properties, the UNI EN 2338-2009 (class of resistance) and the UNI 11035-2:2003 are used. It is retained to be important the use of the Italian code in order to have the timber characteristics proper of the Italian timbers. The structure present two

species of timber, chestnut and larch, in the following tables their mechanical characteristics are presented.

Table 4 - Mechanical properties (UNI 11035-2:2003), Class S2 (intermediate damage)

Species			larch	chestnut
Class			S2	S
Density	ρ_{mean}	kg/m3	600	550
Young modulus	$E_{0,mean}$	MPa	12000	11000
	$E_{90,mean}$	MPa	400	730
Shear modulus	G_{mean}	MPa	750	950
Compression strenght	$f_{c,0,k}$	N/mm^2	24.0	22.0
	$f_{c,90,k}$	N/mm^2	4.0	3.8
Tensile strenght	$f_{t,0,k}$	N/mm^2	19.0	17.0
	$f_{t,90,k}$	N/mm^2	0.6	0.5

In order to define the mechanical properties it is necessary to define the elements class (UNI 11035-2:2003, §5.3). Regarding the elements made in larch, this species belongs to the coniferous and three possible classes are present in the code, the classes are depending on the degradation level. Due to the several restorations of the timber elements, the conditions of the larch beams are not severe, therefore it is chosen the middle-class S2. The code presents just one class S for elements made in chestnut (Table 4).

The element's cross section is defined in the following table:

Table 5 - Element's characteristics

element		cross section (h x w) [m]	species
cantilever	modiglione	0.33 x 0.35	larch
roof ridge	colmo	0.19 x 0.15	larch
columns	colonne	0.25 x 0.25	larch
longitudinal beams	longherrine	0.38 x 0.35	larch
piers	pile	θ 0,5	chestnut
truss beams	copertura	0.25 x 0.28	larch
struts	saettone	0.33 x 0.31	larch
supporting beams	serraglia	0.33 x 0.31	larch

secondary beams	travetti	0.20 x 0.20	larch
bracing beams	controventi	0.15 x 0.15	larch
transversal beams	traversa	0.20 x 0.20	chestnut
additional poles	rostri	Θ 0.3	chestnut

5.5 Timber diaphragm

5.5.1 Literature background

In the out-of-plane direction, the response of the entire structure is dependent on the characteristics of the wooden deck and in particular on its in-plane stiffness and on the quality of the connections. International guidelines on the seismic rehabilitations of buildings (ASCE/SEI 41-06 2007, NTC 2008) and international literature (Piazza 2008, Brignola 2012) clarify the importance of correctly including the diaphragms flexibility and the wall-to-floor connections properties in the analysis of URM buildings. Although several experimental studies are done on masonry structure, the role of the timber diaphragm is fundamental even in the case of timber structures such as the bridge of Bassano.

In Brignola 2012 the in-plane stiffness of timber diaphragms is analysed through experimental data and comparison of the main codes. It is explained how in the ASCE/SEI 41-13 document, depending on the sheathing properties, different wood diaphragms type are defines. In case of linear analysis procedure, values for the shear stiffness G_d are provided.

Table 6 shows the equivalent shear stiffness of the diaphragms G_d is defined as the shear modulus multiplied by the thickness.

Table 6 - Expected stifness Gd values for some diaphragms types (ASCE/SEI 41-06).

Diaphragm type	G_d [kN/mm]
Single straight sheathing	0.35
Double straight sheathing	
Chorded	2.67
Wood structural panels overlay on straight sheathing	
Un-chorded	1.24
Un-blocked, un-chorded	0.87
Un-blocked, chorded	1.58
Blocked, un-chorded	1.24
Blocked, chorded	3.20

However, the diaphragm properties are influenced by the presence of the perimeter chord and by the presence of blocking in correspondence to the edges. The span at the middle of the panel (Δ) has to be evaluated with the formula:

$$\Delta = \frac{v \cdot (L/2)}{G_d}$$

Where L is the span and v is the shear stress multiplied by the thickness.

The Italian technical code for constructions (NTC 2008) does not specify nor does it suggest how to calculate the in-plane mechanical parameter of wood diaphragms. The National Research Council Document (CNR DT 2006/2007), as well as the Eurocode 5, provides just few indications regarding the mathematical hypothesis to assume, without giving any further information.

In Brignola, Pampanin, Podestà (2012) several types of timber diaphragms were tested in order to compare the experimental stiffness with the values proposed by the codes. The ASCE suggestions result to be able to represent the result in case of straight flooring boards but not really in case of more complex systems; moreover it is not able to predict the stiffness contribution related to shear-wall-to-floor connectors.

In Piazza 2008, an experimental analysis was performed: specimens of timber floors were tested in order to determine the in-plane strength and stiffness of the floors typical of the historical buildings in Italy. The test was set up in order to allow the free in-plane deformation of the floor itself subjected to lateral load. The purpose of the research was to calibrate engineered models that can be used for studying existing structures. In case of masonry building it is well known that a key role is played by the horizontal diaphragms in the transmission of the seismic actions, in this view, the in-plane stiffness and the connection between floors and walls are the key factors. Both real size and smaller samples are tested and the floor stiffness was defined in two ways: as the ratio between the force and the displacement at the middle, k and an equivalent shear modulus has been defined in order to get independent results from the geometry:

$$G = \frac{F_T \cdot L}{4 \cdot \Delta \cdot B \cdot t} (MPa)$$

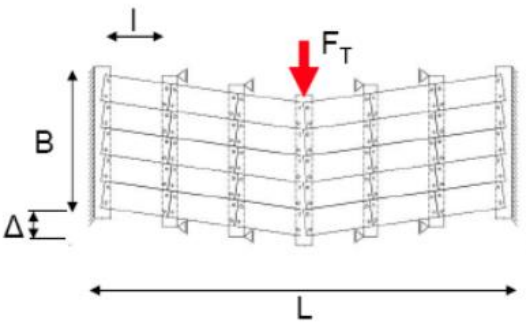


Figure 41 - Piazza et al. 2008, in plane deformation of timber floor

In Figure 42 the results for small specimens are shown in term of force versus the displacement of the central timber beam.

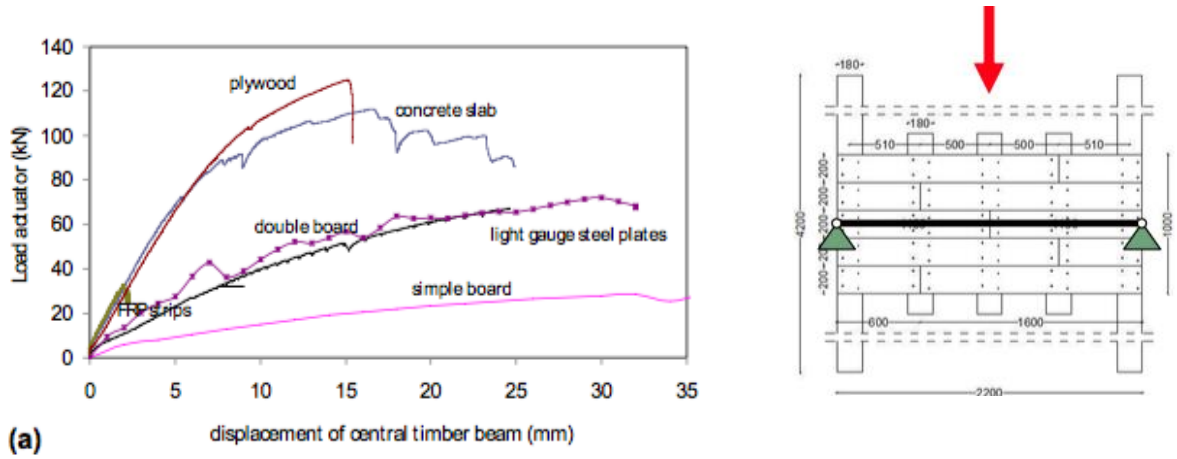


Figure 42 –Timber diaphragm tested for out of plane behaviour. On the left the results in terms of load/displacement and on the right the test set-up (Piazza et al. 2008).

Different types of strengthening were tested, analyzing the experimental results in terms of equivalent shear modulus. It was concluded that, compared with the reference floor system, the double board floor seems to increase the original stiffness more or less ten times the experimental, the concrete slab and plywood planks seem to increase up to one hundred times.

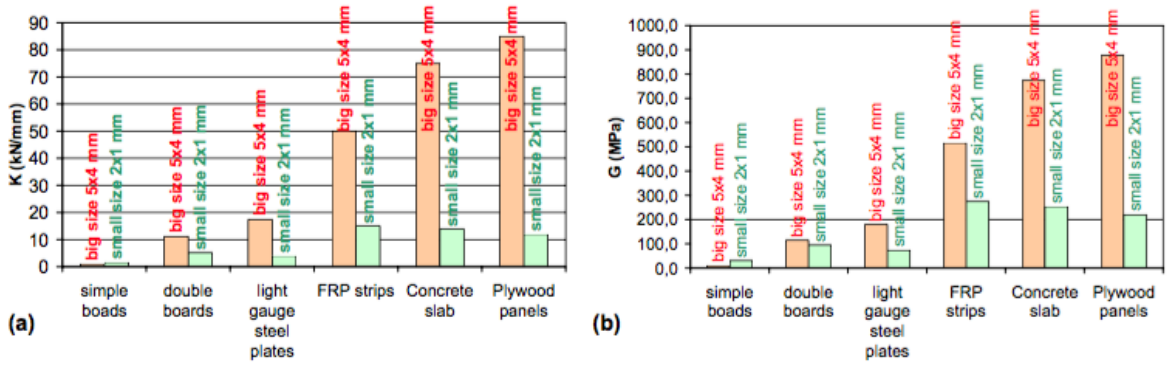


Figure 43 - Experimental results for small size and real size floor specimens in terms of general stiffness parameter k (a) and a equivalent shear stiffness parameter G (b) (Piazza et al. 2008).

5.5.2 Calibrated equivalent system

The cited papers show how important can be the role of the timber floors in the structural behaviour and, on the other hand, how complex is the proper evaluation of the stiffness of the timber boarding. In order to analyse the out of plane behaviour of the Bassano Bridge it was necessary to take in account

the in plane stiffness of the timber diaphragm. A refined model representing each element such as the boards, the secondary beams and all the connection is time consuming and out of the scope of this analysis; this type of modelling was done by a research group at the University of Padova and the values of lateral force and relative displacements were obtained.

The refined model of the deck of the Bassano Bridge consider its actual geometry and the connections made with historical nails with 8 mm diameter and a length of 164 m. Two connections for each board are considered and the connection is done each seven secondary beams. Three cases are considered in order to take in account an increasing level of damage: an original condition and two damaged conditions where the mechanical characteristics were decreased. The table below shows the results in terms of displacement corresponding to a force of 6940,8 N:

Table 7 - Displacements of the deck

	Displacement	stiffness [N/mm]
Original state	289,79 mm	23.95
Deformed state	869,36 mm	7.98
Severe damages	1214, 97 mm	5.71

In the FEM model of the entire Bridge structure it was necessary to define new simplified elements able to represent the stiffness of the timber deck, an equivalent calibrated system is defined. Once the deck was studied through the refined model, it was possible to insert additional elements in the bridge model in order to consider the in plane stiffness of the timber boards. In each span two diagonal elements are inserted in order to represent the role of the diaphragm, the stiffness of those elements is calculated on the base of the refined model results.

In order to define the dimension of the cross section of the new diagonal elements, in Straus 7 a simple model constituted by two longitudinal beams and two secondary beams is drawn with the addition of the two diagonals. Applying a point load on the node 3 (Figure 44) the stiffness is iteratively changed until the displacement on the node 4 is equal to the displacement of the deck's span (obtained by the refined model). In this way the two fictitious diagonal elements are designed in such a way that they represent the same stiffness of the timber board.

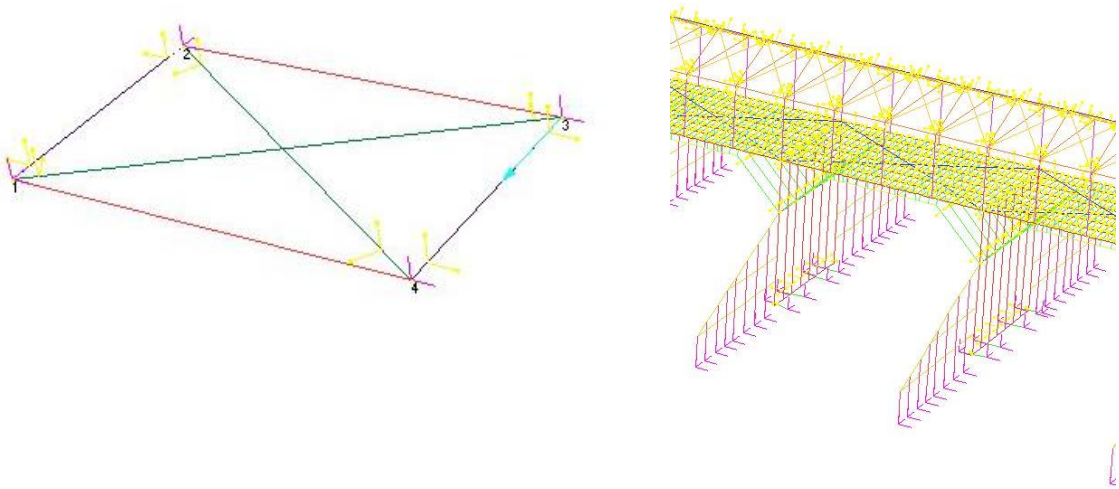


Figure 44 - Equivalent system for the bridge's deck.

In the following tables the results are presented, the stiffness of the diagonal elements is calculated considering the young modulus in all the cases equal to 12 GPa.

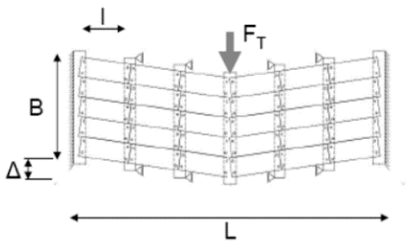
Table 8 - dimensions of the diagonal elements

	Diagonal elements		
	Cross section		Stifness K= EA / L
	area	t x w	
Original state	0.00005 m^2	0.01 x 0.005 m	0.039 kN/mm

In case of the original state, with a thickness of 1 cm, the width of the diagonal element must be 0.5 cm in order to have the defined displacement.

Once the horizontal displacement for a defined force is known, it is possible to calculate the shear modulus of the boarding, G.

Table 9 - Calculation of the deck stiffness G_d

	Ft = 6940,8 N x 2
	B = 8100 mm
	L = 25600 mm
	Δ = 289,79 mm
	t = 40 mm
	G = 0.00095 kN/mm^2 = 0.95 MPa
	G_d = G*t = 0.038 kN/mm

According to the code ASCE/SEI 41-13, in case of a simple boarding, the stiffness G_d is equal to 0.35 kN/mm. This value of stiffness has been analysed with the simplified model in order to obtain an equivalent structure with the two diagonals.

Table 10 - Calculationof the linear stifness according to the plane stifness from ASCE/SEI 41-06

G_d	0.35 kN/mm	Gd= 2F_T 2L / 4ΔB
F_T	6940.8 N	
L	12.8 m	
B	8.1 m	
Δ	0.031 m	

G_d= 0.35 kN/mm	Cross section 0.04 x 0.012 m Area 0.0048 m^2 K= EA/L= 4.5 kN/mm

The values given by the ASCE/SEI 41-06, turn out to give a higher stiffness to the diagonal elements, this option is chosen. Diagonal elements with cross section 0.04 x 0.012 m are chosen for the bridge model. Having a higher level of stiffness, is safer in the seismic assessment and moreover it result more reliable in the overall behaviour of the structure

5.6 Loads and loads combination

Selfweight

Straus 7 software takes in account the self load automatically. What it is necessary to input are the characteristics of the materials, especially cross-section and density. The bridge is mainly made of two

types of timber: chestnuts for the submerged parts and larch for the others. The use of chestnuts for the parts in the water is due to its higher resistance. Referring to the Italian code (UNI 11035-2:2003) the density of larch is 600 kg/m3 instead for the chestnuts is 550 kg/m3.

Non-structural weights

In addition to the self load of the structural element, it is necessary to take in account the weight of the elements such as the roof and the deck of the bridge. Those element are not modelled, therefore their weight is represented by an external distributed load. The roofing system weight is applied to the trusses and the load of the deck is applied to the secondary beam, both as linear distributed loads.

The roof is made with timber boards and tiles, with a weight of respectively 80 kg/m2 and 27 kg/m2. The distance between the trusses is 3.3 m. The deck present a thickness of 12 cm and the density of larch is equal to 600 kg/m^3.

Table 11 - Not structural weights, G2

G2	Roof	1.00 kN/m^2	3.30 kN/m
	Deck	0.70 kN/ m^2	0.28 kN/m

Live loads

It is necessary to take in account the effect of the crowd on the bridge and of the accidental loads that may occur on the structure. We have to consider that the bridge is no more accessible by vehicles but it is still a pedestrian walkway. In order to evaluate this load the Italian code is considered: the class of use for the floor is the C3 and for the roof is H1 (NTC2008- table 3.1.II).

Table 12 - Liveloads (NTC2008)

Location	Use class		Load
Floor	C3	Areas without obstacles where people can free move (museums, expositions rooms, railway stations, gyms… etc.)	5 kN/m2
Roof	H1	Roofs where the access is possible just for maintenance.	0.5 kN/m2

Wind Load

The calculation of the wind action is done referring to the Italian code NTC2008 (§3.3), the wind load is considered to be horizontal, clearly the wind action is dynamic but, as the code suggests, it is possible to consider it as an equivalent static load.

The calculation of this action is of course depending on the location of the structure, on its altitude respect to the sea level. The Veneto region belongs to the first zone.

Table 13 - Wind load data (NTC 2008, 3.3)

$v_{b,0}$	25 m/s
a0	1000 m
k_a	0.01 l/s

The load pressure is given by the following expression:

$$P = q_b\, c_e\, c_p\, c_d$$

In the calculation of the shape coefficient we refer to the code supplement *Circolare 617-2009* for the NTC 2008. In case of a gabled structure, the shape coefficient needs to be evaluated as in the picture below:

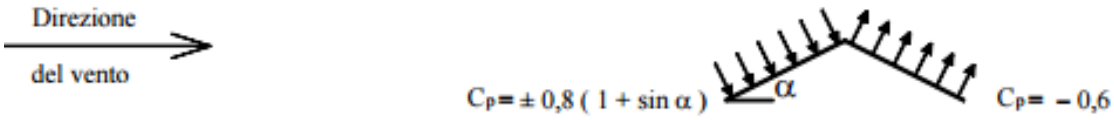


Figure 45 - Influence of the shape in the wind calculation

The inclination of the gable is 25°, therefore Cp= -0,6 and +1.2

Table 14 - Wind load calculation

q_b	Reference kinetic pressure	$=1/2\ \rho\ v_b^2 = 390.6$	N/m^2
ρ	Air density	1.25	kg/m^3
c_e	Exposure coefficient	2.066	-
c_p	Shape coefficient	1.2 and -0.6	-
c_d	Dynamic coefficient	1	-
p	Wind load	970 and -484	N/m^2

The exposure coefficient is calculated depending on the height of the structure and the type of territory is the building located on.

The wind pressure result to be equal to P = + 970 N/m^2 and -484 N/m^2

Snow Load

The snow action is evaluated referring to the Italian code NTC 2008 (§3.4), the snow is consider to be a vertical load action on the roof of the building. Veneto region is belonging to the zone one.

$$q_s = \mu\, q_{sk}\, c_e\, c_t$$

Table 15 - Snow load calculation

q_{sk}	Characteristic value of the snow load	1.5	kN/m^2
c_e	Exposure coefficient	1.0	-
c_t	Thermal coefficient	1.0	-
μ	Shape coefficient of the roof	0.8	-
q_s	Snow load	1.2	kN/m^2

The exposure coefficient is considered equal to one because the topography of the site may be considered as normal (NTC 2008, table 3.4.I). The thermal coefficient is considered as one because it is not present any thermal insulation or any warmer place that can make the snow to melt. The shape coefficient is 0.8 due to the inclination of the roof, which is less than 30°.

The snow load result equal to 1.2 kN/m^2.

Flow rate

The river action is evaluated considering the overflow of the 1966 in order to take in account the most dangerous case, the water action is evaluated at the chapter 4. An equivalent static force is taken in account:

$$F = 19.4 \text{ kN/m}^2$$

The load is applied to the piers until the high of 7 m. The force F is multiplied by the width of the piers in order to obtain a distributed force on those.

Loads combinations

In the Italian code NTC 2008, several combinations of the actions are taken in account. To consider model uncertainties and dimensional variations it is necessary to use partial factors: γ_G and γ_Q for permanent actions and variable actions respectively. The factors ψ are coefficients used for the variable actions, to consider the improbable coexistence of all the variable actions in the same moment (Table 16, Table 17 and Table 18).

Table 16 - Safety coefficients (NTC2008)

γ_{G1}	Permament loads	Favorable	1.00
		Unfavorable	1.35
γ_{G2}	Permanent loads (not structural)	Favorable	0.00
		Unfavorable	1.50
γ_{Q}	Variable loads	Favorable	0.00
		Unfavorable	1.50

Table 17 - Loads and their respective safety and combination coefficients

Load		Type of load	direction	Ψ	γ
Self Weight	G_1	Permanent load	Z	-	γ_G=1; 1.3
Roof Weight	G_2	Permanent load (non structural)	Z	-	γ_G=1; 1.5
Live load, Roof	$LL_{,R}$	Variable load	Z	Ψ=0	γ_Q=0; 1.5
Live load, Floor	$LL_{,F}$	Variable load	Z	Ψ=0.7	γ_Q=0; 1.5
Snow	S	Variable load	Z	Ψ=0.5	γ_Q=0; 1.5
Wind	W	Variable load	Y	Ψ=0.6	γ_Q=0; 1.5
Water flow	R	Variable load	Y	-	γ_Q=0; 1.5

Several types of combinations are present in the code, for both ultimate limit states and serviceability limit states. The fundamental combination (ULS) is:

$$\Sigma\gamma_{G,j}G_{k,jj}+\gamma_{Q,1}Q_{k,1}+\Sigma\gamma_{Q,i}\psi_{0,i}Q_{k,i}$$

Where G represents permanent actions and Q the variable actions.

Table 18 - Loads combinations adopted

Combination 1	1.3 G_1 + 1.5 G_2 + 1.5 R + 0 $LL_{,R}$ + 0.7* 1.5 $LL_{,F}$ + 0.5*1.5 S + 0.6*1.5 W
Combination 2	1.3 G_1 + 1.5 G_2(1,3,5) + 1.5 R + 0 $LL_{,R}$(1,3,5) + 0.7* 1.5 $LL_{,F}$(1,3,5) + 0.5*1.5 S(1,3,5) + 0.6*1.5 W (1,3,5)
Combination 3	1 G_1 + 1.5 G_2(1,3,5) + 1.5 R + 0 $LL_{,R}$(1,3,5) + 0.7* 1.5 $LL_{,F}$(1,3,5) + 0.5*1.5 S(1,3,5) + 0.6*1.5 W (1,3,5)
Combination 4	1.3 G_1 + 1.5 G_2(2,3,5) + 1.5 R + 0 $LL_{,R}$(2,3,5) + 0.7* 1.5 $LL_{,F}$(2,3,5) + 0.5*1.5 S(2,3,5) + 0.6*1.5 W (2,3,5)
Combination 5	1 G_1 + 1.5 G_2(2,3,5) + 1.5 R + 0 $LL_{,R}$(2,3,5) + 0.7* 1.5 $LL_{,F}$(2,3,5) + 0.5*1.5

	S(2,3,5) + 0.6*1.5 W (2,3,5)
Combination 6	1.3 G_1 + 1.5 G_2 + 1.5*0.6 R + 0 $LL_{,R}$ + 1.5 $LL_{,F}$ + 0.5*1.5 S + 0.6*1.5 W
Combination 7	1.3 G_1 + 1.5 G_2(1,3,5) + 1.5*0.6 R + 0 $LL_{,R}$(1,3,5) + 1.5 $LL_{,F}$(1,3,5) + 0.5*1.5 S(1,3,5) + 0.6*1.5 W (1,3,5)
Combination 8	1 G_1 + 1.5 G_2(1,3,5) + 1.5*0.6 R + 0 $LL_{,R}$(1,3,5) + 1.5 $LL_{,F}$(1,3,5) + 0.5*1.5 S(1,3,5) + 0.6*1.5 W (1,3,5)
Combination 9	1 G_1 + 0 G_2 + 1.5 R + 0.6*1.5 W

In order to obtain the maximum forces and stress levels the envelope of the eight combinations is calculated using Straus7.

5.7 Analysed models

The lateral elements at the foundation, called *"Rostri"* in Italian, have been damaged during the floods of the past, especially during the 1966. They are the element most exposed to the water flow and therefore vulnerable.

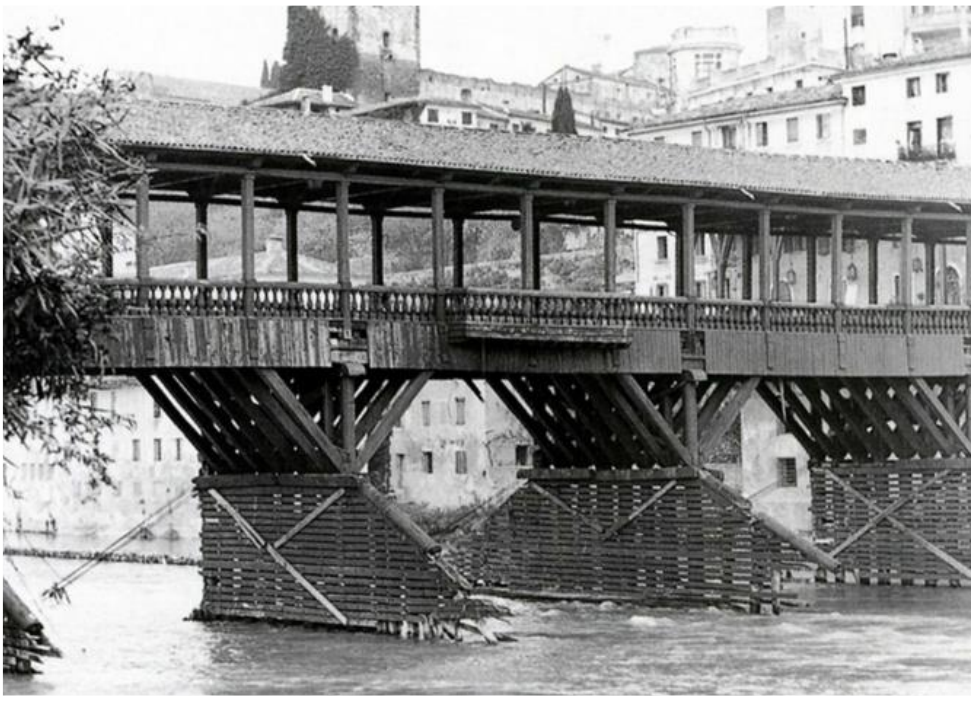


Figure 46 - Damaged structure after the overflow of the 1966

Those parts of the structure are protecting the main central poles against the river. In case of out-of-plane behaviour, those elements make a strong difference in the overall behaviour. Working as

protection and as buttresses of the piers, their effect is important. Therefore, it is of interest to understand their role and the stress level in case of a damaged configuration of the bridge corresponding to the absence of the *Rostri* elements.

The structural analysis are performed in two different models: A and B (Figure 47) in order to understand the effect that the absence of the Rostri can have in the structural behaviour. However, has to be clear that two limit cases are studied, all the cases in between are possible and they represent the gradual states of damage.

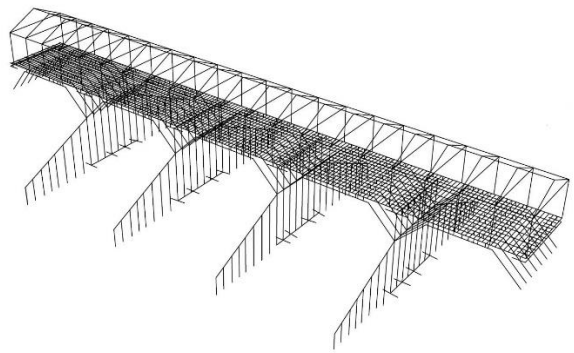
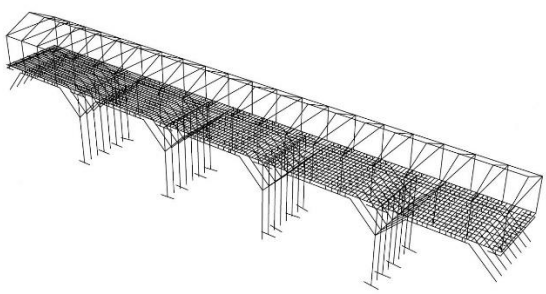
	
A - bridge with *rostri*	B – Bridge without *rostri*

Figure 47 - Analysed models

5.8 Foundations verification method

The main foundation elements, the *Cavezzali* and the threshold beams, are analysed for the ultimate limit states considering the envelope of the nine combinations.

In order to evaluate the safety level of the timber elements we refer to the NTC 2008 (chapter 4.4), to the CNR DT 206/2007 and to the Eurocode 5. The rheological behaviour of wood makes necessary to consider the duration of the loads because the load duration influence the strength and the deformations. In this analysis the fundamental combination has been used and, considering the types of loads affecting the structure, the short-term duration is assumed because evaluated more severe than the long-term action. The Service Class 3 is assumed because of the climatic conditions, this class correspond to high levels of moisture content. The design values of the mechanical properties must be defined multiplying the characteristics values for k_{mod} and dividing or the partial safety factor.

Due to the previous considerations it is possible to define the value of k_{mod} as 0.7;γ_m is equal to 1.5 in case of hardwood.

$$Xd = \frac{Kmod\,Xk}{\gamma m}$$

Table 19 - characteristic and design values of the chestnut strength

	$f_{m,k}$	$f_{v,k}$	$f_{m,d}$	$f_{v,d}$	$f_{c,0,d}$	$f_{c,90,d}$
	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2	N/mm^2
chestnut	28	2	13.07	0.93	10.26	1.73

Due to the homogenous section of the hardwood elements, the resistance to positive and negative bending moments is the same, therefore it is considered the absolute maximum.

Referring to the NTC 2008, in the following table the verifications formulas and procedure and briefly explained.

Table 20 - Verification method, NTC 2008

Compression parallel to the fibers	$\sigma_{c,0,d} > f_{c,0,d}$ Where $\sigma_{c,0,d}$ is the compression stress parallel to the fibers, $f_{c,0,d}$ is the compression resistance parallel to the fibers.	NTC 2008_4.4.8.1.3
Compression perpendicular to the fibers	$\sigma_{c,90,d} > f_{c,90,d}$ Where $\sigma_{c,90,d}$ is the compressive stress perpendicolar to the fibers, $f_{c,90,d}$ is the compression resistance perpendicular to the fibers.	NTC 2008_4.4.8.1.4
Flexure	$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$ $k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$ Where $\sigma_{m,y,d}$ and $\sigma_{m,z,d}$ are the maximum stress levels due to bending in the xz and xy planes respectively. $f_{m,y,d}$ and $f_{m,z,d}$ are the flexure resistances.	NTC 2008_4.4.8.1.6

	k_m is a coefficient taking in account the inhomogeneity of the material, for rectangular sections it is equal to 0.7.	
Shear	$\tau_d \leq f_{v,d}$ Where τ_d is the tangential stress calculated with the Jourawsky method, $f_{v,d}$ is the correspondent resistance to shear	NTC 2008_4.4.8.1.9
Torsion	$\tau_{tor,d} \leq k_{sh} f_{v,d}$ Where $\tau_{tor,d}$ is the maximum tangential stress due to torsion k_{sh} is a coefficient equal to $k_{sh} = 1 + 0.15\frac{h}{b} \leq 2$ in case of rectangular sections. $f_{v,d}$ is the correspondent resistance to shear	NTC 2008_4.4.8.1.10
Shear+Torsion	$\frac{\tau_{tor,d}}{k_{sh} f_{v,d}} + \left(\frac{\tau_d}{f_{v,d}}\right)^2 \leq 1$	NTC 2008_4.4.8.1.11

6. STATIC ANALYSIS, FLOOD CONDITIONS

In the analysis of the bridge the first step is the static approach, the static loads are evaluated as previously explained and the combinations implemented in the software. The aim of this analysis is to understand the safety level of the foundations elements and the role of the lateral elements of the piers, called *Rostri* in Italian. The static analysis is mainly focused on the foundation system because it presents the main damages and structural problems, this part of the structure is the less studied because of the lack of information regarding the submerged structure and due to its changing configuration.

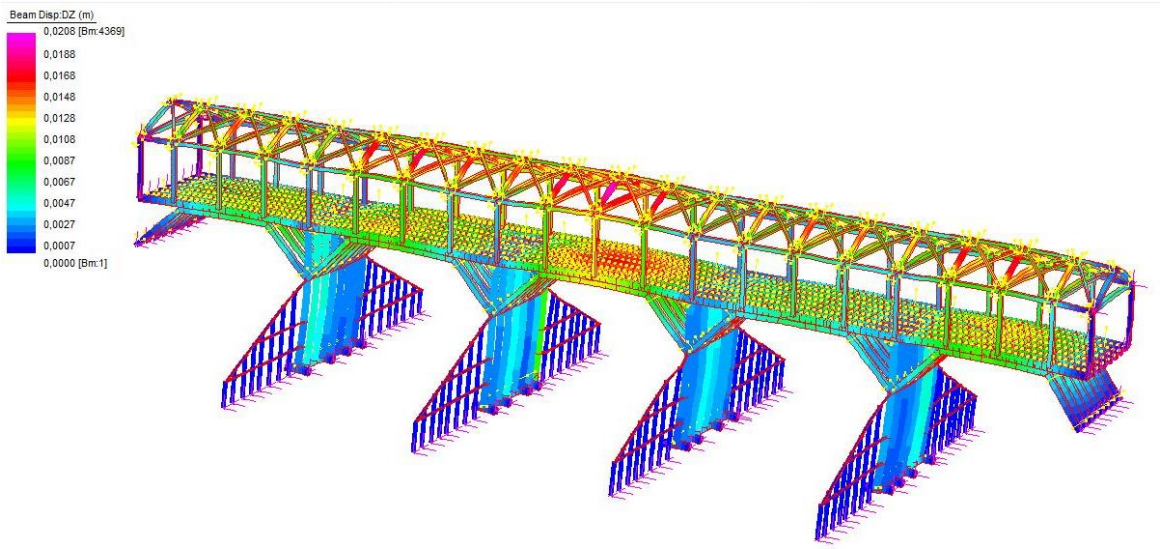


Figure 49 - FEM model, Static analysis (Envelope of the combinations). Displacement in Z direction (solid view of the linear elements).

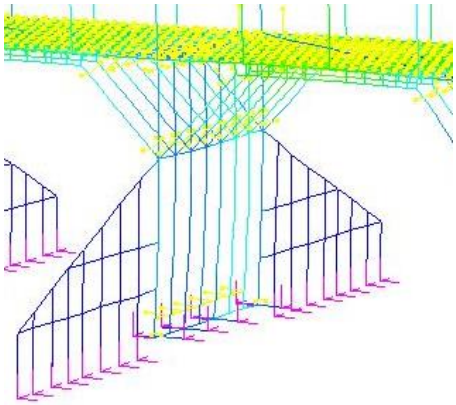


Figure 48 - Static analysis, Displacement in Z

In a general overview of the results of the static analysis, we can see from the picture representing the displacement in the vertical direction (Figure 49 and Figure 48), that the *Rostri* have not a fundamental role to this action: their vertical displacement is almost zero (in blu displacement zero). In the case of only vertical loads the eight vertical poles compose the main reacting structure of each pier. This understanding is compatible with the results of the deformations survey (Figure 50). Observing the deformations it is possible to see how the *Rostri* were not deformed (elements in brown), instead the central poles

have a vertical displacement (in red) and the horizontal boards connecting the poles with the Rostri are rotating due to this differential movement (in yellow).

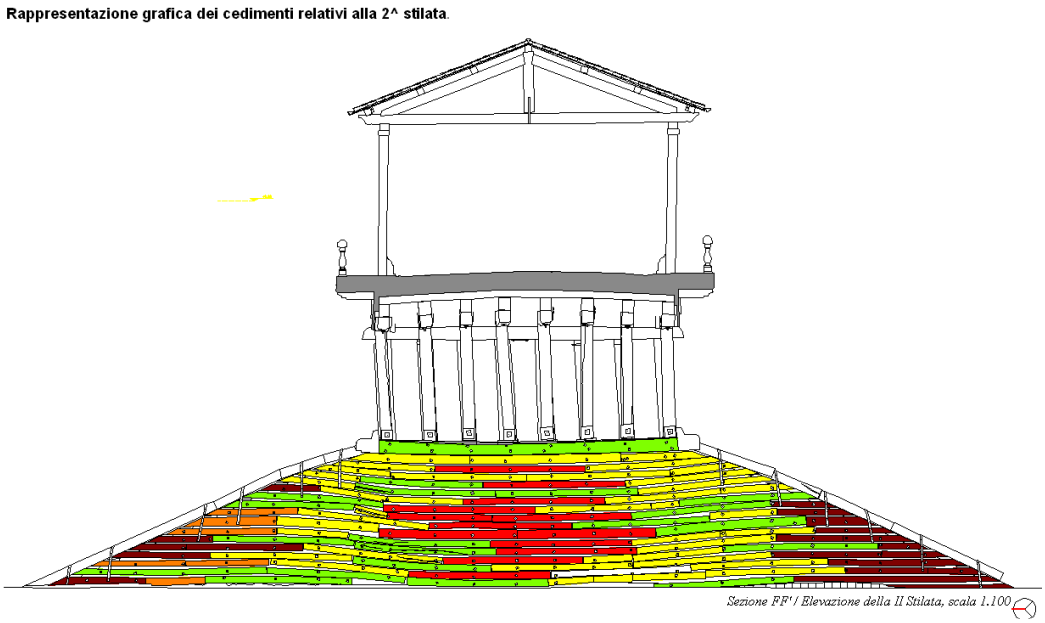


Figure 50 - Deformation survey (only the unsubmerged part is visible)

On the other hand, considering the combination number 9, corresponding to the maximal horizontal forces, the *Rostri* elements turn out to be stressed and the maximal displacement is ad the middle span of the deck.

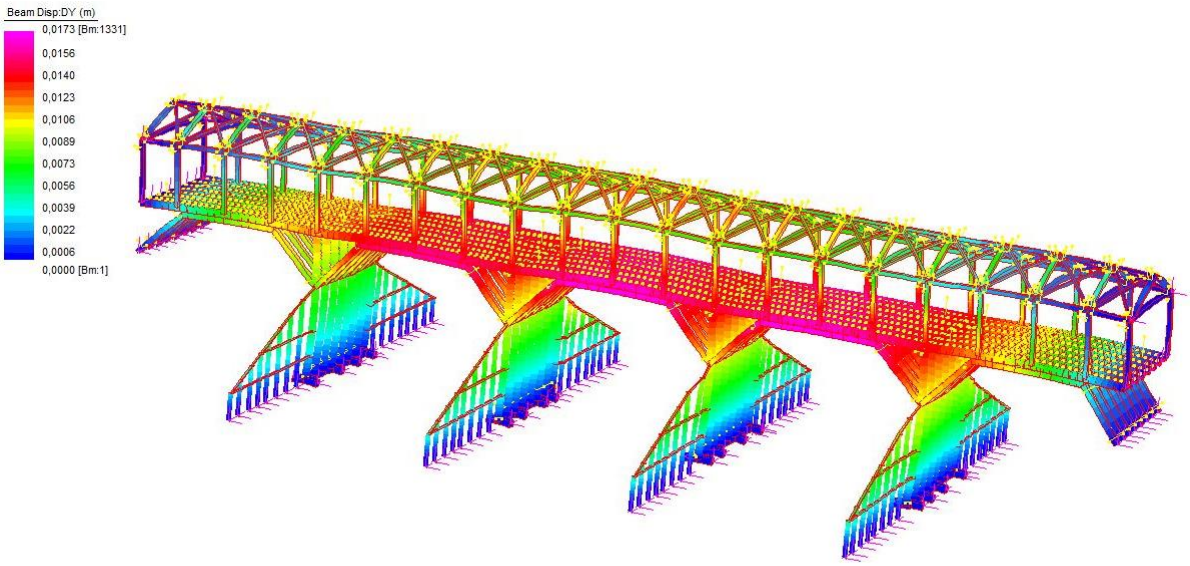


Figure 51 - FEM model, Static analysis (Combination 9). Displacement in Y direction.

The outcome of this primary evaluation is that the Rostri have an important role in the out-of plane behaviour but not in the vertical direction. In the following paragraphs this assumption is better analysed.

6.1 Model A, verifications of the "Cavezzali" beams

The *Cavezzali* are the transversal timber element added in the 1990 in order to reinforce the foundation system already damaged. Those elements are transversal respect to the water flow and they have the role to sustain the longitudinal (threshold) beam. Those elements are 16 and they all have the same geometry, they have a rectangular cross section of 300 mm x 300 mm and a length of 1.8 m.

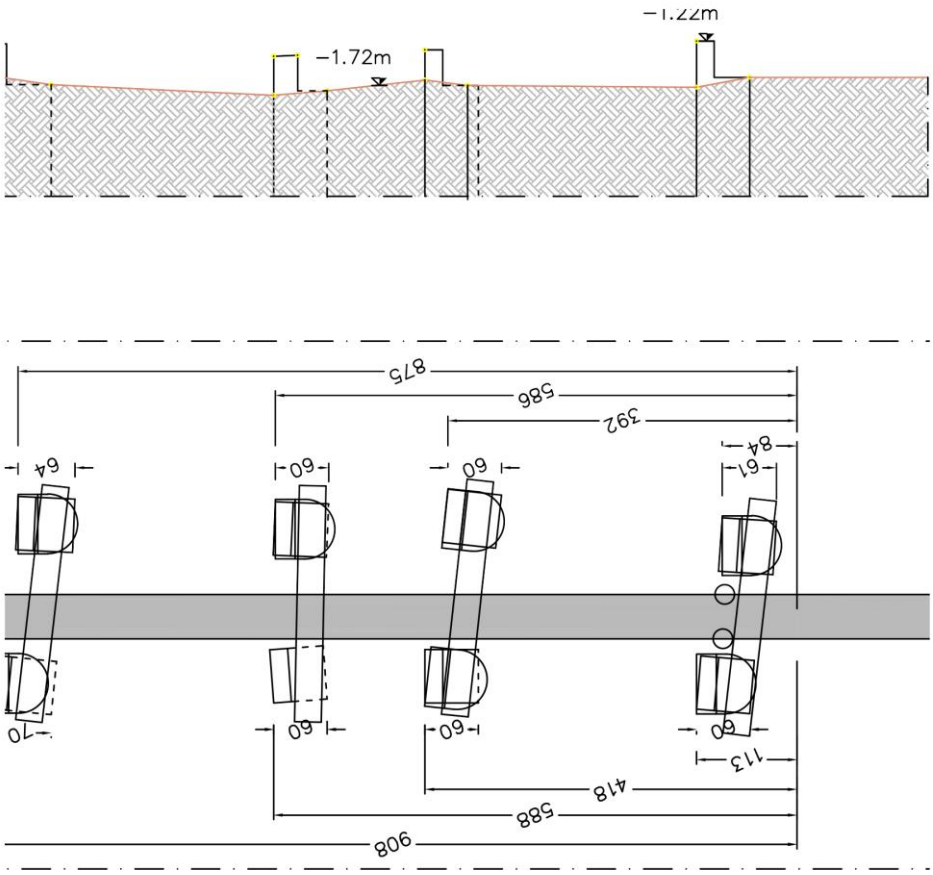


Figure 52 - Geometry of the foundation system

Table 21 - Geometry of the Cavezzali elements

b (width)	400	mm
h (hight)	400	mm
t (length)	1.8	m
I (inertia)	2133333333	mm^4

Considering the rectangular cross section, it is possible to evaluate the maximum bending moment and the maximum shear force.

Table 22 - Maximum shear and bending capacity of the Cavezzali beams

Maximum Bending Moment (M)	139.37	kNm	σ = M*y / I
Maximum Shear Force (S)	99.55	kN	T = 3S / 2bh

These elements present all the same type of forces distribution, are mainly effected by shear force and bending moment in direction z.

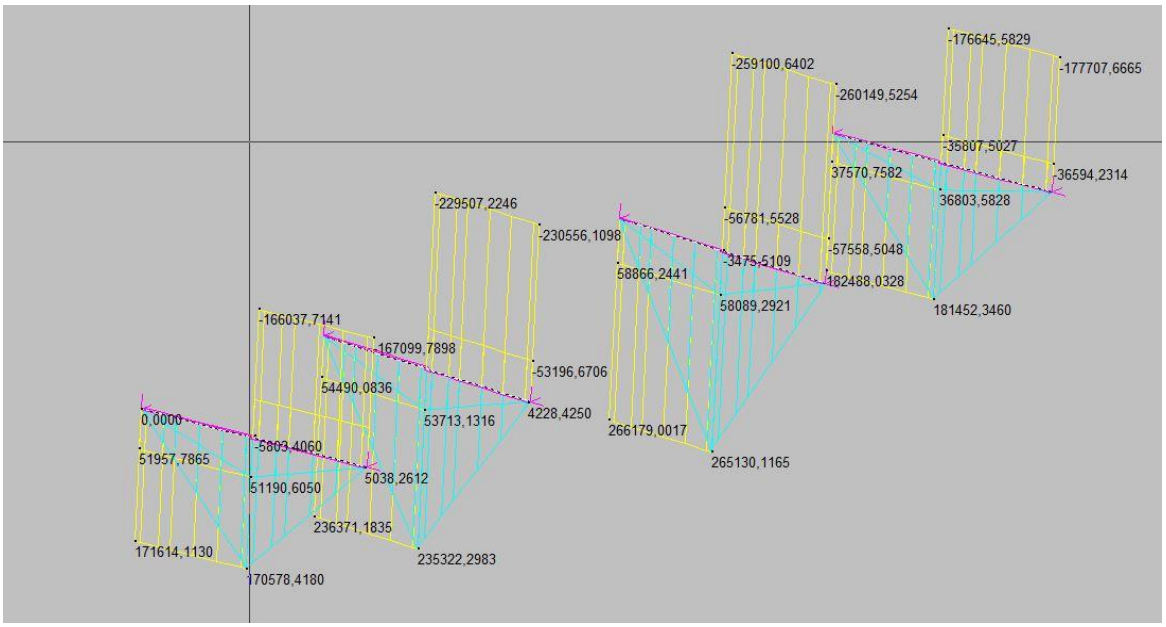


Figure 53 - *Cavezzali* of the third pier, shear and bending moment in direction z (action in N,m)

In the following table (Table 23) The maximum values of shear forces, bending moment and axial force are summarized for each *Cavezzale* of the four piers.

Table 23 - Shear, bending moment and axial force of the Cavezzali

	Element n°	Max Shear Force 1 [kN]	Max Bending Moment 1 [kNm]	Max Normal Force [kN]
Pier 1	1	215.68	196.14	6.52
	2	247.17	221.98	11.83
	3	176.91	159.24	8.77
	4	177.71	143.14	3.57
Pier 2	5	216.79	186.87	4.01
	6	256.54	222.59	9.16
	7	182.59	163.86	9.29
	8	292.68	262.93	4.31
Pier 3	9	171.61	152.05	3.45
	10	260.15	212.26	11.94
	11	260.15	239.09	11.94
	12	177.71	239.09	11.94
Pier 4	13	205.17	189.08	3.55
	14	150.55	136.72	8.48
	15	243.29	221.32	12.01
	16	198.43	180.36	5.10

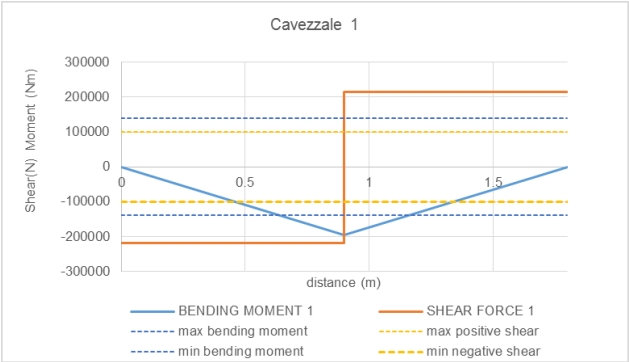
In Table 23 the values higher than the maximum resistance are highlighted in red, the normal force is really low but shear and bending moment present high values. Considering the geometrical section of the elements, the stress levels can be calculated. The stress is compared with the maximums values as defined by the code.

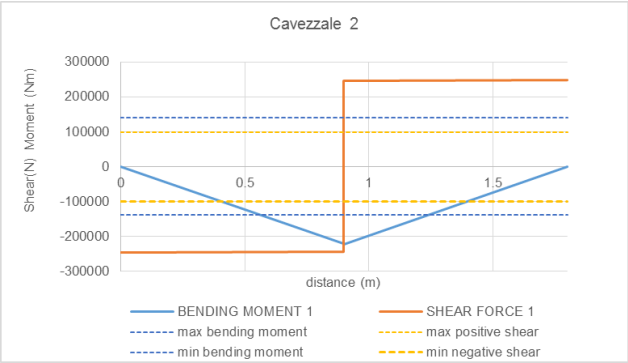
Table 24 - Stress level of the Cavezzali normalized to the code strengths values, in red the over-limit values

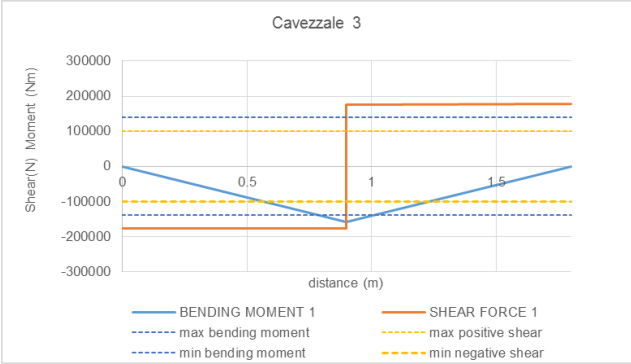
	n°	Max $\sigma_{m,y,d}$/ fm,d	Max $\sigma_{m,z,d}$ / fm,d	Max comb. $\sigma_{m,y,d}$;$\sigma_{m,z,d}$	Max τ_d / fv,d	Max $\sigma_{c,o,d}$ / fc,0,d	Max $\sigma_{c,o,d}$ / $f_{c,0,d}$
Pier 1	1	1,41	0,04	1,43	2,21	1,41	0,04
	2	1,59	0,02	1,61	2,48	1,59	0,02
	3	1,14	0,01	1,15	1,78	1,14	0,01
	4	1,03	0,02	1,04	1,79	1,03	0,02
Pier 2	5	1,34	0,03	1,36	2,18	1,34	0,03
	6	1,60	0,03	1,61	2,58	1,60	0,03
	7	1,18	0,03	1,19	1,83	1,18	0,03
	8	1,89	0,02	1,90	2,94	1,89	0,02
Pier 3	9	1,09	0,04	1,12	1,72	1,09	0,04
	10	1,52	0,04	1,52	2,61	1,52	0,04
	11	1,72	0,03	1,73	2,67	1,72	0,03
	12	1,16	0,03	1,18	2,66	1,16	0,03
Pier 4	13	1,36	0,04	1,38	2,12	1,36	0,04
	14	0,98	0,03	1,00	1,55	0,98	0,03
	15	1,59	0,03	1,61	2,51	1,59	0,03
	16	1,29	0,04	1,32	2,04	1,29	0,04

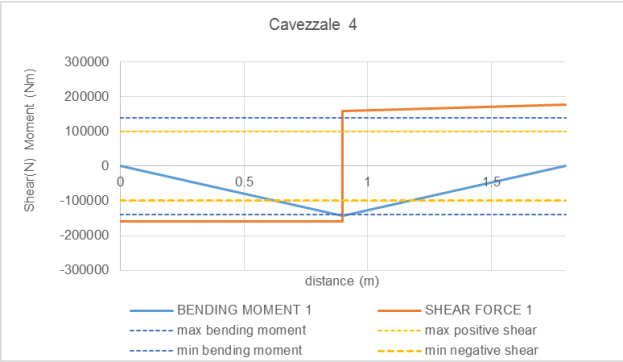
All elements are all not verified for shear, and many are not for bending. It is possible to conclude that those elements are not sufficient for the foundation system, the safety level is not achieved; and hence reinforcement is recommended. In the following table the diagrams of shear and bending moment for each *Cavezzale* are presented (Table 25).

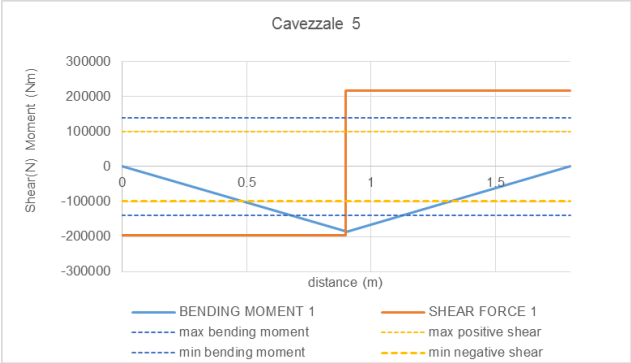
Table 25 - Shear force and bending moment in plane 1

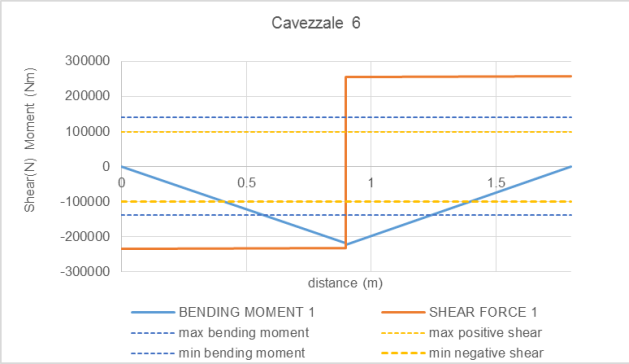


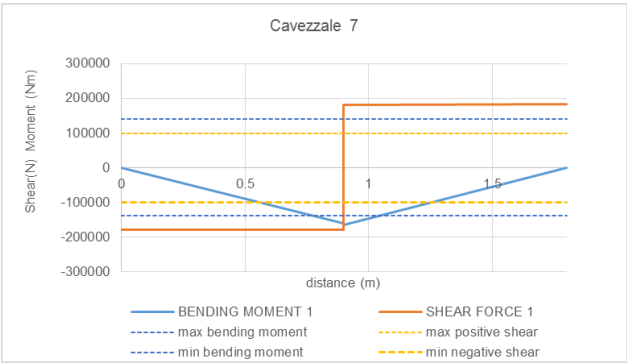


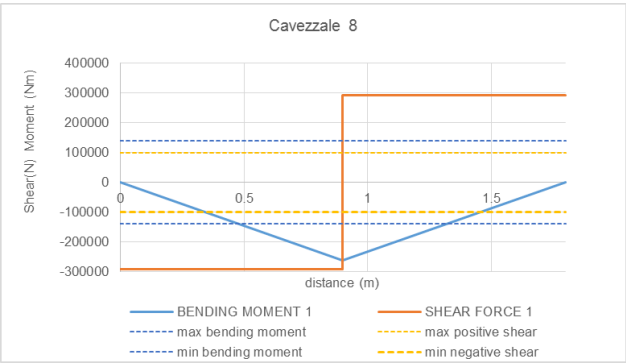


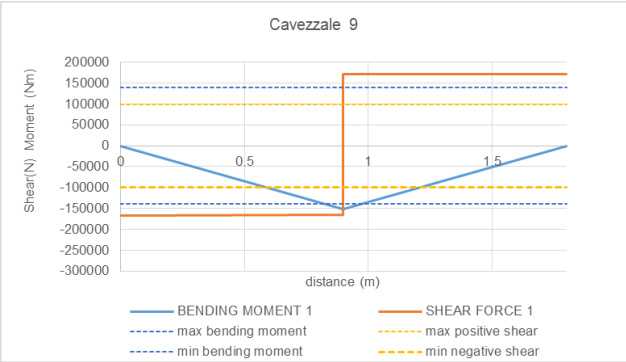


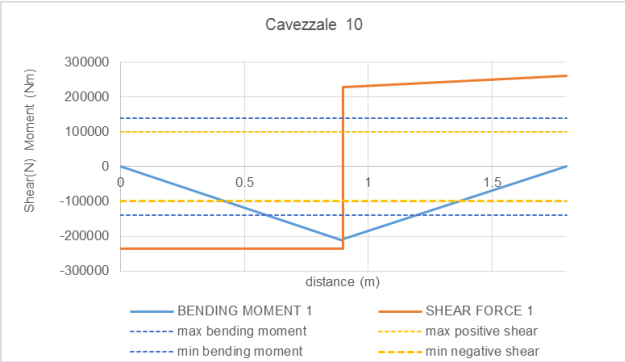


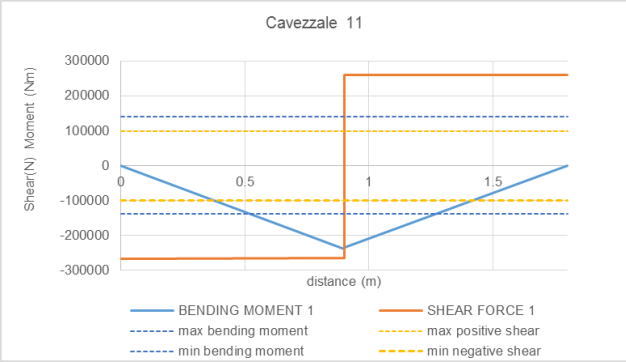


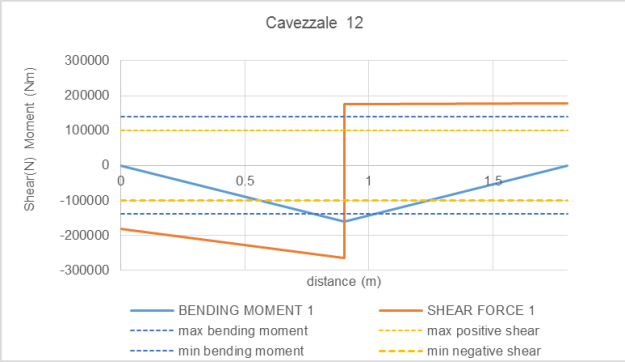


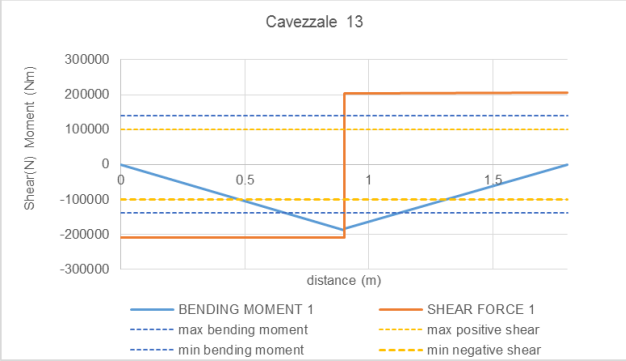


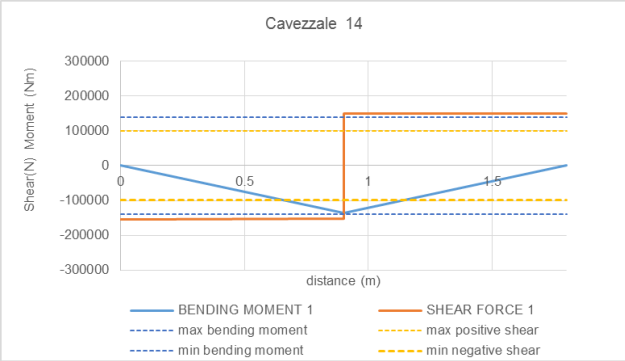


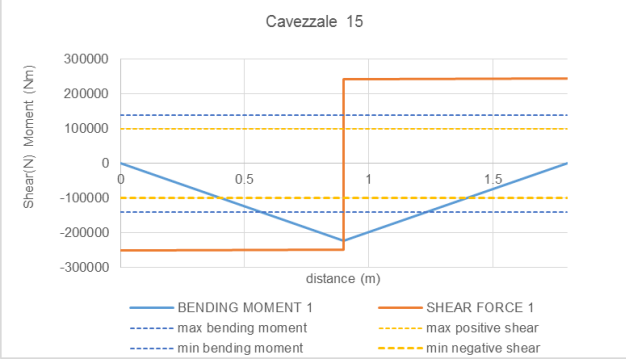


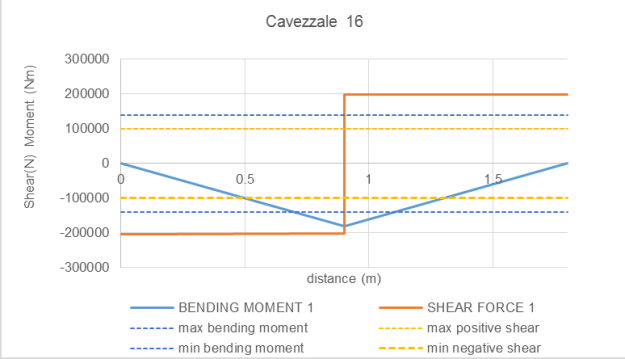












The Cavezzali elements are sustained at the extremities by concrete short poles and at the middle span they present the central load given by the vertical timber poles of the pier. This type of structure generates localized pressure perpendicular to the direction of the fibres. This type of stress may be dangerous in timber elements because of their orthotropic characteristics, timber presents a low resistance in the direction perpendicular to the fibres. Therefore, in addition to the verification of the

stress levels of the entire element, it is necessary to control the localized stress; the forces are almost punctual and they generate high level of compression at the extremities of the beam and in the middle. The concrete elements are not part of the FEM model because of their high level of stiffness respect to the timber ones. Their presence is simplified with the external restrains.

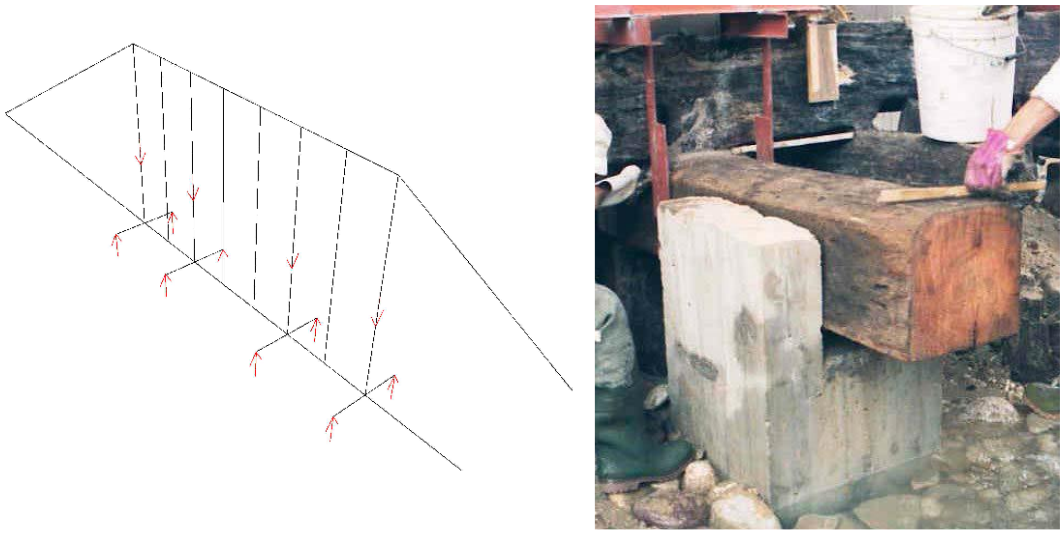


Figure 54 - Localized compressive stress on the Cavezzali beams

At the ends of the beam the stress distribution is considered triangular, instead at the central part the distribution is constant. In the following table it is possible to see the result of this analysis, in red the stress levels higher than the maximum acceptable.

Table 26 - Compression perpendicular to the fibers

	Middle Span F_z	σ_{zm}	**Reaction F_z**	σ_{z1}
	kN	N/mm^2	kN	N/mm^2
Cavezzale 1			202.86	1.88
	-433.36	-1.34	0.00	0.00
			198.99	1.84
Cavezzale 2			207.57	1.92
	-491.07	-1.52	0.00	0.00
			208.72	1.93
Cavezzale 3			164.64	1.52
	-352.27	-1.09	0.00	0.00
			164.11	1.52
Cavezzale 4			145.68	1.35
	-318.09	-0.98	0.00	0.00

			145.92	1.35
Cavezzale 5			207.35	1.92
	-411.99	-1.27	0.00	0.00
			188.43	1.74
Cavezzale 6	-488.72	-1.51	219.34	2.03
			0.00	0.00
			200.11	1.85
Cavezzale 7			186.94	1.73
	-359.53	-1.11	0.00	0.00
			183.29	1.70
Cavezzale 8			246.77	2.28
	-583.07	-1.80	0.00	0.00
			246.88	2.29
Cavezzale 9			173.09	1.60
	-336.61	-1.04	0.00	0.00
			168.20	1.56
Cavezzale 10			211.93	1.96
	-464.82	-1.43	0.00	0.00
			207.06	1.92
Cavezzale 11			233.42	2.16
	-524.23	-1.62	0.00	0.00
			228.65	2.12
Cavezzale 12			181.44	1.68
	-441.77	-1.36	0.00	0.00
			176.18	1.63
Cavezzale 13			186.17	1.72
	-413.68	-1.28	0.00	0.00
			181.52	1.68
Cavezzale 14			155.75	1.44
	-302.63	-0.93	0.00	0.00
			151.93	1.41
Cavezzale 15			205.69	1.90
	-490.77	-1.51	0.00	0.00
			200.44	1.86
Cavezzale 16			184.31	1.71
	-399.79	-1.23	0.00	0.00
			179.75	1.66

The *Cavezzali* beams result to be not verified to the compression perpendicular to the fibres at the ends of the elements. In the central parts, where the vertical poles are loading, they resulted verified.

6.2 Model A, Verifications of the thresholds

The threshold is the timber beam at the base of the poles. As already mentioned, this element was sustained by the soil but nowadays it is under bending due to the changing of the soil conformation and due to the corrosion of the below elements. Above this elements there are the eight piles each pier and below there were sixteen other timber poles, the role of the threshold was to spread the forces. This element is sustained by the Cavezzali elements and nowadays present high levels of stress.

Table 27 - Geometry of the threshold

b (width)	400	mm
h (hight)	500	mm
t (length)	7.7	m
I (inertia)	2666666667	mm^4

Considering the rectangular cross section, it is possible to evaluate the maximum bending moment and the maximum shear force.

Table 28 - Maximum shear and bending moment of the thresholds

Maximum Bending Moment (M)	139.37	kNm	$\sigma = M*y / I$
Maximum Shear Force (S)	124.44	kN	$T = 3S / 2bh$

These elements present all similar type of forces distribution, but the position of the *Cavezzali* in respect to the threshold is not always the same, there are some irregularities due to the construction process of the reinforcement. In Table 29 and Table 30 maximum forces and stresses are presented; in red the stress values higher than the respective resistance.

Table 29 - Shear, bending moment and axial force of the thresholds

El. Number	Max Shear Force [kN]	Max Bending Moment [kNm]	Max Normal Force [kN]	Max Torque k[Nm]
T 1	183.22	106.88	9.06	0.75
T 2	349.33	225.14	15.38	3.72
T 3	300.21	132.22	11.74	1.84
T 4	285.84	127.06	20.18	0.37

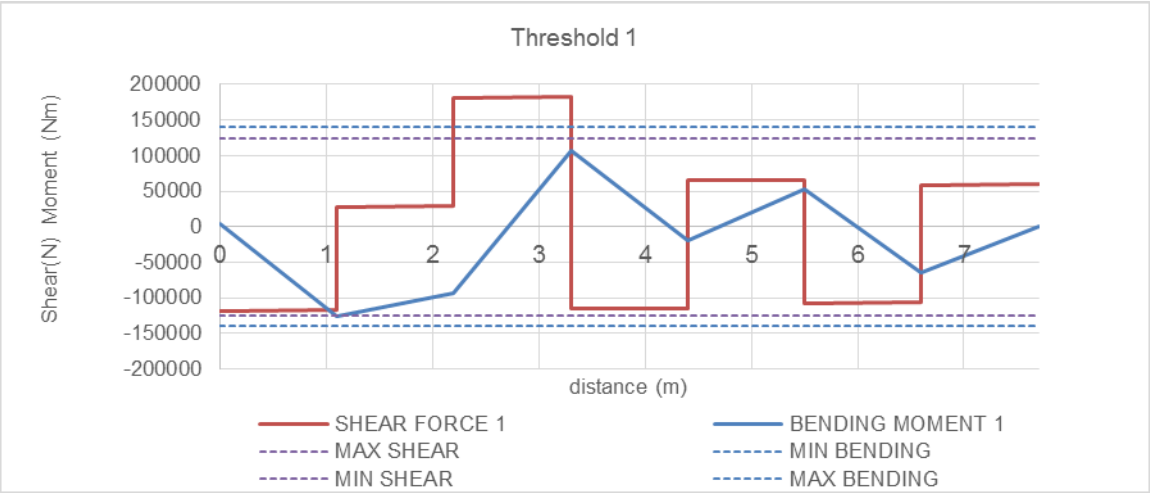
Table 30 - Stress levels of the thresholds

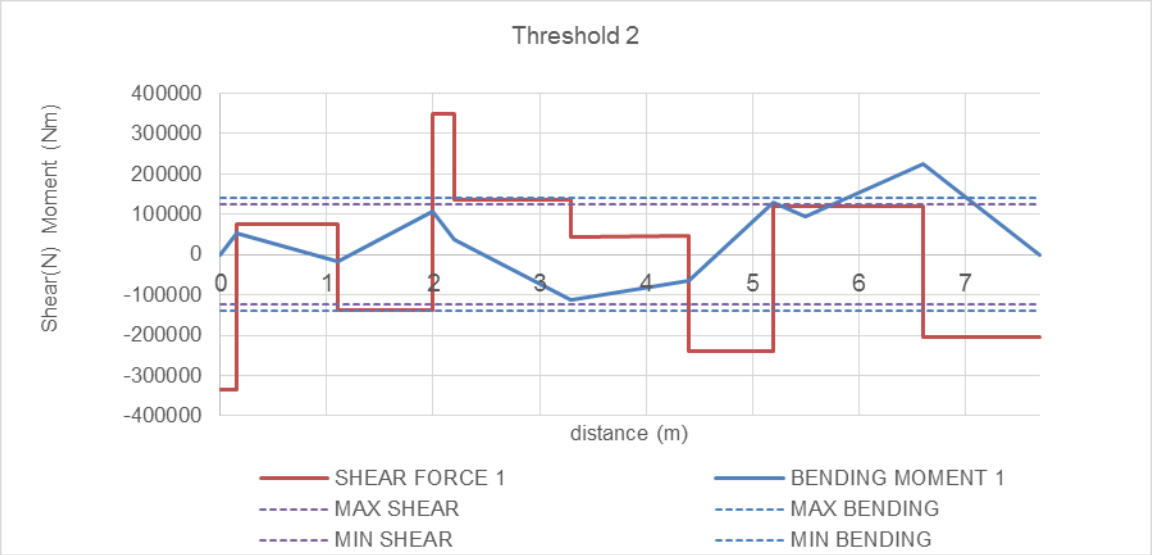
n°	Max $\sigma_{m,y,d}$ / $f_{m,z,d}$	Max $\sigma_{m,z,d}$ / $f_{m,z,d}$	Max comb. $\sigma_{m,y,d}$; $\sigma_{m,z,d}$	Max $\tau_{d,z}$ / $f_{v,d}$	Max $\tau_{d,y}$ / $f_{v,d}$	Max τ_{tor} / f_{torque}	Max τ_d+ τ_{tor}	Max $\sigma_{c,o,d}$ / $f_{c,0,d}$
T 1	0,61	0,04	0,74	1,47	0,08	0,00	2,17	0,05
T 2	1,29	0,03	1,30	2,81	0,10	0,00	7,88	0,08
T 3	0,76	0,04	0,77	2,41	0,12	0,00	5,82	0,06
T 4	0,60	0,04	0,75	2,30	0,08	0,00	5,27	0,10

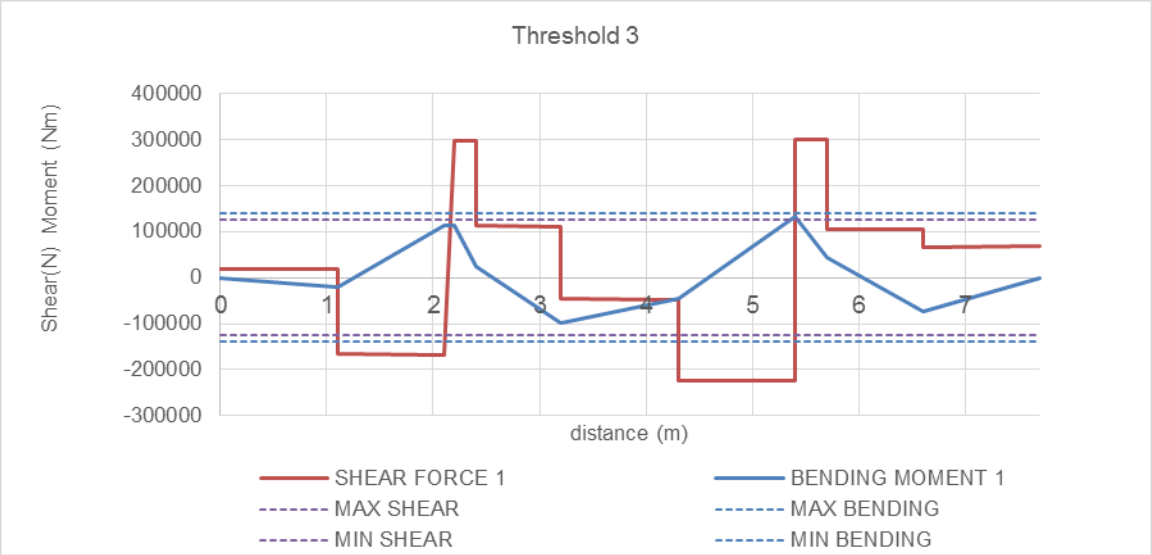
All the threshold result to be not verified to the shear forces and the number 2 is also not verified to flexure. It is interesting to notice how the threshold 2 undergoes the highest actions and actually in the bridge is the most damaged element.

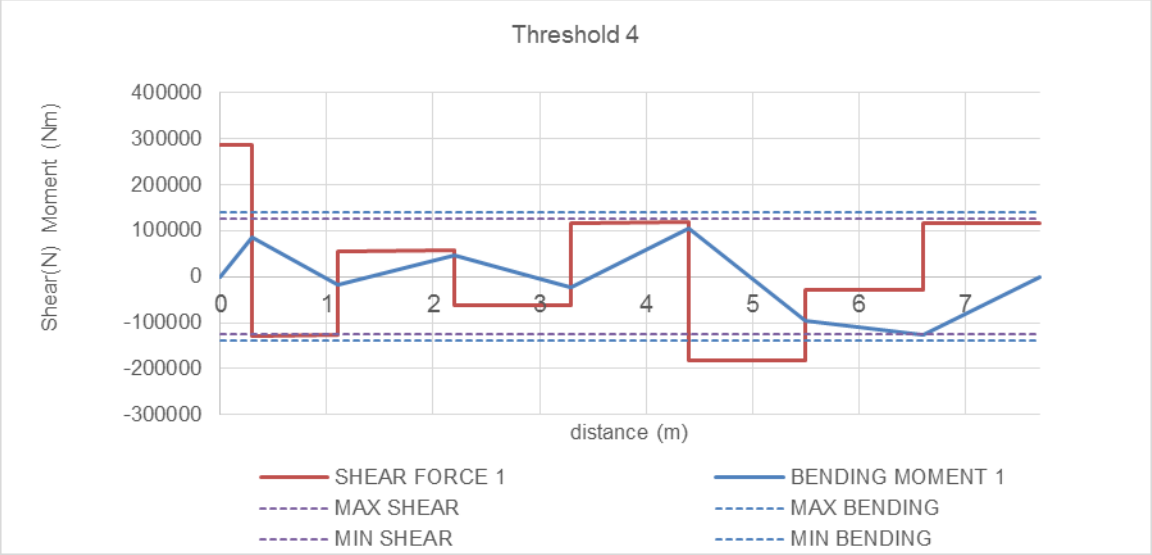
In the following tables the diagrams of shear force and bending moment in the plane one are presented.

Table 31 - Diagrams of shear and bending moment of the thresholds




Threshold 2
Shear(N) Moment (Nm)
400000
300000
200000
100000
0
-100000
-200000
-300000
-400000
0
1
2
3
4
5
6
7
distance (m)
SHEAR FORCE 1
BENDING MOMENT 1
MAX SHEAR
MAX BENDING
MIN SHEAR
MIN BENDING


Threshold 3
Shear(N) Moment (Nm)
400000
300000
200000
100000
0
-100000
-200000
-300000
0
1
2
3
4
5
6
7
distance (m)
SHEAR FORCE 1
BENDING MOMENT 1
MAX SHEAR
MAX BENDING
MIN SHEAR
MIN BENDING


Threshold 4
Shear(N) Moment (Nm)
400000
300000
200000
100000
0
-100000
-200000
-300000
0
1
2
3
4
5
6
7
distance (m)
SHEAR FORCE 1
BENDING MOMENT 1
MAX SHEAR
MAX BENDING
MIN SHEAR
MIN BENDING

Examples of actual state of damage are visible in Figure 55 and Figure 56; the poles loading on the threshold create high levels of shear stress. The numerical results are compatible with the survey information.

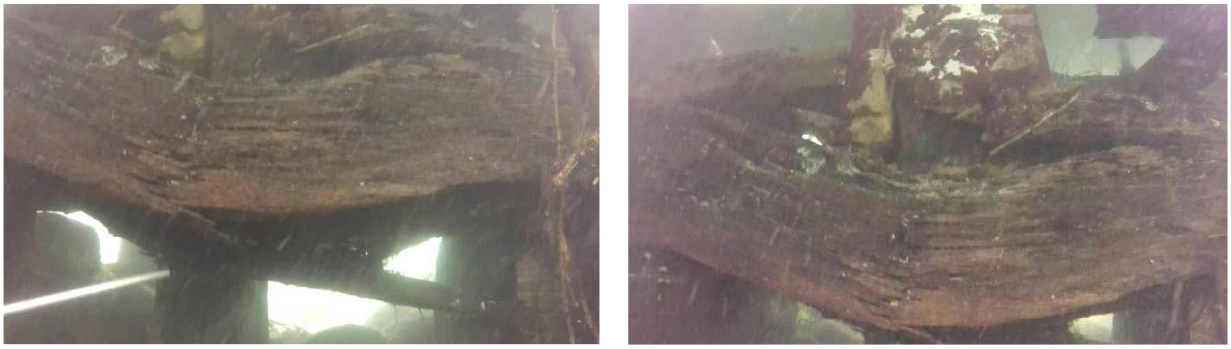


Figure 55 - First Pier, threshold (UNIPD 2015)

Figure 56 - Second pier, threshold (UNIPD 2015)

6.3 Model B, verification of the foundation elements and comparison

The static analyses of the two models give different results in terms of stress levels at the foundation elements. In the Table 32 it is possible to see the results of model B in terms of stresses (normalized to the standards' resistance values), all the values higher than the unit are higher than the limits of NTC 2008. Shear and bending moment in direction Y present the main differences, this is due to the absence of the *Rostri*, those elements in the model A increase the stiffness in the out of plane behaviour and they take the shear in direction Y (mainly due to the river flow). Once those elements are no more present or damaged, the stress level in the direction Y of the Cavezzali beams has an increase. However the level of shear stress in Y direction is still under the limits.

Table 32 – Model B, Cavezzali's stress levels. In red the value higher than the resistance limit (>1).

Cavezzale n°	Max $\sigma_{m,y,d}$/ $f_{m,d}$	Max $\sigma_{m,z,d}$ / $f_{m,d}$	Max comb. $\sigma_{m,y,d}$; $\sigma_{m,z,d}$	Max $\tau_{d,z}$ / $f_{v,d}$	Max $\tau_{d,y}$ / $f_{v,d}$	Max $\sigma_{c,o,d}$ / $f_{c,0,d}$
1	1,28	0,13	1,37	2,01	0,20	0,01
2	1,58	0,11	1,65	2,46	0,16	0,01
3	1,16	0,09	1,22	1,81	0,14	0,01
4	1,20	0,09	1,27	2,11	0,15	0,00
5	1,16	0,12	1,24	1,89	0,19	0,00
6	1,57	0,11	1,65	2,54	0,18	0,00
7	1,08	0,10	1,15	1,69	0,16	0,01
8	2,23	0,10	2,30	3,48	0,16	0,00
9	0,94	0,12	1,03	1,49	0,20	0,00
10	1,50	0,12	1,58	2,34	0,18	0,01
11	1,69	0,12	1,76	2,69	0,18	0,01
12	1,37	0,10	1,44	2,16	0,16	0,00
13	1,22	0,12	1,31	1,91	0,19	0,00
14	0,98	0,10	1,05	1,55	0,16	0,01
15	1,60	0,10	1,67	2,52	0,15	0,01
16	1,45	0,10	1,53	2,29	0,17	0,00

As expected, the Cavezzali elements in the model B present higher levels of stresses than model A, they are not verified to combination of bending moments and to shear. The importance of a strengthening system is therefore more evident. Some of the elements such as Cavezzale 2 or 11 present a stress level more than two times the code limit.

In case of damages of the *Rostri* elements the stress level is going to be between the levels of model A and B; the worst case is resulting to be model B.

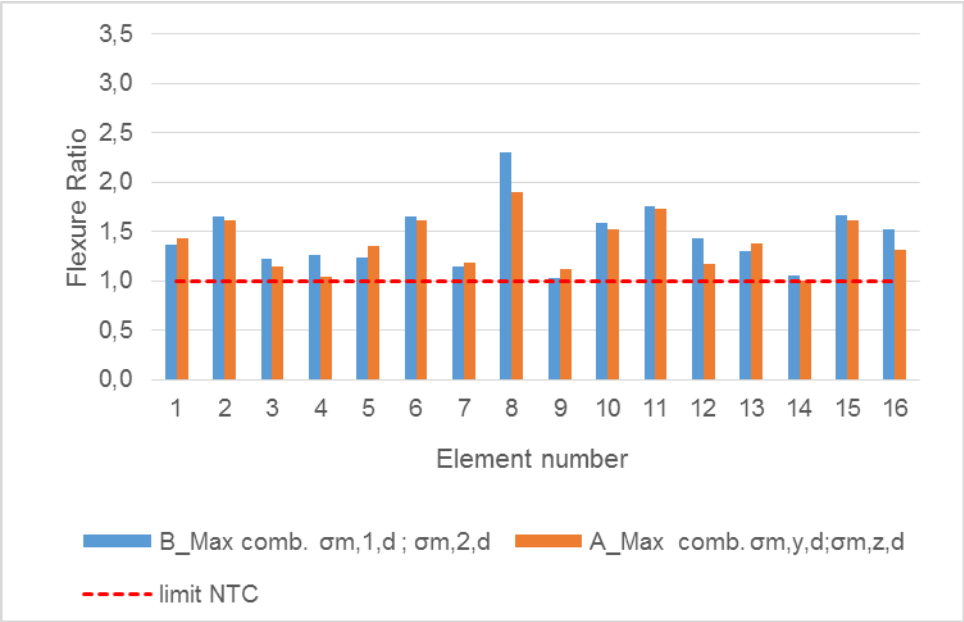


Figure 57 - Comparison between the Flexure ratios of model A and B

In Figure 57 is presented the comparison, between model A and B, of the flexure ratios; this ratio is corresponding to the combinations of flexure in the two planes (y and z). There is a general increase in the values of the damaged configuration (model B).

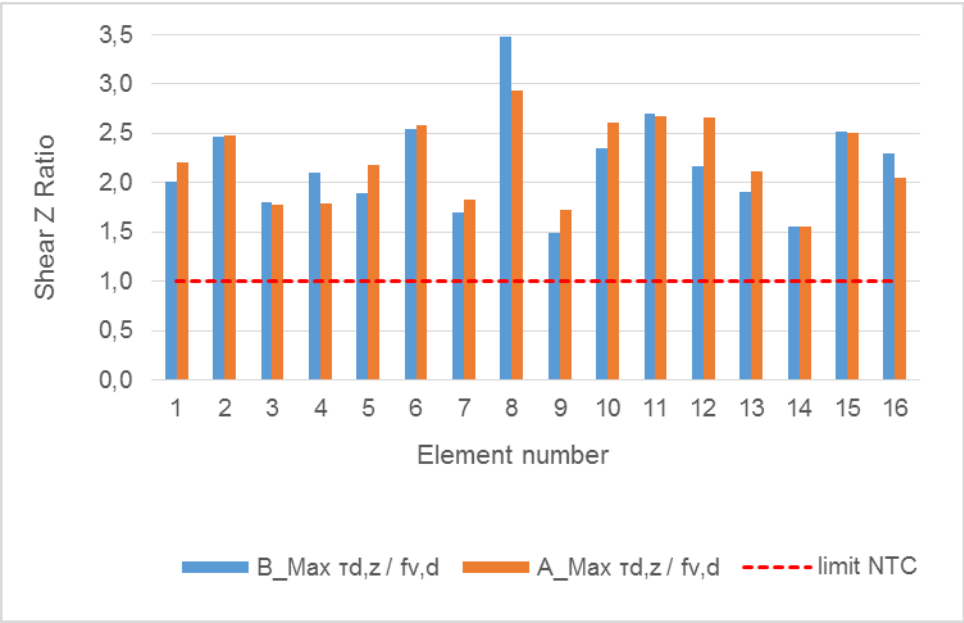


Figure 58 - Comparison between the Shear Z ratios of model A and B

In Figure 58 the results are presented in terms of shear: the ratio between the maximum shear stress and the shear strength is presented. All the results are higher than 1, meaning that the shear stress is always higher than the code level.

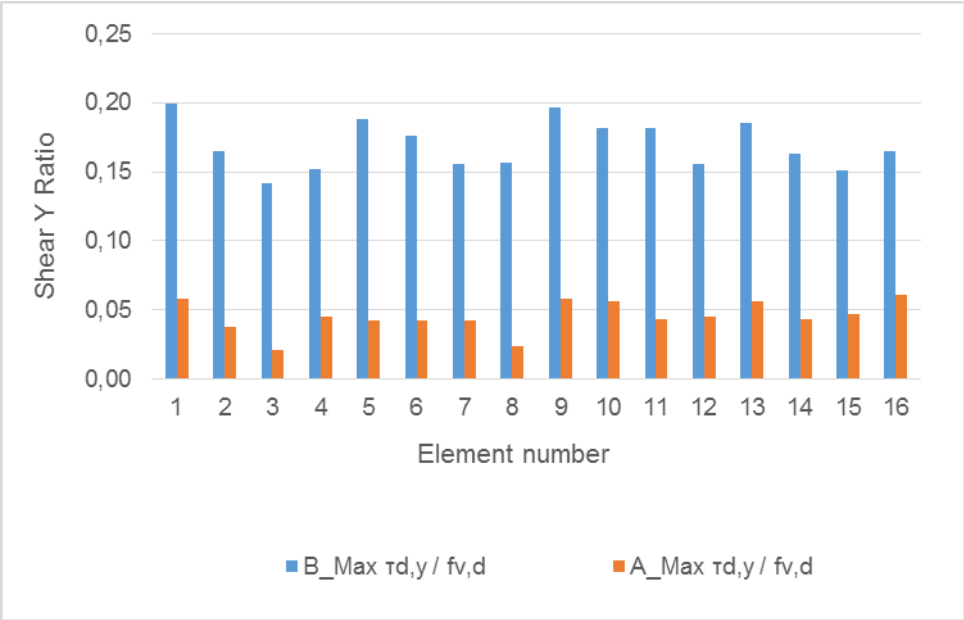


Figure 59 - Comparison between the Shear Y ratios of model A and B

In Figure 59 the comparison between the shear in direction Y is shown. Respect to the comparison in term of flexure or shear in Z, in this case the difference between the two models is evident and clear. Nevertheless, in the worst configuration the shear in Y is at the 20%, the structure does not present problem.

Table 33 – Model B, Threshold's stress levels. In red the value higher than the resistance limit (>1).

n°	Max $\sigma_{m,y,d}$ / $f_{m,z,d}$	Max $\sigma_{m,z,d}$ / $f_{m,z,d}$	Max comb. $\sigma_{m,y,d}$; $\sigma_{m,z,d}$	Max $\tau_{d,z}$ / $f_{v,d}$	Max $\tau_{d,y}$ / $f_{v,d}$	Max τ_{tor} / f_torque	Max comb $\tau_{d;}$ τ_{tor}	Max $\sigma_{c,o,d}$ / $f_{c,0,d}$
T 1	0,60	0,04	0,73	1,46	0,08	0,00	2,14	0,15
T 2	1,67	0,04	1,68	2,76	0,10	0,00	7,61	0,02
T 3	0,78	0,04	0,81	2,43	0,12	0,00	5,88	0,01
T 4	0,72	0,04	0,74	2,02	0,08	0,00	4,09	0,02

The stress levels of the thresholds in the model B are slightly higher but it is not visible a high difference. Therefore, all the elements are still not verified to shear and just the third threshold is not verified to bending moment (Figure 60).

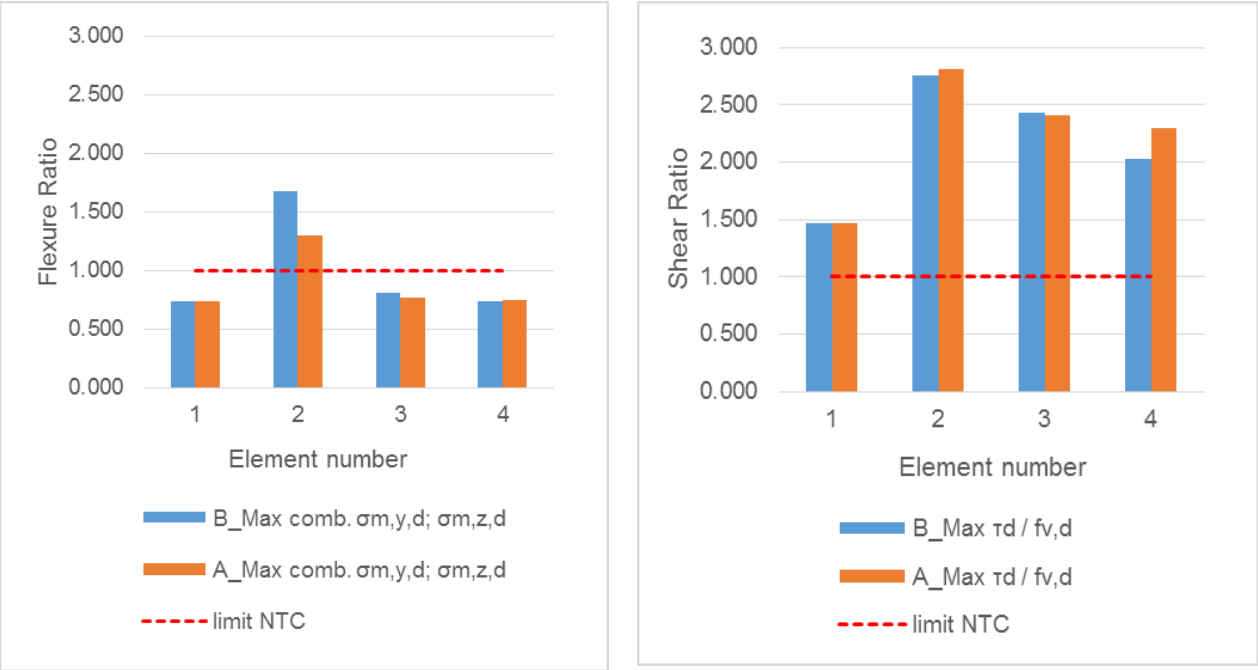


Figure 60 - Threshold beams, comparison of flexure and shear ration in the model A and B

6.4 Out of plane behaviour, comparison between model A and B

Table 34 - Distribution of the Y-reactions in the piers and abutments

	A	B
abutment 1 [%]	17.09	19.59
abutment 2 [%]	17.92	20.53
poles 1 [%]	3.01	14.66
poles 2 [%]	3.02	15.27
poles 3 [%]	4.09	15.54
poles 4 [%]	3.98	14.42
rostri 1 [%]	13.50	-
rostri 2 [%]	13.53	-
rostri 3 [%]	11.83	-
rostri 4 [%]	12.02	-
Fy_tot [kN]	908.55	875.60

The out of plane behaviour is strongly affected by the presence of the *Rostri:* these elements give a high stiffness in the direction of the river flow. It is possible to compare the models A and B in terms of reaction in Y direction (river flow direction) (Table 34).
The abutments in both cases takes around the 20% of the horizontal forces, but the wind horizontal load corresponds to 9-10% of it. In model A, the hydraulic force at the abutments is the 8-9% of the total, instead in case of model B the hydraulic is the 10-10.5%. The poles in presence of the *Rostri* take just the 3% but when the structure does not present those additional elements the poles are taking the 15% of the shear forces.

As discussed in the previous paragraph, in the response to the vertical loads the *Rostri* have not a fundamental role, in case of out of plane the situation is strongly different. The stress level on poles change drastically in presence or absence of those lateral elements.

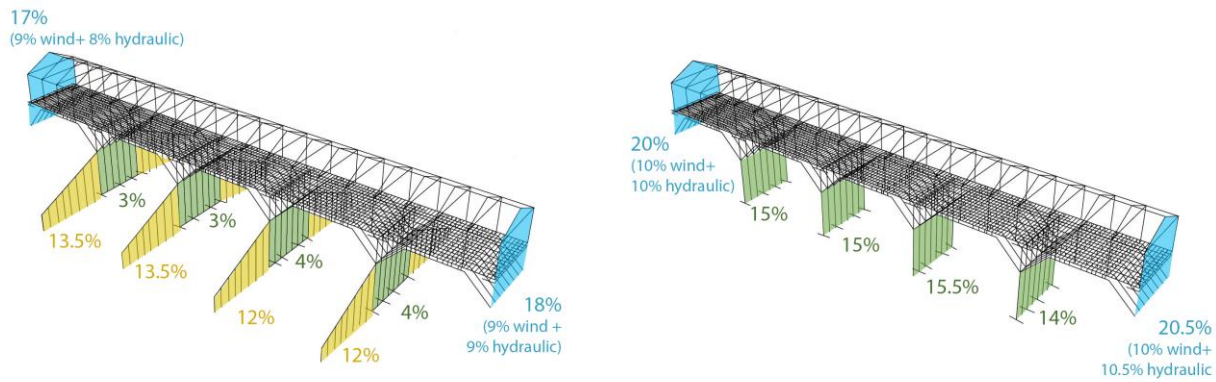


Figure 61 - Distribution of the reactions in Y direction model A (left) and model B (right).

Considering the points at the middle spans and in correspondence to the piers in the FEM model, the displacement in the out of plane direction is analysed. Comparing the two models, A and B, in terms of displacements in Y direction, the difference is evident (Figure 62). The *Rostri* make the structure stiffer and they reduce significantly the displacement. The model A presents a maximum y-displacement equal to 9 cm, instead in the model B the maximum displacement is around 2 cm. What is possible to conclude is that if the bridge was to suffer severe damages due to the river flow, and the *Rostri* are no more effective, the displacement might be in the order of 10 cm at the middle.
In both the model the maximum displacement corresponds to one of the central piers, this is due to the type of loading: the hydraulic force of the river is applied to the vertical pole elements and not at the deck level. The high of the hydraulic loading is due to the data coming from the floods of the 1966.

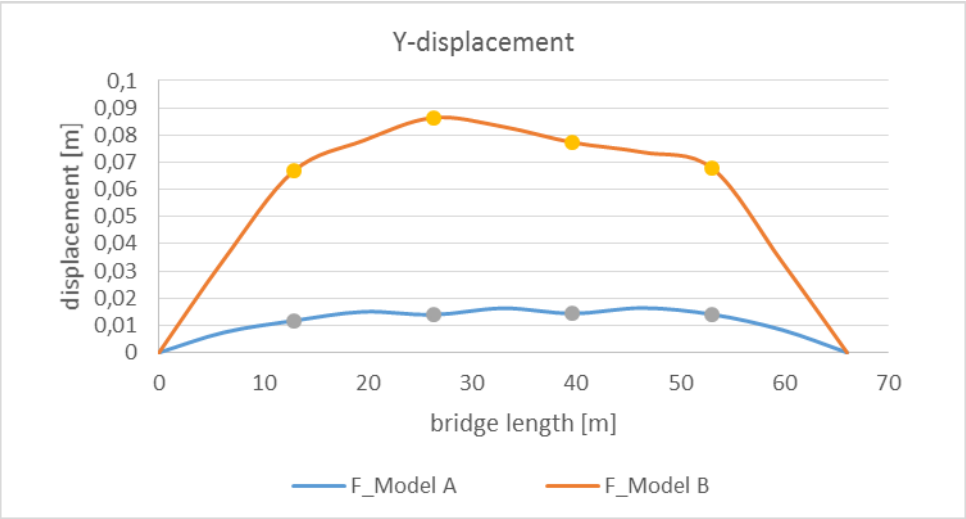


Figure 62 - Displacement of the bridge deck in the out of plane direction

In terms of displacement, the case without Rostri is the one reaching the maximum movement in the Y direction, the direction of the river flow and of the wind. The deformed shape is in accordance with the distribution of the horizontal forces. Model B, respect to model A, has an increase of the horizontal forces at the abutments due to the hydraulic force; accordingly, the larger displacement correspond to model B.

7. STATIC ANALYSIS, DAILY CONDITIONS

In order to have a better understanding of the exceptional situations such as the flood and the earthquake conditions, a daily load conditions has been performed with a static analysis. The adopted combination is the "quasi-permanent":

$$G_1 + G_2 + P + \psi_{21} \cdot Q_{k1} + \psi_{22} \cdot Q_{k2} + \psi_{23} \cdot Q_{k3} + \ldots$$

In this loads combination the structural and not structural loads are computed together with the live-load, notice that there are not safety factor but just combination coefficients.

$$1\ G_1 + 1\ G_2 + 0.6 * LL_{,F}$$

In the case of the daily conditions, the model A is analyzed, no relevant differences are present between the two models and this is due to the absence of horizontal loads.

Table 35 - Verification of the cavezzali beams, daily conditions

	n°	Max σm,z,d/ fm,d	Max σm,y,d / fm,d	D_Max comb. σm,y,d;σm,z,d	D_Max τd,z / fv,d	D_Max τd,y / fv,d
Pier 1	1	0,52	0,00	0,52	0,82	0,00
	2	0,83	0,00	0,83	1,29	0,00
	3	0,59	0,00	0,59	0,93	0,00
	4	0,40	0,00	0,40	0,66	0,00
Pier 2	5	0,39	0,00	0,39	0,63	0,00
	6	0,75	0,00	0,75	1,21	0,00
	7	0,61	0,00	0,61	0,95	0,00
	8	0,65	0,00	0,65	1,01	0,00
Pier 3	9	0,34	0,00	0,34	0,53	0,00
	10	0,78	0,00	0,78	1,22	0,00
	11	0,87	0,00	0,87	1,39	0,00
	12	0,43	0,00	0,43	0,67	0,00
Pier 4	13	0,46	0,00	0,46	0,72	0,00
	14	0,52	0,00	0,52	0,83	0,00
	15	0,83	0,00	0,83	1,31	0,00
	16	0,52	0,00	0,52	0,82	0,00

Table 36 - Verification of the thresholds, daily conditions

n°	Max σm,y,d / fm,z,d	Max σm,z,d / fm,z,d	Max comb. σm,y,d; σm,z,d	Max τd,z / fv,d	Max τtor / f_torque	Max comb τd; τtor
T 1	0,37	0,00	0,37	0,76	1,731E-05	0,58
T 2	0,37	0,00	0,37	1,30	6,228E-05	1,68
T 3	0,38	0,00	0,38	1,25	3,945E-05	1,55
T 4	0,38	0,00	0,38	0,76	9,701E-06	0,58

In the daily conditions, the stress level of the foundation elements is lower than the floods state. Even though, localized high levels of shear are still present. The thresholds 1 and 4 result to be safe, on the other hand the 2nd and the 3rd present high level of shear force, 20% higher than the code limit.
Applying the daily load configuration, the difference between model A and model B are minor, because of the absence of horizontal actions.

The Bridge of Bassano result to be unsafe even at the daily conditions, this is the reason why the bridge presents temporary safety structures. From these results, it is evident how the strengthening plan is nowadays urgent and unavoidable.

8. SEISMIC ANALYSIS

In order to evaluate the response under the seismic action, the spectral analysis is proceeded on the Bassano Bridge. First of all, the natural frequencies of the building were calculated with the Straus 7 software in order to evaluate the behaviour of the structure with the analysis of the main modes. Once the modes are evaluated together with the participant masses for each mode, the seismic action need to be calculated. The horizontal force given from an earthquake, seismic excitation of the ground, can be given from a time history of the ground acceleration, or in the form of a response spectrum. The second is the most common approach and it is used by most of the national codes. The response spectrum used for the bridge is from the Italian code NTC 2008. The response spectrum is used in design and represents the maximum response of a single degree of freedom system, defined with its natural period and its damping. For different damping levels there will be different response curves.

The linear dynamic analysis, or spectral, consist on a few main steps:

- Definition of the natural vibration modes
- Definition of the response spectrum
- Computation of the seismic action's effect for each mode of vibration
- Combination of the effects

It is necessary to take in account all the modes with a significant participation mass, meaning all the modes with a participating mass higher than 5% and to take in account as many modes as the total excited mass is higher than the 85%. This is necessary to obtain a good representation of the actual structural behaviour. Several combinations procedure are possible, in this case the SRSS is applied (sum of the squares).

8.1 Seismic Action and seismic load combinations

Referring to the NTC 2008, §3.2 the seismic action is calculated. The ULS is evaluated, with a probability of 10% in the reference time.

Use Class	II	(NTC §2.4.2)
Nominal Life	50	(NTC §2.4.1)
Type of Soil	B (dense sand or gravel or stiff clay)	(NTC §3.2.II)
Topography	T1	(NTC §3.2.4)
Tr	712 years	

The formulation of the elastic spectra present in the Italian code is:

Table 37 - Equations of the response spectrum (NTC 2008)

$0 \leq T < T_B$	$S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \cdot \left[\frac{T}{T_B} + \frac{1}{\eta \cdot F_0}\left(1 - \frac{T}{T_B}\right)\right]$
$T_B \leq T < T_C$	$S_e(T) = a_g \cdot S \cdot \eta \cdot F_0$
$T_C \leq T < T_D$	$S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \cdot \left(\frac{T_C}{T}\right)$
$T_D \leq T$	$S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \cdot \left(\frac{T_C \cdot T_D}{T^2}\right)$
Where, a_g, maximal horizontal acceleration S, ground factor (stratigraphy and topography) $\eta = \sqrt{10/(5+\xi)} \geq 0.55$, damping factor ξ= 5% damping F_0 , maximal amplification factor T_B,T_C and T_D , periods characterizing the spectra	

To take in account the ductility of the building it is possible to use a simplify method and define a behaviour factor. The elastic spectra has to be divided by the q factor in order to obtain the inelastic spectra.

$$S_d = \frac{S_e}{q}$$

Where q is the behaviour factor and it is equal to:

$$q = k_r \cdot q_0$$

Where k_r is equal to 1 in regular buildings.

In the following paragraph 8.2 the role of the behaviour factor is explained.

The response spectra defined with the parameters explained before, is presented in Figure 63 in terms of period and acceleration (g unit).

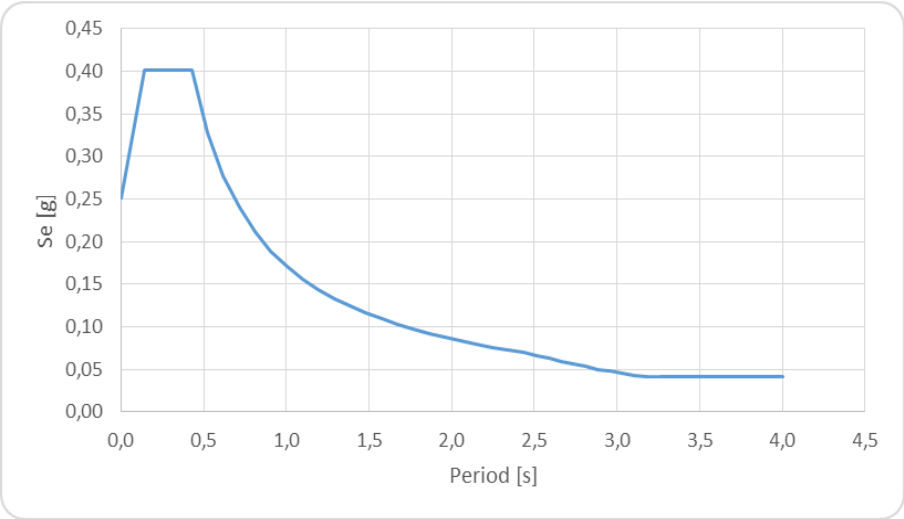


Figure 63 - Response spectra (Bassano del Grappa)

In order to analyze the structure to the seismic action it is necessary to adopt the seismic load combination as explained in the NTC 2008 at the 2.5.3 :

$$E + G_1 + G_2 + P + \psi_{21} \cdot Q_{k1} + \psi_{22} \cdot Q_{k2} + \ldots$$

In this load combination, the wind and the snow action have no role together with the live-load on the roof; only the structural and non-structural self weights and the live load on the deck are considered in addition to the seismic load. The computation of these actions is the same as it was explained in the case of the seismic analysis.

The river flow action in not taken in account because in the normal conditions the river level is very low (around 40 cm) and its velocity and height are not relevant.

Table 38 - Seismic combinations

Combination E.1	$1\ E_y + 0.3\ E_x + 1\ G_1 + 1\ G_2 + 0.6^*\ LL_{,F}$
Combination E.2	$0.3\ E_y + 1\ E_x + 1\ G_1 + 1\ G_2 + 0.6^*\ LL_{,F}$

The earthquake action is schematized in the two main directions of the building X and Y, however the seismic action can have any direction. Therefore, two combinations are computed considering the main action with coefficient 1 and the secondary action on the other direction as the 30%. In the scope of the bridge assessment the combination 1 is adopted for the verifications, nevertheless both the analysis were computed and the combination 2 result to have the lower stress levels.

In order to perform the spectral analysis of the FEM model, the main masses are considered as lumped, within this procedure the modal shapes are easier to define.

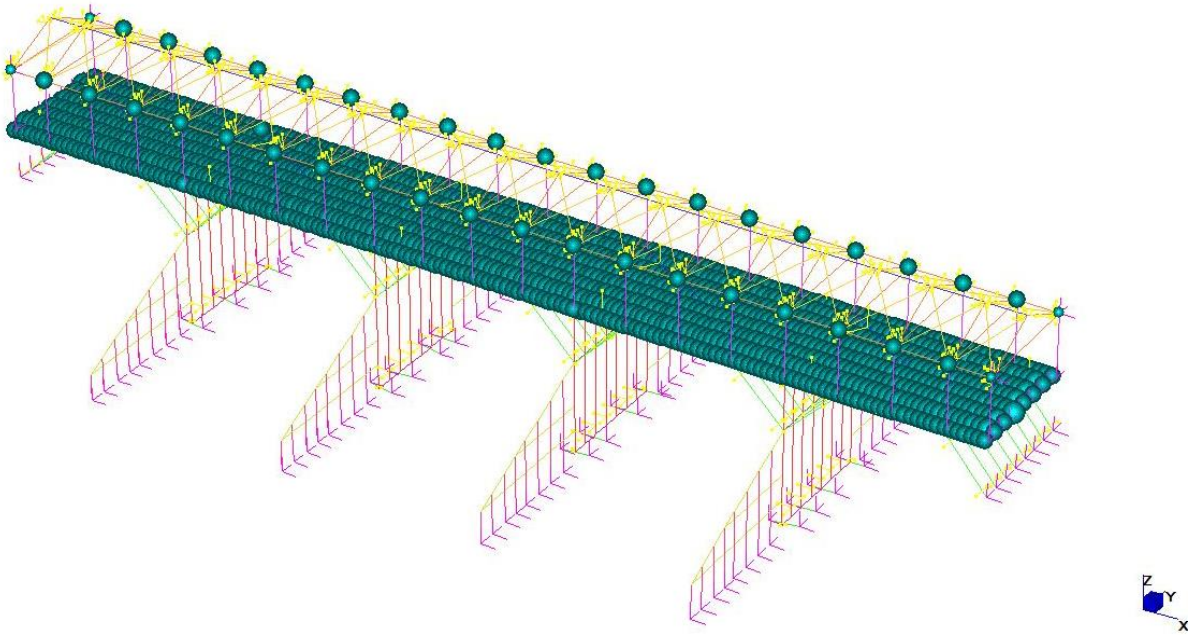


Figure 64 - FEM model, lumped masses

8.2 Behaviour Factor, q

The importance of the evaluation of the behaviour factor is well explained in Ceccotti 2010, where an experimental procedure for the definition of the factor is explained. The behaviour factor q is introduced to reduce the force obtained from the linear elastic analysis in order to takes in account the non-linear response of a structure associated with the material, the structural system and the design procedures. Once the elastic seismic actions are reduced by q, it is allowed to verify the structural elements and connections using the same design values as the relevant static code (Eurocode 5). In practice, the q factor represents the capability of the structure to dissipate energy and to withstand large deformations without ruin. The peak ground acceleration PGA of different earthquakes at which near-collapse status of the structure is reached is divided by the PGA with which the building was designed elastically:

$$q = \text{PGA near collapse} / \text{PGA elasticity}$$

The behaviour factor can be also determined by the reaction side and not from the action side; the q factor is the ratio between the seismic base shear assuming linear-elastic behaviour and the seismic

base shear accounting the non-linear behaviour. The higher the q-factor, the lower the seismic base shear, or alternatively, the more energy a structure is dissipating the higher the q factor.
The Italian code NTC 2008 at §7.7 (seismic design, timber structure) and the Eurocode 8 at §8.3 define two types of seismic-resistant timber structures: a) dissipative behaviour, b) low-dissipative behaviour. In case of structure of the first type, the ductility can be of two classes CD H or CD M (Eurocode 8, table 7.7.I); the buildings belonging to the class H present high energy dissipation and the q factor is between 3 and 5, the class M present low energy dissipation and a q factor between 2 and 2.5. In case of structure low-dissipative the code suggests a behaviour factor lower or equal to 1.5.

Design concept and ductility class	**q**	**Example of structures**
Low capacity to dissipate energy – DCL	1.5	Cantilevers; Beams; Arches with two or three pinned joints; trusses joined with connectors.
Medium capacity to dissipate energy - DCM	2	Glued wall panels with glued diaphragms, connected with nails and bolts; Trusses with doweled and bolted joints; mixed structures consisting of timber framing (resisting the horizontal forces) and non load bearing infill.
	2.5	Hyperstatic portal frames with doweled and bolted joints.
High capacity to dissipate energy - DCH	3	Nailed wall panels with glued diaphragms, connected with nails and bolts.
	4	Hyperstatic portal frames with doweled and bolted joints.
	5	Nailed wall panels with nailed diaphragms, connected with nails and bolts.

Due to the lack of deep information regarding the dynamic behaviour of the bridge, in the spectral analysis of the Bridge of Bassano a q factor of 1.5 is adopted to stay on the conservative side.

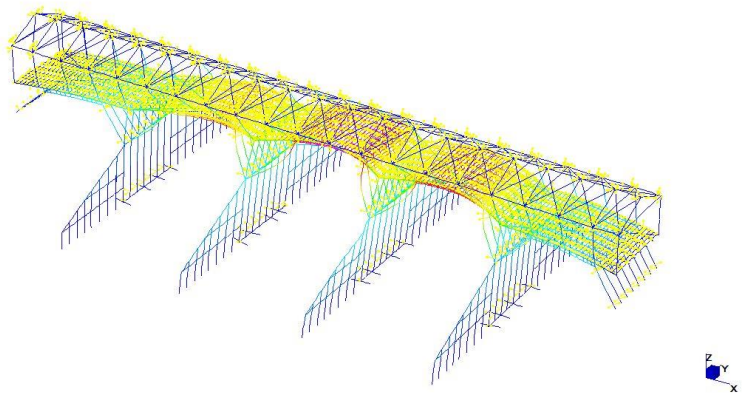
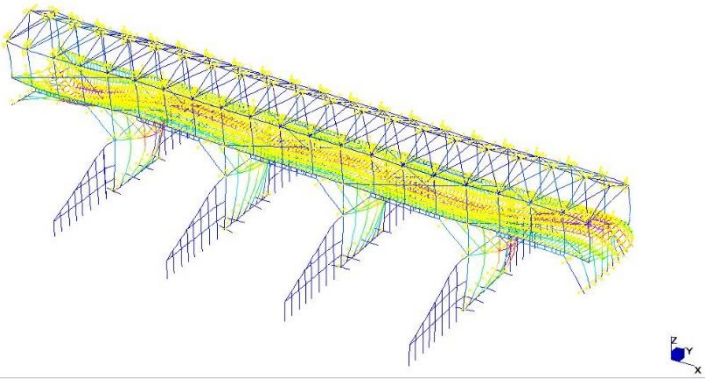
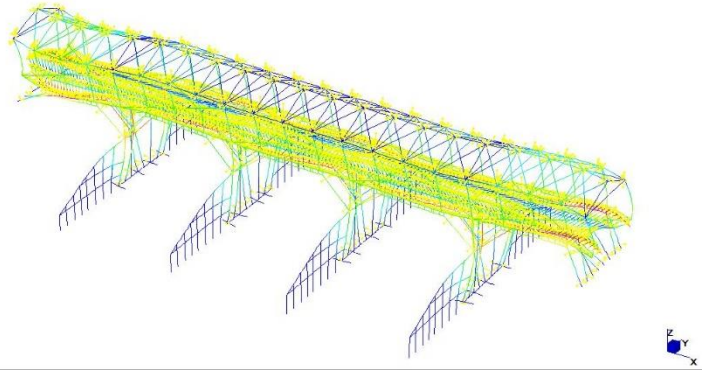
8.3 Natural frequencies analysis

The definition of natural frequencies and the relative vibration modes allows the understanding of the dynamic behaviour of the structure. This step is fundamental in order to perform the spectral analysis based on the mode shapes.

Table 39 - Modes of the model A

Mode	Frequency (Hz)	Modal Mass	PF-X (%)	PF-Y (%)	PF-Z (%)
1	**1.18E+00**	**8.96E+04**	**0**	**64.06**	**0**
2	1.31E+00	8.78E+04	0	0.15	0
3	1.51E+00	8.49E+04	0	8.87	0
4	1.82E+00	7.81E+04	0	0.02	0
5	1.97E+00	1.02E+05	0	5.21	0
6	**2.27E+00**	**1.15E+05**	**60.92**	**0**	**0.02**
7	2.68E+00	1.48E+05	0	0	0
8	2.78E+00	7.45E+04	0	0.01	0
9	3.36E+00	8.93E+04	0	0.52	0
10	**3.52E+00**	**1.69E+05**	**9.20**	**0**	**0**
11	3.99E+00	5.89E+04	0	0	0
12	4.09E+00	7.51E+04	0	0	0
13	**4.31E+00**	**1.45E+04**	**0**	**6.66**	**0**
14	4.49E+00	4.74E+04	0	0	0.04
15	4.54E+00	6.77E+04	0	0.12	0
16	4.55E+00	5.92E+04	0	1.29	0
17	5.03E+00	9.15E+04	2.29	0	0
18	5.29E+00	5.15E+04	0	0	0
19	5.43E+00	3.71E+04	0	0	0
20	5.65E+00	1.46E+05	0	0	0
21	5.73E+00	5.48E+04	1.77	0	0
22	5.83E+00	4.58E+04	1.62	0	0.01
23	5.85E+00	5.57E+04	0.72	0	0.03
24	5.92E+00	6.37E+04	0.04	0	0
25	6.46E+00	6.58E+03	0	0	0

26	6.46E+00	5.59E+03	0	0	0
27	6.46E+00	7.93E+03	0	0	0
28	6.46E+00	5.19E+03	0	0	0
29	6.46E+00	4.40E+03	0	0	0
30	6.62E+00	3.02E+04	0	0	0.05

	Mode 1 1.18 Hz PF 64.06% (Y)
	Mode 6 2.27 Hz PF 60.92% (X)
	Mode 10 3.52 Hz PF 9.20% (X)

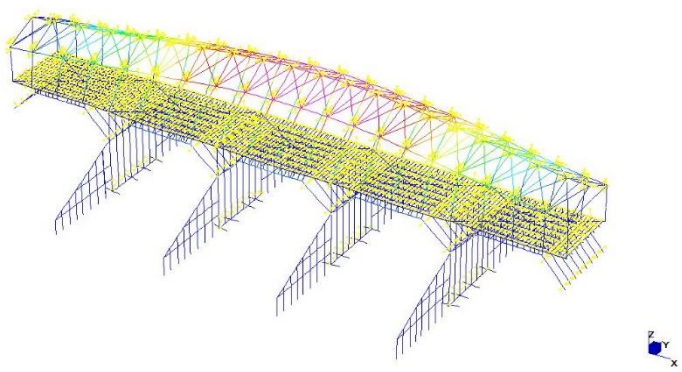
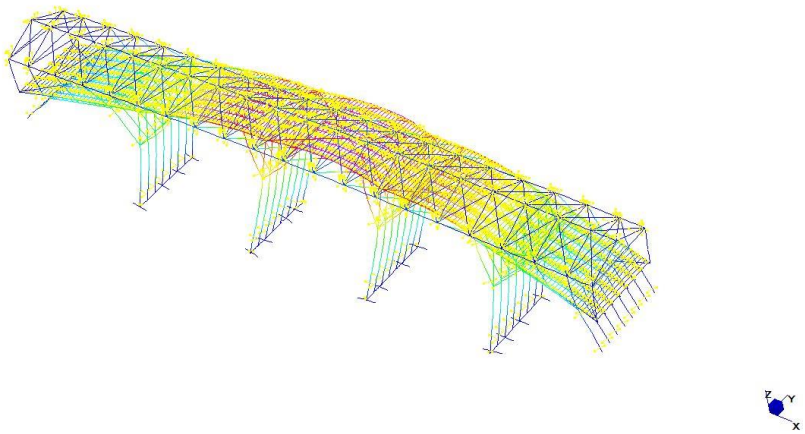
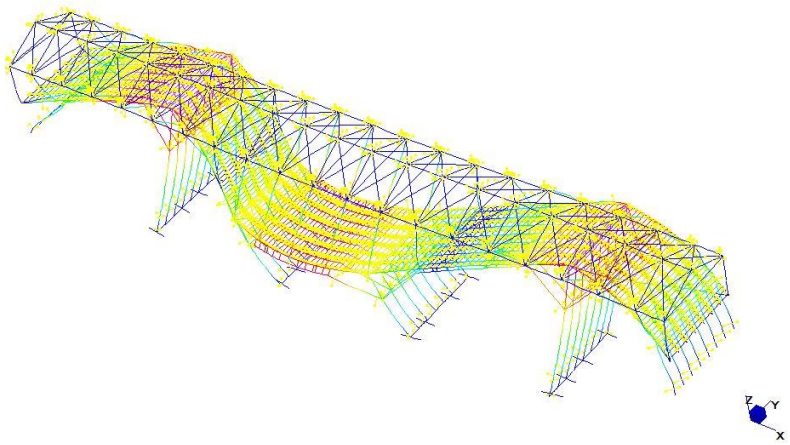
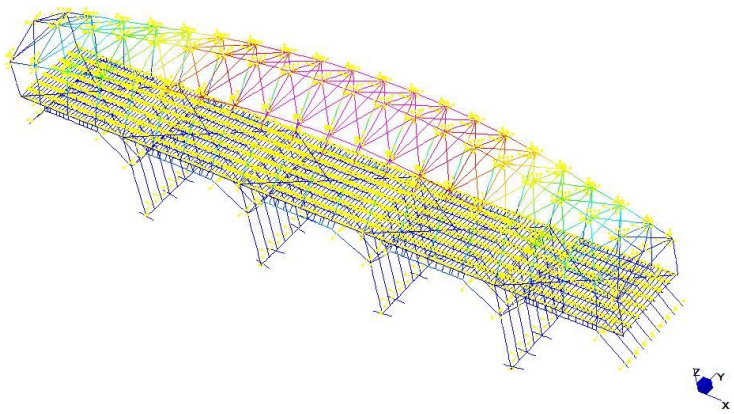
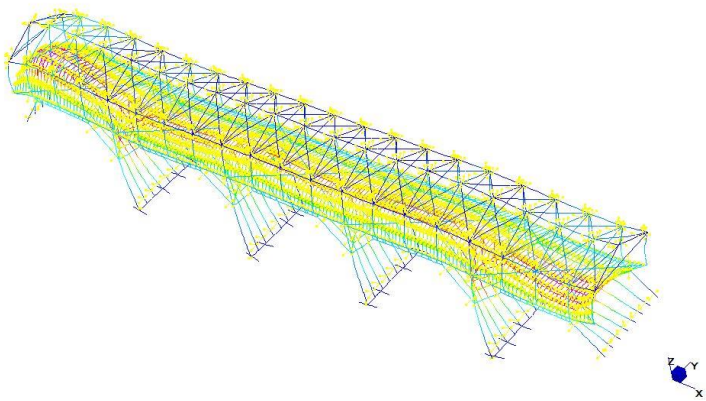
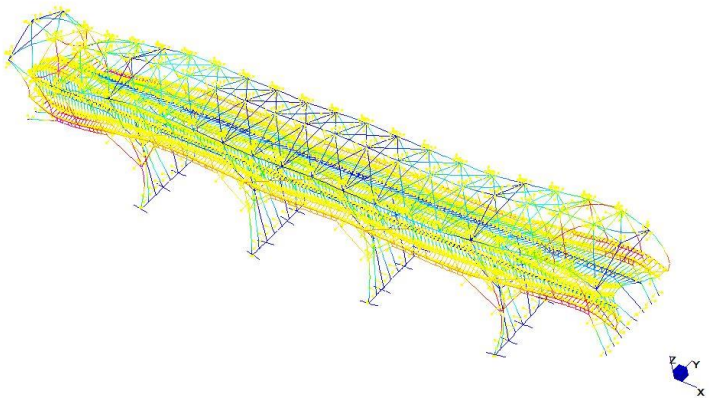
	Mode 13 4.31 Hz PF 6.66% (Y)

Table 40 – Modes of the model B

Mode	Frequency (Hz)	Modal Mass	PF-X (%)	PF-Y (%)	PF-Z (%)
1	**6.44E-01**	**1.20E+05**	**0**	**72.66**	**0**
2	7.76E-01	1.21E+05	0	0.10	0
3	**9.78E-01**	**1.20E+05**	**0**	**7.96**	**0**
4	1.23E+00	9.67E+04	0	0.02	0
5	**1.81E+00**	**1.38E+04**	**0**	**5.18**	**0**
6	1.94E+00	1.21E+05	0	2.44	0
7	**2.25E+00**	**1.24E+05**	**66.85**	**0**	**0.02**
8	2.30E+00	1.06E+05	0.01	0	0
9	2.67E+00	1.47E+05	0	0	0.002
10	2.76E+00	1.66E+10	0	0	0
11	2.77E+00	1.73E+09	0	0	0
12	2.81E+00	1.18E+05	0	2.03	0
13	**3.22E+00**	**1.07E+05**	**7.88**	**0**	**0**
14	3.34E+00	7.35E+04	0	0.	0
15	3.51E+00	1.40E+04	0	0	0
16	3.60E+00	2.61E+10	0	0	0
17	3.65E+00	8.23E+10	0	0	0
18	3.73E+00	4.35E+09	0	0	0
19	3.75E+00	6.85E+04	0	0	0
20	3.84E+00	2.43E+10	0	0	0
21	3.98E+00	6.04E+09	0	0	0
22	4.06E+00	8.08E+04	0	0	0

23	4.13E+00	2.83E+09	0	0	0
24	4.29E+00	3.48E+09	0	0	0
25	4.45E+00	7.67E+07	0	0	0
26	4.46E+00	5.40E+04	0	0	0.05
27	4.52E+00	5.89E+04	0.01	0.01	0
28	4.63E+00	1.92E+07	0	0	0
29	4.67E+00	9.93E+04	0.78	0	0
30	4.80E+00	2.92E+09	0	0	0

	Mode 1 0.64 Hz PF 72.66% (Y)
	Mode 3 0.98 Hz PF 7.96& (Y)

	Mode 5 1.81 Hz PF 5.18% (Y)
	Mode 7 2.25 Hz PF 66.85% (X)
	Mode 13 3.22 Hz PF 7.88% (X)

8.4 Spectral response analysis

Once the natural frequencies and the modal shapes are defined, it is possible to carry out the spectral analysis implementing the response spectrum in the software.

The deformed shapes corresponding to the two spectral combinations are of course similar to the main modal shapes.

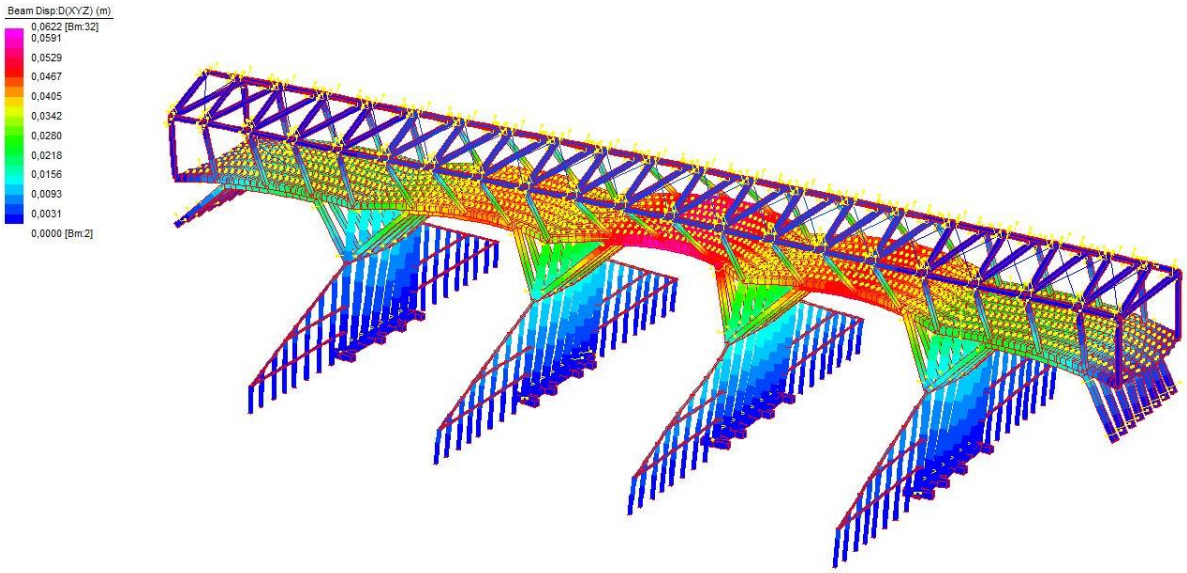


Figure 65 - Model A, displacement (XYZ), combination EQ.1 (solid view of the linear elements).

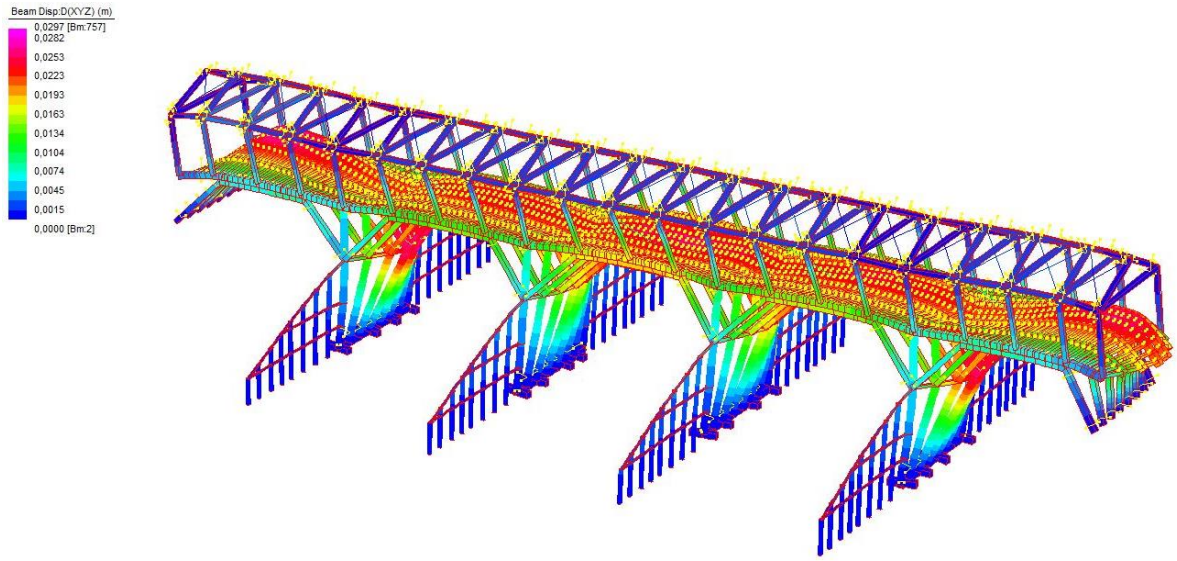


Figure 66 - Model A, displacement (XYZ), combination EQ.2 (solid view of the linear elements).

Figure 65 and Figure 66 show the deformed shape of combination EQ.1 (Y direction) and EQ.2 (X direction) respectively.
It is possible to notice how the roof system does not show high levels of deformations; the masses at that level are low (no live load at the roof is included in the seismic combination).

8.5 Model A, verification of the foundation elements to the seismic action

As well as in the static analysis, the foundation elements are assessed in the case of the seismic action. The combination 1 is considered (maximal in direction Y). In the Table 41 and Table 42 the results are presented: the stress level of the beams is normalized to the code limit level, therefore any value higher than the unit correspond to a stress level higher than the prescribed maximum.

Table 41 - Model A, Spectral Analysis; Cavezzali stress levels. In red the value higher than the resistance limit (>1).

	El. n°	Max $\sigma_{m,y,d}$/ fm,d	Max $\sigma_{m,z,d}$ / fm,d	Max $\sigma_{m,y,d}+\sigma_{m,z,d}$	Max $\tau_{d,z}$ / $f_{v,d}$	Max $\tau_{d,y}$ / $f_{v,d}$	Max $\sigma_{c,o,d}$ / $f_{c,0,d}$
Pier 1	1	0,53	0,01	0,54	0,87	2,24E-02	5,47E-04
	2	0,85	0,01	0,85	1,43	1,59E-02	3,23E-03
	3	0,61	0,01	0,61	1,02	1,56E-02	2,13E-03
	4	0,41	0,01	0,41	0,67	1,58E-02	5,60E-04
Pier 2	5	0,39	0,01	0,39	0,67	9,00E-03	5,08E-04
	6	0,74	0,01	0,74	1,26	1,13E-02	1,35E-03
	7	0,60	0,01	0,61	1,00	1,04E-02	1,31E-03
	8	0,65	0,01	0,65	1,06	1,31E-02	4,79E-04
Pier 3	9	0,34	0,01	0,34	0,56	7,51E-03	2,67E-04
	10	0,77	0,00	0,77	1,31	7,32E-03	1,55E-03
	11	0,88	0,00	0,88	1,44	7,32E-03	1,55E-03
	12	0,86	0,01	0,86	1,44	7,27E-03	1,21E-03
Pier 4	13	0,45	0,01	0,46	0,76	8,07E-03	4,34E-04
	14	0,54	0,00	0,54	0,88	6,24E-03	2,23E-03
	15	0,85	0,00	0,85	1,39	5,02E-03	3,22E-03
	16	0,52	0,01	0,53	0,85	1,55E-02	4,98E-04

The *Cavezzali* elements are not verified to the shear force in direction Z, but their level of bending moment is acceptable.

Table 42 - Model A, Spectral Analysis; thresholds stress levels. In red the value higher than the resistance limit (>1).

El. Number	Max $\sigma_{m,y,d}$ / $f_{m,z,d}$	Max $\sigma_{m,z,d}$ / $f_{m,z,d}$	Max comb. $\sigma_{m,y,d}$; $\sigma_{m,z,d}$	Max τ_d / $f_{v,d}$	Max τ_{tor} / f_{torque}	Max comb. τ_d ; τ_{tor}	Max $\sigma_{c,o,d}$ / $f_{c,0,d}$
T 1	0,35	0,02	0,35	0,69	1,580E-05	0,48	0,01
T 2	0,40	0,01	0,40	1,24	6,527E-05	1,53	0,03
T 3	0,40	0,01	0,40	1,20	4,118E-05	1,45	0,02
T 4	0,39	0,02	0,39	0,82	1,086E-05	0,68	0,01

The thresholds 2 and 3 result to be unsafe due to the shear stress level in direction z; the stress level is 20% higher than the code limit. Even if the level of torsion is low, combined with the shear stress makes the solicitation higher than the limit.

8.6 Model A: comparison between the analyses results

In the following bar graphs, the results from the static analysis and the spectral dynamic are compared with the daily state. The physical meaning of this comparison is to find in the difference in terms of stress levels at the foundation in case of the over flow loading (static analysis) and the seismic loading (dynamic).

In orange the results from the flood state are presented (F), in this case the horizontal action is due to the river flow and in lower percentage to the wind action. On the other hand, in yellow, the results from the spectral analysis (EQ) are presented; in this case, the seismic action is the only horizontal load. In blue, it is possible to see the results from the daily conditions; in this state horizontal forces are not present. The dotted red line is representing the limit stress level given by the NTC 2008.

What comes out is that the river-flow has higher impact on the foundation structure respect to the seismic action, therefore taking in account the river-flow force (equal to the over flow of the 1966) in the assessment procedure would mean take in account the worst case.

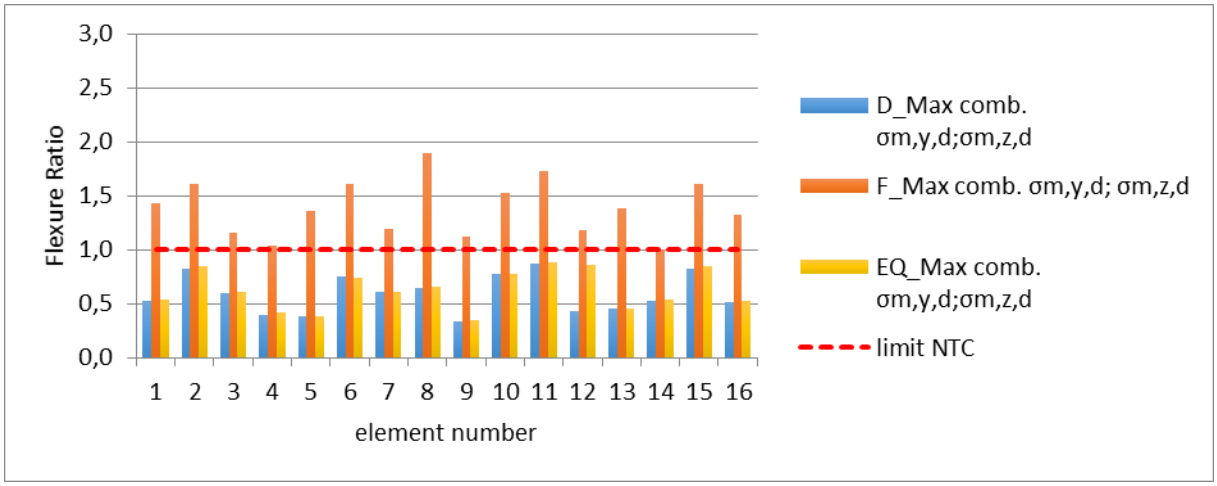


Figure 67 - Comparison of the results between daily, over flow and earthwquake state. Flexure Ratio of *Cavezzali* (model A).

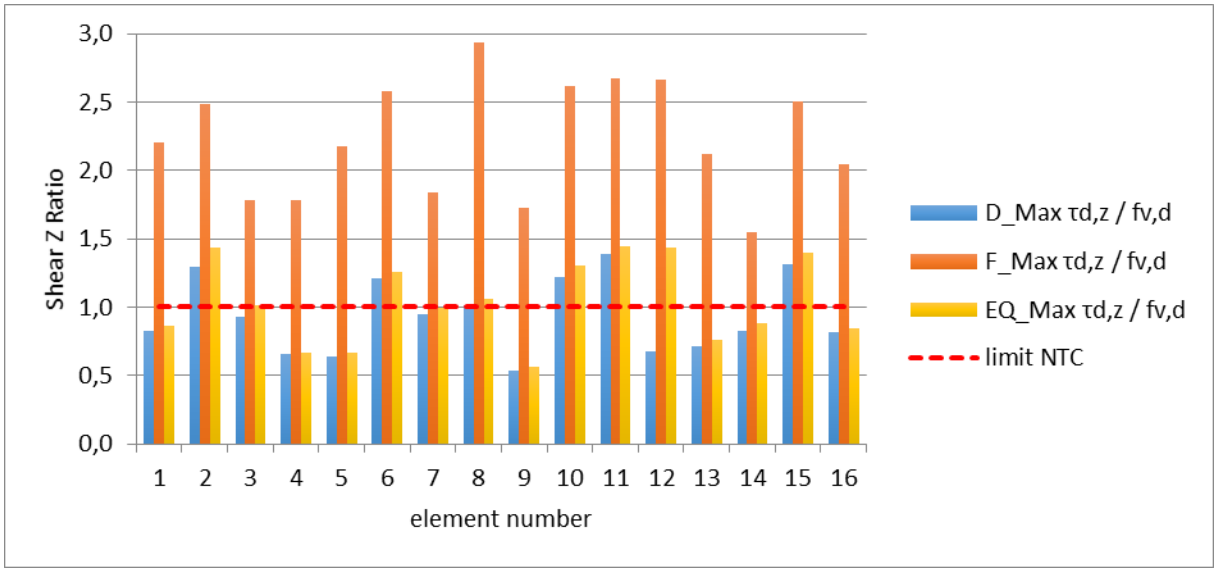


Figure 68 - Comparison of the results between daily, over flow and earthwquake state. Shear Z Ratio of *Cavezzali* (model A).

In terms of bending (Figure 67) and shear stresses in direction Z (Figure 68), the overflow action creates the highest stress level on the Cavezzali beams, the values are greater than the code limit. In this case the seismic conditions and the daily are close, slightly higher is the EQ state.

Concerning the shear stress in the direction Y, the out of plane direction, the stress level for the floods state is still the highest but there is a gap between the earthquake and the daily conditions when no horizontal forces are present. However, in all the three states the shear in direction Y is lower than the code prescription (Figure 69).

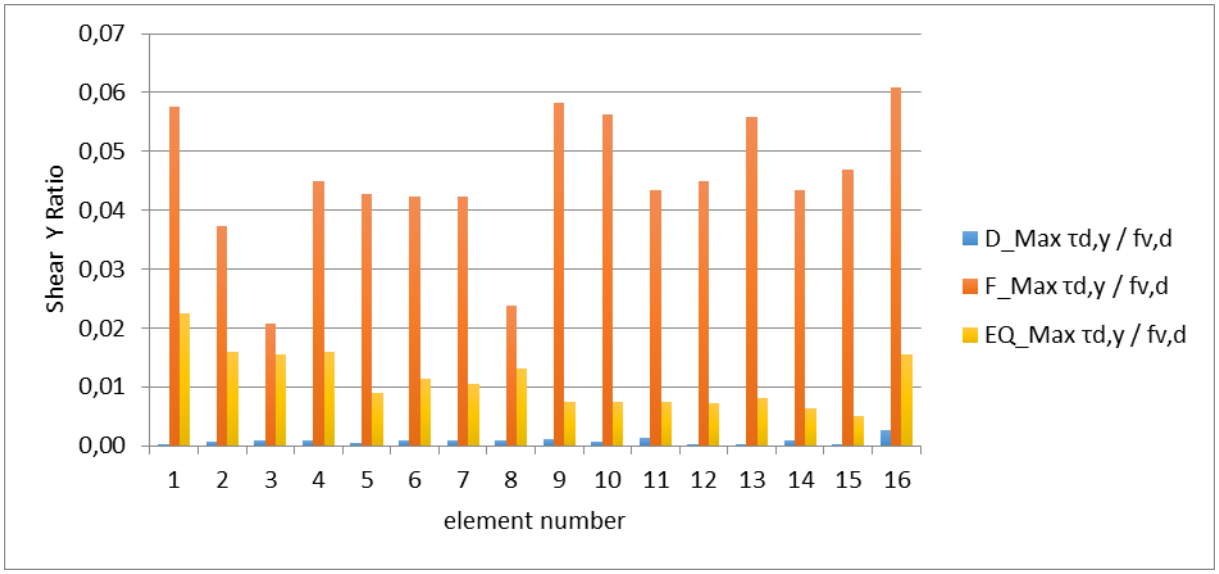


Figure 69 - Comparison of the results between daily, over flow and earthwquake state. Shear Y Ratio of *Cavezzali* (model A).

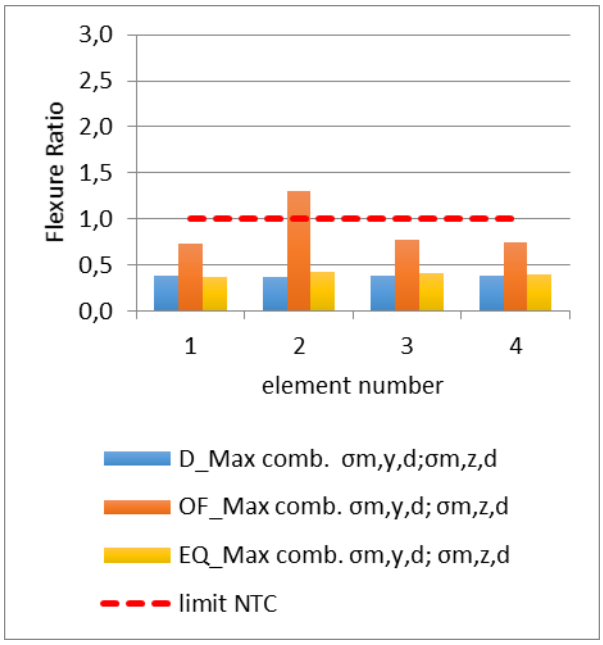


Figure 70 - Comparison of the results between daily, over flow and earthwquake state. Flexure Ratio of the thresholds (model A).

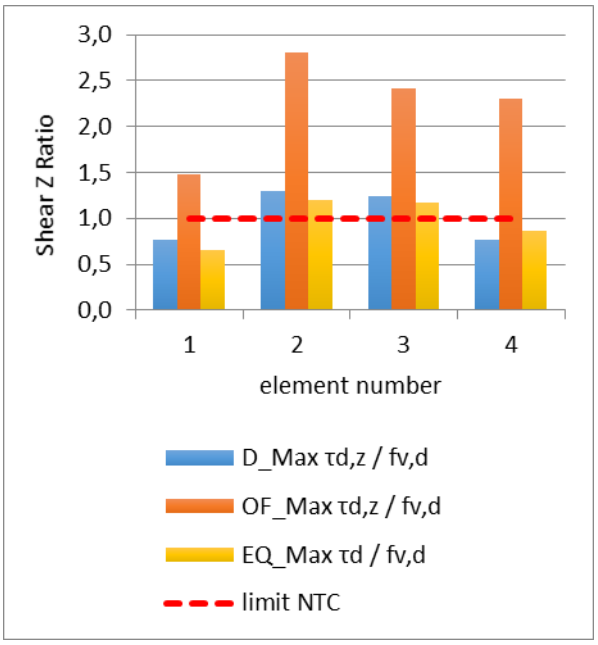


Figure 71 - Comparison of the results between daily, over flow and earthwquake state. Shear Z Ratio of the thresholds (model A).

In Figure 70 and Figure 71 the threshold's stress levels are represented; the seismic action cause lower stress levels than the over-flow loading. In this case, due to the orientation of these elements, the Y direction is not relevant.

8.7 Model B, verification of the foundation elements to the seismic action

In the case of a damaged configuration of the timber bridge, means the absence of the *Rostri* element, the structure is of course more vulnerable and less stiff in the out of plane. Nevertheless, the response spectrum analysis is based on the natural frequencies of the structure and the model A and B present, of course, different dynamic behaviour. Considering the spectra representation of the seismic response, the two model present different acceleration levels due to the different vibration modes (masses and stiffness); for this reasons the seismic actions are not the same for the two bridge configurations. In Table 43 and Table 44 the results of the spectral analysis of the model B are presented, the foundation elements (Cavezzali and thresholds) are verified using the code procedures.

Table 43 - Model B, Spectral Analysis; Cavezzali stress levels. In red the value higher than the resistance limit (>1).

	Element n°	Max $\sigma_{m,1,d}/f_{m,d}$	Max $\sigma_{m,2,d}/f_{m,d}$	Max comb. $\sigma_{m,1,d}$; $\sigma_{m,2,d}$	Max $\tau_{d,z}/f_{v,d}$	Max $\tau_{d,y}/f_{v,d}$	Max $\sigma_{c,o,d}/f_{c,0,d}$
Pier 1	1	0,58	0,05	0,59	1,10	0,07	1,300E-03
	2	0,85	0,05	0,85	1,44	0,07	3,240E-03
	3	0,61	0,05	0,62	1,02	0,06	2,158E-03
	4	0,46	0,05	0,46	0,93	0,09	1,185E-03
Pier 2	5	0,46	0,07	0,47	1,12	0,09	1,373E-03
	6	0,74	0,08	0,75	1,26	0,10	1,644E-03
	7	0,61	0,07	0,62	1,09	0,09	1,970E-03
	8	0,80	0,07	0,81	1,72	0,10	4,915E-04
Pier 3	9	0,41	0,08	0,42	0,96	0,09	9,356E-04
	10	0,77	0,08	0,79	1,28	0,10	1,173E-03
	11	0,88	0,08	0,89	1,45	0,10	1,523E-03
	12	0,51	0,08	0,52	1,13	0,09	7,667E-04
Pier 4	13	0,50	0,05	0,51	1,03	0,06	8,491E-04
	14	0,54	0,04	0,54	0,88	0,06	2,216E-03
	15	0,85	0,05	0,86	1,40	0,06	3,188E-03
	16	0,57	0,05	0,57	1,07	0,06	7,339E-04

The *Cavezzali* elements are not safe due to the shear level. Concerning the thresholds, only the second and the third elements are unsafe to the seismic action in the configuration B.

Table 44 - Model B, Spectral Analysis; thresholds stress levels. In red the value higher than the resistance limit (>1).

El. Number	Max $\sigma_{m,y,d}$ / $f_{m,z,d}$	Max $\sigma_{m,z,d}$ / $f_{m,z,d}$	Max comb. $\sigma_{m,y,d}$; $\sigma_{m,z,d}$	Max τ_d / $f_{v,d}$	Max τ_{tor} / f_torque	Max comb τ_d; τ_{tor}	Max $\sigma_{c,o,d}$ / $f_{c,0,d}$
T 1	0,34	0,02	0,35	0,69	6,476E-06	0,47	0,04
T 2	0,85	0,01	0,85	1,44	9,289E-05	2,07	0,01
T 3	0,44	0,01	0,45	1,18	4,962E-05	1,38	0,01
T 4	0,39	0,02	0,39	0,83	1,904E-05	0,68	0,00

8.8 Model B: comparison between the analysis results

The comparison between the stress levels for the three states (daily D, floods F and earthquake EQ) is proceeded for the model B as well. The results are comparable with the case A, even if the stress levels are generally higher.

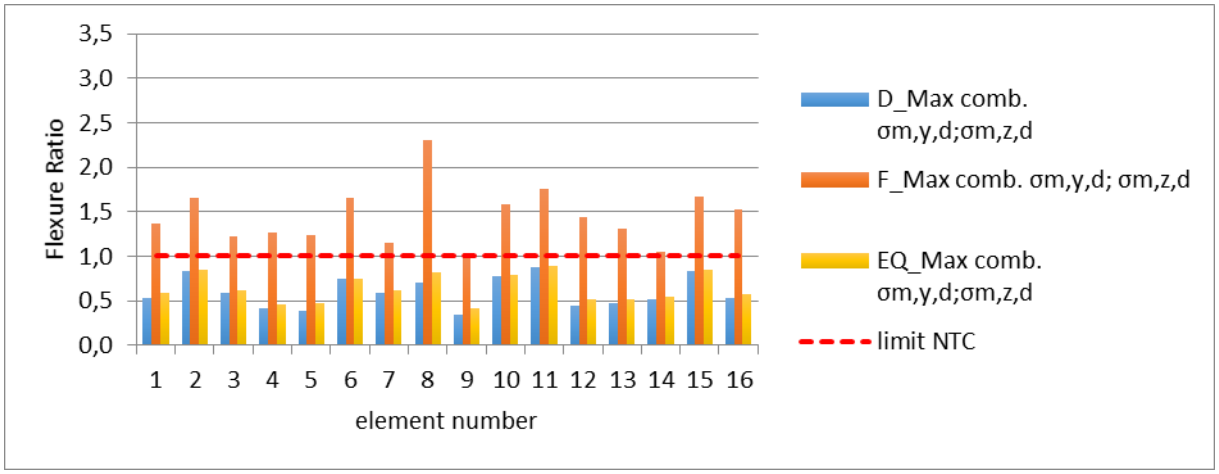


Figure 72 - Comparison of the results between daily, over flow and earthwquake state. Flexure Ratio of *Cavezzali* (model B).

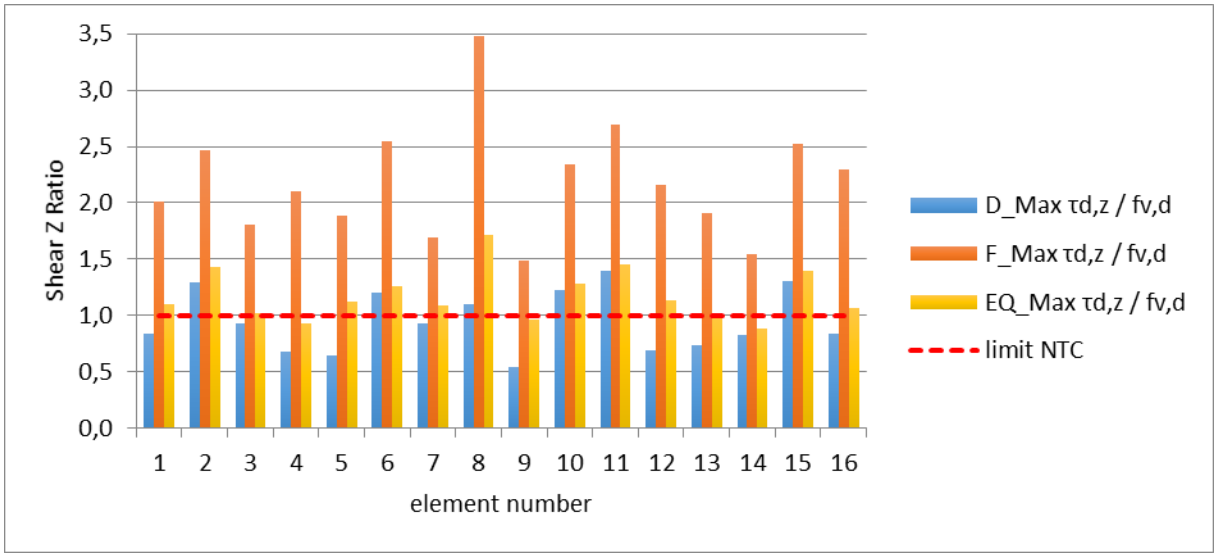


Figure 73 - Comparison of the results between daily, over flow and earthwquake state.Shear Ratio Z of *Cavezzali* (model B).

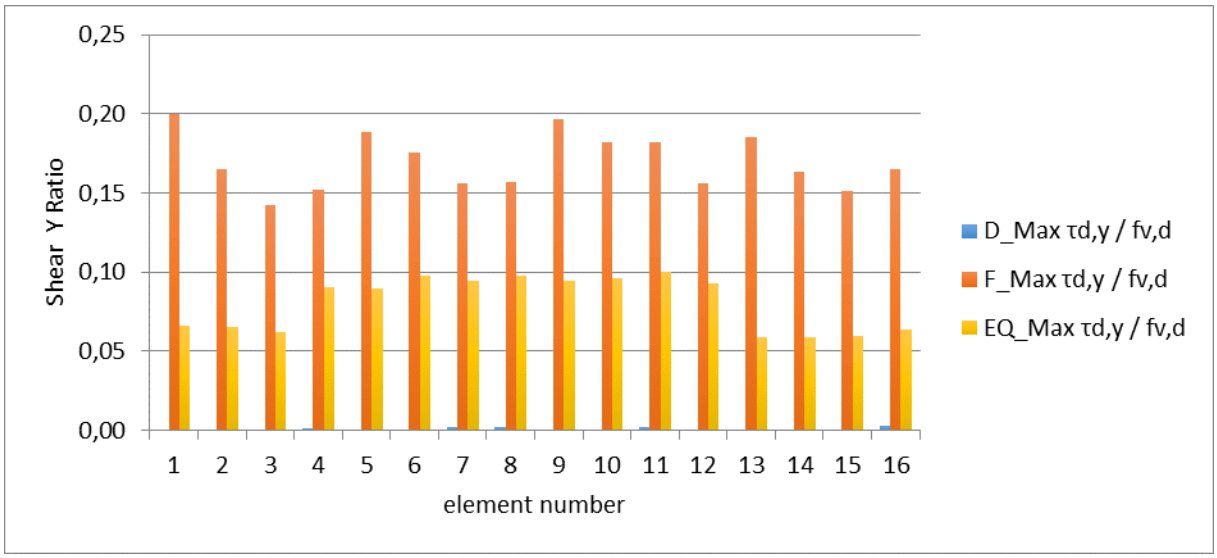


Figure 74 - Comparison of the results between daily, over flow and earthwquake state.Shear Ratio Y of *Cavezzali* (model B).

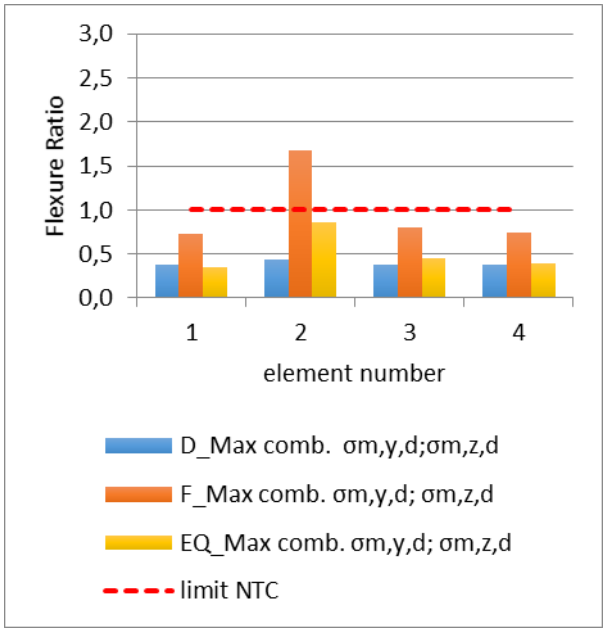


Figure 75 - Comparison of the results between daily, over flow and earthwquake state. Flexure Ratio of the thresholds (model A).

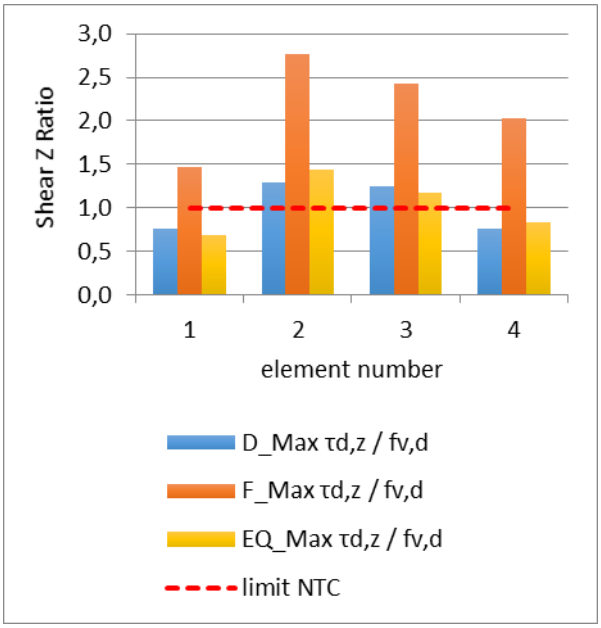


Figure 76 - Comparison of the results between daily, over flow and earthwquake state. Shear Z Ratio of the thresholds (model A).

In Figure 72, Figure 73 and Figure 74 the *Cavezzali*'s stress levels are presented, the most important difference respect to the model A is in the shear force in direction Y, the elements responding to the force in Y direction in model B are less than in model A, therefore the shear force is more concentrated on the Cavezzali beams. In the out of plane direction the result in terms to shear force are to really higher in the model B than in A. In Figure 76 and Figure 75 the stress levels of the threshold are presented, the relationship between the different states is similar to the case A, but a slightly higher of stress is visible.

Due to the analyses results, it is possible to conclude that at the flood state, in absence of the *Rostri* elements, the highest level of stress at the foundation is reached.

8.9 Out of plane behaviour, comparison between model A and B

Considering eleven control points at the deck level, the displacement in the out of plane direction is analysed. In case of the earthquake action the maximum displacement for the model A (with Rostri) is equal to 6 cm, instead in case of model B the maximal displacement is 9.5 cm. Any damaged situation is going to be in between of those values.

The peak are located at the middle span, between the piers, this is due to the lower stiffness of the deck. The seismic action, differently from the hydraulic, is distributed on the entire structure, therefore the deck presents higher displacements.

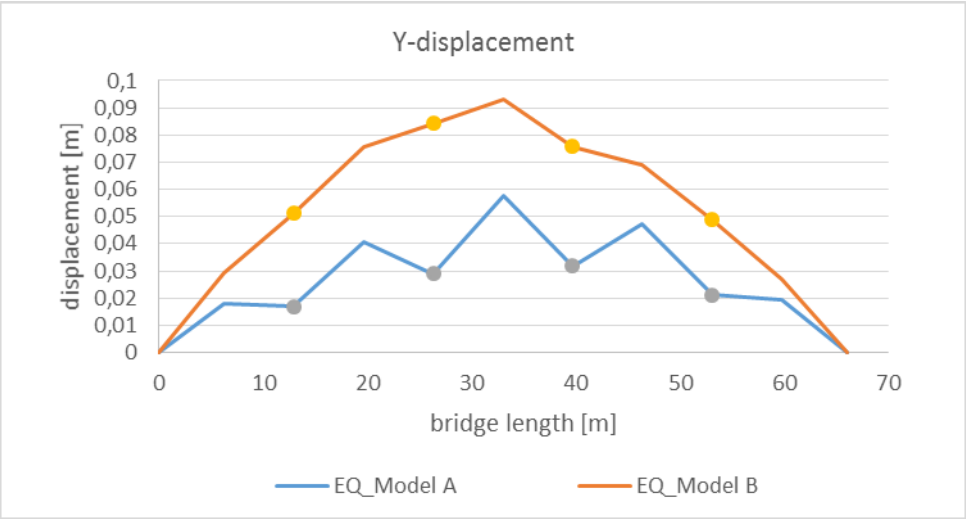


Figure 77 - Displacement in the out of plane direction (Y), Earthquake action (dots represent piers)

In the case of the model at the actual state (A), it is evident the presence of the Rostri and their high level of stiffness in the out of plane behaviour.

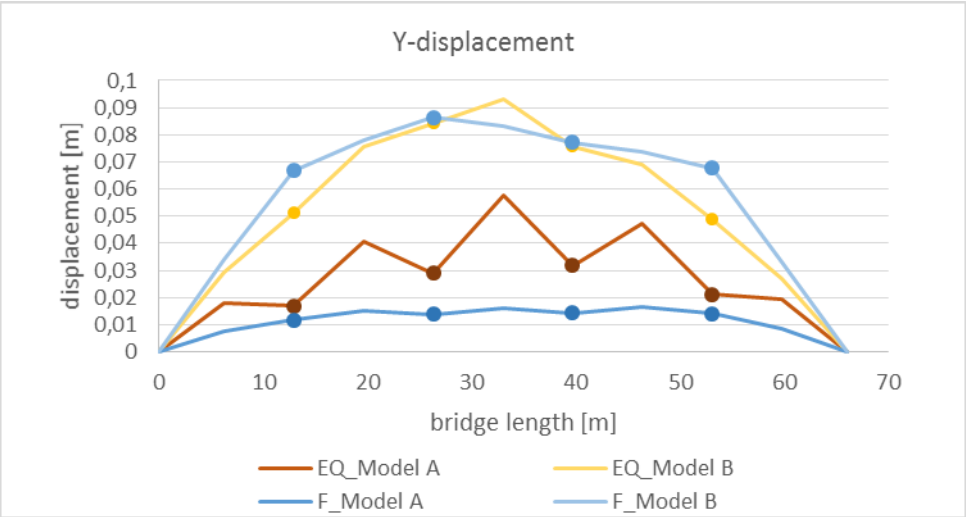


Figure 78 - Comparison between the displacement in the case of Floods (F) and Earthquake (EQ) (dots represent piers)

Comparing the floods and the earthquake displacement results (Figure 78), it is possible to see the difference in the peak location. In the earthquake case the displacement is slightly higher than the flood case and corresponding to it, the shear forces at the top of the piers are higher as well.

It is necessary to consider that in the Floods case the wind is taken in account and its load has an important role at the roof level; on the other hand, the seismic combination does not take in account the wind load.

There is high difference between the two total horizontal forces, in case A (with the Rostri) the total force is equal to 832 kN , instead in model B the total force is 376 kN. This is due to the difference in the seismic forces, the two structures present different stiffness (the first modes are far from each other in terms of frequencies). In the Floods case the same loads were present in both the models, in the earthquake case the loading are different.

In Figure 79 the first modes of the two structures (A and B) are presented together with the acceleration-displacement spectra; it is visible how model B has a lower request in acceleration.

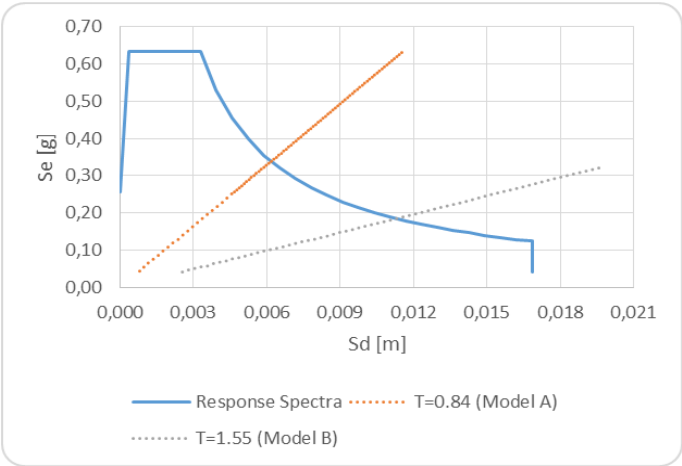


Figure 79 - First mode of model A and model B compared to the response spectra

In Table 45 some of the results are compared between Floods and Earthquake case, the flood case has higher horizontal force Fy than the earthquake. The difference between the two cases (F and EQ) in terms of vertical force Fz is due to the different type of combinations; as prescribed by the code in the case of seismic action many variable loads may be considered as zero.

Table 45 - Comparison between Floods and Earthquake cases

	A	B
EQ	Total Fy = 833 kN	Total Fy = 377 kN
	Total Fz =4730 kN	Total Fz =4780 kN
F	Total Fy = 908 kN	Total Fy =875 kN
	Total Fz =9128 kN	Total Fz =8759 kN

9. FUTURE STEPS, DYNAMIC FREQUENCY ANALYSIS

The experimental investigation is fundamental in the diagnosis of an historical building, it is advisable to collect as many information, related to the structural behaviour of the building, as possible. The experimental data are at the base of a realistic behavioural model of historical buildings. In the non-destructive tests, the dynamic testing (and successive modal analysis) can be consider a very effective tool; moreover the dynamic test is the only ND way to experimentally measure parameters related to the global structural behaviour.

The experimental modal analysis consist on the field measuring and analysing the dynamic response of the structure when excited by an input, this analysis combine vibration test data and analytical methods to determine modal parameters of the structure (frequencies, damping, mode shapes). One of the most important outcome of this test is the validation of FEM models in their elastic range for successive structural verifications; nevertheless, it may be useful in monitoring, in checking repair efficiency or in general in troubleshooting of structures experiencing problems in response.

The vibration test gives information regarding the structural behaviour, especially when failure thresholds are likely to occur and where weak points or defect of the structure are located. The test consists on the application of an adequate exciting source and the recording of vibration by means of acceleration sensors. The choice of the excitation mechanism can make the difference in the test results, experience and a deep knowledge of the structure are fundamental. Shakers, impact hammers, drop weight systems, ambient vibration can be used. The response transducers are able to transform a physic quantity (displacement, velocity, acceleration…etc.) into a proportional electrical signal, ready to be processed. Displacement might be the better choice in case of low frequency response cases such as engineering structures. However, the use of accelerometers is more common because it is more cost effective; it is possible, anyway, to calculate the displacement by numerical integration of the acceleration records. What is possible it to do is an output-only identification technique (or ambient vibration), this method is widely applied in large civil engineering structures such as bridges and towers in which artificial excitations and the determination of forces constitutes a problem. The output-only technique requires that excitements are reasonably random in time and in the physical space of the structure. The robust results associated with the facilities and the economic low cost test and real operating conditions during its daily use makes this technique very popular.

Test procedure:

1. Modal analysis with a FEM model to have a preliminary idea about the frequency values and the modal shapes
2. Definition of the number of points to measure
3. Localized signal measure in order to characterize the signal to noise ratio and to have an idea about the frequencies involved.

4. Setup of the measurements. The measuring duration has to be defined, an empirical rule is to consider 2000 times the highest natural period of interest (the lowest frequency).
5. Preliminary check, to avoid low signal or data losses
6. Test

The University of Sydney has done a study of several timber bridges in Australia (Crews 2004), those bridges were in excess of 50 years old and many are in degraded conditions but they are highly valued, not just for economic reasons but also for social and historical value placed on them by the rural community. A campaign of dynamic frequency analysis was done, 69 bridges were tested. The aim of the project was to research and develop a cost – effective and accurate method for assessing the structural condition and predicting the load capacity of timber bridges, based on analysis of fundamental frequency of the superstructure to determine the global stiffness of a bridge deck.
Accelerometers were attached underneath the bridge girders and the vibration response and natural frequency of the bridge superstructure is measured when a calibrated sledgehammer is used to hit the unloaded deck and, than again with a relatively small mass applied at the middle span. The difference in response allows the load carrying capacity of the bridge to then be calculated, using a reliability based strength model, derived from extensive testing of aged timber girders. Thanks to these test campaign, some considerations were possible regarding the degradation effects. The increase of the traffic volume and loading intensity can cause cumulative incremental damage. The influence of these effects on the rate of deterioration of timber bridges can be particularly significant. Such damage to the strength and stiffness, in general, may not be identifiable from visual inspections, even by experienced staff. It is important not only the discovery of any damage but also the determination of the effects that such damage have on the structural integrity of the bridge.

Figure 80 - Crews 2004, example of tested bridge

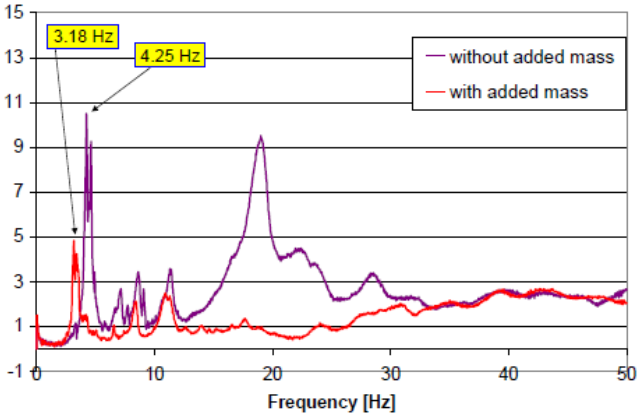


Figure 81 - Crews 2004, example of frequency result

The project result is a new method, based on dynamic response of timber bridges to an impact load, to measure the flexural stiffness of timber bridges. With a statistically based approach, the knowledge of flexural stiffness can be converted into an estimate of the load carrying capacity. The method result to be quick and cost-effective (less than 20% of traditional load testing).

The problem of a proper choice in the excitation mechanism was experienced in many cases. The University of Ostrava analysed a timber Bridge on the Bata's channel in Czech Republic (Cechakova 2012), the bridge should bear heavy and special load conditions but the natural degradation of the glued laminated timber beams is an important issue. The structure was experimentally tested and a FEM model was created. The bridge was loaded dynamically using a car passing over the bridge, the speed of the vehicle was 15 km/h, the record frequency of measured data was 100 Hz. Based on the measured values of deflections, the dynamic factor of the load was determined as 1.16. Unfortunately, the load used was too small for dynamic analysis of the structure. The vibration frequencies evoked by the testing vehicle did not reach the Eigen frequencies of the bridge.

In Chun 2015 the structural analysis of the historic timber bridge of Wenxing is presented. The dynamic characteristics of the structure were analysed and a FEM model was done with both the undamaged and damaged configuration. On site survey revealed that the bridge occasionally swayed obviously when people walked on it. The dynamic analysis of Wenxing Lounge Bridge was carried out and the main modes of vibrations were analysed. The basic frequency of walking is approximately 1.8–2.5 Hz, the predominant frequency of Wenxing Lounge Bridge was 2.02 Hz and the second natural frequency was 2.69 Hz. So resonance between Wenxing Lounge Bridge and people walking on the bridge occurred easily. This is a problem that may easily come up in this type of historical structure, what is suggested in Chun 2015 is that the vibration under influence of walking action or wind action shall be researched in the future. It is a very important factor that affects structural safety of this kind of bridges.

A dynamic frequency analysis is suggested for the timber Bridge of Bassano in order to validate the FEM model. Have a good understanding of the entire structure behaviour, especially in what concern the dynamic behaviour might be fundamental. The FEM model has necessarily some simplifications and it has to take in account the damage state in an approximate way; this type of testing may help in the evaluation of the effects done by the natural damage.

10. CONCLUSIONS

After the damage survey and the monitoring performed in 2015, it was evident that a structural assessment of the Palladio's timber bridge was required. The high level of deformation and decay is a concern for both the safety of the pedestrians and for the conservation of the historical monument, which is a symbol of the town of Bassano del Grappa.

This thesis project is focused on two fundamental aspects: the assessment of the foundation system, which was modified many times during the bridge's history; and the analysis of the role of the Rostri elements, which affects the structure especially in the out of plane behaviour of the structure. Based on the analysis carried out in this thesis, the Bridge was determined to be unsafe because of the high stress levels at the foundations. Even if temporary structures are present, a retrofitting plan is urgent and necessary.

The results of the three analyses, performed with different loads-states, shows that the minimum safety level is not reached. Applying the quasi-permanent combination, the foundation elements are overstressed, meaning that the daily state is not safe. In addition to that, it has to be accounted that the 90's heavy floor has been already removed decreasing the daily state stress. The flood state resulted in high levels of shear stresses at the base of the structure; unfortunately, this accidental load is not so rare: many times the structure was damaged by the river flow. Finally, performing the linear dynamic analysis, the earthquake's effect was analysed. Comparing the stresses at the foundations, this condition resulted in lower stress levels than the flood case, but still many elements are not safe in respect to the code's limits.

The reason why the foundation system is nowadays in these unsafe conditions may be found in the decay process, in the natural riverbed modification and, furthermore, in the changes of the structure done during past reconstructions and refurbishments. The results of the analysis, showing the unsafe level of stresses, are comparable with the strong level of damage of the foundation timber elements.

The second main aspect of the project is the role of the triangular-shaped portion of the piers, called *Rostri*. In order to understand their structural role two FEM model were analysed: one with and one without these elements. Performing the same analysis for both the cases, it turned out that in the second case the stress levels at the base are higher, especially in the out of plane direction (Y). In the case of vertical static loads, the role of the *Rostri* is negligible; on the other hand, if the structure is horizontally loaded (such as in floods or earthquakes) their role becomes evident. The presence of the *Rostri* makes the structure stiffer in the out of plane direction; therefore, the displacements are much lower. The analyses of the two models, with and without the Rostri elements, represent the two limit cases, where all possible damage states will fall in-between the two extremes. Therefore, it is possible

to conclude that it is necessary to include the Rostri elements in the seismic analysis; this because a higher stiffness correspond to a higher seismic force. In case of flood state analysis, instead, it is safer not to include the Rostri in the model and take in account the possible damaged configuration of the bridge.

The flood and the earthquake states are compared, but it is important to notice the difference in their load distribution and in the nature of the load itself: the flood load is mass-independent and applied only at the piers, instead the earthquake action is mass-dependent and applied to the entire structure.

To conclude, it has been demonstrated that the worst scenario, in terms of stress levels at the foundation, is the flood-state in the absence of the Rostri elements at the piers. The performed analyses and element verifications, together with the understandings on the structural behaviour of the bridge are one step in the direction for conservation, in the endeavour to maintain our cultural heritage and preserve it to the future generations.

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