STRUCTURAL MODELLING, ANALYSIS AND EVALUATION OF THE HISTORIC BUZLUPINAR BRIDGE AND RECOMMENDATIONS FOR ITS RECONSTRUCTION

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
MIDDLE EAST TECHNICAL UNIVERSITY

BY

EZGİ ÇABUK

IN PARTIAL FULFILMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF MASTER OF SCIENCE
IN
RESTORATION IN ARCHITECTURE

SEPTEMBER 2015

Approval of the Thesis:

STRUCTURAL MODELLING, ANALYSIS, AND EVALUATION OF THE HISTORIC BUZLUPINAR BRIDGE AND RECOMMENDATIONS FOR ITS RECONSTRUCTION

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ABSTRACT

STRUCTURAL MODELLING, ANALYSIS, AND EVALUATION OF THE HISTORIC BUZLUPINAR BRIDGE AND RECOMMENDATIONS FOR ITS RECONSTRUCTION

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September 2015, 171 pages

The Historic Buzlupınar Bridge is a timber pedestrian bridge, located in Buzlupınar Village, Rize, Turkey, which was constructed early in the 20th century and partially collapsed in 2008 at about 100 years of age. The wind loading was deemed responsible for the collapse of the bridge as stated by the locals. General Directorate of Highways has attributed special attention to Buzlupınar Bridge due to its rare value as a timber pedestrian bridge and has taken decision in 2012 to be rebuilt it again.

In this thesis, the original architectural and structural characteristics of the bridge were studied as well as its current condition and interaction with the environment. Structural analysis and evaluation of the bridge was carried out both with hand calculations and Finite Element Model to investigate its structural behaviour under live loads, wind loads, and earthquake loads. The FEM was formed with SAP2000 using frame, shell, and solid elements. The critical failure modes for overturning of the bridge were checked. Material tests were done on new timber members for the reconstruction studies to determine the mechanical properties of the timber elements. Controlled shear and tension tests were carried out using nailed connections which will be used in the reconstruction project. Structural proposals were made for the reconstruction project considering the original structural and architectural features of the bridge. Furthermore, stabilization, monitoring, and maintenance recommendations were made, which would be helpful to prolong the service life of the structure.

The investigation and reconstruction studies of the bridge are conducted by the General Directorate of Highways. The measured surveys, documentation of the bridge and reconstruction projects were made by Mukaddes Ataman (Bender Restoration). The proposals and recommendations made within the scope of this thesis might be helpful for the reconstruction project; however, provided as-is and does not burden the liability of structural design or restoration work.

Keywords: Buzlupınar Bridge, timber bridge, structural analysis, finite element modelling, conservation of historic bridges, reconstruction

ÖZ

TARİHİ BUZLUPINAR KÖPRÜSÜNÜN YAPISAL MODELLEME, ANALİZ VE DEĞERLENDİRME ÇALIŞMASI VE YENİDEN İNŞASI İÇİN ÖNERİLER

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Eylül 2015, 171 sayfa

Tarihi Buzlupınar Köprüsü Buzlupınar Köyü, Rize, Türkiye'de bulunan ahşap bir yaya köprüsüdür. Köprü 20.yy'ın başlarında inşa edilmiş ve 2008 yılında, yaklaşık 100 yaşındayken kısmen yıkılmıştır. Yöre halkı, köprünün rüzgâr yükleri etkisi ile yıkıldığını belirtmektedir. Ahşap yaya köprülerinin özgün bir örneği olması nedeniyle Karayolları Genel Müdürlüğü tarafından Buzlupınar Köprüsü'ne özel ilgi gösterilmiş ve 2012 yılında köprünün yeniden inşa edilmesine karar verilmiştir.

Bu tez kapsamında köprünün özgün mimari ve yapısal özellikleri, mevcut durumu ve çevresi ile olan ilişkisi incelenmiştir. El hesapları ve Sonlu Elemanlar Metodu ile oluşturulan analitik model ile köprünün canlı yükler, rüzgâr yükleri ve deprem yükleri etkisindeki davranışını incelenerek yapısal analiz ve değerlendirme çalışmaları yapılmıştır. Köprünün analitik modeli SAP2000 programı ile frame, shell ve solid elemanlar kullanılarak oluşturulmuştur. Kritik olan durumlar incelenerek köprünün devrilme tahkiki yapılmıştır. Köprünün yeniden inşa çalışmalarında kullanılacak olan ahşaplar üzerinde malzeme testleri yapılarak ahşap elemanların mekanik özellikleri belirlenmiştir. Yeniden inşa projesinde kullanılacak olan çivili bağlantılar üzerinde kesme ve çekme deneyleri yapılmıştır. Köprünün özgün yapısal ve mimari

özelliklerine bağlı kalınarak yeniden inşa projesi için öneriler yapılmıştır. Ek olarak, yeniden inşa sonrasında köprünün hizmet ömrünü uzatmak için uygulanabilecek izleme ve bakım önerilerinde bulunulmuştur.

Köprü ile ilgili inceleme çalışmaları ve yeniden inşası için yapılan çalışmalar Karayolları Genel Müdürlüğü kontrolünde yürütülmektedir. Köprüye ait projeler ve belgeleme çalışmaları Mukaddes Ataman (Bender Restorasyon) tarafından hazırlanmıştır. Bu tez kapsamında yapılan çalışmalar, öneri ve tavsiyeler yeniden inşa projesine katkı sağlayabilir, bununla birlikte sunulan öneriler ve tavsiyeler yapısal tasarım ve yeniden inşa çalışmalarında yazarlara ve kurumlara bir sorumluluk yüklemez.

Anahtar Kelimeler: Buzlupınar Köprüsü, ahşap köprü, yapısal analiz, sonlu eleman modelleme, tarihi köprülerin korunması, yeniden yapım

ACKNOWLEDGEMENTS

The author would like to express her sincere gratitude to her supervisor Prof. Dr. Ahmet Türer for his invaluable guidance, insight, encouragement and support throughout the research. It was a great opportunity to work with him.

The author would also like to express her gratefulness to her co-supervisor Prof. Dr. Neriman Şahin Güçhan for her advice, constructive criticism and encouragement in every phase of the study; and also for advising to study such an interesting subject.

The author would like to express her sincere thanks to Prof. Lucio Nobile for his supports and understanding.

The author would also like to thank to Halide Sert, Süheyla Yılmaz and the Historic Bridges Department of the General Directorate of Highways to give her the opportunity to study on this subject. Also, Mukaddes Ataman (Bender Restoration), Zülfikar Halifeoğlu and Ahmet Ali Kork are gratefully acknowledged for sharing the information, documentation, and studies about the bridge with the author.

Thanks also to Emrah Erdoğan for his support, sharing his knowledge and answering every question patiently. Thanks to Serap & John Adair and Nevin Çabuk for all their helps. And heartfelt thanks Feyyaz, Vahit, Apocan, Ezgi, Alican, Ali, Onur, Anıl Şen, Şengül, Anıl Poyraz for their understanding and invaluable friendship.

Lastly, the author wants to thank to her parents and brother Can for their endless love, support and understanding.

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CHAPTER 1

INTRODUCTION

1.1 The Importance of Historic Buildings and Their Maintenance as Cultural Heritage

The term cultural heritage includes the tangible and intangible assets which have survived from past to present; such as the structures, places, artefacts, and the values attributed to them by people. Additionally, it includes the interaction between these assets and people, and people's beliefs, customs, and traditions.

Humans have been building structures for thousands of years with various construction techniques, using different materials. These structures are the witnesses of the civilisations; they mirror the life styles of the people and inform us about the original and traditional construction techniques pertain to a particular time and place, technical developments and the characteristics of period that they were built. They are not important only for their ages; in addition, the interaction between people and structures is another issue what makes the structures valuable in the context of cultural heritage. The structures gain a seat in people's memories as long as people use them, a special interaction forms between the users and structures in time. Sometimes for economic reasons, sometimes to keep their memories alive and immaterial reasons, sometimes to satisfy their needs; for various purposes, people have tended to protect the structures and prolong their service life so the conservation and restoration of historical buildings have been an important issue for the mankind for many years; both to maintain their functionalities and protect them because of their cultural, historic, and symbolic values.

Since past century, national and international charters were published to guide about how to approach to the conservation issues. The first stage of conservation is thoroughly understood the structure and condition of the architectural heritage. Today, the historic structures can be analysed and evaluated properly with multidisciplinary approaches (D'ayala & Forsyth, 2007). In ICOMOS Principles for the Analysis, Conservation and Structural Restoration of Architectural Heritage (2003), the importance of multidisciplinary approach in conservation was emphasized too. To understand and evaluate a structure completely is only possible with a work done by a multidisciplinary team. The role of engineer here is to investigate the structural features and condition of the property and provide structural safety of the structure in the current situation and for the future use (D'ayala & Forsyth, 2007).

Conservation of historic structures is possible with continued use, maintenance and repairs. In the same document, ICOMOS Principles for the Analysis, Conservation and Structural Restoration of Architectural Heritage (2003), it is also stated that the interventions should be done with respect to the authenticity and integrity of the structure, and before any interventions the current condition of the structure should be examined and documented; the structural analysis, safety evaluation, the causes of the decay, and damage of the structure should be examined comprehensively.

Historic structures present some difficulties in diagnosis, analysis, and restoration due to their material characteristics and construction techniques, which make it difficult to evaluate them according to the modern structural codes. It is desirable to make recommendations and implementations both rational and confirmed with the historical context (ICOMOS, 2003).

1.2 Selected Case Study

In the recent times, the prototype construction techniques and materials replacing the traditional construction techniques and materials which are special for a particular place or time. Therefore, the artefacts built with the traditional construction techniques, craftsmanship, designs and materials are very precious historic documents, and these artefacts should be examined and documented thoroughly. As stated in 5th article of the Nara Document of Authenticity (1994) “The diversity of cultures and heritage in our world is an irreplaceable source of spiritual and intellectual richness for all humankind.” and to protect and enhance this diversity is crucial for the development of humanity.

All the repairs and interventions in historic buildings should be done regarding its authenticity, without damaging and making any alterations on the original architectural, structural and material features of the structure; the aim should be protect the existing form of it. Reconstruction of an architectural cultural heritage is only permissible if the structure is partially or completely destroyed and it takes part in the common memory of people and essential in terms of its contributions to the cultural environment and the reconstruction project should be based on the existing documentation about the structure; such as the ruins, original measured surveys, photographs, oral or visual sources (ICOMOS Türkiye, 2013).

In this study, the historic Buzlupınar Bridge which has collapsed in 2008, was investigated structurally and proposals were developed for its reconstruction project in regard to the conservation principles. After the collapse of the bridge its remaining elements have lost their strength in time and it became functionless. The materials tests on the remaining elements of the bridge showed that the timber elements have substantially lost their strength.

Buzlupınar Bridge is located in Buzlupınar Village, Çayeli, Rize. It exists more than 100 years as learned from the local villagers. It was completely built with wood, with a rare construction technique which is quite seldom in its region and in Turkey. Moreover, when the historic bridges in Turkey are considered, relatively small part of them are built with wood (Tarihi Köprüler, 2009, p.29). Therefore each of them is a very valuable historical document like the Buzlupınar Bridge. In addition to its age and documental value, as learned from the local villagers, the village in which the bridge is located was named after the Buzlupınar Bridge so the bridge has become significant for the local people in time.

Bridges not only span distances and connect two sides together but also they connect the lifes of the people to each other, they connect together the past and the future. Because they are used by different generations in time; they become a part of the common memory of the public and witness to their lifes.

In the case of Buzlupınar Bridge, most of the locals are not pleased to see their bridge collapsed because it has traces from their past, their childhood and/or youth. They want the bridge to be known and seen by their children since it gave its name to the village. Consequently, the bridge has a meaning for the locals to become usable again. Moreover, there is interaction between the bridge and the landscape. It was constructed with wood which is the most common construction material in that region and probably constructed by the local craftsmen; it typifies authentic characteristics and limits of the landscape.

Due to the above mentioned reasons, responsible authorities decided to rebuild the bridge using its original construction technique and structural system. Then original construction technique and the structural system of the bridge were examined from remaining part of the bridge, the measured surveys and old photographs; aiming to use the original remaining materials if it is possible. The studies focused on the investigation of the structural system of the bridge, determining the causes of the collapse, modelling, structural analyses of the bridge, and making proposals for the reconstruction project.

The measured surveys, material tests on the remaining elements of the bridge, and documentation about the bridge were made in 2012 by the General Directorate of Highways and these were used by the author in structural analysis studies and in defining the architectural and structural features of the bridge. It was resolved to change the original remaining timber elements of the bridge and rebuilt it by General Directorate of Highways, since the results of the material tests done on the remaining timbers showed that the remaining timber elements have already lost their strength and were not able to carry the loads act on it safely. In section 3.3 the information about the material tests and mechanical properties of the wood are given in details. In 2014, in order to determine the original structural features of the bridge, the existing elements of the bridge were disassembled; the documentation about the dismantling process was also obtained from the General Directorate of Highways and used for the studies done within the scope of this thesis.

1.3 Methodology

The present study was conducted in 4 stages; first is the field survey and interviews with the locals, second is the literature search and examination of the related documents and photographs, third is the structural investigation of the structure including material tests, fourth is the hand calculations and analytical modelling, and recommendations for the reconstruction, consolidation, monitoring and maintenance of the bridge.

The field survey was done in October, 2013. The aim of the field survey was to investigate the structural condition of the bridge, determine and understand its architectural and structural features. For this purpose, the connections between the existing structural elements and formation of the structural geometry of the bridge were investigated in detail. The biological formations on the existing wooden elements were examined visually; photographical documentation was done and simple sketches of the structural details were made in the scope of the field survey. In addition, the relationship of the bridge with its surrounding area was also investigated and interviews were made with the local people, inhabitants of Buzlupınar Village, in order to learn the history of the bridge and their opinions about the bridge.

Unfortunately, there are very few written sources about the Buzlupınar Bridge and most of them contain of the same information gathered from the oral interviews during the field survey done by the author. Therefore the literature research was mostly made about the history and classification of bridges in general and similar bridge examples from Turkey and the world. In addition to these, analytical modelling of historical structures and the testing procedures for timber and nails were also studied.

Chapter 1 presents the introduction of the thesis. In section 1.1, the purpose of the conservation of historical structures is clarified briefly. The conservation principles followed in this study and the selected case study are introduced in section 1.2.

Chapter 2 focuses on bridges in general and timber bridges in particular, and conservation of historic bridges. A brief description of the historical development and classification of the bridges are made in section 2.1. Wooden bridge examples from Turkey and the world, similar with the Buzlupınar Bridge are given in section 2.2. Section 2.3 mentions about the conservation of historic bridges and conservation approaches.

In Chapter 3, the case study is presented. General information about the location and history of Buzlupınar Bridge is given in section 3.1. In section 3.2 and section 3.3, the structural and architectural features, structural condition and material properties of the Buzlupınar Bridge is described with the help of the measured surveys and unpublished reports which have prepared by the General Directorate of Highways. The original architectural and structural features of the bridge are studied carefully; the construction stages and assembly details of the original structure are described and illustrated with simple sketches. Section 3.4 mentions about the disassemble process of the cantilever beams of the bridge, and the reconstruction project of the Buzlupınar Bridge which have prepared by the General Directorate of Highways.

Chapter 4 focuses on the structural investigation of the bridge based on the dimensions given in the restitution stage and in the restoration project. The conservation approaches defined for the case of Buzlupınar Bridge are defined in section 4.1. Also, brief information and examples are given about the use of analytical modelling in structural investigation of historical timber structures. The structural analysis studies were done in accordance with the building codes such as TS 498: Design Loads for Buildings and Eurocode 5: Design of Timber Structures. In Section 4.2, the structural investigation begins with hand calculations, which were done in order to check if the dimensions given in the restoration project and the material properties satisfy the safety requirements under different loading conditions; which are the live loads and wind loads. The hand calculations were focused on the mains beams, cantilever beams and effect of wind forces on the bridge; the hand calculations were also helpful to determine the weak parts of the structure.

In the scope of this study, material tests were done to determine the mechanical properties of the timber and nails that will be used on the reconstruction project. The results of the tests are given in section 4.3. The material parameters gathered from these tests were used in the analytical model of the bridge.

The finite element model of the bridge was formed with SAP2000, using frame, shell and solid elements. General information about the formation of the model geometry, the mechanical properties used on modelling and loading conditions are presented in details in section 4.4. Since the mechanical properties of the ground were not known clearly, two different models were formed; in one of them the ground was modelled with solid elements, and in the other one was modelled with rigid supports. The same loading conditions with the hand calculations were assigned to both models as well as the seismic loads. Several analysis were performed under live loads, wind loads and earthquake loads and the critical connections of the bridge were examined using the outputs of these analysis.

Structural recommendations for the reconstruction project are noted in different sections of this study where it was necessary in compliance with the analysis results. In chapter 0, these recommendations are summarized and some technical points are pointed out which have to be respected during the reconstruction process. Monitoring and maintenance recommendations are the other subjects mentioned in this chapter, which are necessary to prolong the service life of the bridge.

CHAPTER 2

BRIDGES AND CONSERVATION OF HISTORIC BRIDGES

In this chapter, the historical development of bridge construction and structural types of bridges, construction materials and usages are described briefly. Afterwards, a short classification is made for timber bridges and similar timber bridge examples with Buzlupınar Bridge are given from Turkey and world. Conservation approach and principles for historic bridges are described and defined.

2.1 Historical Development and Classification of Bridges

Bridges are the structures which have existed since the beginning of human civilization. They support the social and cultural development of settlements, towns, and cities since ancient times. Charles Whitney defines bridges as follows; "They span obstructions in his path and open new routes of communication." (Whitney, 2003). Bridges are the substantial examples of the human genius which combines art, beauty and structural efficiency. Another explanation for bridges from Italian architect Palladio is as follows:

> "The convenience of bridges was first thought upon because many rivers are not fordable by reasons of their largeness, depth, and rapidity: upon which account which may be well said, that bridges are a principal part of the way; and are nothing else but a street or way continued over water. Bridges are therefore ought to have the self-same qualifications that are judged requisite in all other fabrics: which are that they shall be convenient, beautiful, and durable." (Whitney, 2003, p.27)

For many centuries, countless of bridges have been built. It is not possible to know when the first bridge was built and used but it is certain that, the first bridges shaped as a result of the needs of humankind and their desire to dominate the nature. At the beginning they might have been used only for crossing over the narrow rivers or canyons. With the changing needs and developments in technology, now the modern bridges cross over wider rivers, deeper valleys, even they connects cities, countries, and continents (Denison & Stewart, 2012; Whitney, 2003).

Bridges had a great progress throughout the history. They have been built in different structural types with various materials such as stone, timber, rope, bamboo, kiln fired brick, iron. Natural stone arches or fallen tree trunks were the models for the earliest bridge builders and probably, the first materials used for the bridge construction were the natural stones, branches, tree trunks, and vine. Together with the development of tools and inventions of new materials, the construction materials were shaped in the required sizes, wider distances could have been spanned and different types of bridges have been constructed (Brown, 2001).

Table 2.1 Classification of bridges according to construction materials, structural types and usages (Denison & Stewart, 2012)

Materials	Structural Types	Usages
• Wood • Stone • Organic materials: rope, bamboo, root, vine • Brick • Iron • Steel • Concrete • Glass	• Beam bridges • Arch bridges • Truss bridges • Cantilever bridges • Suspension bridges • Cable-stayed bridges • Hybrid bridges	• Pedestrian • Aqueducts • Vehicular • Rail • Military

Whether the primitive bridges or the modern ones, bridges are subjected to the same kind of loads; *dead loads* arising from the own weight of the structure, *live loads* arising from the traffic passing over it, and the environmental loads such as wind and snow. The forces, caused by these loads, acting on the structures are tension, compression, shear, and bending forces, and the ability of a material to withstand these forces is the strength of that material (Brown, 2001). These forces can act singly or combined on a structure and produce different effects on the structure depending on its materials and structural type. Like in all structures, to use the compatible materials in the convenient structural technique is one of the most important points in bridge building to build more efficient and safer structures.

Fallen tree trucks or a stone across a river are the first and primitive examples of the beam bridges which can be simply defined as a horizontal beam supported at each ends (Brown, 2001). The vertical loads over it generate shear, tension and compression forces as shown in Figure 2.1. The horizontal tension and compression forces balance each other and the vertical shear force is shared by the piers at both ends.

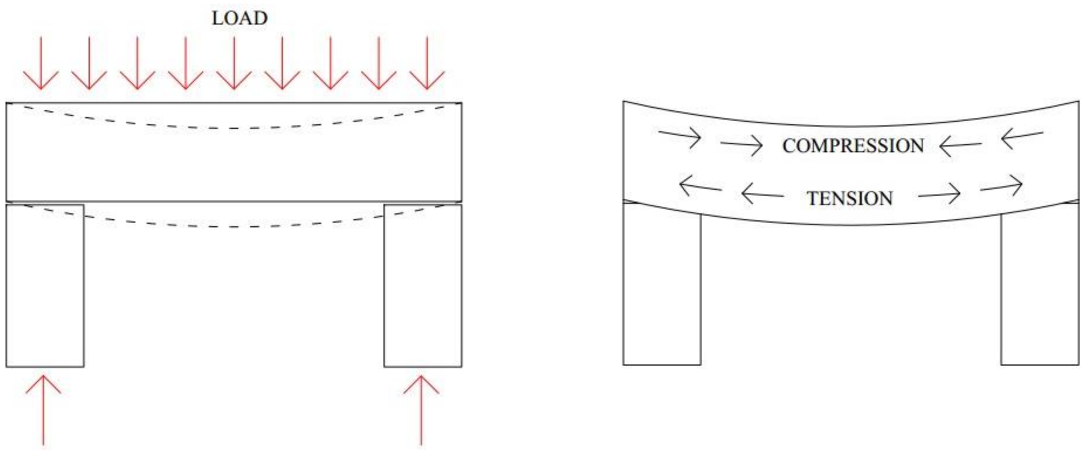


Figure 2.1 Forces acting on a beam bridge (drawn by Ezgi Çabuk)

Primarily, wood and stone was used to build this type of bridge whereas with the developments in material science and technology, the modern ones are built with iron, steel, and reinforced concrete.

The arch bridge was probably arised from the attempt of human beings to replicate the natural arches which is a strong structural form. This structural type was widely used all over the world in bridge design from the ancient times until the industrial revolution. In an arch bridge the vertical loads generate compression force which is transferred to the supports at both ends, called abutment, and then to the ground. Vertical and horizontal reactions occur on the abutments and these forces prevent the motion of the structure. The stone which is placed at the top point of the arch is called keystone and it is responsible to transfer the vertical forces into lateral forces through the arch (Denison & Stewart, 2012).

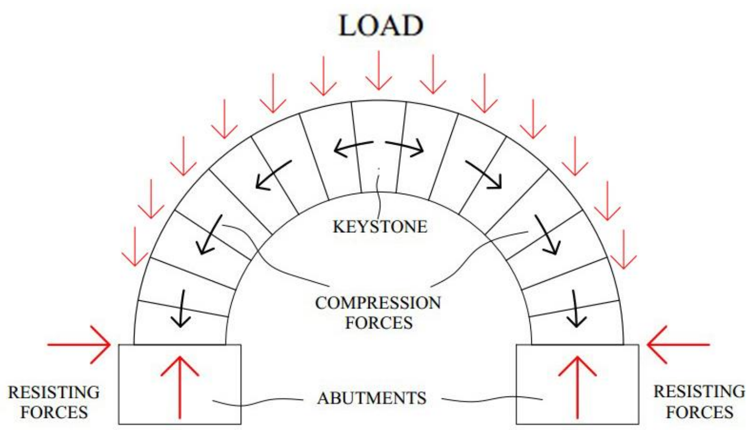


Figure 2.2 Forces acting on an arch bridge (drawn by Ezgi Çabuk)

The first examples of arch bridges were built with stone, which has high compressive strength so works well in compression. In time examples were built also with wood, brick, iron, steel, and prestressed concrete. (Denison & Stewart, 2012)

Building arch bridges was the practical way for spanning large distances for many years. However there were many structural types to discover yet. In the 16 century, Italian architect Palladio described the truss bridge in his *I Quattro Libri dell' Architettura*. Truss is the structural type which consist of triangles formed with the connected straight members. The straight members are subjected to both tension and compression forces and these forces are balanced in a truss. There are various types of trusses however they all have the advantage of the strength and a rigidity of a triangle (Brown, 2001).

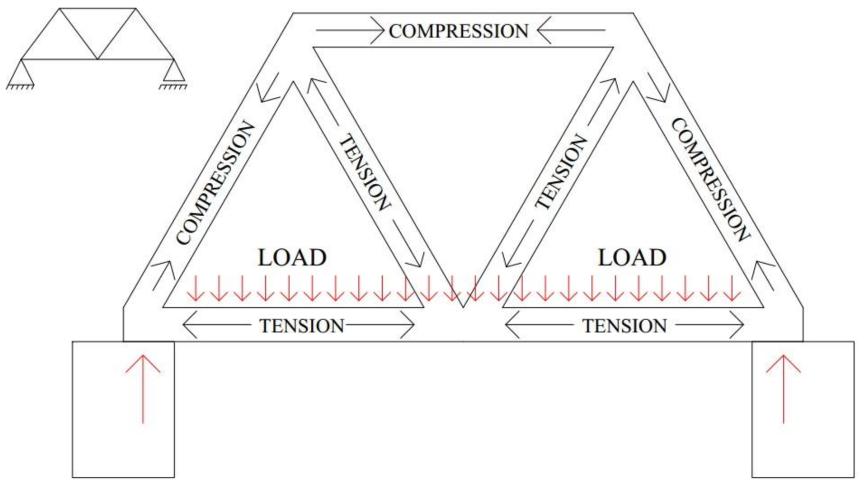


Figure 2.3 Forces acting on a truss bridge (drawn by Ezgi Çabuk)

The first examples of truss bridges were constructed with wood. With the industrial revolution, the iron and steel replaced the wood in 19^{th} century.

Another bridge type is the cantilever bridge which is a developed form of beam bridges to span wider distances (Brown, 2001). Cantilever is a beam fixed one end and the other end is free. In the simplest form, the cantilever type of bridge consists of a beam placed on the free ends of cantilever beams those placed oppositely.

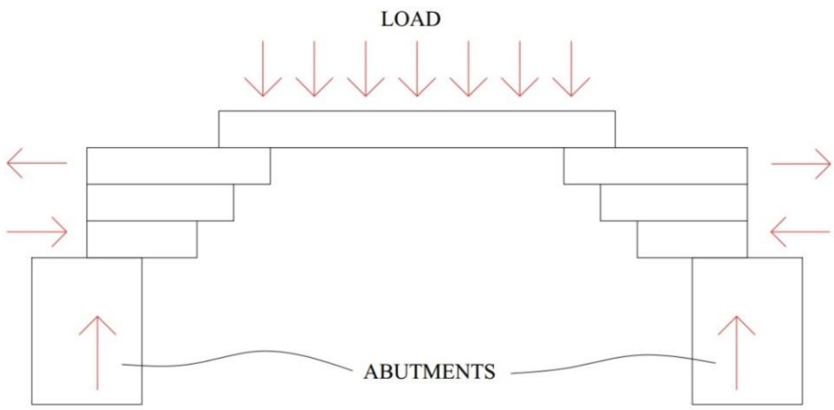


Figure 2.4 Simplest form of a cantilever bridge and forces acting on it (drawn by Ezgi Çabuk)

In addition to the primitive examples for cantilever bridges which were built with stone and wood, in 19th century more complex iron and steel cantilever bridges were designed and build in many different types generally with more than one span.

Beam bridges and arch bridges are not the only bridge types which have existed for long time. Suspension bridges also date back to the early times. Basically, a suspension bridge is formed with a rope hanging between the supports (Brown, 2001). For the last two century the suspension bridge design has showed a great progress thanks to the developments in material science and construction technology. The modern suspension bridges are based on the strength of suspension cables which are hanging over the towers called pylon and are anchored to the ground at the ends, and the deck of the bridge is hanged on this suspension cables with the vertical cables called hangers (Denison & Stewart, 2012).

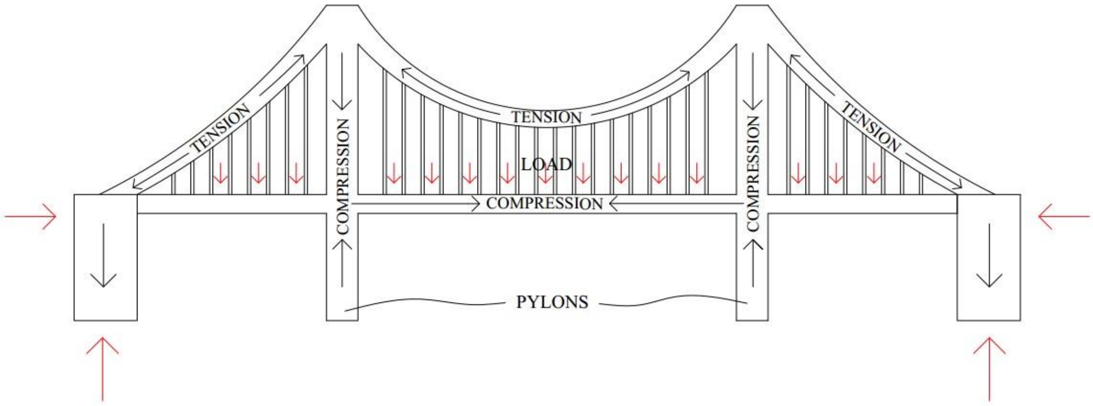


Figure 2.5 Forces acting on a suspension bridge (drawn by Ezgi Çabuk)

In the early examples of the modern suspension bridges the suspending cables were made from iron chains and wire cables, it is followed by the use of steel cables at the end of the 19th century (Denison & Stewart, 2012).

Not long time ago, at the end of the 19th century, the cable stayed bridges showed up. Although appearing similar to the suspension bridges, they have quite different structural mechanism. In cable stayed bridge, the deck is supported by the iron or steel cable hanging over a tower. This structural type can be used for building small bridges or span large distances and it can be built repeatedly throughout the span (Denison & Stewart, 2012).

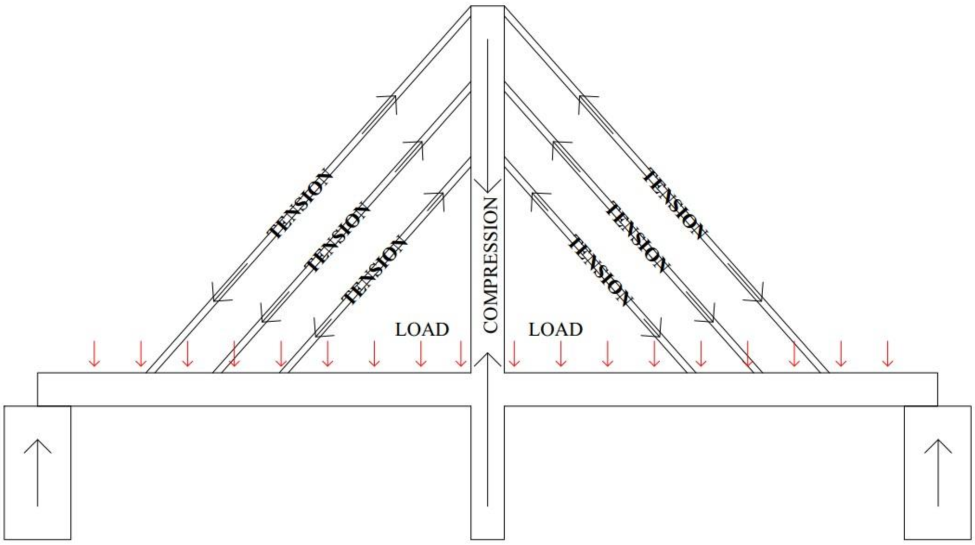


Figure 2.6 Forces acting on a cable-stayed bridge (drawn by Ezgi Çabuk)

In reality, most of the bridges do not have a single structural system; many of them were built as a combination of two or more structural types, or built in a single structural type then integrated with another structural type to meet the changing conditions which have occurred throughout the time. This type of bridges is named as *hybrid bridges* and can be constructed as many different combinations like arch and beam, arch and truss, cable stayed and beam (Denison & Stewart, 2012).

Whereas the main purpose of building a bridge is to span a distance, most of them were built for a special use such as pedestrian bridges, aqueducts (which are built to carry the water), vehicular bridges, rail bridges and military bridges. Pedestrian bridges and aqueducts have been built from the ancient times to modern days. In 18th and 19th centuries, vehicular bridges and rail bridges designed with increase in usage of cars and trains, and countless examples have been built until today. In addition to these, for thousands of years bridges were used for military purposes. Military bridges are transportable and can be constructed and deconstructed easily.

When Buzlupınar Bridge is considered, it can be classified as a combination of a beam bridge and cantilever bridge. The cantilever beams of the Buzlupınar Bridge behaves similar as in the cantilever bridge form and the main beams of it are simply supported by the cantilever beams, so the mid span of the bridge behaves similar as a beam bridge.

2.2 Structural Classification of Timber Bridges and Examples from Turkey and the World

Timber has been widely used as construction material from the first examples of bridge building to the modern designs. There are several advantages of timber as a construction material. It has a high strength to weight ratio; it is natural, renewable and sustainable, has low embodied energy content during manufacture; and with regular surface treatments and protection, longer service life can be ensured easily. It is also ideal for the applications where aesthetics and beauty is important (Mettem, 2011).

Since the primitive ones built with tree trunks to the modern examples built with industrial timber, timber bridges have been built in several structural types; beam, arch, truss, cantilever, suspended, and in hybrid form such as trussed arches. Timber bridges are generally built for pedestrian, animals, cyclists or light vehicles; however with the technological developments they become suitable for relatively higher loads (Mettem, 2011).

In this part of the study, comprehensive study has been made; examples of timber bridges are given from Turkey and the world which have similar architectural and structural features with Buzlupınar Bridge in order to better understand its structural and architectural features.

1st example from Turkey is the Hapsiyaş Bridge which is also known as Kiremitli Bridge shown in Figure 2.7. It is located in Trabzon, approximately 55 km away from the Buzlupınar Bridge. Hapsiyaş Bridge was built in 1935 and restored in 2002. It is a covered bridge; the cover is formed with timber posts and roof placed on top of the posts. There are cantilever beams at each side of the bridge located on stone masonry abutments and the main beams are supported by these cantilever beams.

Figure 2.7 Hapsiyaş Bridge[1]

Other examples from Turkey are Yurtpınar Bridge, Çaylı Güvem Bridge, Taşlıgedik Bridge, and Bayramören Bridge.

Yurtpınar Bridge is located in Yurtpınar Village, Bayramören, Çankırı; was built in 19th century; which is shown in Figure 2.8. It is a covered bridge with cantilever beams placed on masonry abutments, the gaps between the cantilever beams are filled with stones and main beams are supported by these cantilever beams. The cover is formed with posts, horizontal timber beams which connect the posts together and the roof. There are timber bracings which connect the posts and deck piles, and prevent the transversal motion of the bridge, which can be said to work in similar fashion with the 'L' shaped bracings in Buzlupınar Bridge.

[1] retrieved from: http://myu.blogcu.com/zamana-direnen-kopru-hapsiyas-koprusukiremitli-kopru-of-cay/6104851 , 17 March 2015

Figure 2.8 Yurtpınar Bridge[2]

Çaylı Güvem Bridge is located in Çaylı Village, Çerkeş, Çankırı; shown in Figure 2.9. It was built in late Ottoman Period. It is also a covered bridge; the cover is formed with timber posts, horizontal timber beams between them and roof on top. There are timber bracings which connect the posts and deck piles and prevent the transversal motion of the bridge.

[2] retrieved from: http://www.kulturportali.gov.tr/turkiye/cankiri/kulturenvanteri/yurtpinarkoyu-koprusu, 17 March 2015)

Figure 2.9 Çaylı Güvem Bridge[3]

Taşlıdegik Bridge is located in Çaykara, Trabzon. As shown in Figure 2.10, it is a covered bridge; the cover is formed with timber posts, transversal elements, and roof on top of them.

Figure 2.10 Taşlagedik Bridge[4]

[3] retrieved from: http://www.cankiri.gov.tr/index.php/post/view?id=3314, 17 Mart 2015

[4] retrieved from: http://www.tasligedik.net/site/news.php?readmore=937, 17 March 2015

The last example from Turkey is the Bayramören Bridge, located in Bayramören, Çankırı, Turkey. The bridge was built in 19^{th} century. The abutments and piers of the bridge are formed with overlapping cantilever beams located horizontally with right angles on top of the stone masonry walls as shown in Figure 2.11. It is covered bridge and the cover is formed with posts, horizontal timber beams which connects the posts together, and roof on the top. There are transversal timber elements located between the posts and deck piles to prevent the longitudinal and transversal motion of the bridge.

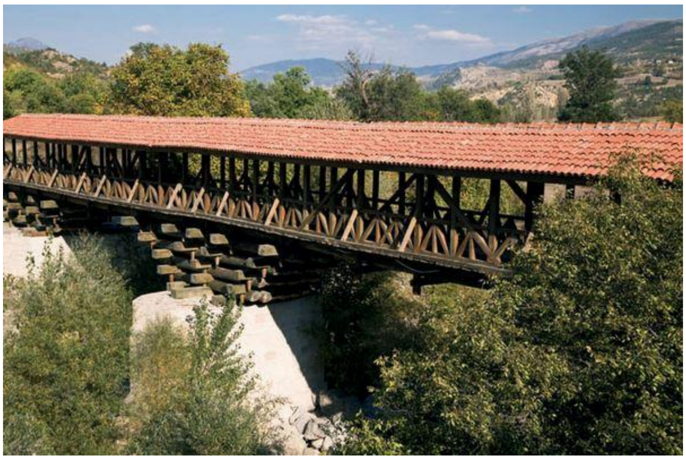


Figure 2.11 Bayramören Bridge[5]

The given examples were helpful to see the construction techniques and architectural details of the timber covered bridges in Turkey which are similar with Buzlupınar Bridge and to determine the common structural and architectural features. In the given examples, the main beams are supported by cantilever beams or bracings and in all of them the longitudinal or transversal movement of cover is limited by using bracings as in the Buzlupınar Bridge.

[5] retrieved from: http://arsiv.kesfetmekicinbak.com/atlaskitap/kitapdetay.aspx?kitapid=46&parentid=16, 17 March 2015

The examples from other countries are located in China, Nepal, Bhutan, and Papua New Guinea. Unfortunately, detailed information about their names, history, and location could not be reached.

First example is the bridge in Papua New Guinea; the main beams of this bridge are simply supported by the cantilever beams which are located at both ends as shown in Figure 2.12-a. There are horizontal beams which is perpendicularly located between the cantilever beams.

The bridges in Tingri, Tibet China and in Nepal are formed with cantilever beams located on the masonry stone abutments at both end. As shown in Figure 2.12-b, Figure 2.12-c, and Figure 2.12-d the main beams are supported by the cantilever beams. One of the bridges in Nepal is in Dudh Kosi River and the other is on the Gunsa River. The main beams of the bridges shown in Figure 2.12-a, Figure 2.12-b, and Figure 2.12-c are simply supported by the cantilever beams at each end as in the Buzlupnar Bridge. In Figure 2.12-d, the main beams are extending from one side of the span to the other side; supported by the cantilever beams at two points.

There are three examples from Bhutan shown in Figure 2.12-e, Figure 2.12-f, Figure 2.12-g; two of them are in Punakha and the other is in Paro district. All of them are covered bridges; the cover is formed with timber posts, horizontal beams between them, and roof piles on top. In first example from Bhutan, the main beam is supported by the cantilever beams which are anchored in stone masonry abutments. In the second and third examples from Bhutan, the cantilever beams which are placed on the stone masonry abutments support the main beams. The buildings over the cantilever beams at both sides prevent the overturning of the cantilever beams.


a
b
c
d
e
f
g
h

Figure 2.12 a) Papua New Guinea[6], b)Tingri, Tibet[7], c) Dudh Kosi River, Nepal[8], d) Gunsa Bridge, Nepal[9], e)Punakha, Butan[10], f)Paro, Bhutan[11], g)Punakha Bhutan[12], h) Çengyan Bridge, China[13]

The last example is the Chengyang Bridge in China, which was built in 1912. As shown in Figure 2.12-h, this is also a covered bridge; there are overlapping cantilever beams supporting the main beams, located perpendicular to each other placed on the stone masonry abutments and piers.

By the help of these examples it was seen there are similar bridge examples with the Buzlupınar Bridge in the world, built with traditional construction techniques. There must be multiple other examples in Turkey and the world which could not be mentioned in this study.

2.3 Conservation of Historic Bridges

Historic bridges draw interests and contribute to the characteristics of the natural landscape. They are valuable examples for cultural heritage but unlike the ruins or the artefacts exhibited in museums they should sustain their functionalities. Over the

[6] http://www.panoramio.com/photo/73986904, 18 March 2015

[7] http://en.wikipedia.org/wiki/Log_bridge#/media/File:Tibetan_log_bridge.JPG, 18 March 2015

[8] http://www.fotolibra.com/gallery/393117/new-bridge-dudh-kosiriver, 18 March 2015

[9] http://www.tibetrelieffund.co.uk/gunsa-bridge/, 18 March 2015

[10] http://www.panoramio.com/photo/77237300, 18 March 2015

[11] https://www.windhorsetours.com/wind/images/gallery/bhutan/sights_and_places/paro/large/Paro_Dzong.jpg, 18 March 2015

[12] http://nordenadventures.com/images/all/punakha.jpg, 18 March 2015

[13] http://en.wikipedia.org/wiki/Chengyang_Bridge, 18 March 2015

years, it has begun to be preferred to conserve and repair the old bridges than demolishing and rebuilding them (Tilly, 2002).

In “Conservation of Bridges” by Graham Tilly, the advantages are defined as follows; to conserve an old bridge has numerous advantages than building a new one, it is generally less expensive than the total cost of demolishing and rebuilding the bridge, it has understood most of the historic bridges are structurally stronger than previously thought; the stone arch bridges could perform for thousands of years without requiring a significant maintenance or repair. With conserving the existing bridges instead of building new ones the consumption of raw materials is also prevented; less damage is given to the natural sources. In addition, the historic bridges have people’s interest; sometimes they have emotional connections with people, they witness to the time and show traces from a specific period in the past (Tilly, 2002).

In the “Context of World Heritage Bridges” by Eric DeLony, it is stated the authenticity of a bridge includes the spirit and character of it as well as its construction technique, material and technical properties. To define a bridge as a cultural heritage it must be authentic and meet one or more of the following criteria;

- It is unique in terms of design, materials and craftmenship,

- It has influenced the engineering theory, transportaion, communucation, technology and construction within an area,

- It is typical example in terms of engineering and technology specific to a particular period or a stage in bridge construction (Delony, 1996).

The Mehmet Pasa Sokolovic Bridge in Visegrad, Bosnia and Herzegovina is one of the bridges in UNESCO World Heritage List. The bridge was constructed at the end of 16th century. It has a universal historic value; it represents a significant stage in civil engineering and bridge architecture, it has witnessed to the interaction between different cultures and ethnicities for centuries, and it is an inseparable part of the landscape. The bridge was damaged in a flood in 1896; in addition further damages took place during the World Wars, it was reconstructed in 1950s after some temporary repairs (UNESCO World Heritage Centre, 2007).

Another example is the Old Mostar Bridge, Stari Most; which is also located in Bosnia and Herzegovina and in UNESCO World Heritage List. The bridge was constructed in 16th century. It is an extraordinary example of bridge construction and technology and a valuable part of the cultural landscape as well as being a symbol of coexistence of people from different cultures, ethnicities and religions. In 1990 during the civil war, the bridge was destroyed. In 2004, the bridge was reconstructed based on the original and authentic documents and multiple analysis with authentic materials and the ruins of the original bridge are exhibited in a museum (UNESCO World Heritage Centre, 2005).

The conservation approach mentioned in section 1.1 should be applied to conservation of historic bridges certainly. On the conservation process, the most important thing is to well understand the existing features of the structure such as the defects, decays, load transfer mechanism of the structure and examine the structural details such as joist and connection details of the structural elements. (Hume, 2007).

The aim of conservation should be to protect the current condition of the structure if it is not in danger and there is no need to reinforcement. The behaviour of the structure should be examined under the loading conditions that the structure is exposed or could be exposed in the future and unnecessary reinforcements should be avoided.

D’ayala and Forsyth says the structural performance a structure is defined by “structural appraisal” which comprises the load carrying capacity and stability of structural components, overall stability and serviceability of the structure. The structural appraisal studies should include the research and inspection of the existing documents related to the building, measurement and investigation of the structural elements, determination of the defects and decays on the structure, structural analysis, material tests, interview with locals in order to gathering information about the structure (D’ayala & Forsyth, 2007).

When these principles are applied for historic bridges, they can be summarized as follows; understand the structural and architectural features of the bridge with visual inspections and help of the documentation related to the bridge, determine the decays, past deformations on the structure and material properties and structural analysis. In addition, the meaning of the bridge for the local people and the value of the bridge within the landscape should be understood. The cultural landscape comprises the diversity of interactions between the property, people and nature. The protection and continued use of cultural landscape contributes the sustainability of the area, increases the value of the landscape. The bridges should be evaluated within the landscape in terms of craftsmanship, architecture, construction technique and materials.

CHAPTER 3

BUZLUPINAR BRIDGE

The aim of this chapter is to define Buzlupınar Bridge considering its location, history, structural and architectural features, material characteristics, and structural condition with the help of field surveys done by the author, measured survey studies of the bridge and documentation made by Mukaddes Ataman (Bender Restoration) then shared by the General Directorate of Highways. In addition, the factors affecting the structural condition of the bridge and material properties of the timber elements are mentioned briefly.

3.1 General Information about the Location and History of Buzlupınar Bridge

The historic Buzlupınar Bridge is on Madenli Stream in Buzlupınar Village, in Çayeli district of Rize province. It has the 41°00'25.2"N and 40°46'59.0"E geographical coordinates and it is approximately on the 11.5th km of the Çayeli-Kaptanpaşa road, 11 km away from the coastline as shown in Figure 3.1.

Although the exact date of its construction is not clear, it is known that the transportation between the Buzlupınar Village and the main road was provided over the Buzlupınar Bridge, therefore the bridge is expected to exist since the beginning of the settlement in Buzlupınar Village. The bridge is approximately 2.20 m in width and it has 21.80 m span length between its abutments, and it was used for pedestrian, animal and load transportation.

Figure 3.1 Location of Buzlupınar Bridge at Eastern Black Sea Region (Google earth 7.1.2.2041, accessed on 22 December 2014)

Figure 3.2 Location of Buzlupınar Bridge and its surrounding (Google Google earth 7.1.2.2041, accessed on 22 December 2014)

The bridge was burned out during a fire in 1906 as learned from the local villagers. After the fire, it was reconstructed by the inhabitants but it is not known certainly if this bridge is the replica of the former one or some of the elements and materials of the former bridge was used in this reconstruction.

In 1906, the bridge was entirely built with chestnut tree wood elements which are nailed to each other with wrought iron nails; and stones were used on abutments. As learned from the statements of a local villager, Ahmet Ali Kork (Personal communication, October, 2013), and seen on the photos gathered from him and Neriman Şahin Güçhan, the bridge had a timber cover in its original state until the year 1985. In 1985, the original timber roofing was decayed and it was replaced with sheet metal roofing by Eyüp Ensar Saral, a local villager from the neighbour Yeşiltepe Village (A.A. Kork, personal communication, October, 2013). The old photographs of the bridge taken in 1970s, 1980s and 1990s are shown in Figure 3.3, Figure 3.4, and Figure 3.5.

Figure 3.3 Buzlupınar Bridge in 1970s (A.A Kork archive)

Figure 3.4 Buzlupınar Bridge in 1980s, with the sheet metal roofing (A.A Kork archive)

Figure 3.5 Buzlupınar Bridge in 1990s (A.A Kork archive)

In 2003 a reinforced concrete bridge known as Istoponos Bridge by the locals was opened to use next to the Buzlupınar Bridge, which is also used by vehicles (A.A. Kork, personal communication, 24 December, 2014). It can be said that after that the Buzlupınar Bridge has lost its function in the course of time.

As reported by the villagers, in 1998 two stone quarries started to operate on the valley in which the Buzlupınar Bridge was located. One of the quarries is known as Seslidere stone quarry, approximately on the 10th km of the Çayeli-Kaptanpaşa road, about 1 km away from the bridge and the other is known as Buzlupınar stone quarry which is approximately on the 12,3th km of the Çayeli-Kaptanpaşa road and about 0,5 km away from the bridge (Figure 3.2). The activities of these quarries have led to changes in the natural environment of the valley and the bridge was exposed to heavier wind loads than ordinary as alleged by the locals. However, there is not certain information or research done to prove this. As reported by the locals with the effect of the wind load, the covering and one of the main beams fell down into the river on the downstream side on 23 March 2008. When the meteorological data was investigated, it was determined the wind load was blowing from west-south-west with a velocity of 4,5m/sec[14] on that day; which would cause the bridge to collapse towards downstream.

Hereby it must be pointed out the collapse of the bridge was probably resulted from its prolonged exposure to the high wind forces, not an instant wind force. As for the meteorological data gathered from the stations in Rize and Pazar, the dominant wind directions between the years 2003 and 2013 were WNW and S respectively. The condition of the bridge right after the collapse is shown in Figure 3.6. Most of the timber elements which fell down into the river went adrift and got lost. Some of the roof covering sheets and the main beam which fell down were taken out of the river by the inhabitants but after a while the main beam faded away. It must be pointed out

[14] Meteorological data of the speed and direction of the wind gauged at the stations in Rize (station number: 17040) and Pazar (station number: 17628). The data is provided by the Turkish State Meteorological Service.

here, the strength of the timber changes in time, effected by the humidity and changes in climate conditions. In case of Buzlupınar Bridge, when the bridge was collapsed in 2008, over 100 years of lifetime, probably the strength of the timber was decreased and the timber elements were deteriorated; these factors were reduced the resistance of the bridge against wind loads. So the wind forces are not only responsible for the collapse of the bridge; the age of the bridge must be taken into account too.

Figure 3.6 Buzlupınar Bridge after the Destruction (A.A. Kork, 2008)

After this incident in 2008, only two main beams and some of the timber deck elements of the bridge stayed in their original places. Following the collapse of the covering, the remaining timber elements of the bridge were directly exposed to rain, snow, sunlight and other atmospheric conditions; naturally, these conditions accelerated the deterioration and degradation of the timber elements and consequently caused loss in strength.

After the destruction of the timber bridge on 23 March 2008, the headman of the Buzlupınar Village in that period made an application to the Rize Provincial

Directorate of Culture and Tourism to preservation and registration of the bridge as a cultural heritage on 07 April 2008. On 12 December 2008 the bridge was registered as a cultural heritage by the Trabzon Cultural and Natural Heritage Preservation Board. In the following years after the registration of the bridge, investigations studies have made by the General Directorate of Highways which is in charge of the maintenance and repair of historic bridges (Tarihi Köprüler, 2009). All the studies done or will be done about bridge are under the control of Trabzon Cultural and Natural Heritage Preservation Board.

The measured surveys of the bridge which have been prepared in 2012 and the other documentation about the bridge were shared with the author by the General Directorate of Highways in December 2013. The information about the original structural elements and details were gathered in January 2014, thanks to the photos gathered from Neriman Şahin Güçhan's archive (Figure 3.8, Figure 3.10, Figure 3.11, Figure 3.12, Figure 3.14, and Figure 3.15), which were taken by Neriman Şahin Güçhan during a trip to Eastern Black Sea Region in 2007, one year before the bridge has collapsed. These photographs were quite helpful to understand the original structural form of the bridge and connection details of the timber elements; thus the following studies were made considering the original structural features.

Figure 3.7 Buzlupınar Bridge in 2013 (E. Çabuk, October 2013)

In October 2013, when the first field survey to the bridge done by the author, the bridge was already in a derelict condition as shown in Figure 3.7; it was not able to function as there were only two main beams and a few elements of the deck standing, the cover was completely lost. As shown in Figure 3.8, biological deterioration was formed on the remaining timber elements and they were quite decayed by fungal and insect attacks.

(a)

(b)

(c)

(d)

(e)

(f)

Figure 3.8 (a), (b), (c), (d), (e), (f) biological formations and insect damage on the timber elements (E. Çabuk, October 2013)

3.2 Existing and Original Structural and Architectural Features of Buzlupınar Bridge

As previously mentioned, the first step of the conservation studies is to determine the current condition and define the structural and architectural features of the structure. The overall condition, load transfer mechanism, functions and details of the structural elements should be well understood before the structural analysis studies. In this section the existing features of the bridge and details of the structural elements are investigated. The construction technique of the bridge is defined with the help of the old photographs and measured surveys.

3.2.1 Existing Features

The Buzlupınar Bridge is formed with a simply supported mid-span which was positioned horizontally on the timber cantilever beams which slide on each other upwards with an angle. The dimensions of the bridge were approximately 2.20 m in width, approximately 21.80 m long total span between two abutments and 13.5 m long simply supported mid-span. The mid-span of the bridge comprised of 3 main beams. Their lengths changed between 14.35 m to 14.50 meters and their diameters changed between 21 cm to 25 cm along their lengths. The lengths of the main beams and the total span of the bridge are shown in Figure 3.9. The cantilever beams have squarely sections whereas the main beams have circular sections.

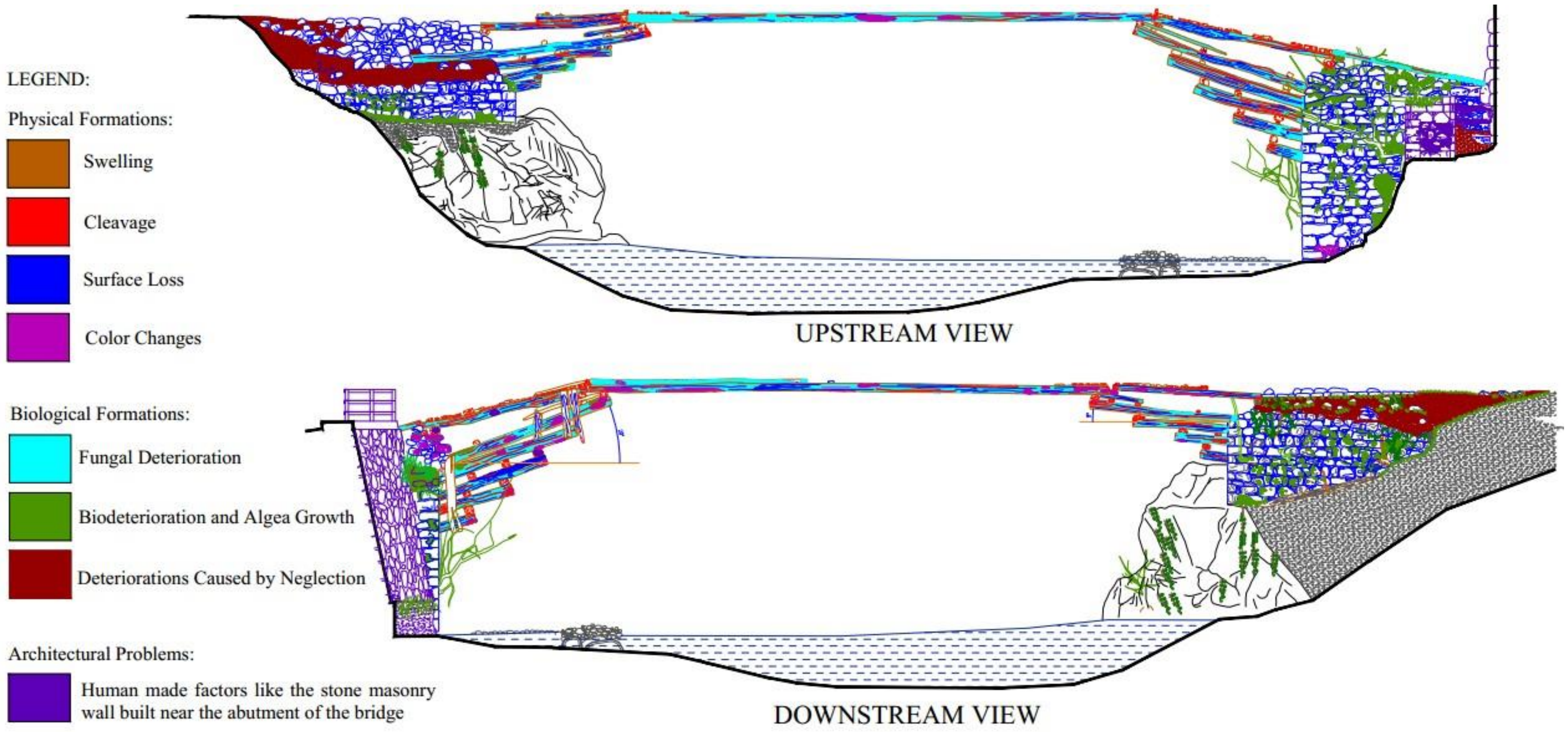


Figure 3.9 Measured surveys of the bridge made in 2012 (KGM, 2012, Ankara)

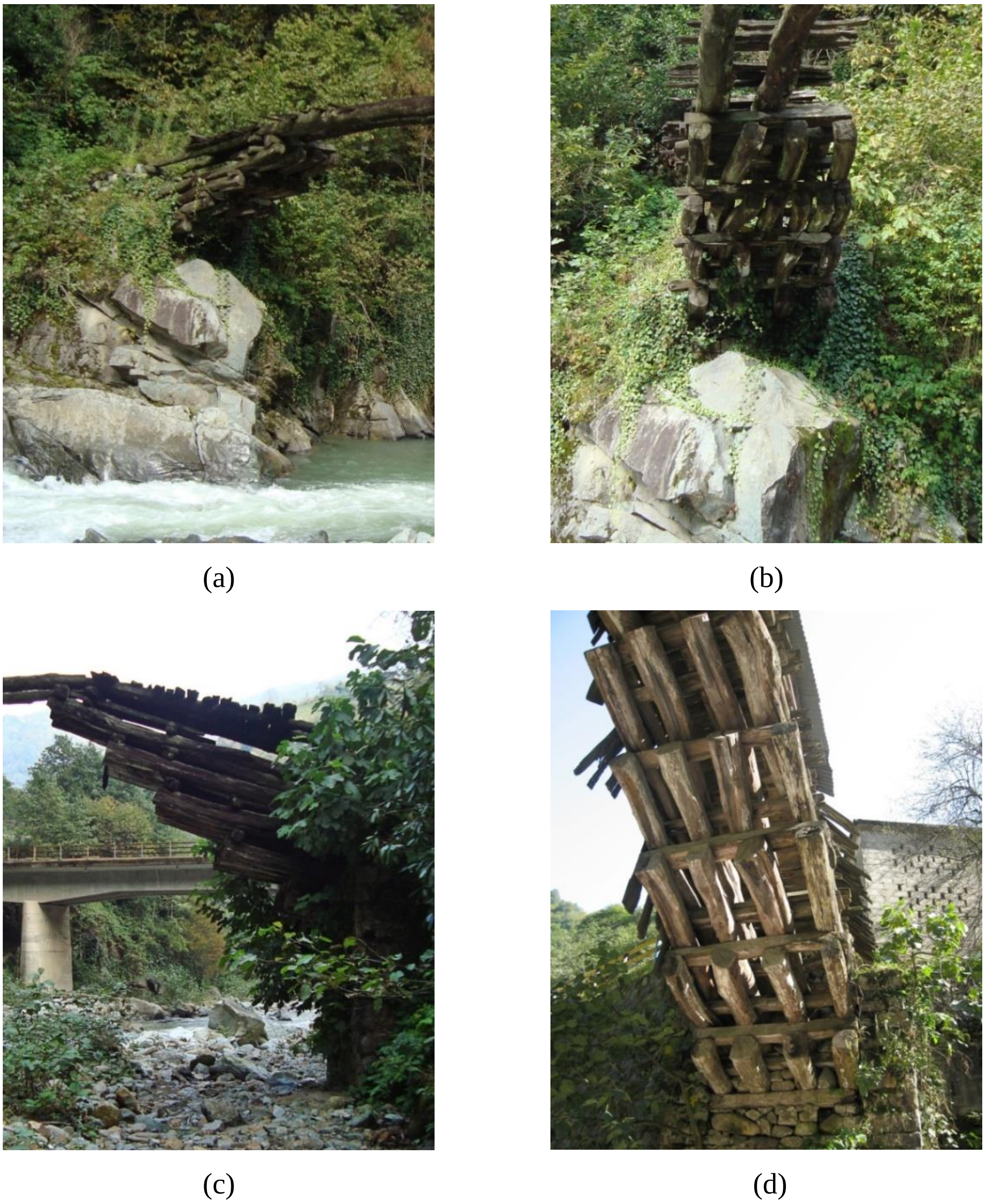


Figure 3.10. (a), (b) The cantilever beams placed on natural rock (E. Çabuk, October 2013) (c), (d) The cantilever beams placed on stone masonry wall. (E. Çabuk, October 2013; N. Şahin Güçhan, 2007)

The cantilever beams form the abutments of the bridge. One of the abutments is built on ground by piling stones, shown in Figure 3.10-c and Figure 3.10-d; and the other is on natural rock, shown in Figure 3.10-a and Figure 3.10-b. One of the cantilever beam groups is composed of 4 timber beams placed side by side in 5 rows at different elevations with horizontal angle of approximately 20^0 and the other is formed by 3 rows of 4 beams and 1 row of 7 beams at different elevations with a horizontal angle of approximately 5^0 . Perpendicular beams are also stacked between the longitudinal ones with the center filled with stones, thus the cantilever beams are stabilized. All the cantilever beams and horizontal beams between them are nailed to each other with wrought iron nails.

As it can be seen from the photos of the 2007 year, in its original state the bridge had a timber cover which was formed with timber posts and horizontal timber beams which links the timber posts horizontally across the bridge shown in Figure 3.11. L shaped timber bracings were linked between the timber posts and the deck of the bridge rigidly as so this L shaped bracings which is shown in Figure 3.11 and Figure 3.12 were transferring the moment from the posts to the deck and main beams of the bridge. Transversal bracings were also used to prevent the motion of the cover of the bridge on the longitudinal direction. The timber frame system was covered with an approximately right angled gable shaped timber roof.

Figure 3.11 Posts, L shaped elements and roof elements (N. Şahin Güçhan, 2007)

Figure 3.12 L shaped elements and transversal elements (N. Şahin Güçhan, 2007)

The connections of the timber elements were made by the wrought iron nails of various sizes and lengths up to approximately 60 cm; some of the posts, horizontal and transversal elements, and the roof elements were constructed as interlaced also nailed in several points to each other. The details of the connections are shown in Figure 3.13 and Figure 3.14.

Figure 3.13 Nailed connection details of the cantilever beams, decks and main beams. (Ezgi Çabuk, October 2013)

Figure 3.14 Connection deteails of the posts and roof elements (N. Şahin Güçhan, 2007)

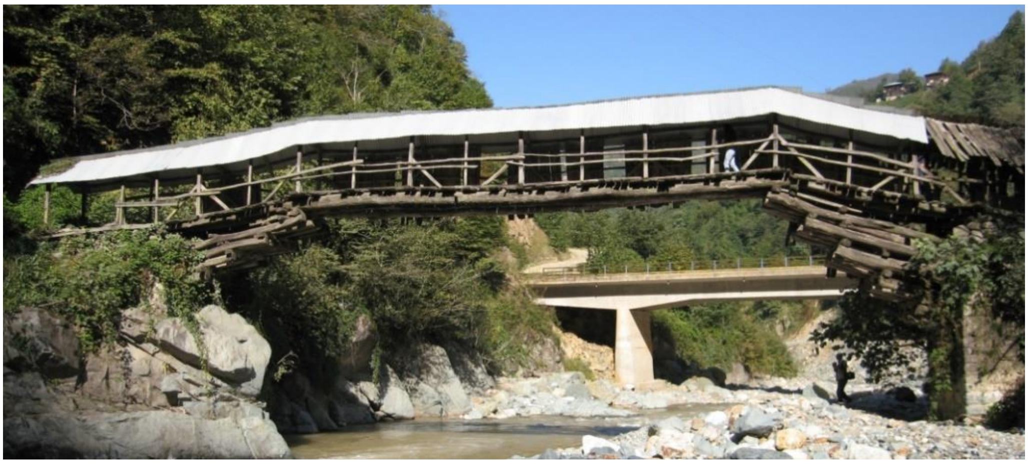


Figure 3.15 Upstream side view of Buzlupınar Bridge before the destruction (N. Şahin Güçhan, 2007)

Figure 3.16 Downstream side view of Buzlupınar Bridge before the destruction (N. Şahin Güçhan, 2007)

As seen on Figure 3.15 and Figure 3.16, the downstream side of bridge was covered with thin timber sheets whereas the upstream side was not. Although the function of these timber sheets is not clarified, it is estimated the covering was probably installed to prevent the wind that comes from downstream side.

3.2.2 Details of the Original Elements and Construction Technique of the Bridge

The bridge was completely built with wood materials. As it can be defined from the photos of 2007 year the bridge consisted of 17 types of elements in different shapes and dimensions. In this section, these elements are described separately and named from A to N as shown in Figure 3.17. The cantilever beams are shown with A_1, they are located on a masonry wall on side A and on natural rock on side B. They are formed with squarely sectioned logs; their diameters change from 25 cm to 32 cm at side A and from 16 cm to 24 cm at side B; and their lengths change from 5.35 m to 9.85 m at side A and from 7.25 m to 9.87 m at side B. There are 20 cantilever beams on side A and 19 cantilever beams on side B.

A_2 shows the cantilever beams which are extending from the entrances of the bridge to the main span. There are 3 of them at each side of the bridge. Their lengths change between 4 m to 5.25 m and their diameter changes between 15 cm to 19 cm.

There are 3 main beams on the bridge which are shown with C on Figure 3.17. The main beams have circular sections; their diameter changes between 20 cm to 25 cm and their length is approximately 14.50 m. They are placed on the cantilever beams simply supported at ends.

The side lintels (shown with D, in Figure 3.17) are placed above the deck plates; between the posts at each side of the bridge through its length. They have squared sections (10 cm*15 cm) and their lengths changes between 3 m to 3.5 m.

The L shaped bracings (shown with E, in Figure 3.17) connect the posts and deck plates; they are placed on the tie beams. The width of the bracings is approximately 10 cm; the approximate length of their short leg is 70 cm whereas the long leg is 1 m. The forces are transferred from the posts to the deck plates via L shaped bracings; besides, the bracings prevent the transversal motion of the bridge.

There are 22 posts on the bridge shown with J in Figure 3.17, placed on tie beams vertically and connected with the side lintels. They have squared sections (10 cm*15 cm) and their lengths are approximately 2.30 m. Besides the posts, there 16 studs (shown with I, in Figure 3.17) on the bridge placed vertically on the side lintels between the studs in 10 cm*10 cm*230 cm dimensions.

There are 11 tie beams on the bridge which are shown with F in Figure 3.17, placed horizontally on the main beams. They are approximately 2.2 m long; their width and height are 15 cm and 20 cm, respectively.

There are squared sectioned (10 cm*10 cm) transversal bracings connecting the posts and side lintels. They are shown with G in Figure 3.17. Their lengths are approximately 1.50. There are also horizontal beams shown with H in Figure 3.17, connecting together the studs, posts and bracings. They are in 5 cm * 10 cm dimensions and their lengths change between 4 m to 7 m.

The roof elements are shown with K_1, K_2, K_3, K_4, L_1, and L_2 in Figure 3.17. The elements named as L_1 (purlins & wall plates) are placed on the posts and studs at each side of the bridge through its span. Both of L_1 and L_2 have squared sections, 8 cm*12 cm and 8 cm*8 cm respectively. Their lengths changes between 3 m to 3.5 m.

K_1 is a curved joist placed between the two opposite posts. There are 11 joists on the bridge. Its width and height are approximately 10 cm and 20 cm, respectively. K_2 is the hanger which is placed on joists K_1. It is approximately 50 cm in length with a squared section of 10 cm*10 cm. The rafters are shown with K_3 and K_4. They have squared sections, 10 cm*10 cm; they are in 1.50 m long and placed transversally between the posts and ridges (L_2). There are 22 rafters on the bridge in total.

The width and height of the deck covering deck plates are approximately 15 cm and 5 cm respectively. They are placed on the main beams and their length is 2.20 in average. The lengths of roof covering plates show differences, change between 1.60 and 1.75 m in average. Their thickness is about 2 cm whereas their width is approximately 11 cm.

Hereby, it must be noted that the numbers of the elements were determined with the help of the old photos of the bridge and measured surveys, so there might be small margin of error in these numbers.

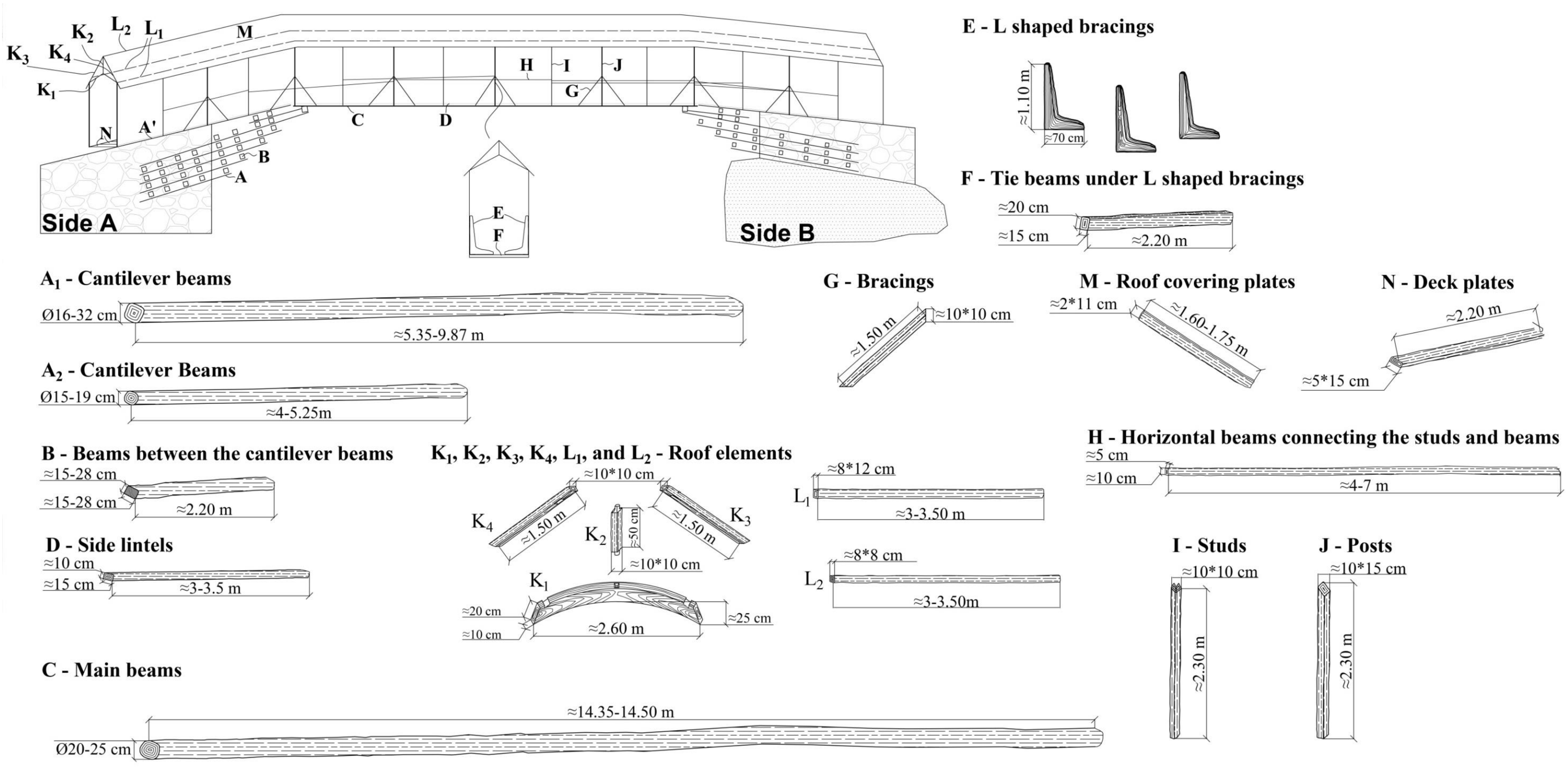


Figure 3.17 Elements of the bridge and their dimensions (Drawed by Ezgi Çabuk to illustrate the grain directions of the timber elements)

It was not able to define the construction process of such timber bridge as the author did not witness such an experience before. But by assessing the gathered information about the Buzlupınar Bridge a hypothesis about its construction process can proposed as described in following paragraphs.

In the case of Buzlupınar Bridge, the construction of the bridge possibly started with the construction of the abutments. In this case, a natural rock (which is located where the river narrows relatively) was chosen as one of the abutments and for the other one stone masonry wall was built opposite to the natural rock. The inside of the stone masonry walls was filled with rubble stones and clay mortar, detail of it is given in Figure 3.18.

Figure 3.18 Construction details of the stone masonry abutment (General Directorate of Highways' archive, 2014)

The construction possibly continued with the placement of the cantilever beams (A_1) on the abutments at each side. They horizontally overlap on top of each other. There are square sectioned horizontal beams (B) approximately 2.20 m long, placed between the cantilever beams. The cantilever beams and horizontal beams are nailed to each other with wrought iron nails where they overlap. At side A, there are 5 rows each consists of 4 logs and at side B there are 4 rows one consists of 7 logs and each of the other three consist of 4 logs. The gaps between the cantilever beams are filled with earth and stones as shown in Figure 3.19 and Figure 3.20. Construction stages of the cantilever beams are shown in Figure 3.21 and Figure 3.22, coded from 1-a to 1-l respectively.

Figure 3.19 Stone infill between the cantilever beams which were placed on stone masonry wall

Figure 3.20 Stone infill between the cantilever beams which were placed on natural rock

When constructions of the cantilever beams are completed as shown in Figure 3.23-a, the main beams (C) are placed on top of them, supported by the cantilever beams at both ends as shown in Figure 3.23-b. The main beams are nailed to the horizontal beams and cantilever beams with wrought iron nails as sketched in Figure 3.23; in addition, 3 cantilever beams (A_2) are placed at either side of the main beams, shown in Figure 3.23-c. The construction stages of the cantilever beams and placement of the main beams on top of them are shown in Figure 3.21, Figure 3.22, and Figure 3.23.

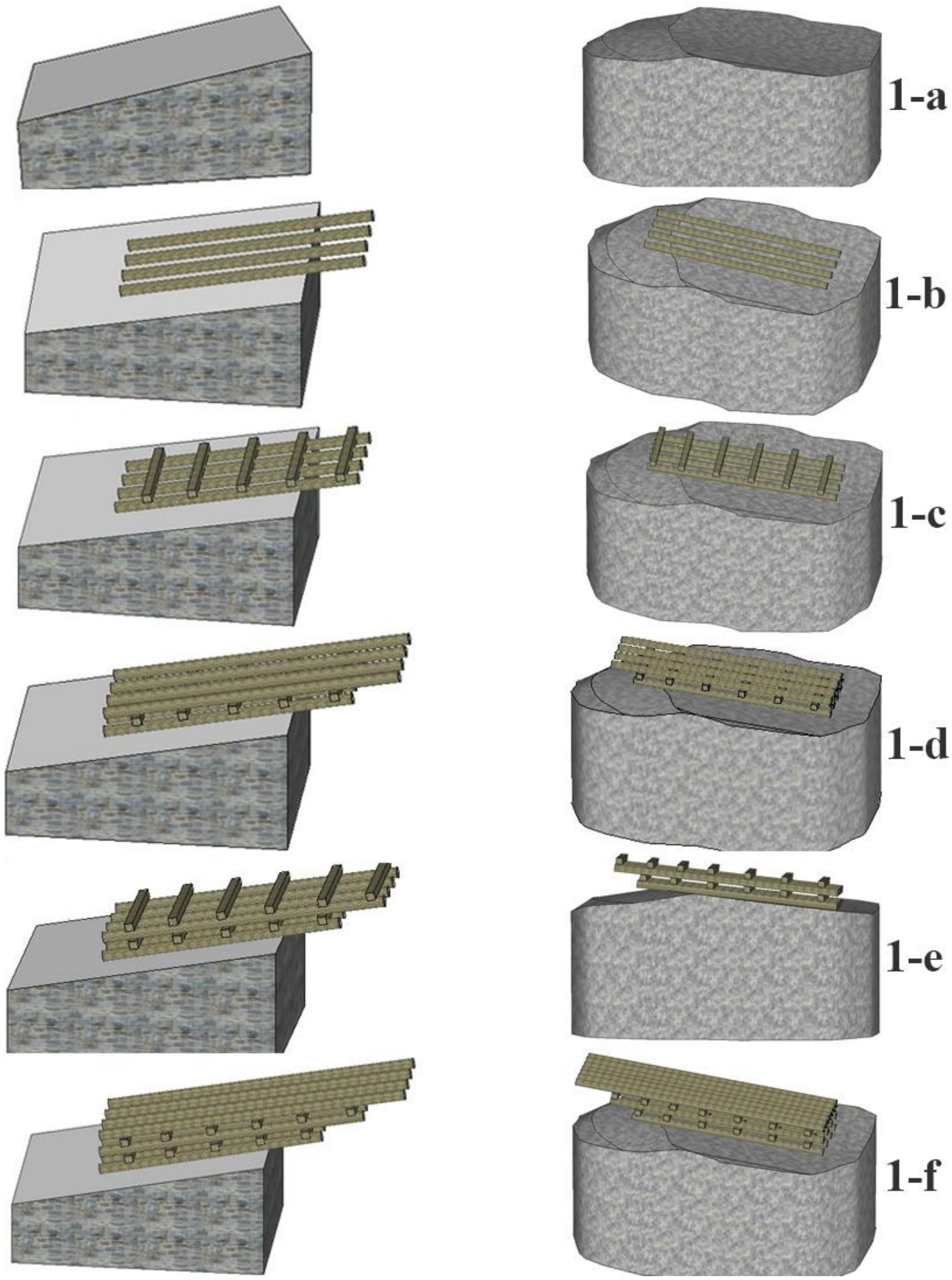


Figure 3.21 Construction stages of the cantilever beams -1 (Made by Ezgi Çabuk)

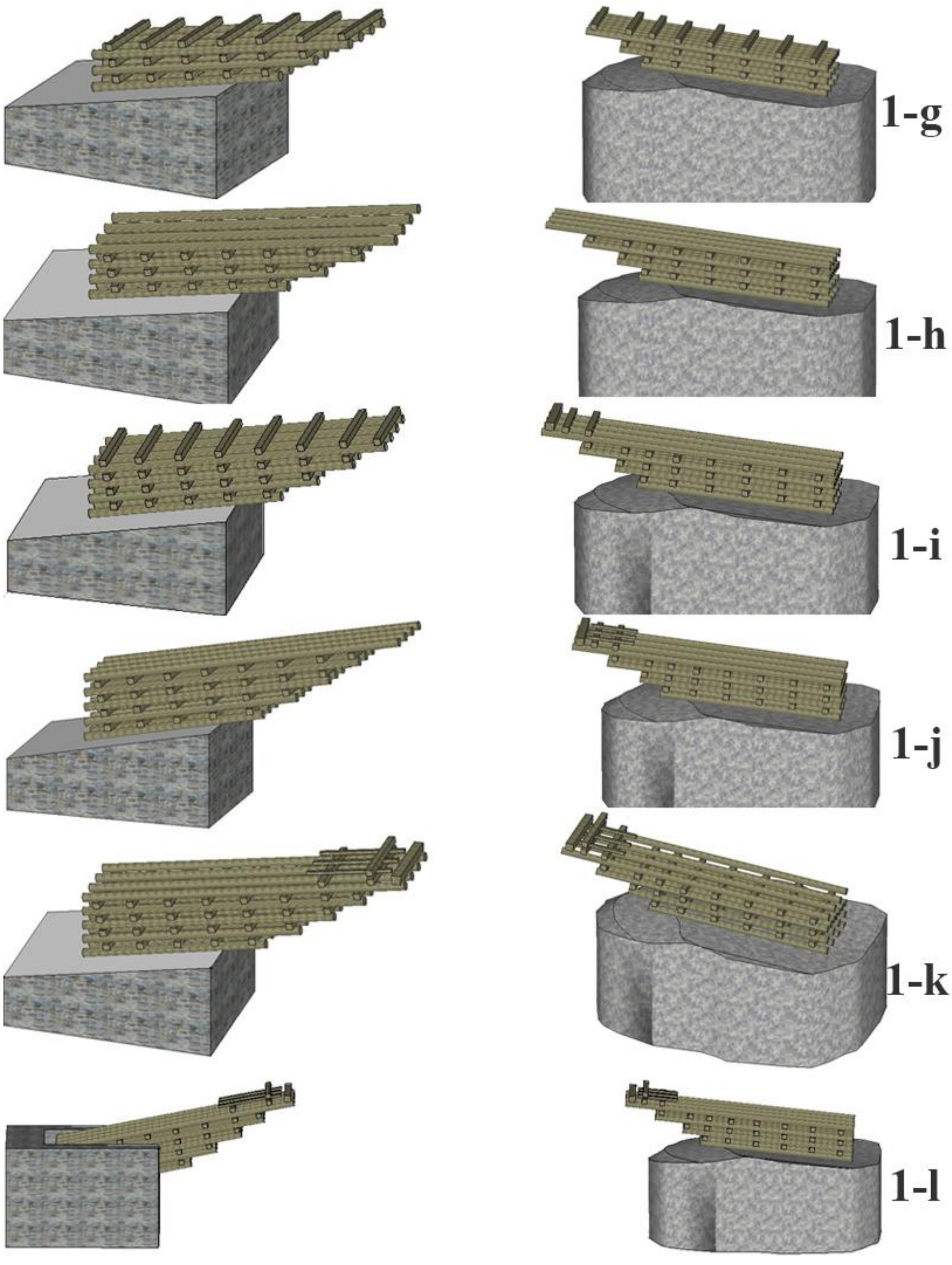


Figure 3.22 Construction of the cantilever beams -2 (Made by Ezgi Çabuk)

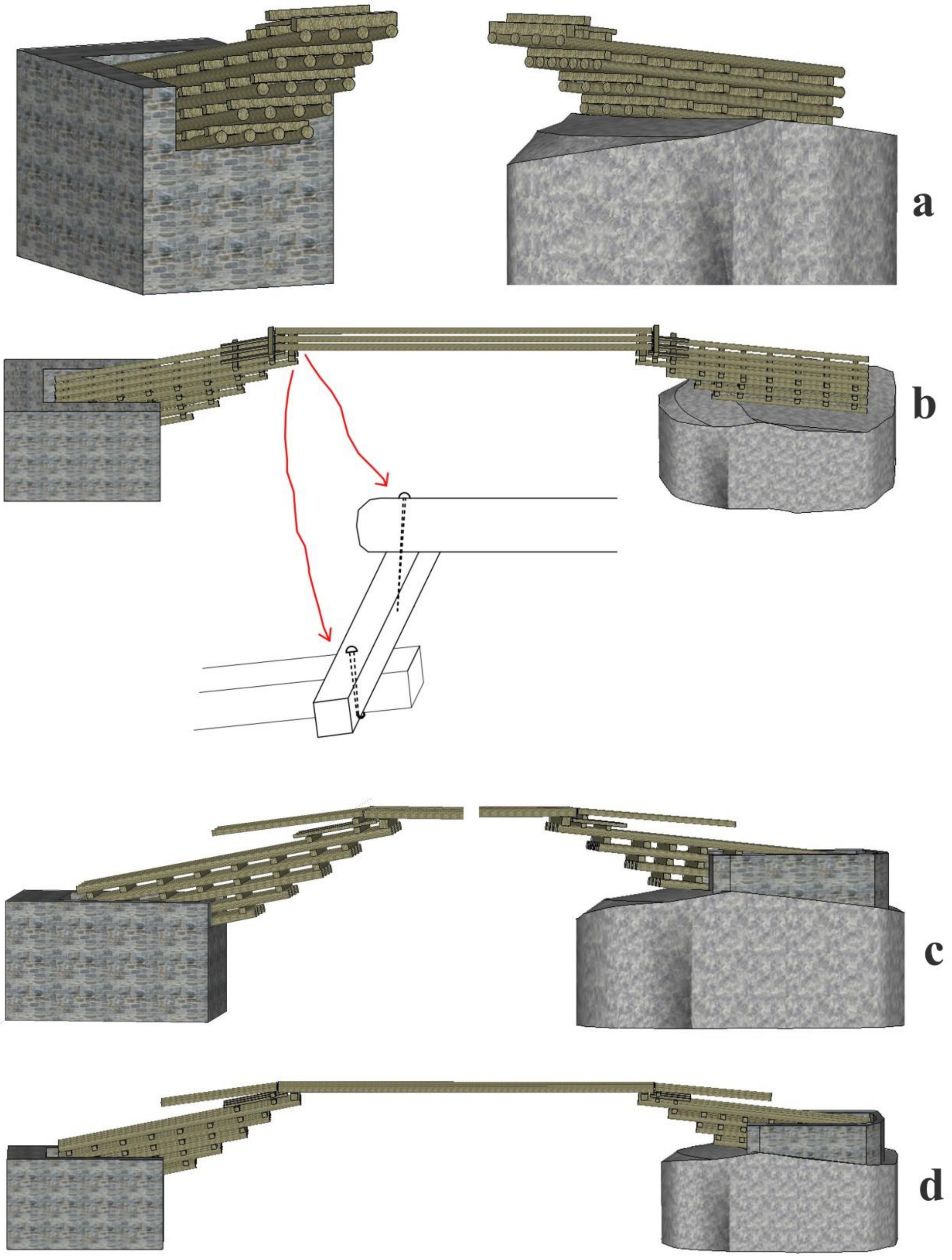


Figure 3.23 Placement of the main beams on top of the cantilever beams, and connection details of the main beam and cantilever beams (Made by Ezgi Çabuk)

The deck plates and tie beams are placed on main beams and cantilever beams at each side as shown in Figure 3.25-e. As defined from the old photos of the bridge they are nailed to the main beams with 1-2 nails at each connection; and as defined from the measured surveys and photos again, the diameter of the original nails changes between 0.5 cm to 1 cm and their lengths were approximately 18 cm-20 cm. Then the posts are assembled, there is like a mortise and tenon type of connection between the posts and tie beams, the lower edge of the posts are put into the holes on the tie beams as shown in Figure 3.25-f. Meanwhile the side lintels are placed throughout the bridge as shown in Figure 3.25-g. There is a lapped connection between the side lintels and posts as sketched in Figure 3.25. The connections are secured with nails at several points.

On the next step, the studs are put into the mortises on the side lintels as shown in Figure 3.27-h. The mortise and tenon type of connection is repeated for the assembly of bracings; they are placed transversally between the side lintels and posts, and the connections are secured with nails as shown in Figure 3.27-i. Then the horizontal beams which connect the studs and posts together are assembled, nailed to the studs, post, and bracings at several points as shown in figure Figure 3.27-j.

At this stage, the L shaped bracings, shown in Figure 3.24, are placed above the tie beams, nailed to the tie beams and vertical beams, as sketched in Figure 3.26 and shown in Figure 3.27-k. As it is understood from the old photographs of the bridge, the short legs of L shaped bracings were pounded to the tie beams with 1 or 2 nails and their long legs were pounded to the vertical beams with 2 or 3 nails. The L shaped bracings were produced from monoblock wood, such as the branched parts of a tree. (see Figure 3.24) The fibres of them extend in the direction of the legs by rotating 90^0 at the edge. Thus, it is seen that the craftsman/carpenter was aware of the performing of trunk.

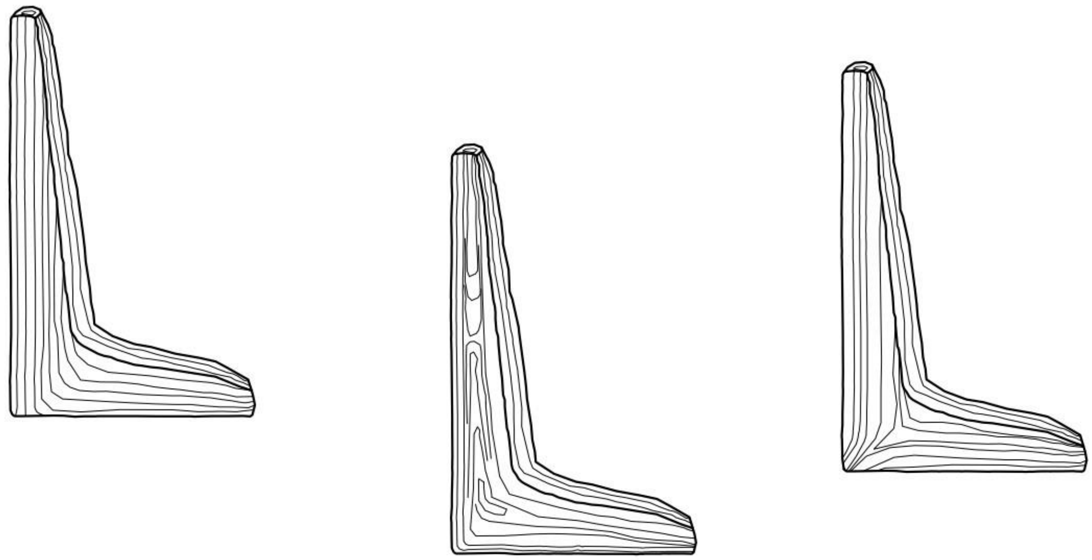


Figure 3.24 Details of the grain directions of the L shaped bracings (Drawn by Ezgi Çabuk)

When the above mentioned works are finished, as seen in Figure 3.27-k, the construction continues with the assembly of the roof elements. Although the order of construction stages of the roof elements are not yet certain, the details of the connections of between them are given in Figure 3.29. These details were determined from the old photos of the bridge and measured surveys. It begins with the assembly of the joists (K_1) between two posts in the transverse direction (Figure 3.28-l). The mortise and tenon type of connection is repeated again between the posts and joists. This is followed by assemble of the hangers (K_2) over the chords and placement of the wall plates along the whole length of the bridge as in Figure 3.28-m and Figure 3.28-n. The tenons at the edges of the hangers are put into the mortice on the joists and wall plates are put into the mortises at the edges of the joist as shown in Figure 3.29 . Then the rafters (K_3 and K_4) are assembled as shown in Figure 3.28-o; first K_3 is interlaced to the hanger, then K_4 is interlaced to both K_2 and K_3 as illustrated in Figure 3.29. All of these elements, joist, rafters and wall plates, have interlaced connections with each other and nailed to each other additionally. The construction stages of the roof elements up to now are shown in Figure 3.29. When these installations are completed, it continues with the placement of purlins and ridge (L_1 and L_2, repectively) as seen in Figure 3.30-p and Figure 3.30-r. The purlins are nailed to the rafters. With the installation of the purlins, the construction of the structural elements of the bridge is completed as shown in Figure 3.30-s. Finally, the roof covering plates assembled with nailing to the purlins. In addition to these, on the covered side of the bridge, a second line of horizontal beams is constructed between the upper parts of posts and studs in order to support the side covering plates, and the plates are nailed to these two lines of horizontal beams.

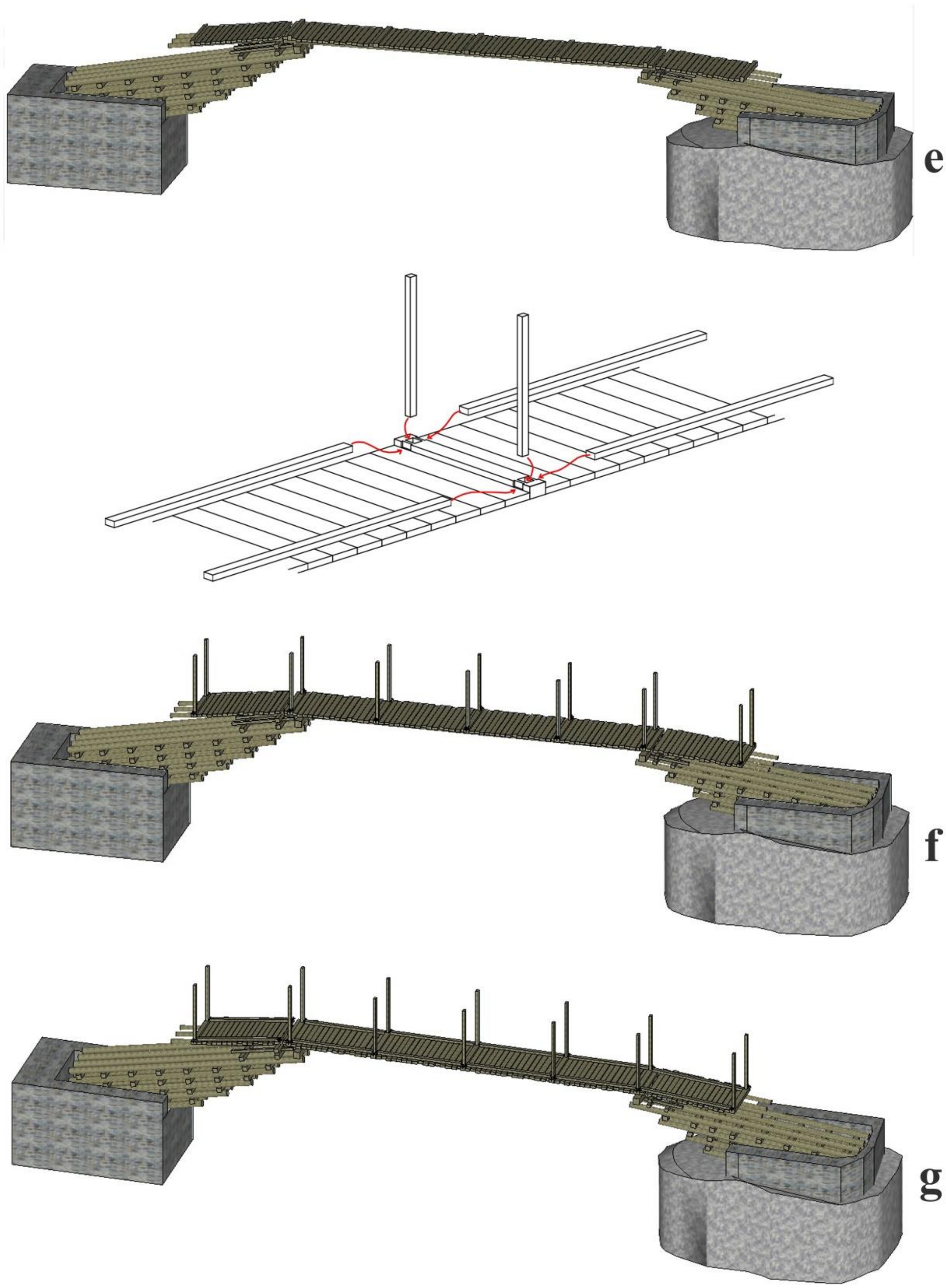


Figure 3.25 Assembly and connection details of the deck plates, tie beams, side lintels and posts (Made by Ezgi Çabuk)

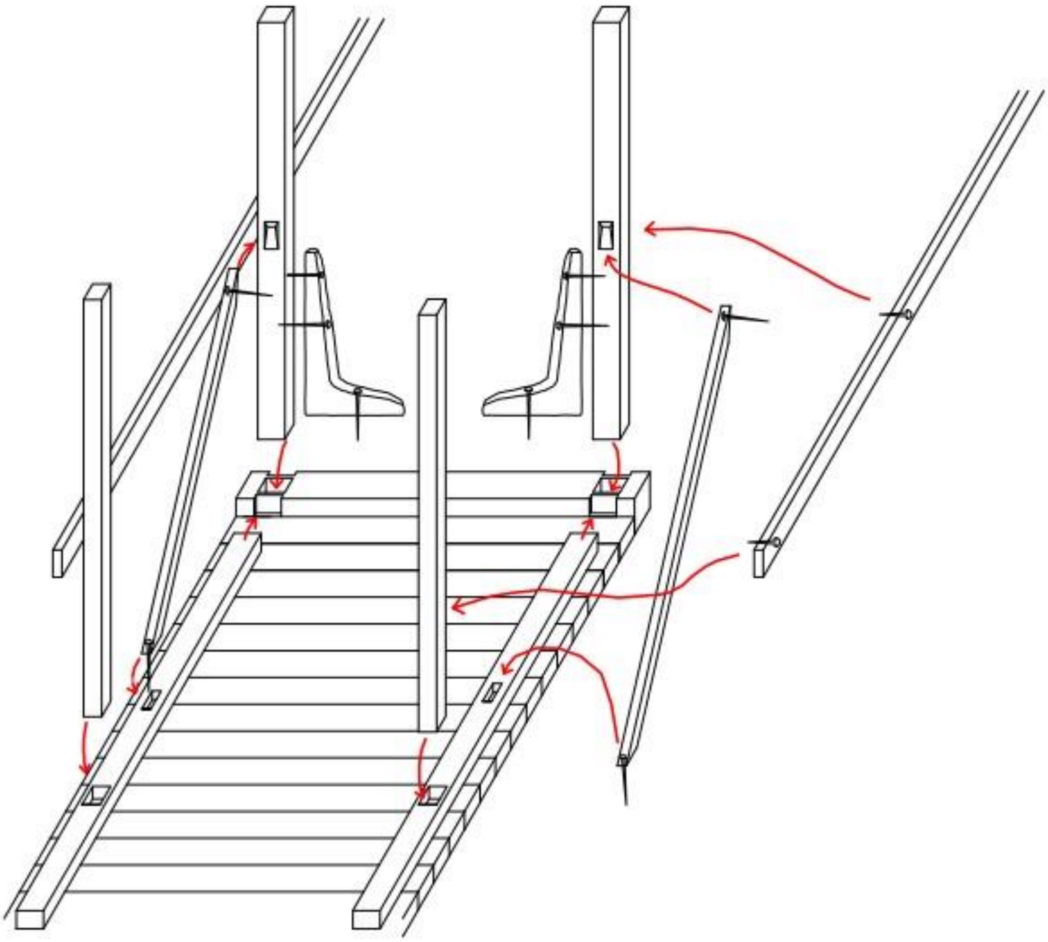


Figure 3.26 Connection details of the tie beams, studs, posts, L shaped bracings, bracings and horizontal beams to each other. (Drawn by Ezgi Çabuk)

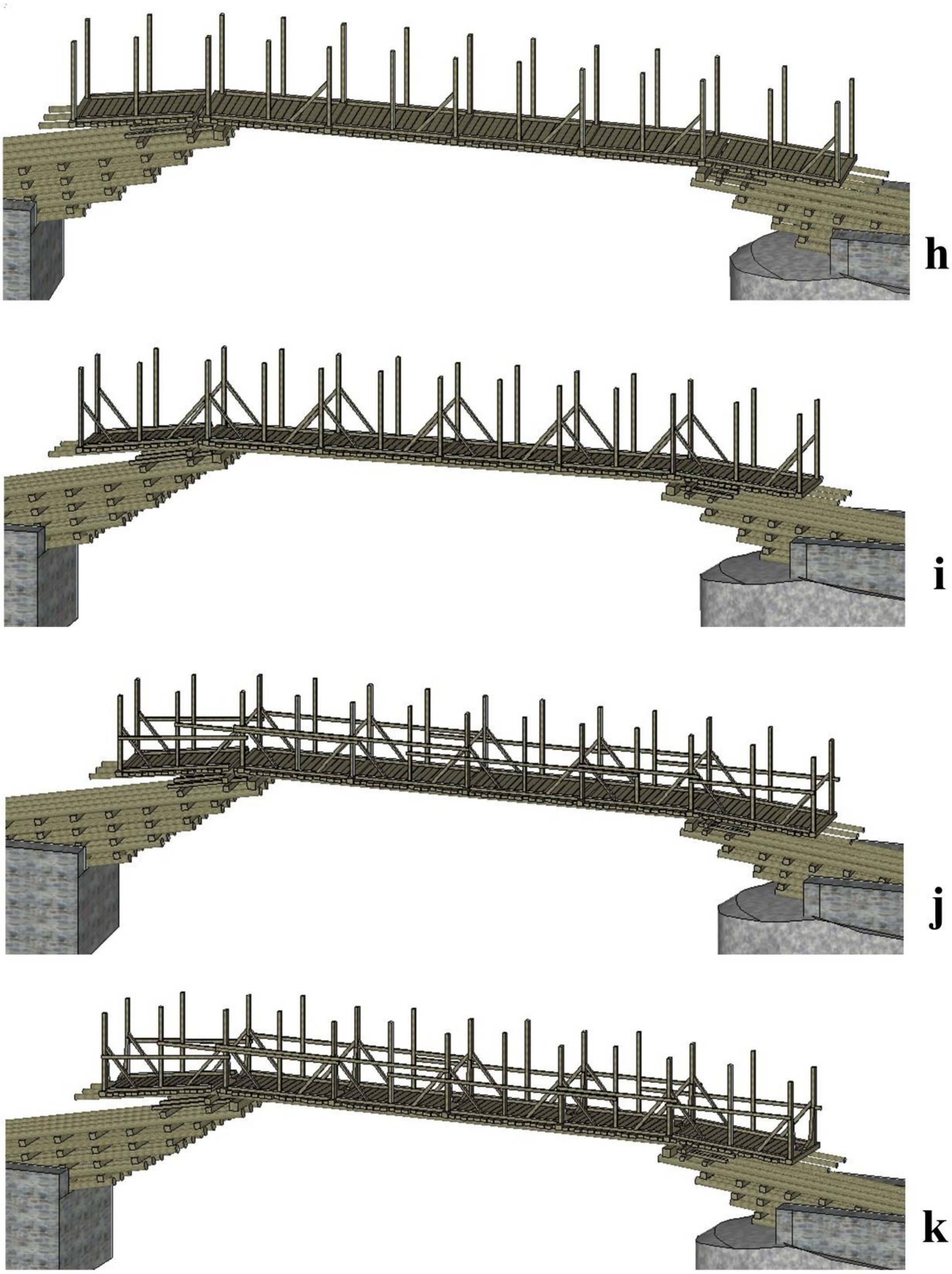


Figure 3.27 Assembly of the studs, bracings and horizontal beams connecting posts, studs and bracings together (Made by Ezgi Çabuk)

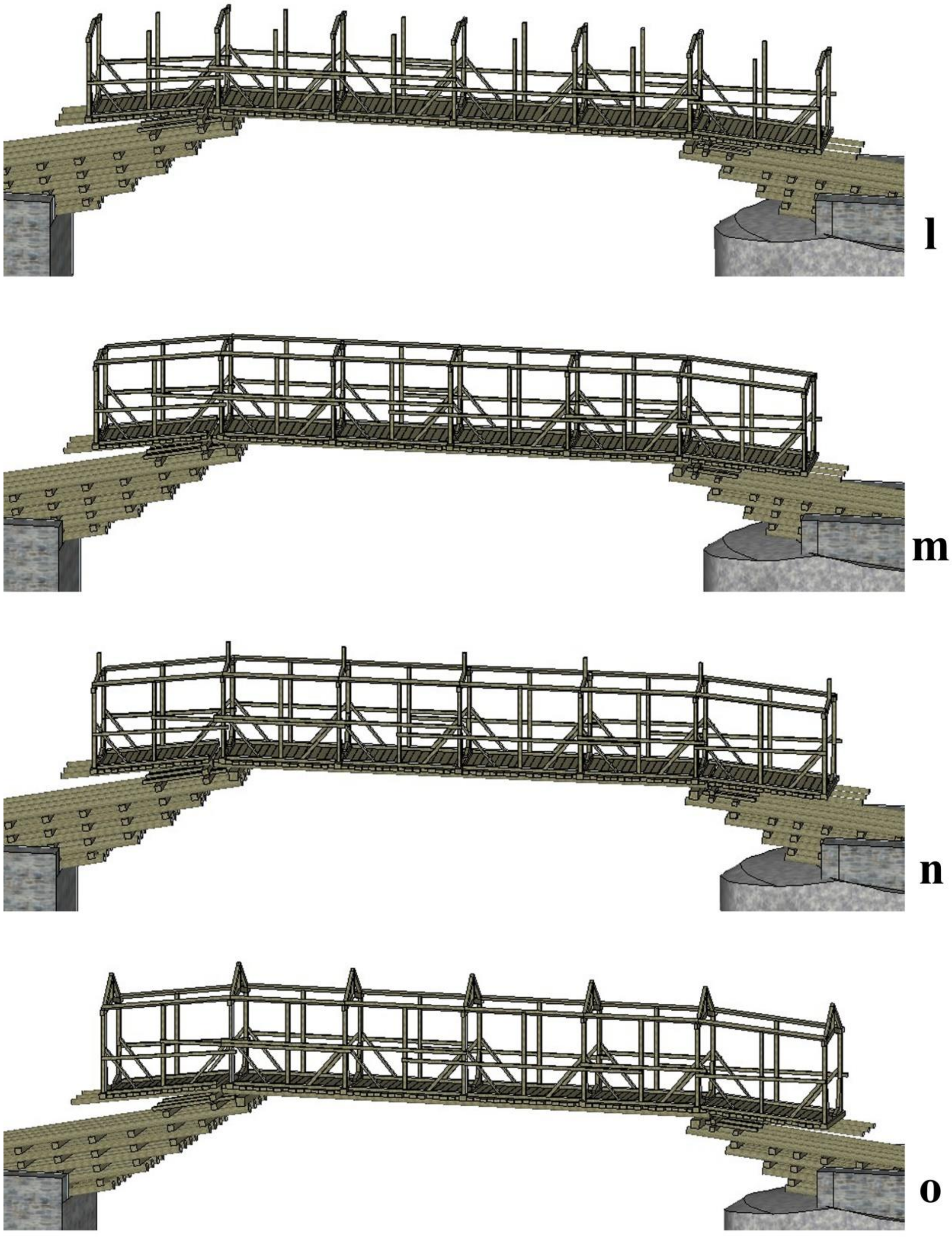


Figure 3.28 Construction of the roof elements -1 (Made by Ezgi Çabuk)

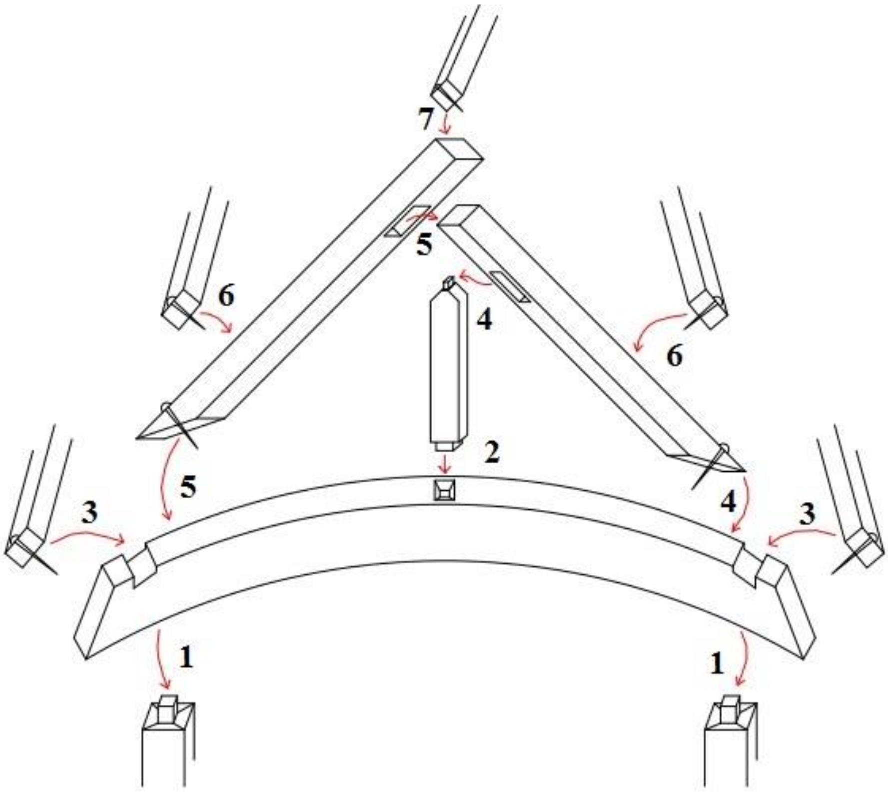


Figure 3.29 Connection details of the roof elements (Drawn by Ezgi Çabuk)

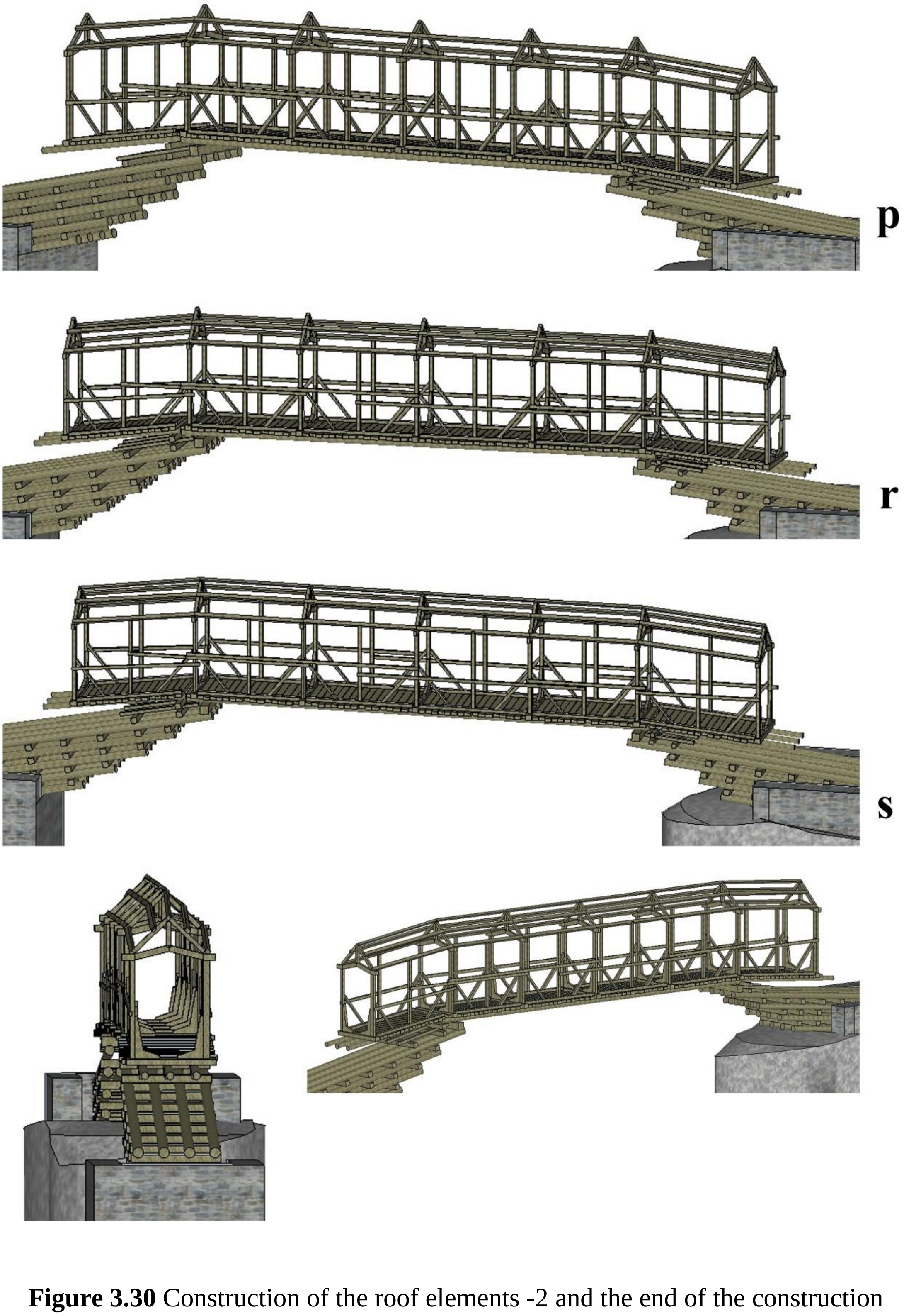


Figure 3.30 Construction of the roof elements -2 and the end of the construction

(Made by Ezgi Çabuk)

3.3 Material Characteristics of Collapsed Buzlupınar Bridge, Material Tests and Mechanical Properties of Timber Elements

In 2012, parallel to the field survey and several material tests[15] have made by the General Directorate of Highways to determine the characteristic and mechanical properties of the timber elements of Buzlupınar Bridge. Wood type, bending strength perpendicular to grains, compression strength parallel to grains, specific gravity, and humidity values of the timber elements were determined. These studies were carried by the cooperation of the General Directorate of Highways and Forest Industrial Engineering Department of Karadeniz Technical University. The results of material test are given at an unpublished report have prepared by the General Directorate of Highways in 2012.

The specimens for timber tests and laboratory investigations were taken from the remaining timber cantilever beams and deck plates. Together with the parameters given below, fungal and insect damage on timber elements have been found and decay in the interior parts of the timber elements has been determined with the studies carried out by micro hammer and resistograph devices. The specific weight of the wood was determined as 0,8±0,51 g/cm^3. The strength values were determined using the maximum load values, the results are as shown in Table 3.1.

[15] The material tests were done by Forest Industrial Engineering Department of KTÜ (KGM, 2012, Ankara)

Table 3.1 Results of the wood tests made in 2012 in Forest Industrial Engineering Department, KTÜ,Trabzon

	Number of specimens	Arithmetic mean (N/mm^2)	Standard deviation	Min value (N/mm^2)	Max value (N/mm^2)
Bending strength perpendicular to grains	30	36.77	15.59	5.52	64.81
Compressive strength parallel to grains	30	34.76	5.67	23.53	42.29

In the report,these values are compared with the physical and mechanical parameters of sound chestnut wood which is not exposed to any defects or rot, given in the book named Ağaç Malzeme Teknolojisi written by A. Berkel, in 1970. The book has been viewed by the author to compare the values with the test results but the parameters of chestnut wood could not be found on the book. It is not stated in the report if any special method used to determine or estimate the parameters for chestnut wood; however the un-weathered values of the parameters are stated in the material test report are as follows:

- Bending strength –perpendicular to grains: 75.51 N/mm^2
- Compressive strength –parallel to grains: 49.03 N/mm^2

The author also has compared the test results with the same parameters of un-weathered chestnut wood given in two different sources. At the first source[16] the parameters are as follows:

[16] http://www.wood-database.com/lumber-identification/hardwoods/sweet-chestnut (accessed on 17 March 2015)

- bending strength: 71.39 N/mm^2
- compression strength: 43.79 N/mm^2

At the second source[17] the parameters are as follows:

- Bending strength: 77.49 N/mm^2
- Compressive strength –parallel to grains: 51.79 N/mm^2

It is clearly seen from these comparisons there is a distinguishable loss in the bending strength and compressive strength values of the timber elements of the Buzlupınar Bridge. Since more than half of the bridge collapsed and the residual strength of the remaining timber elements was inadequate, the bridge was decided to rebuild using the same kind of wood, regarding its original construction technique.

3.4 Reconstruction Project of Buzlupınar Bridge Prepared by the Bender Restoration Firm

As mentioned in the previous sections, the cover of the bridge was collapsed, remaining materials were quite damaged and has lost their strength. As a result of the documentation studies of the bridge and the material test, it has been decided to rebuild the bridge again in the same construction technique and using the same kind of materials.

In the summer of 2014, the two remaining main beams and the cantilever beams which form the abutments were disassembled. By this means the actual lengths and construction details of the cantilever beams, original form and dimensions of the nails were learnt, which is quite important information for the structural analysis studies.

[17] Maçka Çatak Bölgesi Anadolu Kestanesi (*Castanea Sativa* Mill) Odununun Bazı Özellikleri (Ay & Şahin, 2002)

The original nails are shown in Figure 3.33 and Figure 3.34. Also in Figure 3.31 and Figure 3.32, some views of the cantilever beams are shown from the dismantling process.

Figure 3.31 The total lengths of the cantilever beams were learned with the disassemble process (General Directorate of Highways' archive, 2014)

Figure 3.32 Disassemble process of the cantilever beams (General Directorate of Highways' archive, 2014)

Figure 3.33 Original nails used the construction of the bridge -1 (General Directorate of Highways' archive, 2014)

Figure 3.34 Original nails used the construction of the bridge -2 (General Directorate of Highways' archive, 2014)

With the information gathered when the cantilever beams were disassembled and the old photographs of the bridge which were provided by Neriman Şahin Güçhan, the measured surveys and projects were revised in September 2014. A view from the reconstruction project is given in Figure 3.35. The structural analysis studies of the bridge were made based on the dimensions and material properties given on the revised projects.

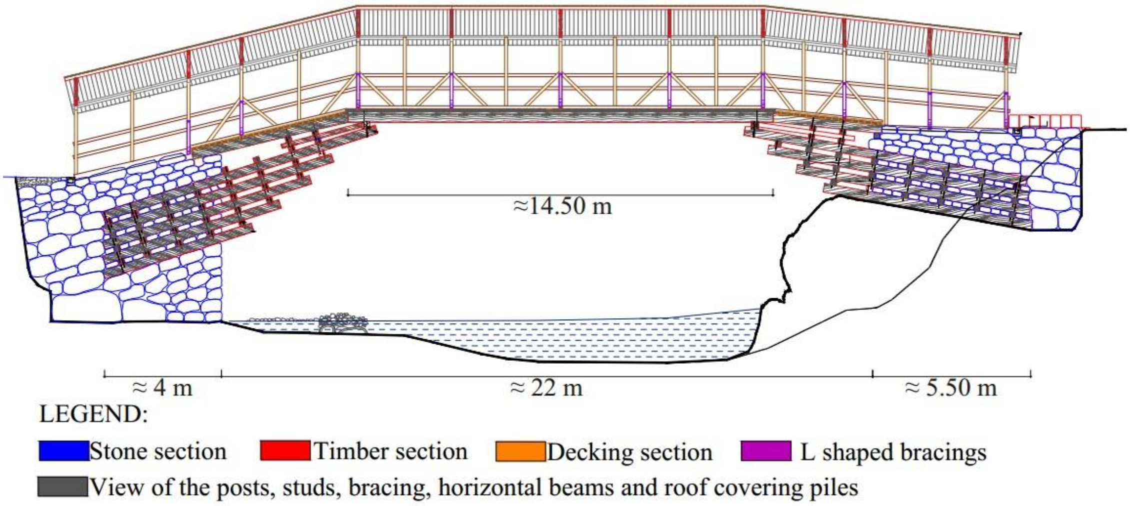


Figure 3.35 Restoration project of the Buzlupınar Bridge (KGM, 2012, Ankara)

CHAPTER 4

ANALYTICAL MODELLING AND STRUCTURAL ANALYSIS OF BUZLUPINAR BRIDGE

In this chapter structural analysis of the bridge is done with hand calculations and finite element model constructed with SAP2000 using the dimensions given in the reconstruction project. The behaviour of the bridge is investigated under live loads, wind loads and earthquake loads. Structural safety of the critical connections is checked.

In order to determine their mechanical properties, material tests are done on the new timber and nails which are going to be used in reconstruction project; the obtained values are used on the structural analysis calculations. In addition, a conservation approach is defined specific to Buzlupınar Bridge and proposals are made for the reconstruction project considering the conservation principles.

4.1 Analytical Modelling and Structural Analysis of Timber Structures and Bridges and a Conservation Approval Specific to Buzlupınar Bridge

Structural analysis means to determine the stresses and deformations formed in the members of a structure caused by the existing loading or could be caused by the expected loads on the structure; and then to compare these results with the safety and serviceability requirements given in the structural design codes. If the results are not satisfying, the sizes of structural members should be revised until the requirements are satisfied (Kassimali, 2005). The structure needs to prove that it is strong enough to carry the given condition of loading within the safe deformation boundaries (Seward, 2014).

Until the 17th century, structural analysis was based on trial and error and the past experiences of the engineer. In the middle of the 17th century the structural analysis began to be made more scientifically with the improvements in knowledge of maths and mechanics. In 1950s, the computers got involved to human's life, it became possible to solve and analyse the large structures and complex problems easily and in short time with computers (Kassimali, 2005). At the same period, the finite element method appeared as a developed form of previous structural analysis methods (Tong & Rossettos, 1977).

The finite element method is a numerical technique used to solve complex and large problems by dividing a continuum to subdivisions, finite elements, which are connected to each other at their boundaries with special joints called nodes. In civil engineering, this method is applicable to analyse trusses, frames, plates, shear walls, bridges, concrete structures; estimation of stability of buildings, frequencies and modes; stress waves, and their reactions to aperiodic loads. (Rao, 2005)

SAP2000 is a structural finite element analysis program to analyse and design structures, which enables to model numerous types of structures from 2D frames to 3D complex structures and analyse them under different types of loads and conditions such as wind loads, seismic loads. Historic buildings may show discontinuities in material properties and structural types, therefore there could be difficulties to model historic structures with SAP 2000 since it is designed to model and analyse modern structures with modern materials. In such a case, some simplifications and modifications can be done in ways that do not change the structural behaviour of the structure to make it easier to prepare the analytical model.

F.Fanous and friends have been used FEM to analyse the Moxley Covered Timber Bridge which was built in 1883 and the Zacke Cox Covered Timber Bridge which was built in 1908. The intent was to compare the displacement and strain values obtained as results of field testing and the analytical model. For this purpose, a two dimensioned idealized model of the Moxley Covered Timber Bridge and a three dimensioned idealized model of the Zecke Cox Covered timber bridge were formed using FEM. In both cases, the outputs of the analytical model and the results of the field testing were compared well with each other (Fanous, Rammer, & Wipf, 2013).

Another example for the use of FEM in analysis of historic timber structures is the structural assessment of a wooden bell tower which was built in 19th century. The timber bell tower is surrounded with masonry wall; when the ring bell, the timber structure was having very large displacements and impact with the masonry wall. To prevent this situation, an analytical model of the tower was formed using FEM and the weak points and connections of the structural system were determined; and a reinforcement proposal was made for the wooden bell tower (André, Galimard, & Morlier, 2003).

Within the scope of the structural analysis studies of the Buzlupınar Bridge, several hand calculations were made to determine the stresses produced on main beams, overturning forces acting on cantilever beams, shear forces between the cantilever beams and the horizontal beams between them, factor of safety of the bridge against wind loads; at the same time a FEM of the bridge was prepared with SAP2000. The results of the hand calculations were compared with the outputs of SAP2000, in addition a seismic analysis was made using SAP2000.

In this study, both the hand calculations and finite element modelling studies were done with respect to the original structural form of the bridge. The dimensions of the elements were taken from the reconstruction project since the bridge is supposed to be reconstructed as given in the project. However, in cases where there are differences between the measurements given in the reconstruction project and original ones, the calculations were repeated for both dimensions. In such a case, if convenient, it is recommended to use the elements in their original dimensions as long as they satisfy the safety requirements. As stated in ICOMOS Principles for the Preservation of the Historic Timber Structures (1999), the aim of the preservation is to maintain the authenticity of the structure. The problems related to the structure should be solved aiming minimum intervention; existing elements of the structures should be retained, if alterations are inevitable they should be done due respect to the original characteristics, historical and aesthetic values of the structure, considering the relevant conditions and requirements.

When these principles are adapted to the case of Buzlupınar Bridge, first of all it is recommended to define the residual strength of each of the existing cantilever beams and main beams, and if there are any of them satisfying the strength requirements, use them on the reconstruction project. The cantilever beams which do not satisfy the strength requirements should be altered with new timber elements. The new timber should be the same wood with the original one and the elements should be produced and shaped as the former one. The other elements of the bridge including the main beams, deck plates, posts, horizontal and transversal beams, roof elements, roof covering piles and side covering piles should be reproduced since they had been demolished. These elements should be produced based on the measured surveys and photographs of the bridge considering the grading, numbers and shapes of the original ones, any tentative approaches should be avoided.

The interlaced and nailed connections should be constructed in the same technique with the original ones as long as there is no need to any reinforcement. The nails that will be used on the reconstruction project will be produced as in the same physical characteristics with the original ones. On the other hand, it is recommended to make admissible changes in their form, size and length where necessary, in order to satisfy the structural safety requirements.

4.2 Hand Calculations and Simple Checks

The purpose of the hand calculations was to determine the structural behaviour of the bridge under different load cases and forces that the bridge could be exposed during its lifetime and check the competency of dimensions of its structural elements. To that end, the calculations were focused on bending moment and shear forces generated on the main beams; overturning forces and shear forces acting on the cantilever members due to the own weight of the bridge and the live loads; and also the effect of the wind forces on the structural safety of the bridge. Considering the results of the hand calculations, structural proposals and recommendations were done where necessary.

The hand calculations were based on allowable stress design (ASD) method, in which the strength of the material is reduced by multiplying with a factor to increase the safety and it must be ensured the stresses developed on the structural elements due to the loading do not exceed the reduced strength of the materials. In this study, the strength values of the materials were obtained with the material tests which is mentioned in section 4.3. For the calculations, the strength values at the proportional limit were used which are far below the ultimate strength values, and this values were reduced with partial factors using equation (4.3) given in Eurocode 5. Therefore reduced values were considered as sufficient enough to ensure the desired safety conditions for the structure and the loads acting on the structure were not multiplied with any factors.

4.2.1 Bending of the Main Span and Deck Cover Plates

Two different assumptions were made to determine the maximum load that could be acting on the bridge. Since the bridge is known to be used by pedestrians and animals, firstly it was presumed that the bridge is full with bovine animals, each is 500 kgf and 1,5 m length and they are passing over the bridge with 0,5 m intervals. The bridge is approximately 22 m long; that is maximum 11 bovine animals could be on the bridge at the same time. The main beams are approximately 14.5 m long, so that maximum 7 bovine animals could be on the main beams of the bridge at same time. In this case, the total live load acting on the main span of the bridge is equal to;

$$\mathrm{LL} = 7 * 500 = 3500\ \mathrm{kg} = 34.32\ \mathrm{kN}$$

The load per unit length of the main span is equal to;

$$\mathrm{LL} = \frac{34.32}{14.5} = 2.36\ \mathrm{kN/m}$$

Another live load condition taken into account is that the bridge is full with people. In TS 498: Design Loads for Buildings; the design live load value for the places that people could concentrate is given as 5 $\mathrm{kN/m^2}$. The active width of the bridge is approximately 1.85 m, and then the distributed load on the bridge is equal to:

$$\mathrm{LL} = 5 * 1.85 = 9.25\ \mathrm{kN/m}$$

Since 9.25 kN/m is bigger than 2.36 kN/m, 9.25 kN/m was taken as the design live load for the bridge to be on the safe side. The self-weight of the bridge was calculated approximately 6670 kg ≈ 4.51 kN/m; then the total load that the bridge is exposed to is equal to:

$$\mathrm{LL} + \mathrm{DL} = 9.25 + 4.51 = 13.76\ \mathrm{kN/m}$$

The maximum moment generated on the main beams is calculated with the equation (4.1). Since the span distance between two supports is equal to 13.5 m, the maximum moment is equal to:

$$M_{max} = \frac{P * L_{span}^2}{8} \tag{4.1}$$

$$M_{max} = \frac{13.76 * 13.5^2}{8} = 313.47\ \mathrm{kN.m}$$

As shown in Figure 4.1, the diameter of the main beams is given as 40 cm and the height of the deck plates are given as 10 cm in the restoration project of the bridge. These dimensions were used for calculate the maximum stresses on the main beams both in cases of the main beams are compositely working with the deck plates or not.

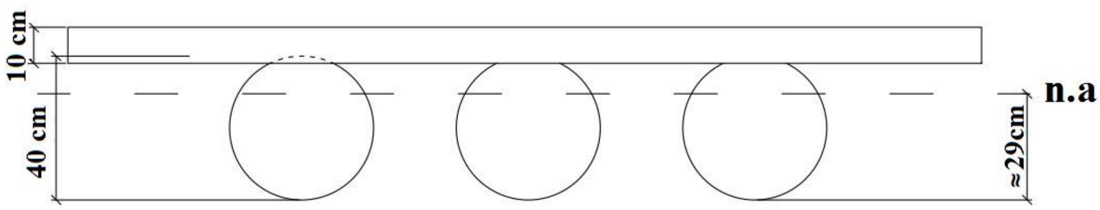


Figure 4.1 Natural axis of the compositely working main beams and deck plates

The stresses on the main beams for both situations are calculated using equation (4.2) as given above:

$$\sigma = \frac{M}{I} * y \tag{4.2}$$

The moment of inertia of the composite structure is equal to 1337309 cm^4.

$$\sigma_{top} = \frac{31347}{1337309} * (40 + 10 - 29) = 4.92\ \mathrm{MPa} \quad \text{compression}$$

$$\sigma_{\text{bottom}} = \frac{31347}{1337309} * (29) = 6.79 \text{ MPa} \quad \text{tension}$$

If the main beams and deck plates do not work compositely; the moment of inertia is equal to 376991 cm^4. Then the stresses are equal to:

$$\sigma_{\text{top}} = \sigma_{\text{bottom}} = \frac{31347}{376991} * (20) = 16.63 \text{ MPa} \quad \text{tension \& compression}$$

The bending strength value of the timber which is going to be used for reconstruction project of the bridge is determined as 47.34 MPa which is the mean strength values obtained using the load values on the proportional limit. This value should be reduced to design strength value with multiplying with the partial factors. In Eurocode 5, the equation to calculate the design strength value as in equation (4.3).

$$X_d = k_{\text{mod}} * \frac{X_k}{\gamma_m} \tag{4.3}$$

Kmod and γ_m values were taken from the related Eurocode standard.

$$X_d = k_{\text{mod}} * \frac{X_k}{\gamma_m} = 0{,}7 * \frac{47.34}{1.3} = 25.50 \text{ MPa}$$

The design value of the bending strength of timber is bigger than the stress values generated on the main beams for both cases; which means, the diameter of the main beams given in the restoration project are enough to carry the loads that the main beams could be exposed.

The diameters of the main beams are given as 40 cm as stated previously, the original dimensions of the main beams changes between 21 cm to 25 cm along their lengths. As mentioned in section 1.2, if reconstruction is inevitable, like in this situation, the new structure should be built in the same architectural characteristics and same form using the same kind of materials with respect to the original structure. To this end, the calculations were repeated for different diameter values of main beams in order to obtain stress values less than the design bending strength, 25.50 MPa. If the main beams and deck plates are working compositely then the minimum diameter of the main beams can be 23 cm; in that case 24.14 MPa tension and 11.24 MPa compression stresses generates on the main beams and deck plates. On the other case, if the main beams and deck plates are not working together, the minimum diameter of the main beams can be 35 cm then the stress values changes to 24.50 MPa both in tension and compression; both are less than the design bending strength, 25.50MPa.

Another stress generated on the main beams is the shear stress. The distributed load acting on the main span is equal to 13.76 kN/m as calculated previously. Then the total load on the mean beams is equal to:

$$\mathrm{DL} + \mathrm{LL} = 13.76 * 14.5 = 199.52\ \mathrm{kN} \approx 200\ \mathrm{kN}$$

Since there are three main beams and they are simply supported at two ends, each end bears total of 100 kN and 33.4 kN/beams shear force is generated in each beam. The shear stress is calculated with the equation given in equation (4.4).

$$\tau = \frac{V * Q}{I * t} \qquad (4.4)$$

$$\tau = \frac{V * \frac{4r}{3\pi} * \frac{\pi r^2}{2}}{\frac{\pi r^4}{4} * 2r} = \frac{V * 4}{3\pi r^2} = \frac{4 * 33.4}{3 * \pi * 0.2^2} = 354.39\ \mathrm{kN/m^2}$$

The shear strength of chestnut wood is given as 7400 kN/m^2 in the referenced source.[18] The shear force generated on each main beam is far less than this value, so the dimensions of the main beams given on the restoration project are enough to satisfy structural safety of the bridge in terms of shear strength. If the diameter of the main beams were taken as 23 cm which is the minimum value that satisfies the bending strength requirements, then the shear stress is calculated as 1075.07 kN/m^2, which is still less than 7400 kN/m^2.

The next analysis made for the main beams and deck plates was to determine the shear flow between them to decide how many nails should be used to connect the deck plates to the main beams. The distances of the deck plate that each beam should carry is given in Figure 4.2.

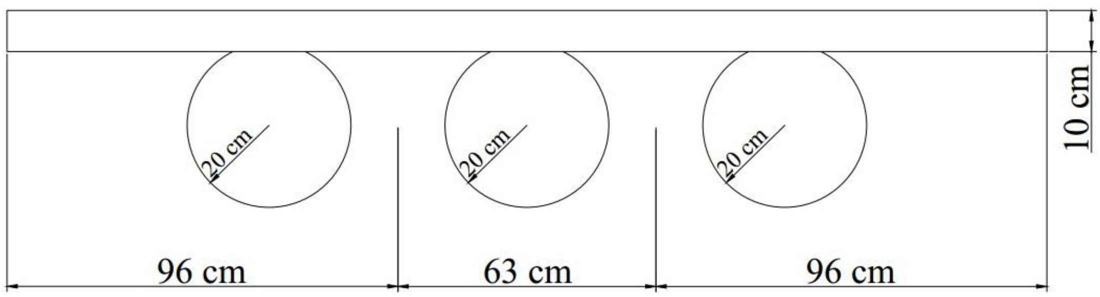


Figure 4.2 The distances of the deck plate carried by each beam

The shear flow was calculated for the sides since the dimension of the sides is bigger than the part which is in the middle. The shear flow was calculated with the equation given in (4.5).

$$q = \frac{V * Q}{I} \tag{4.5}$$

[18] "Wood Strengths", 2008, retrieved from, http://www.woodworkweb.com/woodwork-topics/wood/146-wood-strengths.html, 21st April 2015.

$$q = \frac{33.4 * 0.96 * 0.1 * (0.45 - 0.31)}{I} = \frac{33.4 * 0.0134}{0.004738} = 94.74\ \mathrm{kN/m}$$

$$q = 0.95\ \mathrm{kN/cm}$$

The width of a deck plate is approximately 22 cm; hence the shear force that each deck plate is subjected to is equal to 21 kN. The design shear capacity of a nail was determined as 4 kN as a result of the nail strength tests given in section 4.3. Therefore, at least 6 nails should be used to connect the deck plates to the main beams which are placed at sides.

Originally, the deck plates are connected to the main beams with 1-2 nails in each connection as defined from the old photographs of the bridge and previously stated in section 3.2. Although the main principle of this thesis is to rebuild the bridge in compliance with the original architectural and structural characteristics of it, small differences are allowed as long as kept in minimum to satisfy the safety requirements for the reconstruction project. For this reason, it is proposed to use 6 nails on the connections at the first 2.5 m on both sides of the span. It is admissible to use 4 nails on the connections between the 2.5 m-5 m on both sides; on the remaining connections 2 nails will be adequate. In addition, when the shear forces generated on the nails connecting the main beams and deck plates are considered (which is described in section 4.4.4), it is seen the shear forces increases on the connections between the main beams and tie beams, and the deck plates at each side of the tie beams. Therefore, it is recommended to use 4 nails on the connections between each tie beam and main beams. Also, 6 nails should be used between the deck plates on both sides of the tie beams and the main beams. The nail layout is given in Figure 4.6.

The minimum required spacing between the nails were checked based on the requirements given in Eurocode 5: Design of Timber Structrues. According to the standard, the timber should be pre drilled when the characteristic density of the timber is greater than 500 kg/m^3, and the diameter of the nail exceeds 8 mm (BSI, 2004). Additionally, in another article it is stated for nailed timber to timber connections the timber should be pre drilled if the thickness of it is smaller than the values given below in equation (4.6) and equation (4.7) which is valid if the timber is sensitive to splitting (here t is the minimum thickness of the timber to avoid pre dwelling, ρ_k is the characteristic timber density in kg/m3 and d is the nail diameter in mm) (BSI, 2004):

$$t = \max \begin{cases} 7d \\ (13d - 30)x\,\frac{\rho_k}{400} \end{cases} \tag{4.6}$$

$$t = \max \begin{cases} 14d \\ (13d - 30)x\,\frac{\rho_k}{200} \end{cases} \tag{4.7}$$

The mean value of characteristic density of the timber was defined as 0.5 g/cm^3 with the material tests. The t values calculated for both equations are given below,

For equation (4.9); $t = \max \begin{cases} 70\ mm \\ 125\ mm \end{cases}$

For equation (4.7); $t = \max \begin{cases} 140\ mm \\ 250\ mm \end{cases}$

For both cases (except 70 mm) the thickness of the deck plates, which is 100 mm, is bigger than these values, so the timber should be pre drilled according to these calculations. The maximum diameter of the drilled hole is also given in section 10 of Eurocode 5, which should not exceed 0.8 d, where d is the diameter of the nail. In the case of Buzlupınar Bridge, the diameter of the pre drilled holes shoul not exceed 8 mm. In the standard (Eurocode 5) it is also stated the pointside penetration length should be min 6d for nails other than smooth nails. This means, the pointside penetration length is 6 cm for the nails connecting the deck plates to the main beams.

The distances between the nails were determined using the equations given in section 8.3 of Eurocode 5 for pre drilled holes, which are given below:

$$a_1 = (4 + |cos\alpha|)d$$

$$a_2 = (3 + |sin\alpha|)d$$

Here a_1 shows the spacing of nails within one row parallel to grain, a_2 shows the spacing within one row perpendicular to grain. α, is the angle between the force and the grain direction and d is the diameter of the nail.

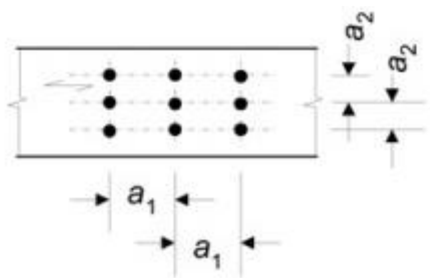


Figure 4.3 Spacing parallel to grain in a row and perpendicular to grain between rows (Eurocode 5, Section 8.3.1.2, p.69)

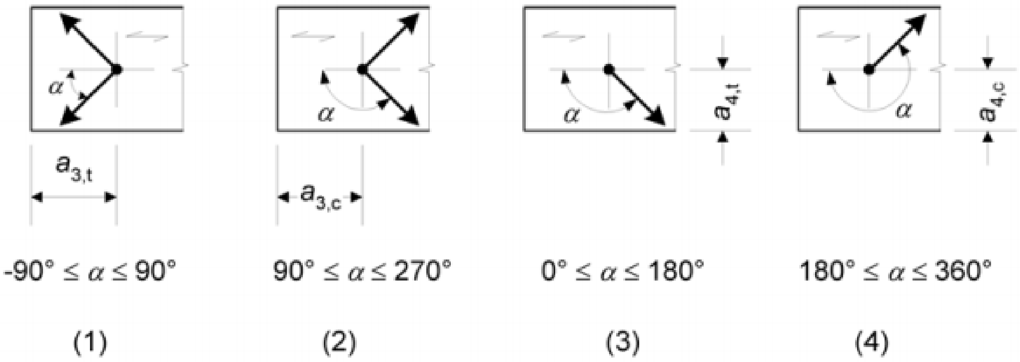


Figure 4.4 Edge and end distances (1) Loaded end, (2) Unloaded end, (3) Loaded edge, (4) Unloaded edge (Eurocode 5, Section 8.3.1.2, p.69)

The equations for a_3 and a_4 given in Eurocode 5 are as follows:

$$a_3(loaded\ end) = (7 + 5cos\alpha)d$$

$$a_3(unloaded\ end) = 7d$$

$$a_4(loaded\ edge) = (3 + 4sin\alpha)d \qquad d \geq 5mm$$

$$a_4(unloaded\ edge) = 3d$$

For shear forces, α is equal to 90° in 2-2 direction and 180°(or 0°, the critical one was used one the calculations) in 3-3 direction; and the diameter of the nail is 10 mm. Using these values minimum distances were defined as given below.

For shear forces in 2-2 direction:

$$a_1 = (4 + |cos90|)10mm = 4\ cm$$

$$a_2 = (3 + |sin90|)10mm = 4\ cm$$

$$a_3(loaded\ end) = (7 + 5cos90)10mm = 7\ cm$$

$$a_3(unloaded\ end) = 7 * 10mm = 7\ cm$$

$$a_4(loaded\ edge) = (3 + 4sin90)10mm = 7cm$$

$$a_4(unloaded\ edge) = 3 * 10mm = 3\ cm$$

For shear forces in 3-3 direction:

$$a_1 = (4 + |cos180|)10mm = 5\ cm$$

$$a_2 = (3 + |sin180|)10mm = 3\ cm$$

$$a_3(loaded\ end) = (7 + 5cos0)10mm = 12\ cm$$

$$a_3(unloaded\ end) = 7 * 10mm = 7\ cm$$

$$a_4(loaded\ edge) = (3 + 4sin0)10mm = 3\ cm$$

$$a_4(unloaded\ edge) = 3 * 10mm = 3\ cm$$

These results show that the minimum spacing between the nails should be 5 cm in the row parallel to grains and 4 in the row perpendicular to grains. The minimum value of end distance is 12 cm and edge distance is 7 cm.

The spacing of the nails was also checked for the tension forces. It was assumed the nails behave together, and imaginary section was determined as shown in Figure 4.5

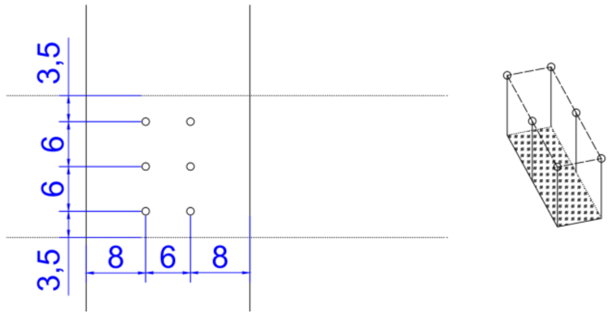


Figure 4.5 The imaginary section around the nails for the tension capacity calculations and the surfaces of timber resisting against tension forces

Dimensions of the imaginary section are 6 cm * 12 cm and the penetrated length of the nails was taken as 9 cm. Shear forces are generated on the timber at the edges of the given area in Figure 4.5 and tension forces are generated at the bottom. The areas where shear and tension forces are generated are equal to:

$$A_{shear} = (12 + 24)x9 = 324\ cm^2$$

$$A_{tension} = 12x6 = 72\ cm^2$$

A_{shear} is multiplied with shear strength of the timber whereas $A_{tension}$ is multiplied with tension strength of the timber (perpendicular to grain) as given in equation (4.8) to calculate the capacity of the timber against tension forces. The shear strength of chestnut perpendicular to grains could not be determined, but it is estimated the shear strength of a wood perpendicular to grains is greater than its shear strength parallel to grains. In the calculations for all the areas where shear forces are generated the shear strength was taken as 7400 kN/m^2=0.74 kN/cm^2 [19] and the tension strength perpendicular to grains was taken as 3200 kN/m^2 = 0.32 kN/cm^2 (United States Department of Agriculture Forest Service, 2010)

$$capacity\ of\ timber = A_{shear} x\tau + A_{tension} x\sigma \tag{4.8}$$

$$capacity\ of\ timber = 324x0.74 + 72x0.32 = 262.80\ kN$$

The tension capacity of one nail was determined as 3.24 kN and 10.08 kN when epoxy applied (section 4.3). Hence the capacity of 6 nails is equal to 19.44kN and 60.48 kN when epoxy applied. It is clearly seen the tension capacity of nails is smaller than the capacity of the timber. As a result, it can be said the spacing between the nails are adequate to meet the generated forces in tension too.

[19] "Wood Strengths", 2008, retrieved from, http://www.woodworkweb.com/woodwork-topics/wood/146-wood-strengths.html, 21st April 2015.

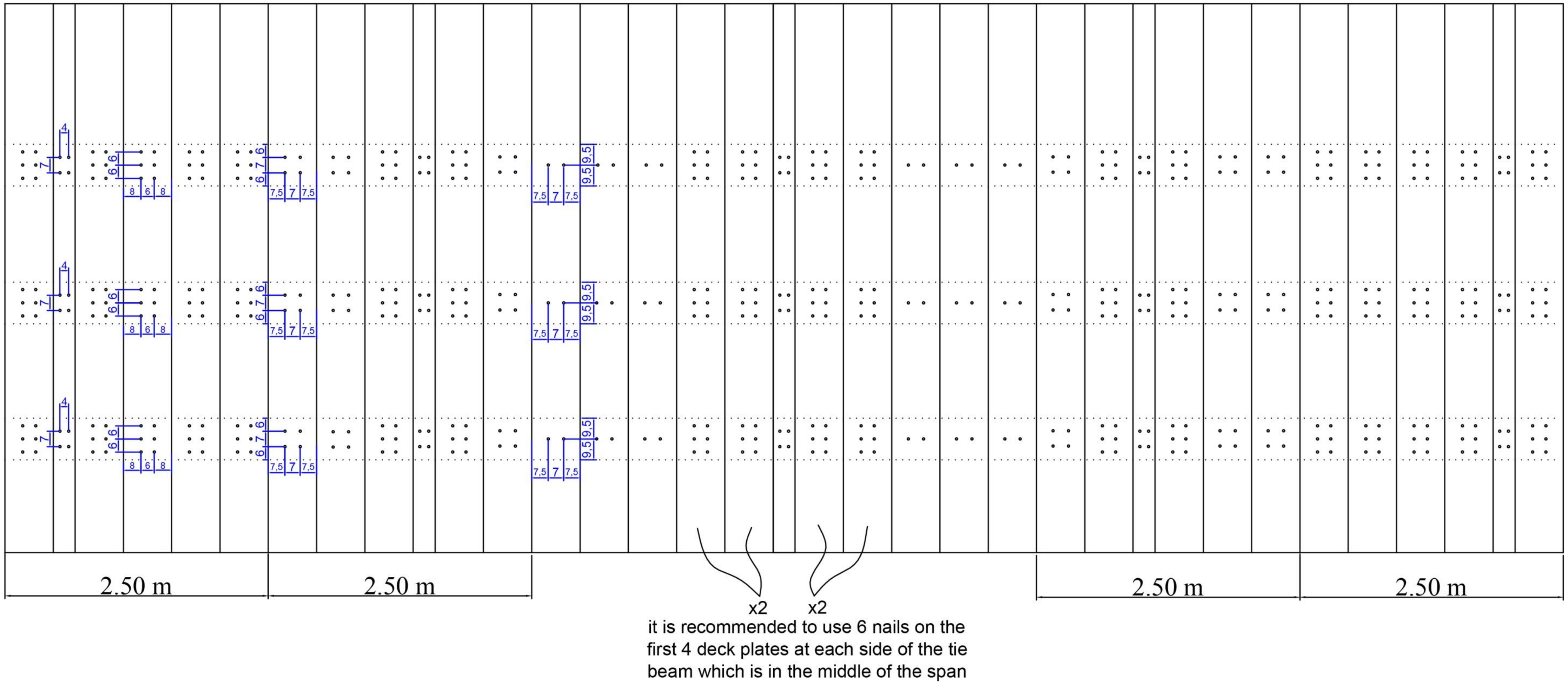


Figure 4.6 Layout of the nails which connect the deck plates to the main beams

4.2.2 Axial and Shear Forces and Overturning Moment Acting on the Cantilever Members

The loads acting on the bridge are transferred to the ground as shown in Figure 4.7; the total load is shared by two supports formed by the cantilever beams; then it is transferred to the ground over the cantilever beams.

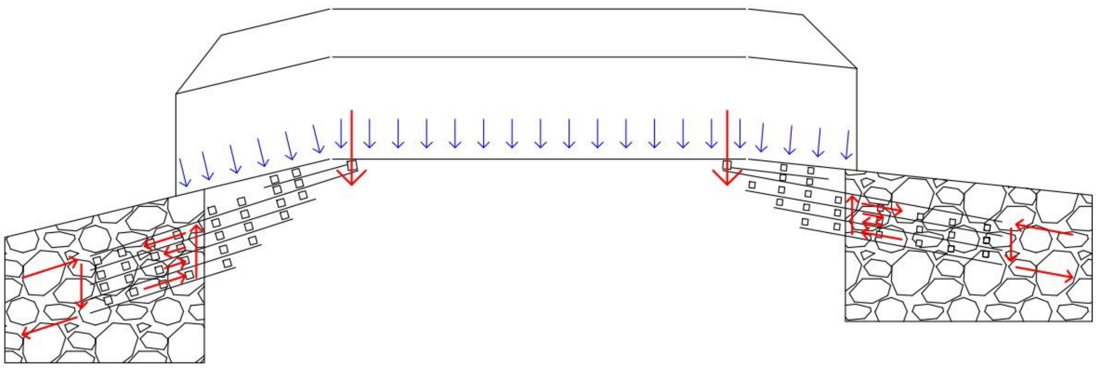


Figure 4.7 Load transfer mechanism of the bridge (Drawn by Ezgi Çabuk)

Because of the horizontal beams which are perpendicularly placed between the cantilever beams, all of the cantilever beams behaves together as a single structural member. As previously defined, the maximum live load that the bridge could be exposed to presumed to be bovine animals, each is approximately 500 kg and 1.5 m tall. The total span of the bridge is approximately 22 m; if the bovine animals are passing over the bridge with 0.5 m intervals then maximum 11 bovine animals could be on the bridge at the same time. The total live load on the bridge in this situation is equal to:

$$\mathrm{LL} = 11 * 500 = 5500\ \mathrm{kg} \ = 55\ \mathrm{kN}$$

The design load for the places where people could concentrate on given in TS 498 is 5 kN/m^2. The active width of the bridge which means that it is possible to stand on is approximately 1.85m, and then the live load per unit length of the b1ridge is equal to:

$$\mathrm{LL} = 5 * 1.85 * 22.4 = 207.20\ \mathrm{kN}$$

Since 207.2 kN is far greater than 55 kN, 207.2 kN was used as the total design live load value. The total weight of the bridge was calculated approximately 10304 kg = 103 kN and the total load on the bridge is equal to:

$$\mathrm{DL} + \mathrm{LL} = 207.2 + 103 = 310.2\ \mathrm{kN}$$

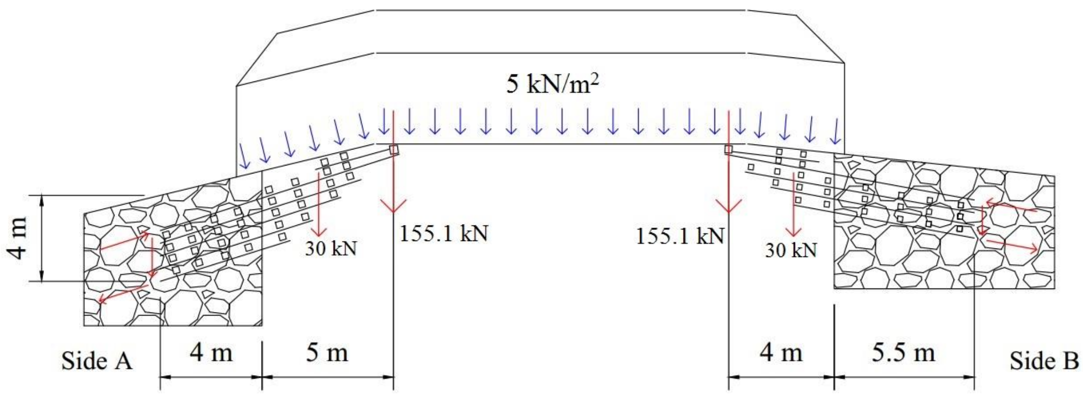


Figure 4.8 The total load acting on the bridge and the dimension used for calculations (Drawn by Ezgi Çabuk)

As previously defined, it was assumed the total load acting on the bridge is almost equally shared between the two supports at each ends. In this case, each group of cantilever beams supports 155.1 kN (Figure 4.8). The own weight of the cantilever beams out of the soil at each side is calculated as 25 kN and taken as 30 kN at the calculations to be on the safe side. The calculations were made for the side A since the length of the cantilever beams inside soil is less than in side B. The dimensions and loads used on calculations are as shown in Figure 4.8. Accordingly, the overturning moment formed on the cantilever beams is equal to:

$$M_o = 155.1 * 5 + 30 * 2 = 840 \text{ kN.m}$$

The resisting moment against the overturning moment is formed by the soil, which is 4 m in deep and length, and 3 m in width. The specific gravity of the soil is taken as 15 kN/m^3 since there are some discontinuities on the soil because of the timber beams. In this circumstance, the resisting moment formed by the soil is equal to:

$$M_r = 4 * 4 * 3 * 15 * 2 = 1440 \text{ kN.m}$$

The factor of safety against overturning is:

$$FS = \frac{M_r}{M_o} = \frac{1440}{840} = 1.7$$

FS=1.7 is quite admissible, however since the lengths of the cantilever beams inside soil do not affect the view of the bridge, it is proposed to increase the length of the cantilever beams inside soil 1 m in part A.

Another calculation for cantilever beams was done for the shear force between the cantilever beams and horizontal beams placed between them. The force acting on each row is shown in Figure 4.9 and the forces acting in each cantilever beam is shown in Figure 4.10.

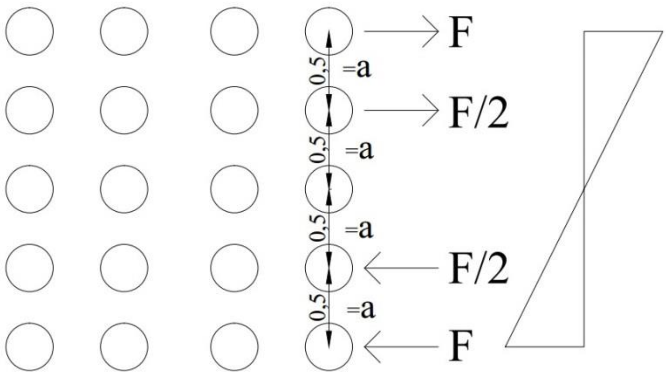


Figure 4.9 Axial forces acting in each row of cantilever beams (cross section)

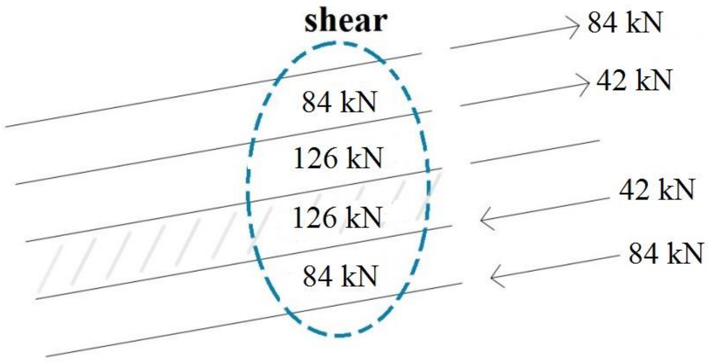


Figure 4.10 Axial and shear forces acting on each cantilever beam

$$\mathrm{M} = 840\ \mathrm{kN.m}$$

$$2\left(\mathrm{F} * 2\mathrm{a} + \frac{\mathrm{F}}{2} * \mathrm{a}\right) = 840\ \mathrm{kN.m}$$

$$\mathrm{F} + \frac{\mathrm{F}}{4} = 420\ \mathrm{kN}$$

$$\mathrm{F} = 336\ \mathrm{kN}$$

Since there are 4 beams at each row; the axial force acting on each beam is equal to 84 kN at top and bottom rows, 42 kN at the inner rows, and 0 at the beams which are in the middle row. The more critical part of cantilever beams-horizontal beams connection is shown with shaded area. In this part of the system, there are 6 nails as for the restoration project; this means each nail should bear 21.4 kN. The strength of the nails that are going to be used on this part of the bridge is defined as 50 kN as a result of the nail tests. If this value is decreased with a safety factor of 2, then the strength value becomes 25 kN. The factor of safety for this situation is equal to:

$$\mathrm{FS} = \frac{25}{21.4} = 1.17$$

The load transfer capacity of the soil between the cantilever beams horizontal beams was not taken into account for this calculation. When it is considered, value of factor of safety is expected to be increased.

4.2.3 Wind Forces Acting on the Bridge

As stated by the locals, the bridge had collapsed in 2008 because of the heavy wind that blew through the valley in which it is located. Therefore, detailed hand calculations were made about the wind forces acting on the bridge according to the parameters given in TS 498: Design Loads for Buildings.

According to the measured surveys of the bridge, the deck plates and main beams are 8 m above the streambed elevation. The height of the bridge was increases with the roof covering, thus the total height was taken as 9-20 m above the ground elevation. For that height interval, the velocity of the wind is given as 36 m/s and the velocity pressure is given as q=0,8 kN/m^2.

Different assumptions were made about the form of the cover type of the bridge for the wind analyses. These assumptions can be divided into two as one side and two sides covered forms of the bridge. Two sides covered analyses were made both for assuming there was no gap between the side covering plates and deck plates so the bridge behaves as a box as given in Figure 4.11; and there was gap between the side covering plates and deck plates so the wind can blow through the bridge as given in Figure 4.13, with chancing the percentage of the side covering. The maximum value of factor of safety was calculated as 0.65 in either of the cases of two sides covered form of the bridge. The graphs showing the change in factor of safety with the change in the percentage of the covered sections for these two situations are given in Figure 4.12 and Figure 4.14.

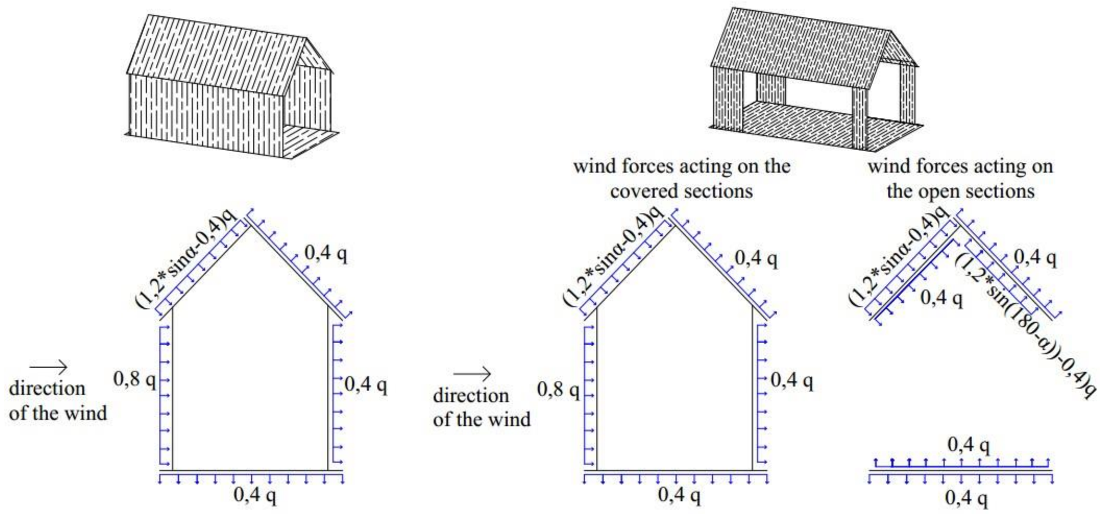


Figure 4.11 The wind forces acting on the two sides covered form of the bridge with no gap between the side covering plates and deck plates

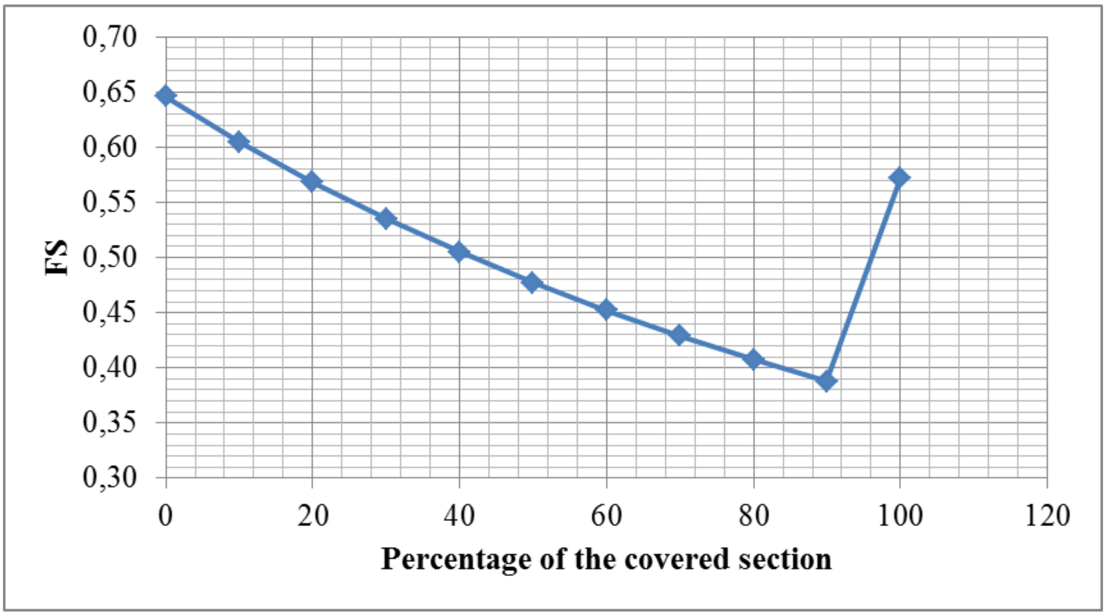


Figure 4.12 FS values changing with the percentage of the covered section for the situation given in Figure 4.11

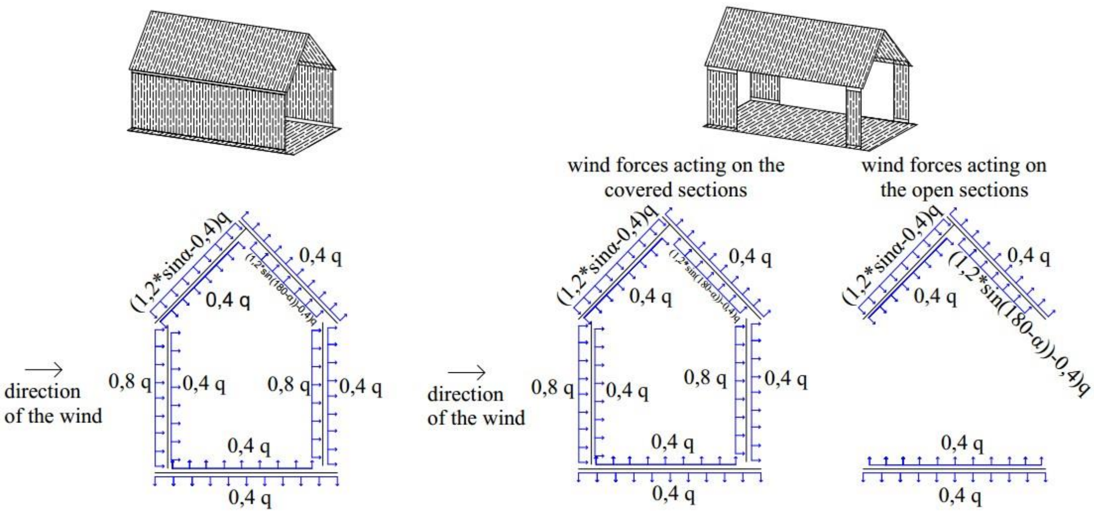


Figure 4.13 The wind forces acting on the two sides covered form of the bridge with gaps between the side covering plates and deck plates

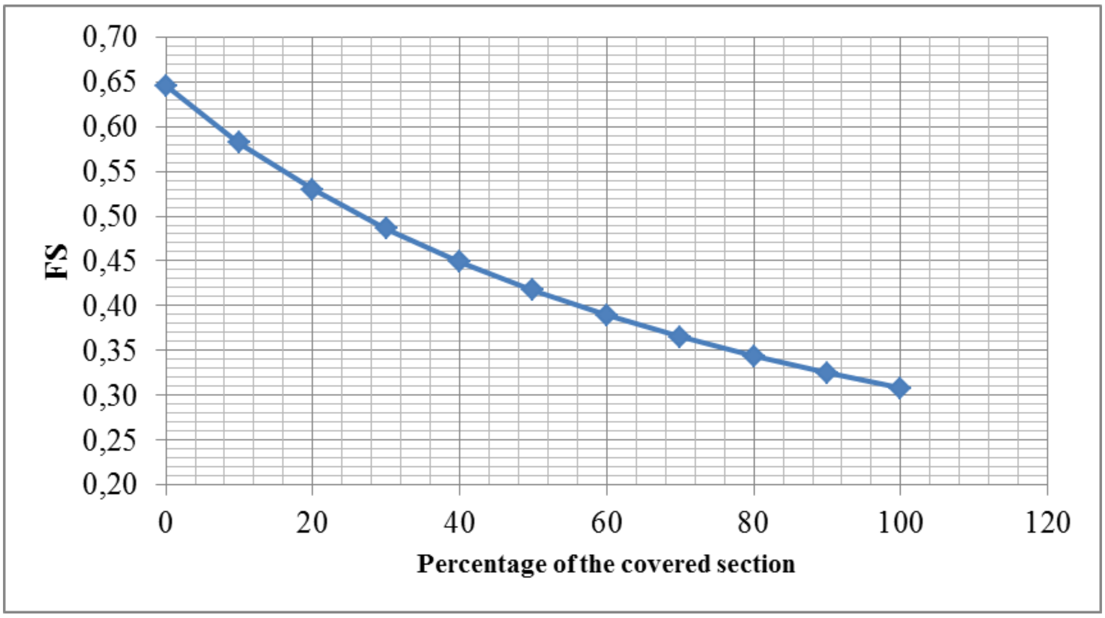


Figure 4.14 FS values changing with the percentage of the covered section for the situation given in Figure 4.13

The hand calculations for the one side covered form of the bridge were done for the same cases with the two sides covered form of the bridge; assuming there is no gap between the covering plates and deck plates so that blowing effect occurs inside the bridge and assuming there was a gap between the covering plates and deck plates so the wind can blow through the bridge. The wind forces acting on the bridge for the one side covered form are given in Figure 4.15 and Figure 4.17. For the one side covered form of the bridge, the highest value of the factor of safety was calculated as 0.72 in the case that there was no gap between the deck plates and side covering plates when the side is covered 100%. The graphs showing the relation between the factor of safety and percentage of covered section for these assumptions are given in Figure 4.16 and Figure 4.18.

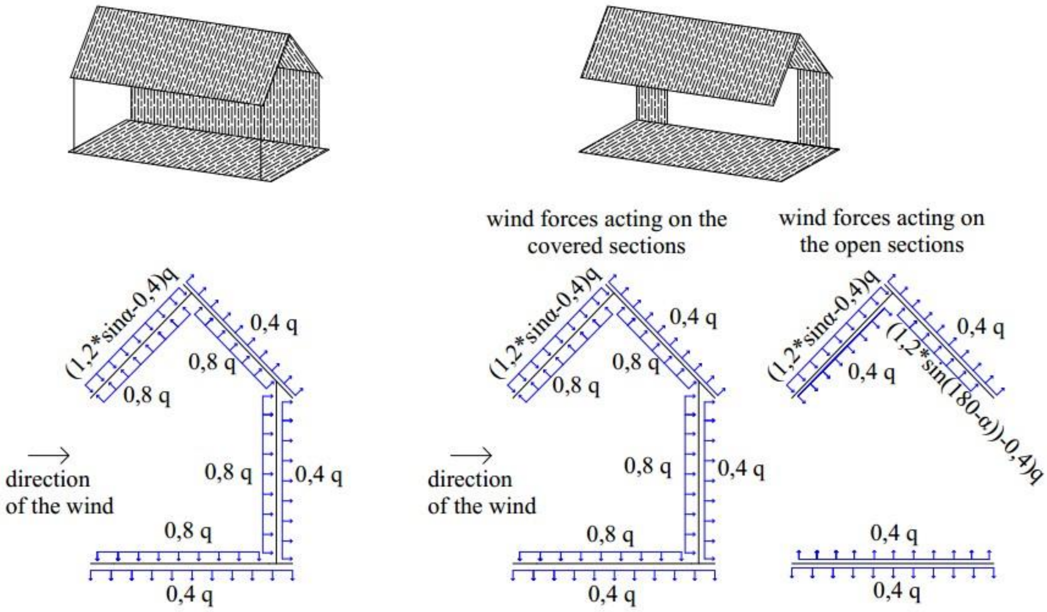


Figure 4.15 The wind forces acting on the one side covered form of the bridge with no gaps between the side covering plates and deck plates

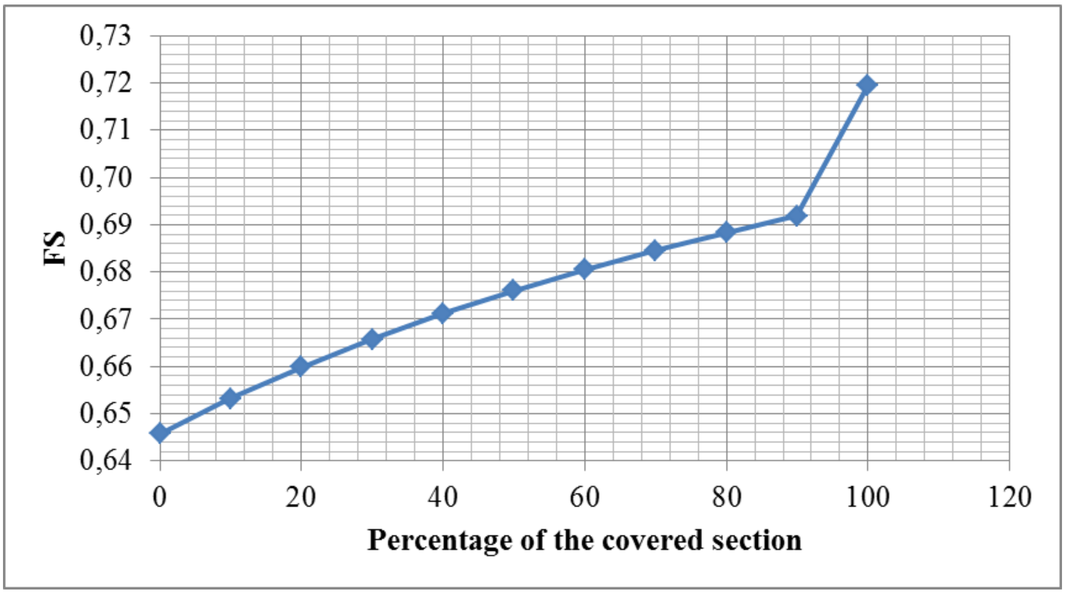


Figure 4.16 FS values changing with the percentage of the covered section for the situation given in Figure 4.15

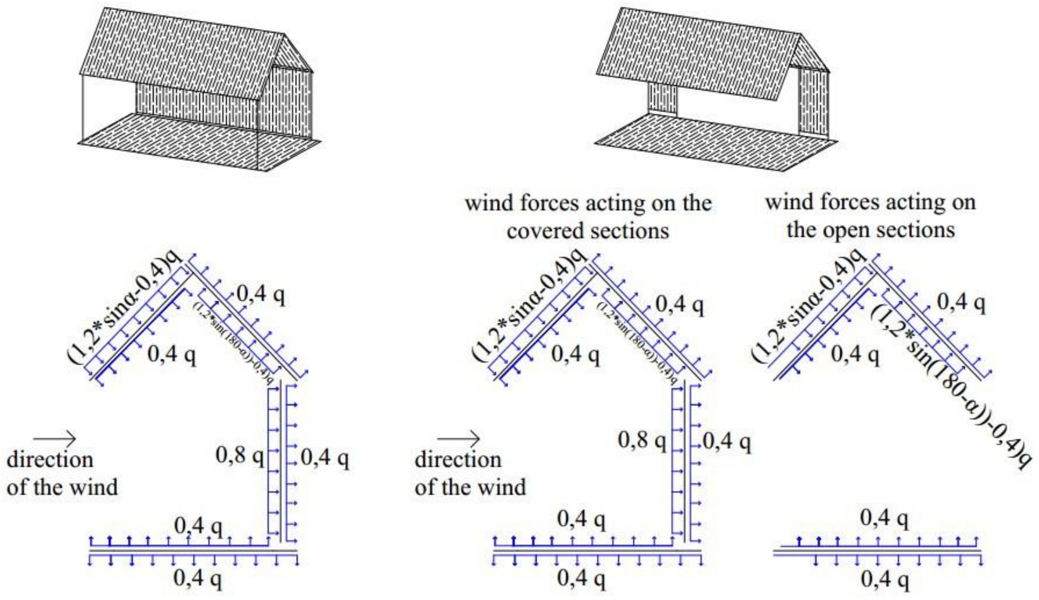


Figure 4.17 The wind forces acting on the one side covered form of the bridge with gaps between the side covering plates and deck plates

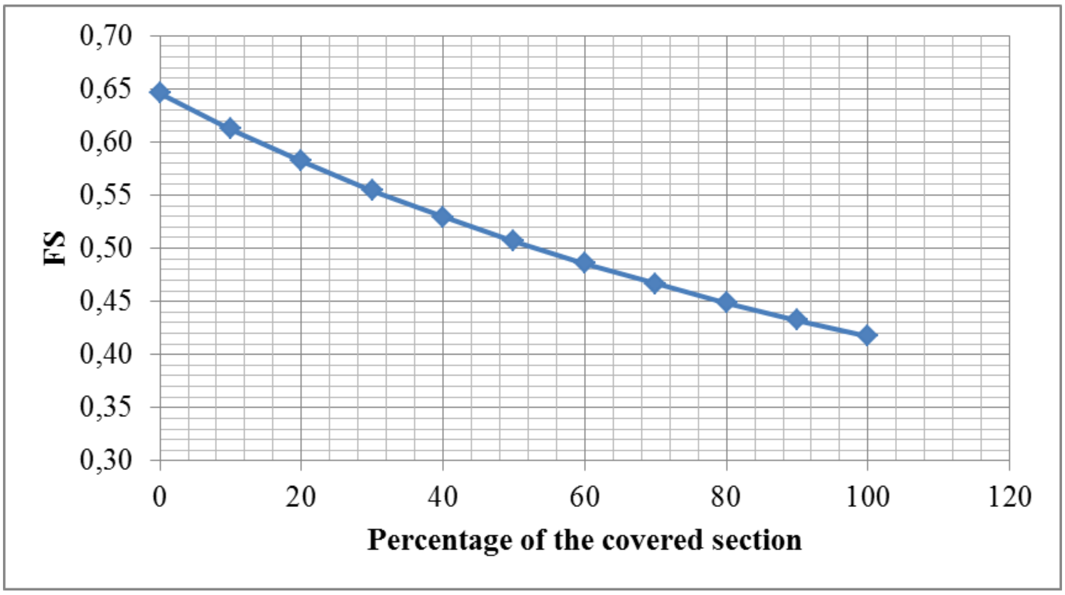


Figure 4.18 FS values changing with the percentage of the covered section for the situation given in Figure 4.17

The hand calculations for the one side covered form of the bridge were also done presuming the covering of the bridge was rotating with the effect of the wind loads, as it happened when the bridge was collapsed. These calculations were made assuming there are gaps between the deck plates and side covering plates, considering both the different percentages of the covered section and rotation of the bridge with the wind forces. The graph showing the estimated factor of safety values for this situation is given in Figure 4.19.

In its original form, there were gaps between the side covering plates and deck plates as shown on the old photos. Also if the difficulty of constructing the plates without any gaps is considered; the one side covered form with gaps between the covering plates and deck plates is the most realistic form of the bridge. In this situation the highest value of factor of safety was calculated as 0.65, which means the overturning moment generated by the wind forces is bigger than the resisting moment. The reconstruction project should be based on the original structural and architectural features of the bridge, further calculations and SAP2000 analyses were based upon the one side covered form of the bridge with gaps between the side covering plates and deck plates. From the calculations done up to now, it is clearly seen the bridge is not safe against wind forces; therefore the structural proposals for the reconstruction project of the bridge concentrated on strengthening the bridge against wind forces. Special care was given to the analysis of the connections between the deck plates and main beams.

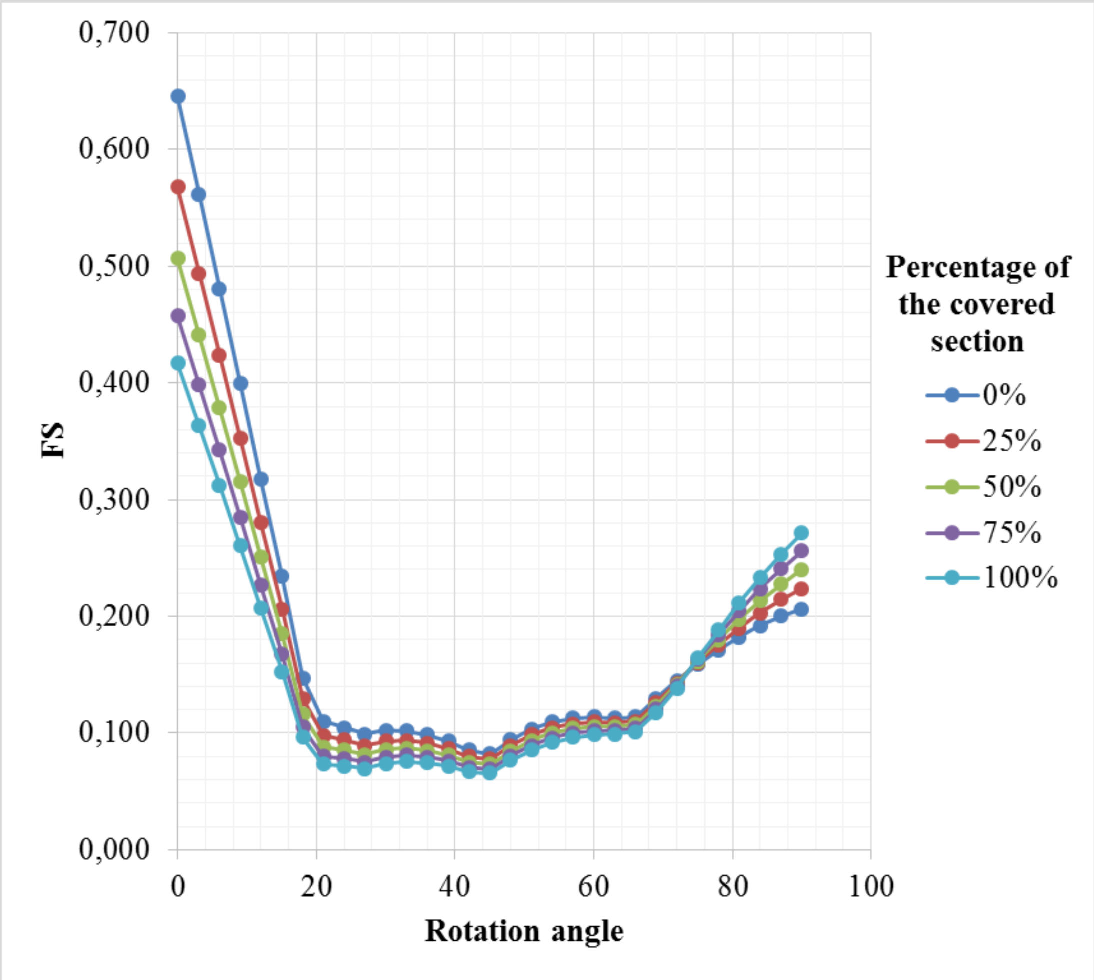


Figure 4.19 FS values changing with the rotation of the bridge for different percentages of the covered section, for one side covered form covered form of the bridge with gaps between the deck plates and side covering plates

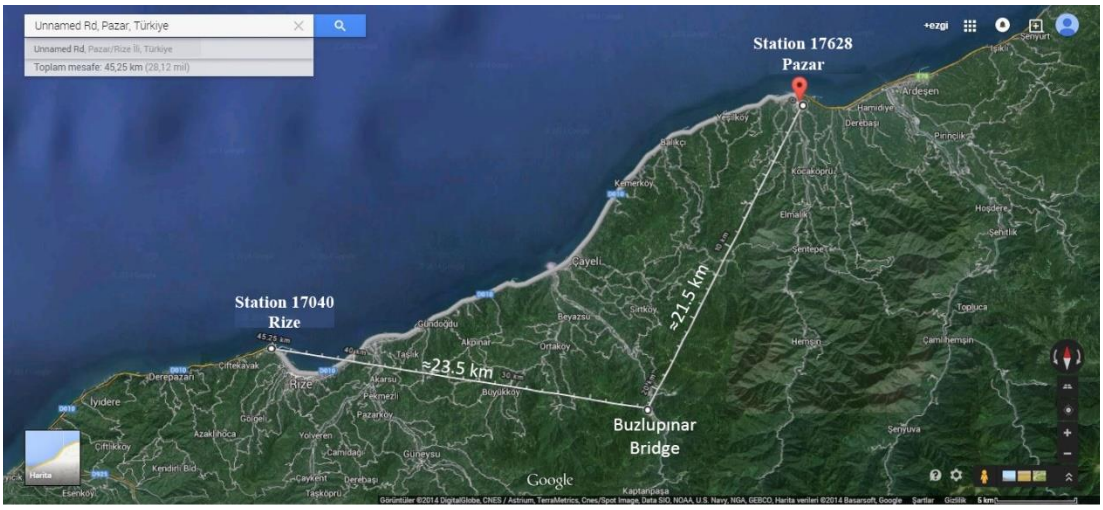


Figure 4.20 The meteorological stations in Rize and Pazar (retrieved from https://www.google.com/maps in 17 December 2014)

In the second part of the study, meteorological data consisting of mean and maximum velocities of the wind forces in Rize for the years from 2003 to 2013 were gathered from Turkish State Meteorological Service. Both the mean and maximum velocities were measured in two different meteorological stations shown Figure 4.20. A retrospective statistical analysis was done using the maximum velocity data measured in both of the stations in order to estimate the approximate velocity of the wind for 100 years return period. The velocity values are given in Table 4.1.

Table 4.1 Maximum Velocity Values for 100 Years Return Period

	Station in Rize		Station in Pazar	
Velocity (m/s)	Day	Year	Day	Year
0				
2	1,08	0,00		
4	1,93	0,01		
6	3,44	0,01	2,39	0,01
8	6,14	0,02	4,49	0,01
10	10,95	0,03	8,42	0,02
12	19,53	0,05	15,78	0,04
14	34,84	0,10	29,59	0,08
16	62,15	0,17	55,48	0,15
18	110,88	0,30	104,02	0,28
20	197,81	0,54	195,04	0,53
22	352,89	0,97	365,70	1,00
24	629,57	1,72	685,68	1,88
26	1123,16	3,08	1285,64	3,52
28	2003,74	5,49	2410,57	6,60
30	3574,72	9,79	4519,81	12,37
32	6377,37	17,46	8474,62	23,20
34	11377,37	31,15	15889,87	43,50
36	20297,47	55,57	29793,42	81,57
37	27110,77	74,23	40796,26	111,69
38	36211,11	99,14	55862,49	152,94
39	48366,18	132,42	76492,76	209,43
40	64601,38	176,87	104741,87	286,77
41	86286,29	236,24	143423,50	392,67

The velocities of the wind for 100 years return period were determined as 38m/s according to the data of the station in Rize and 36.6 m/s according to the data of the station in Pazar. Hence the velocity pressures are defined as 0.90 kN/m^2 and 0.84 kN/m^2 using the equation (4.9). The wind analyses were repeated for 100 years return periods using these velocity pressures and factors of safeties for both situations were determined as seen in Figure 4.21 and Figure 4.22.

$$q = \frac{V^2}{1600}\ \mathrm{kN/m^2} \tag{4.9}$$

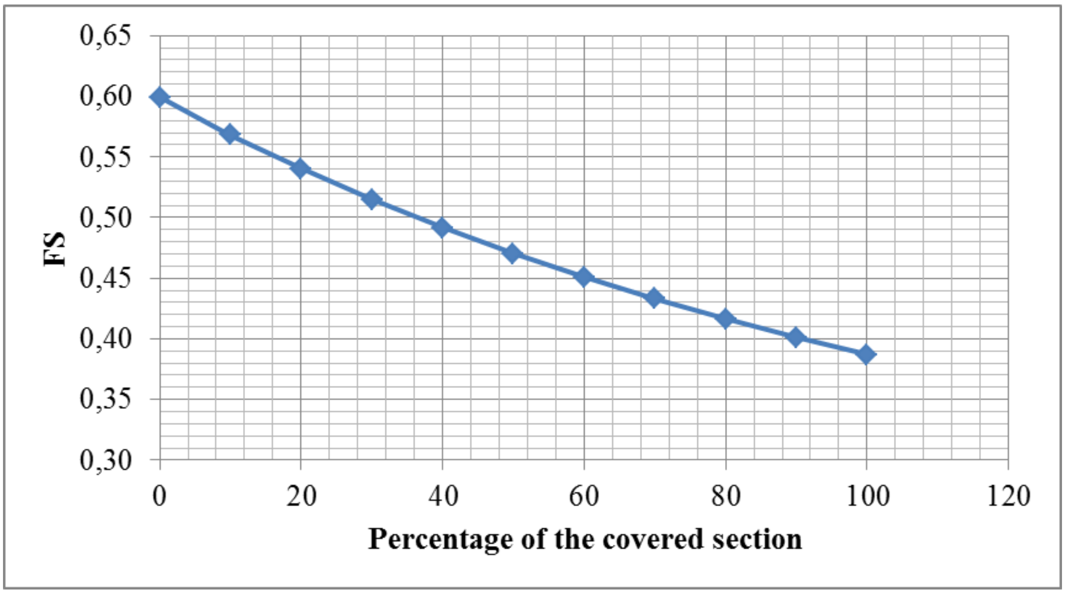


Figure 4.21 Factor of safety values for one side covered form of the bridge with gaps between the deck plates and side covering plates, with velocity pressure valueof 0.90 kN/m^2

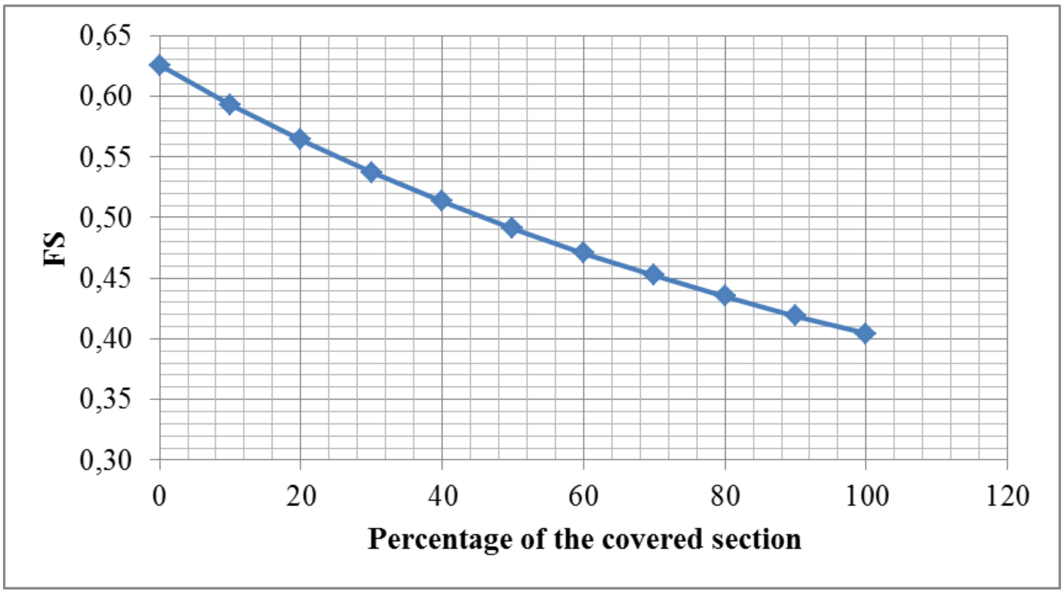


Figure 4.22 Factor of safety values for one side covered form of the bridge with gaps between the deck plates and side covering plates, with velocity pressure value of 0.84 kN/m^2

4.3 Material Tests on New Timber and Nails to be used for the Reconstruction

Material tests were done to determine the mechanical parameters of the timber and nails which are going to be used for the reconstruction project of the bridge. The timber tests were done in accordance with the ASTM-D143. Bending, compression parallel to grains, compression perpendicular to grains, and tension parallel to grains tests were on the timber specimens. The timber and nail specimens which are going to be used on the reconstruction project were supplied by Zülfikar Halifeoğlu.

4.3.1 Bending

Four timber specimens were prepared in 25 mm*25 mm*350 mm dimensions (shown in Figure 4.23) as stated in ASTM-D143 and allowed to dry until their weight fixed at a constant rate. The specimens were placed to the testing apparatus with a span distance of 300 mm as shown in Figure 4.24. The load is applied from the centre of the specimen on its transversal surface with a velocity of 1.3 mm/min. The results of the tests are given in Table 4.2, Table 4.3, and Table 4.4; the failure types of the specimens are shown in Figure 4.25, Figure 4.26, Figure 4.28, Figure 4.30, and Figure 4.32; load-deflections graphs are given in Figure 4.27, Figure 4.29, Figure 4.31, Figure 4.33.

Figure 4.23 Bending test specimens

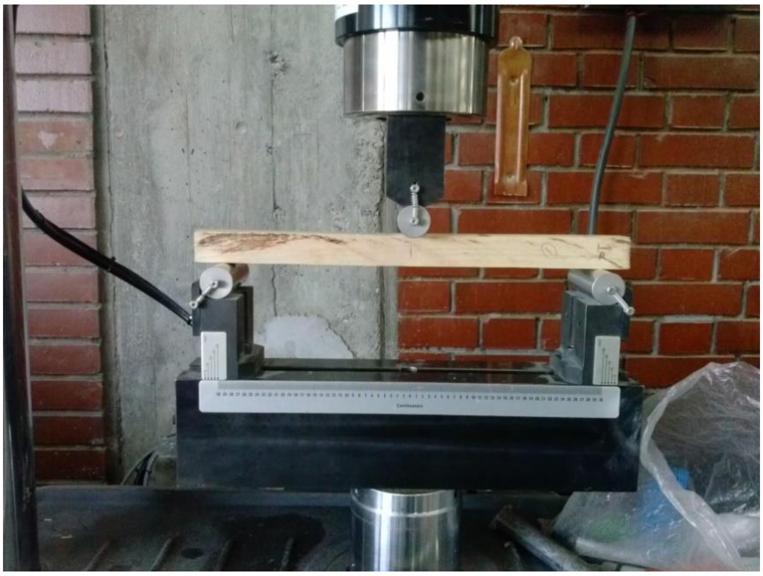
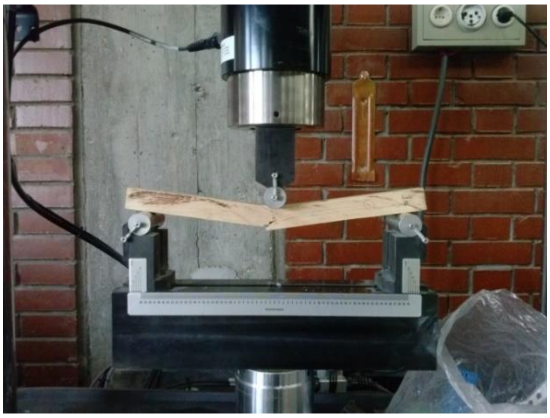
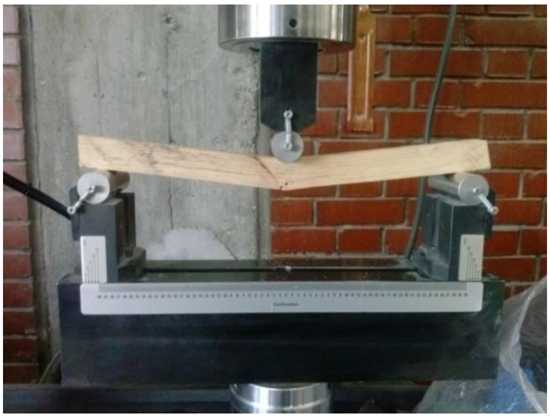


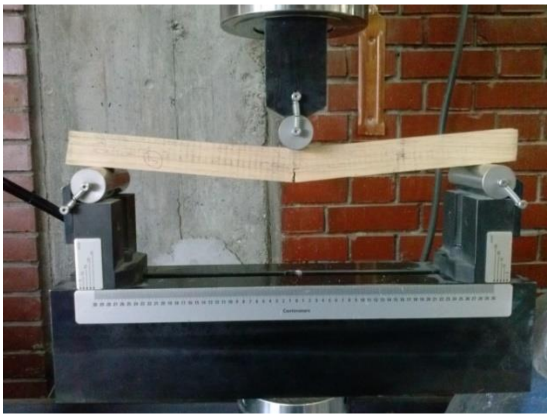
Figure 4.24 Testing apparatus



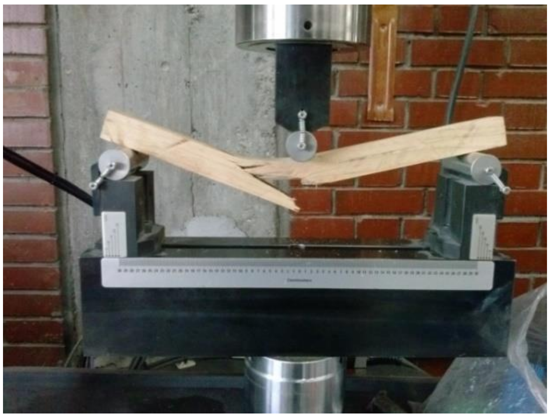
Specimen 1



Specimen 2



Specimen 3



Specimen 4

Figure 4.25 Failure type of the specimen 1, 2, 3, and 4 after testing (simple tension under bending)

Figure 4.26 Failure type of the specimen 1 after testing (simple tension under bending)

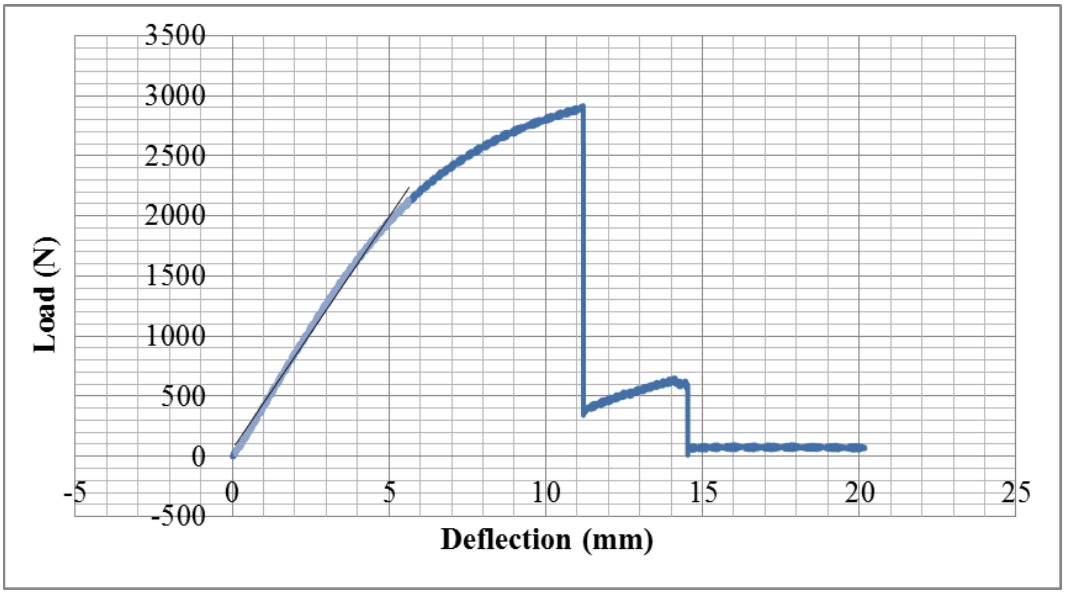


Figure 4.27 Load-Deflection graph of specimen 1

Figure 4.28 Failure type of specimen 2 after testing (simple tension under bending)

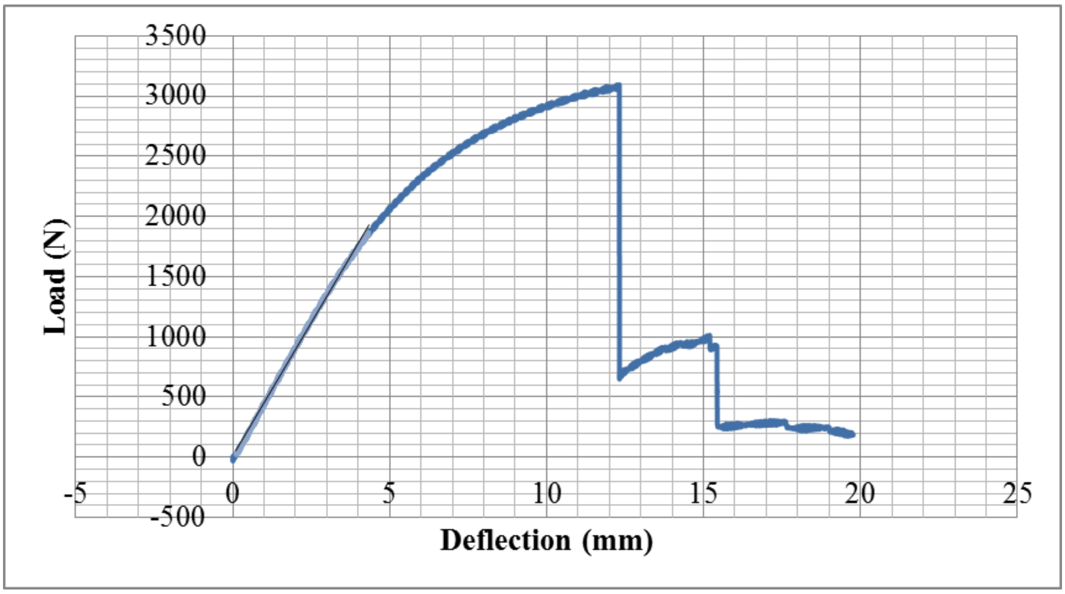


Figure 4.29 Load-Deflection graph of specimen 2

Figure 4.30 Failure type of the specimen 3 after testing (simple tension under bending)

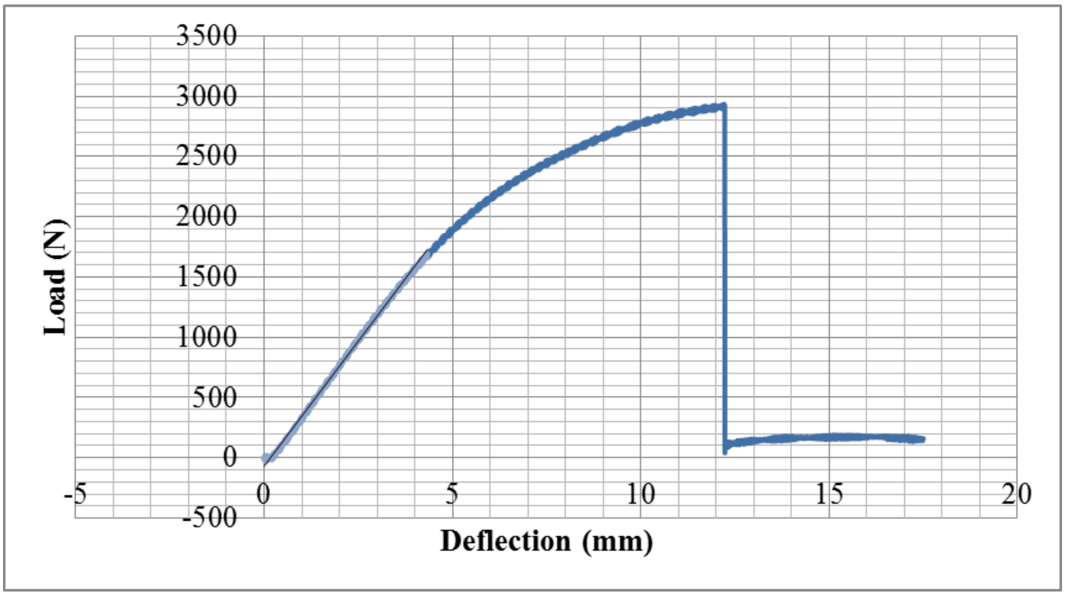


Figure 4.31 Load-Deflection graph of specimen 3

Figure 4.32 Failure type of the specimen 4 after testing (simple tension under bending)

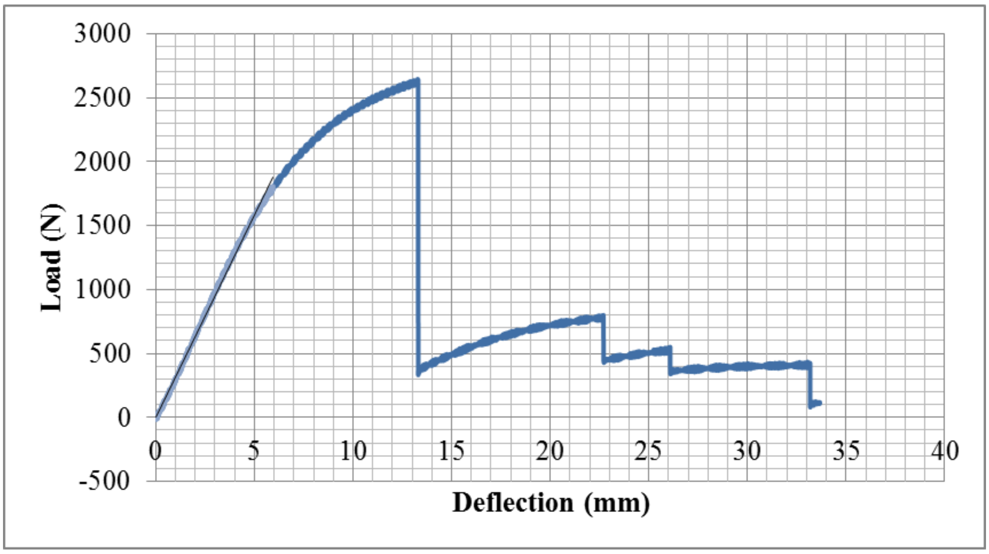


Figure 4.33 Load-Deflection graph of specimen 4

The bending strength and elastic modulus of the specimens were calculated using equations (4.10), (4.11), (4.12), and (4.13). For all of the specimens; the cross-section is 25*25; b= 25 mm, d= 25 mm, length of the speciman is 350 mm; L= 350 mm and span length is 300 mm; Lspan= 300 mm.

$$\text{Bending stress} = \sigma = \frac{M * c}{I} \quad \textbf{(4.10)}$$

$$M = \frac{P * Lspan}{4} \quad \textbf{(4.11)}$$

$$\frac{b}{2} = \frac{25}{2} = 12.5 \text{ mm}$$

$$I = \frac{d * b^3}{12} = \frac{25 * 25^3}{12} = 32552.08 \text{ mm}^4$$

Table 4.2 Bending strength values of the specimens in bending

Specimen No	Load (N)	Moment (N.mm)	Bending Strength (N/mm2)
1	1689.72	126729.00	48.66
2	1571.57	117867.75	45.26
3	1656.55	124241.25	47.71
4	1656.47	124235.25	47.71

$$\delta = \frac{P * L_{span}^3}{48 * E * I} \quad \textbf{(4.12)}$$

$$E = \frac{P * L_{span}^3}{48 * I * \delta} \quad \textbf{(4.13)}$$

Table 4.3 Mean value of the bending strength and standard deviation

Mean Strength (N/mm2)	Standard Deviation
47.34	1.45

Table 4.4 Modulus of elasticity values at proportional limit

Specimen No	Maximum Elongation (mm)	Modulus of Elasticity (N/mm2)
1	4.16	7018.84
2	3.54	7671.39
3	4.27	6703.79
4	5.36	5340.26
Mean	4.33	6683,57

4.3.2 Compression Parallel to Grain

Four timber specimens were prepared in 50 mm*50 mm*200 mm dimensions and dried until their weights fixed at a constant rate. The load is applied at a rate of 0.003 mm/mm with a testing apparatus as shown in Figure 4.34. The deformed shapes of the specimens are shown in Figure 4.35, Figure 4.36, Figure 4.37, and Figure 4.38. The stress-strain graph of the specimens is shown in Figure 4.39. Load-compression and stress-strain graphs of each specimen are given in Figure 4.40 and Figure 4.41.

Figure 4.34 Testing apparatus for the compression test

Figure 4.35 Specimen 1 (shearing)

Figure 4.36 Specimen 2 (shearing)

Figure 4.37 Specimen 3 (brooming and shearing)

Figure 4.38 Specimen 3 (brooming at bottom)

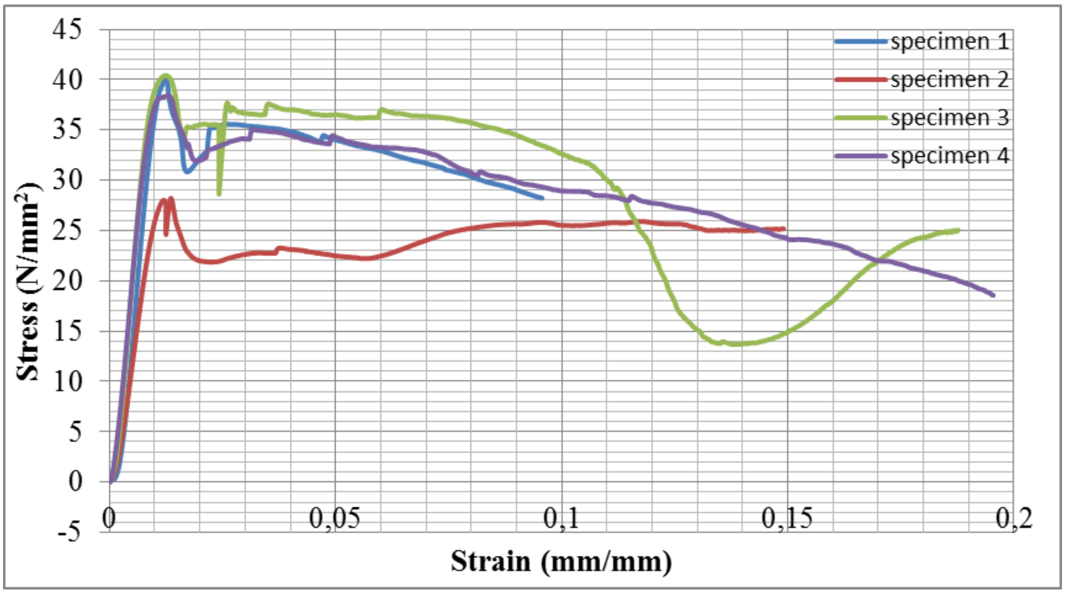
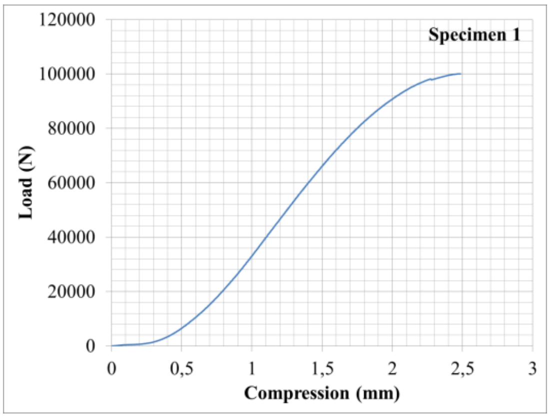


Figure 4.39 Stress-Strain graph of the specimens in compression parallel to grains



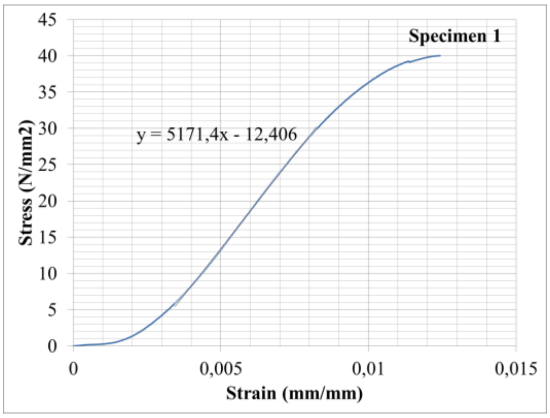


Figure 4.40 Load compression and stress-strain graphs of the specimens 1 in compression parallel to grains test

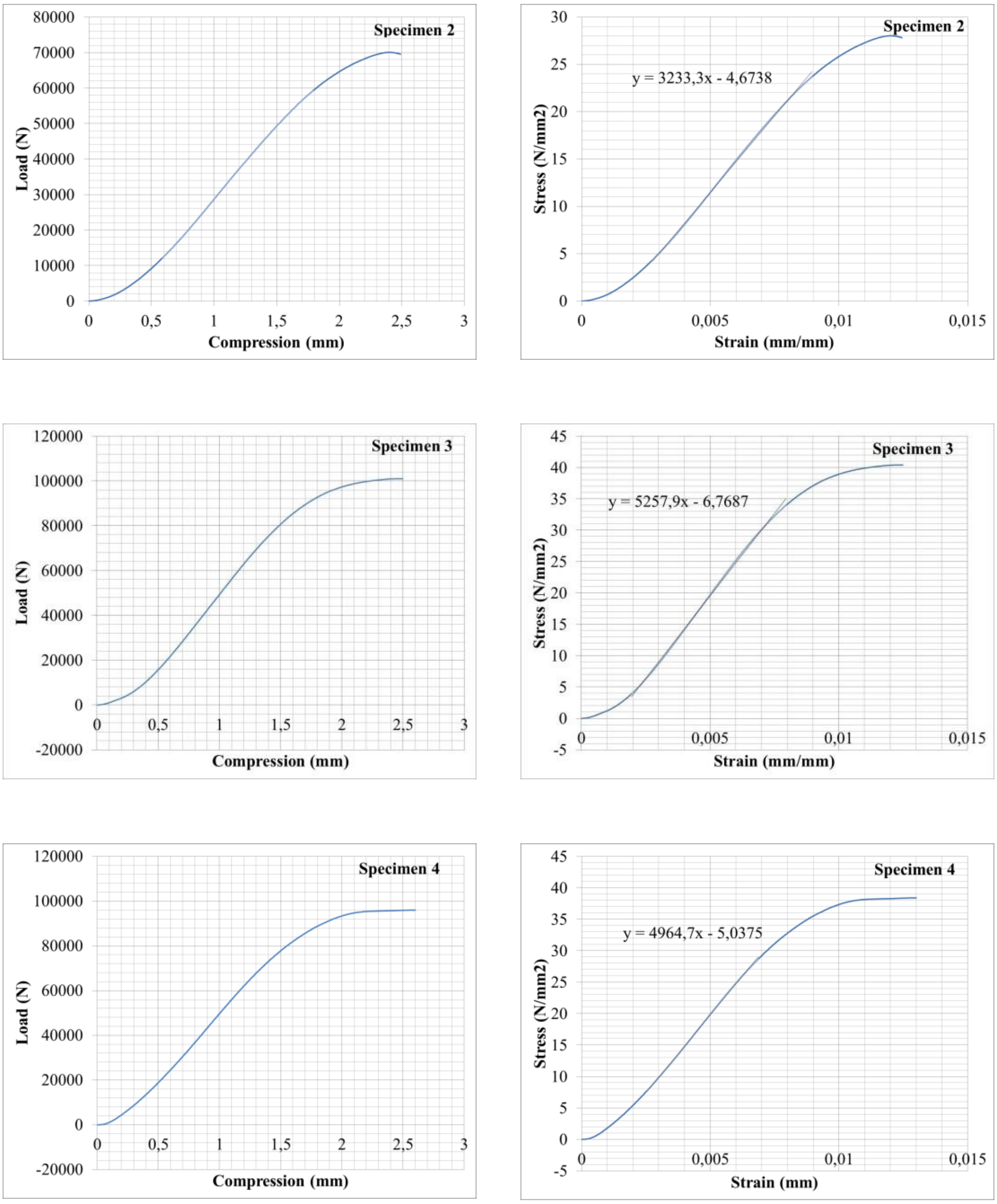


Figure 4.41 Load compression and stress-strain graphs of the specimens 2, 3, and 4 in compression parallel to grains test

The strength values given in Table 4.5 and Table 4.6 were calculated using equation (4.14). The modulus of elasticity values are given in Table 4.7. For all specimens:

$$A_o = 25 * 25 = 2500\ mm^2$$

$$\sigma = \frac{P}{Ao} \tag{4.14}$$

Table 4.5 Maximum load and stress values at proportional limit

Specimen No	Load (N)	Compression Strength Parallel to Grains (N/mm2)
1	89532.00	35.81
2	64127.22	25.65
3	90538.54	36.22
4	88068.60	35.23

Table 4.6 Mean Stress and Standard deviation

Mean Strength (N/mm2)	Standard Deviation
33.23	5.07

Table 4.7 Modulus of Elasticity of the values at proportional limit

Specimen no	Modulus of Elasticity (N/mm^2)
1	3645.07
2	2594.93
3	4184.33
4	3969.29
Mean	3598.41

4.3.3 Compression Perpendicular to Grains

The compression test were made on four 50 mm*50 mm*150 mm specimens as specified in ASTM-D143. The load was applied through a metal bearing plate 50 mm in width, placed at centre of the specimen on its radial surface with a velocity of 0.305 mm/dk; the testing apparatus is shown in Figure 4.42. Load-deflection readings were taken up to 2.5 mm compression as stated in the standard. For the specimens 2 and 3, loading was continued after 2.5 mm deflection. The deflected shapes of the specimens are shown in Figure 4.43, Figure 4.44, Figure 4.45, and Figure 4.46. The load-compression curves for all specimens are shown in Figure 4.48 and for the specimens 2 and 3 are shown in Figure 4.48.

Figure 4.42 Testing apparatus for the compression test

Figure 4.43 Failure type of the specimen 1 after loading

Figure 4.44 Failure type of the specimen 2 after loading

Figure 4.45 Failure type of the specimen 3 after loading

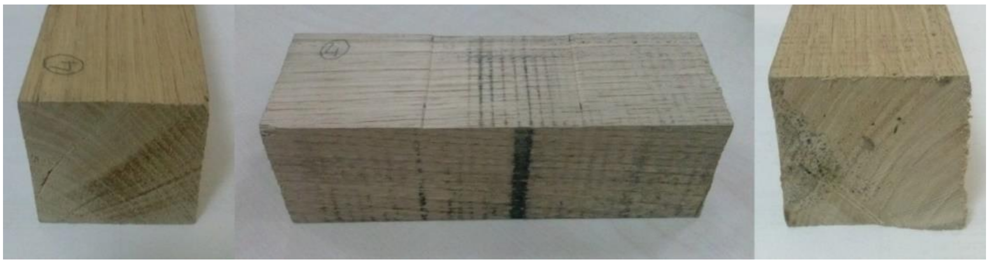


Figure 4.46 Failure type of the specimen 4 after loading

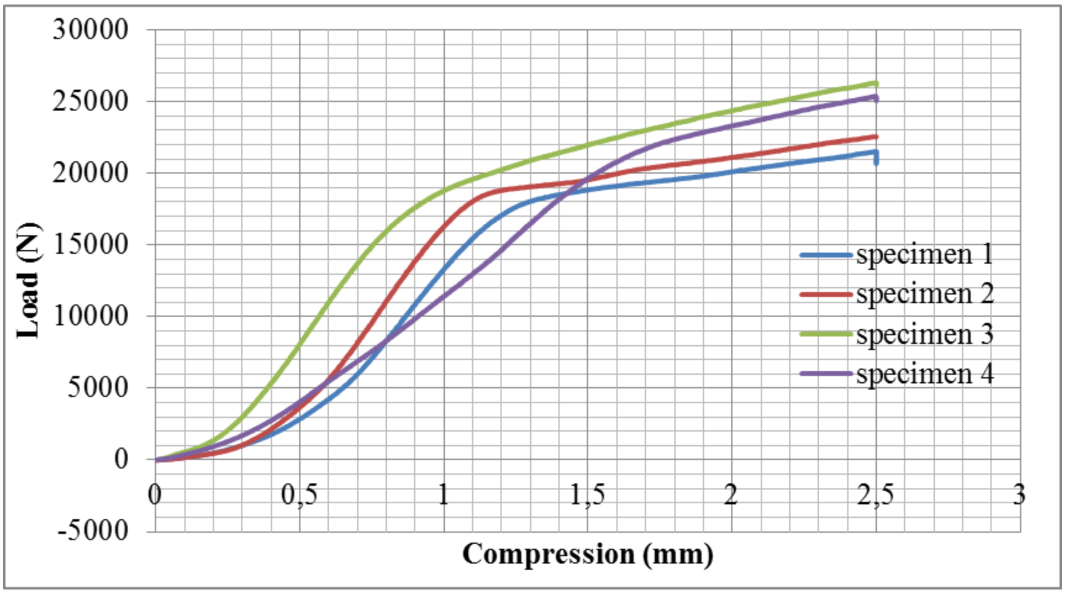


Figure 4.47 Load-compression curves of the specimens 1, 2, 3, and 4 in compression perpendicular to grains

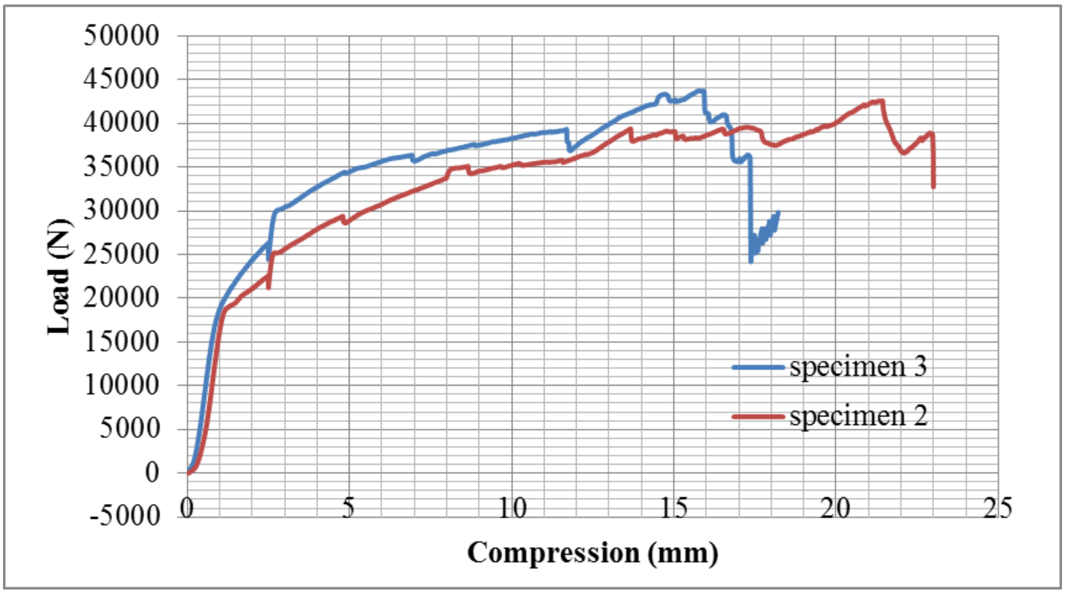


Figure 4.48 Load-deflection curves of the specimens 2 and 4 in compression perpendicular to grains

4.3.4 Tension Parallel to Grains

Since there were no wood samples big enough to have specimens with the dimensions stated in the standard, a new design was made for the samples as shown in Figure 4.49. The velocity of loading was 1 mm/min for this test. The testing apparatus for tension test is shown in Figure 4.50 and the deformed shapes of the specimens are shown in Figure 4.51, Figure 4.52, Figure 4.53, and Figure 4.54.

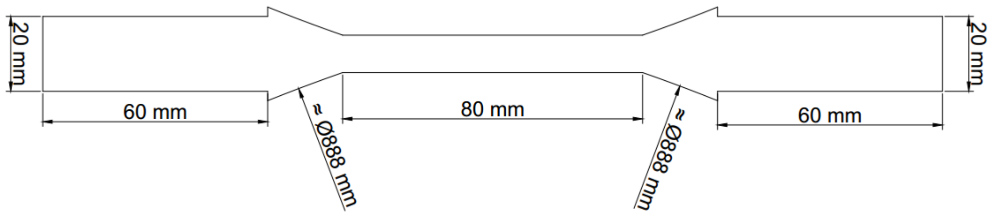


Figure 4.49 Dimensions of the new specimen design for tension parallel to grains test

Figure 4.50 Testing apparatus for the tension test

Figure 4.51 Failure of the specimen 1

Figure 4.52 Failure type of the specimen 2

Figure 4.53 Failure type of the specimen 3

Figure 4.54 Failure type of the specimen 4

The deformations were measured with a gage for the specimens 2, 3, and 4. The load-elongation graph of all specimens is shown in Figure 4.55 and stress-strain graphs of specimens 2, 3, and 4 are given in Figure 4.56, Figure 4.57, and Figure 4.58 respectively. Their modulus of elasticity values are given in table Table 4.8.

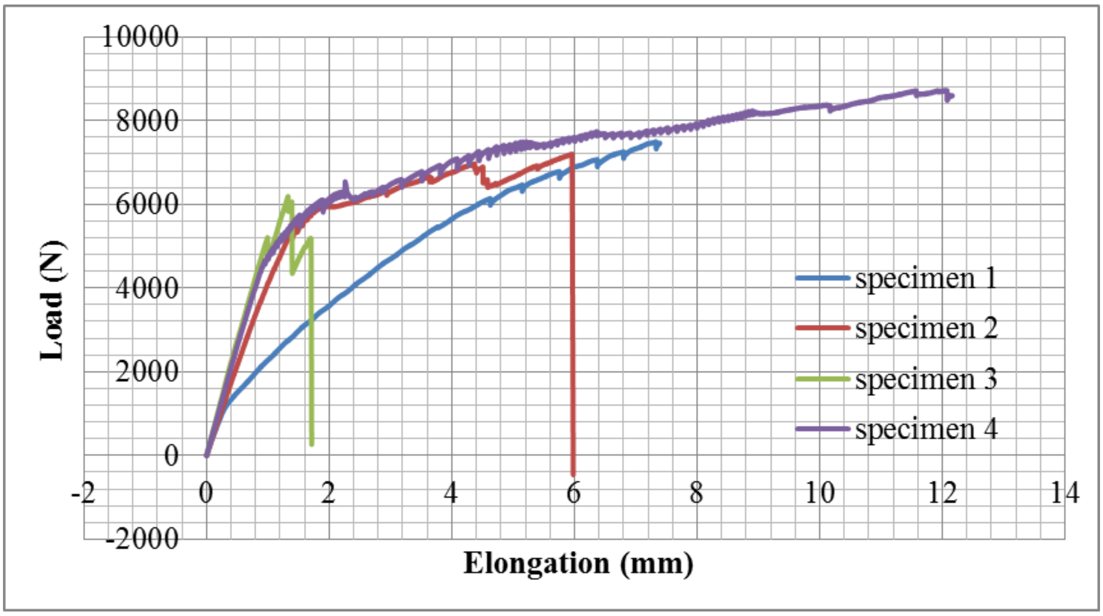


Figure 4.55 Load-elongation curves of the specimens 1, 2, 3, and 4 in tension parallel to grains

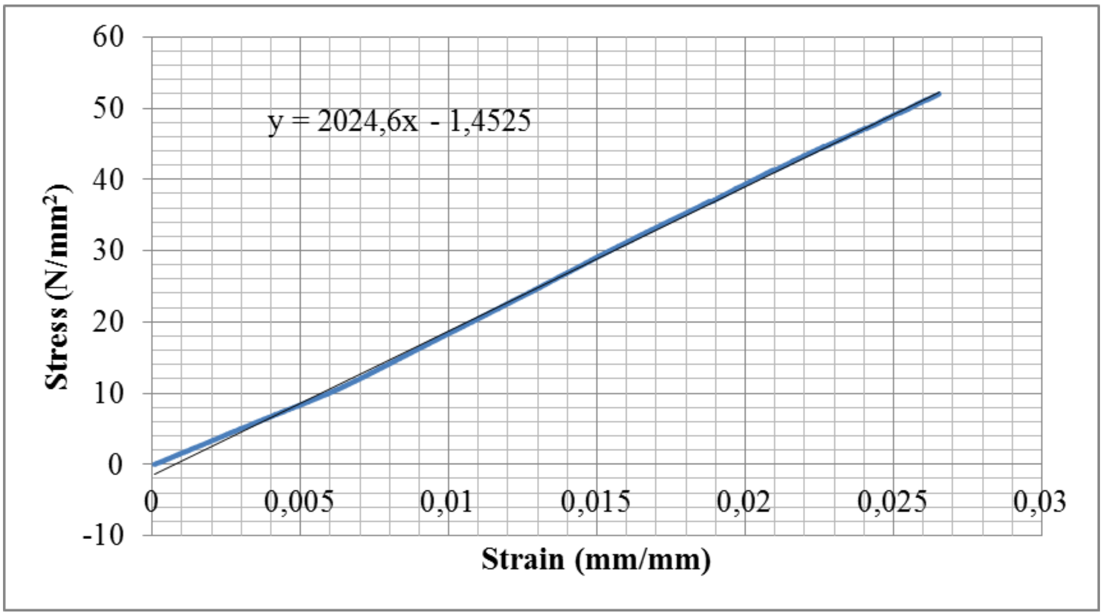


Figure 4.56 Stress-strain graph of specimen 2 in tension parallel to grains

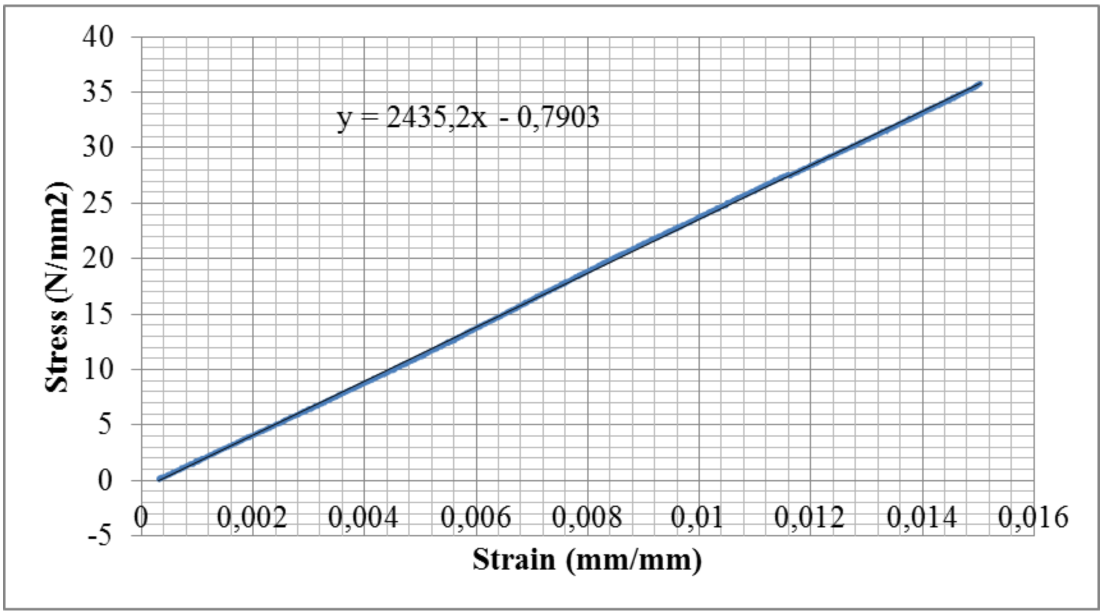


Figure 4.57 Stress-strain graph of specimen 3 in tension parallel to grains

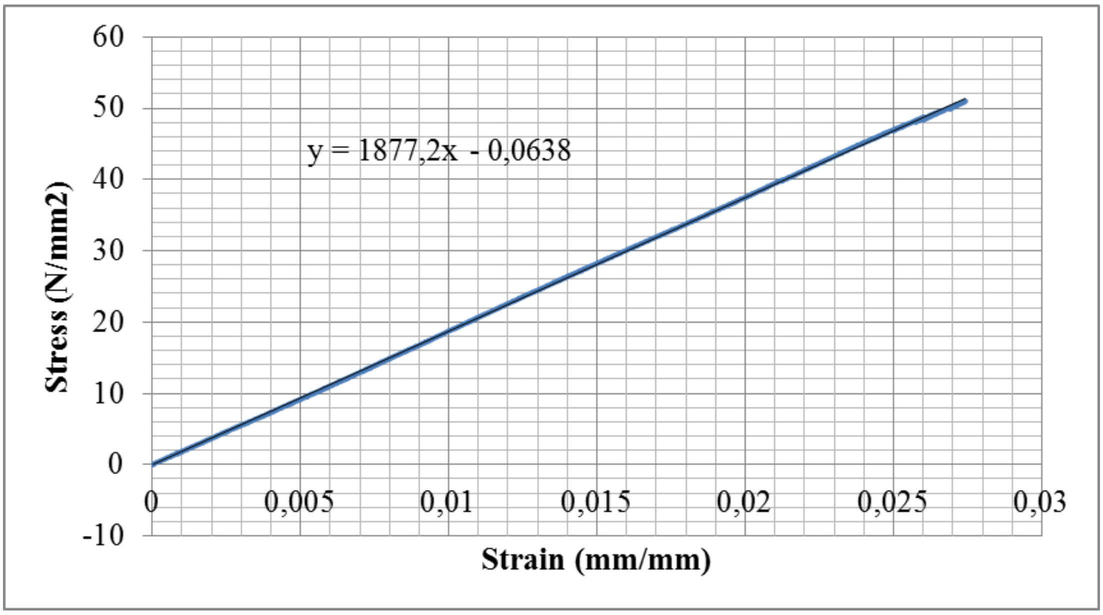


Figure 4.58 Stress-strain curve of specimen 4 in tension parallel to grains

Table 4.8 Modulus of elasticity values of the specimens in tension parallel to grains

Specimen No	Modulus of Elasticity (N/mm2)
2	2024.6
3	2435.2
4	1877.2

The mean strength values of timber specimens in bending, tension and compression at proportional limit and ultimate limit are given together in Table 4.9. These values were used on the structural analysis studies for the reconstruction of the bridge.

Table 4.9 Strength values of the timber in bending, compression parallel to grains and tension parallel to grains

	Mean Strength at Proportional Limit (N/mm2)	Standard Deviation	Mean Ultimate Strength (N/mm2)	Standard Deviation
Bending strength	47.34	1.45	82.8	5.26
Compression strength parallel to grains	33.23	5.07	36.71	5.85
Tension strength parallel to grains	49.73	2.26	73.05	10.2

4.3.5 Nail Tests

Together with the tests done to determine the mechanical parameters of timber, shear and tension test were done on the actual-sized timber specimens, which were connected together with wrought iron nails similar with the ones that will be used for the reconstruction of the bridge. There were three different, nailed specimens; one was composed of three logs which were nailed together, similar with the connection between the main beam, cantilever beam, and the horizontal beam between them. The other two were consist of two logs nailed together. One of them was similar with the connection between the main beams and deck plates; and the other was similar with the connection between the cantilever beams and horizontal beams. Firstly, shear test were done on the three logs nailed specimen and the two logs nailed specimen which was similar with the connection between main beams and deck plates. Then tension tests were done on the two logs nailed specimen which was similar with the connection between the cantilever beams and horizontal beams between them.

The three logs specimen was tested with the testing apparatus shown in Figure 4.59. The cross-section of the nail connecting together the logs was 18.40 mm*19.10 mm and the ultimate load it could resist on was approximately 10000 kg = 100 kN as shown in Figure 4.61.

Figure 4.59 Testing apparatus of the three logs nailed shear test

Figure 4.60 Failure type of the three logs nailed shear test

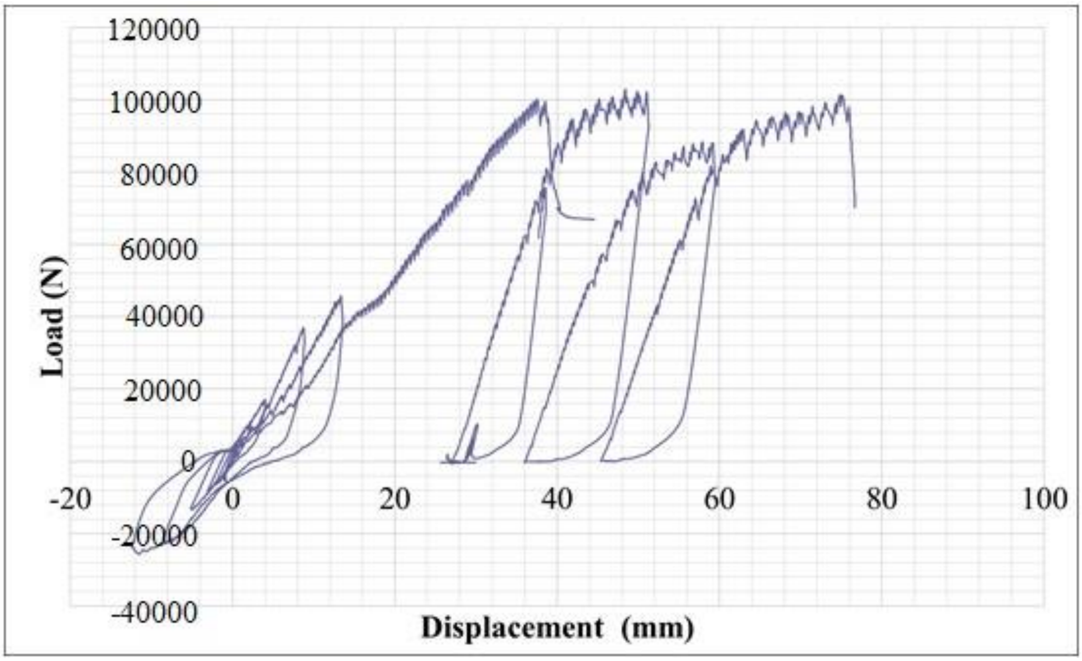


Figure 4.61 Load-displacement graph of the nail for three logs nailed shear test

The second shear test was done on the two logs nailed specimen similar with the connection between the deck plates and main beam. The cross-section of the nail was 9.45 mm*9.60 mm and the maximum load it can resist in shear is approximately 10 kN as shown in the load-displacement graph given in Figure 4.66. The testing apparatus of this test is as in Figure 4.62, failure types are shown in Figure 4.63 and Figure 4.65.

Figure 4.62 Testing apparatus of the two logs nailed shear test

Figure 4.63 Failure type of the two logs nailed shear test

Figure 4.64 Testing apparatus for the two logs nailed shear test

Figure 4.65 Failure type of the nail after the two logs nailed shear test

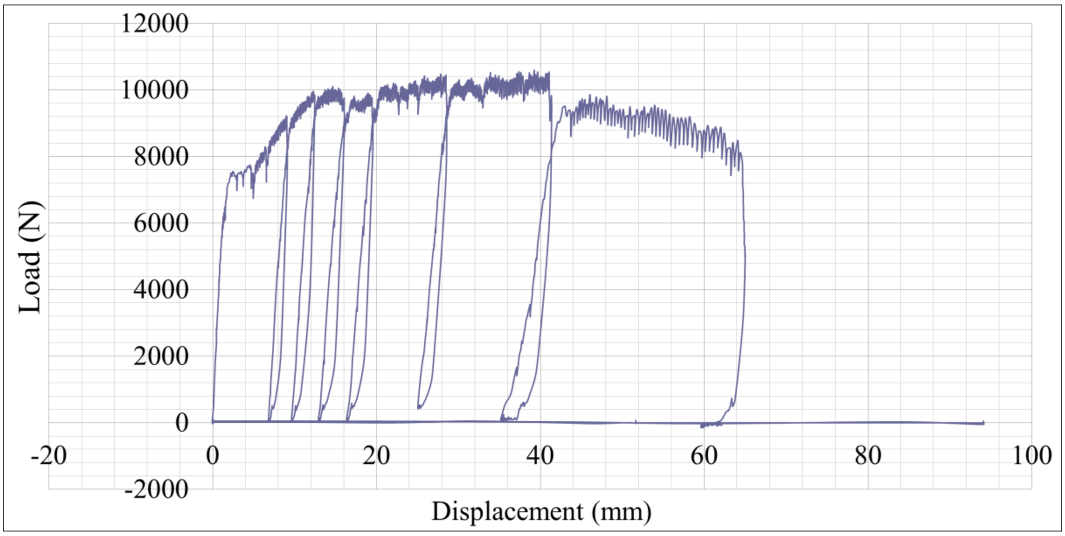


Figure 4.66 Load-displacement graph of the nail which connects deck plates and main beams in shear

For the other two logs nailed specimen, the timber specimen (lower log) was predrilled before the nail was pounded. Firstly, a spherical hole with a smaller radius than the nail's radius was drilled on the lower log, epoxy was applied and then the nail was pounded. For the tension tests, the testing apparatus given Figure 4.67 was used; one of the logs was pulled up while the other was standing firm. During the test, when the upper log was pulling upwards, it was observed the nail scrolled through the upper log. After a while, when the nail was about to project from the log the test was terminated. The maximum load that the nail can resist is approximately 20 kN, the load-deflection graph for this test is given in Figure 4.71. Following that, the upper log was taken apart the specimen and another tension test was done with the nail and lower log using the testing apparatus shown in Figure 4.68 and load - displacement graph for this testing is given in Figure 4.72.

Figure 4.67 Two logs nailed tension specimen for the first tension test

Figure 4.68 Two logs nailed tension specimen for the second tension test

Figure 4.69 Displacement of the upper log during testing

Figure 4.70 Displacement of the upper log at the end of the tension test

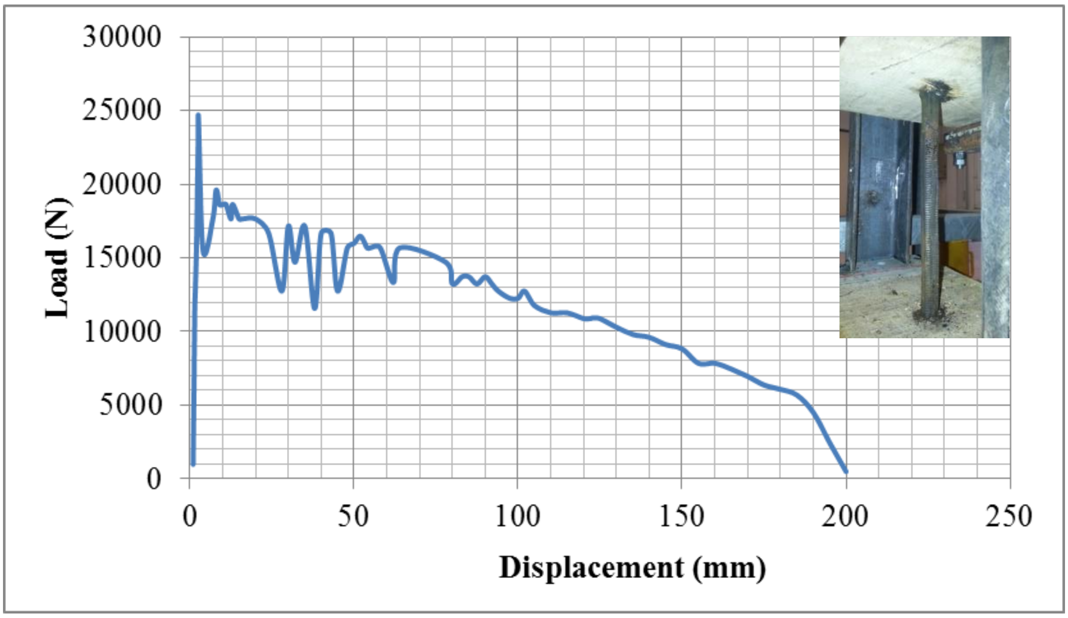


Figure 4.71 Load-displacement graph of the two logs nailed tension test

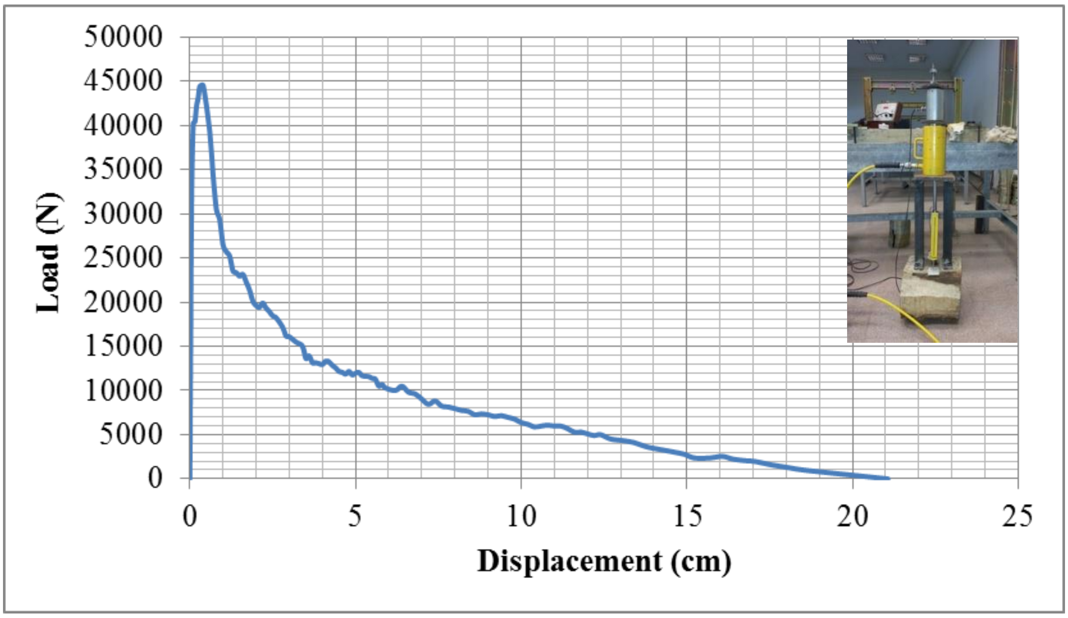


Figure 4.72 Load-displacement graph of the tension test in which the nail was pulled out of the log

In addition to the shear and tension test done with the nailed specimens, tension tests were made on the 18.40 mm*19.10 mm and 9.45 mm*9.60 mm cross-sectioned nails. The testing apparatuses (Figure 4.75 and Figure 4.76), failure types (Figure 4.77 and Figure 4.78), load-deformation graphs (Figure 4.73 and Figure 4.74), and results of these tests (Table 4.10) are given below. The ultimate load that 9.45 mm*9.60 mm cross-sectioned nail could resist is approximately 35 kN and the load at yielding point is equal to 25 kN. For the 18.10 mm*19.10 mm cross-sectioned nail; the ultimate load is equal to 190 kN and the load at yielding point is equal to 114 kN.

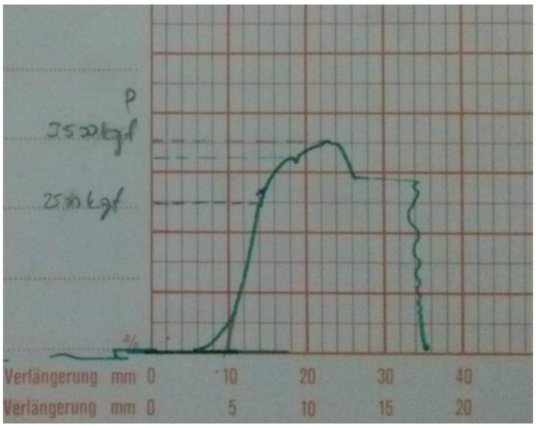


Figure 4.73 Load-deformation graph of the tension test of the 9.45cm*9.60cm cross-sectioned nail

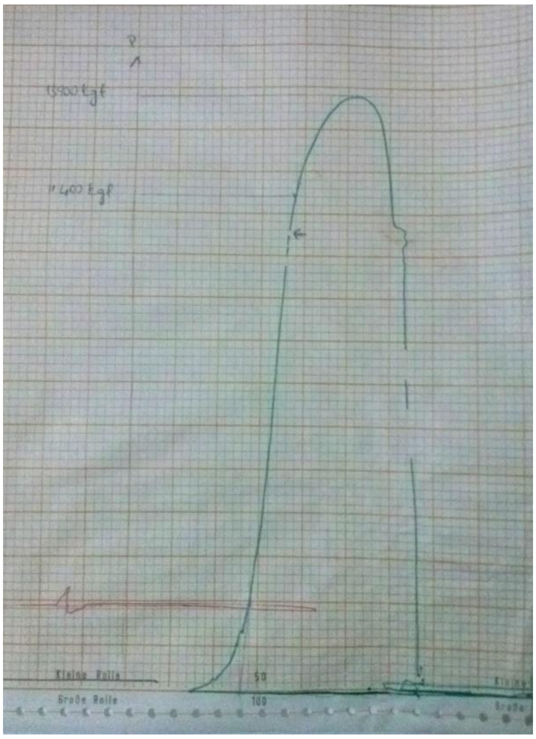


Figure 4.74 Load-deformation graph of the tension test of the 18.40*11.60cm cross-sectioned nail

Table 4.10 Tensile and yielding strength values of the nails

	Dimensions	Ao (mm2)	Ultimate Load (N)	Yielding Load (N)	Tensile Strength (N/mm2)	Yielding Strength (N/mm2)
nail A	18.4*19.1 mm	351.44	136312.44	111795.81	387.87	318.11
nail B	9.45*9.60 mm	90.72	34323.28	24516.63	378.34	270.24

Figure 4.75 Testing apparatus for the 9.45 mm*9.60 mm cross-sectioned nail

Figure 4.76 Testing apparatus for the 18.40 mm*19.10 mm cross-sectioned nail

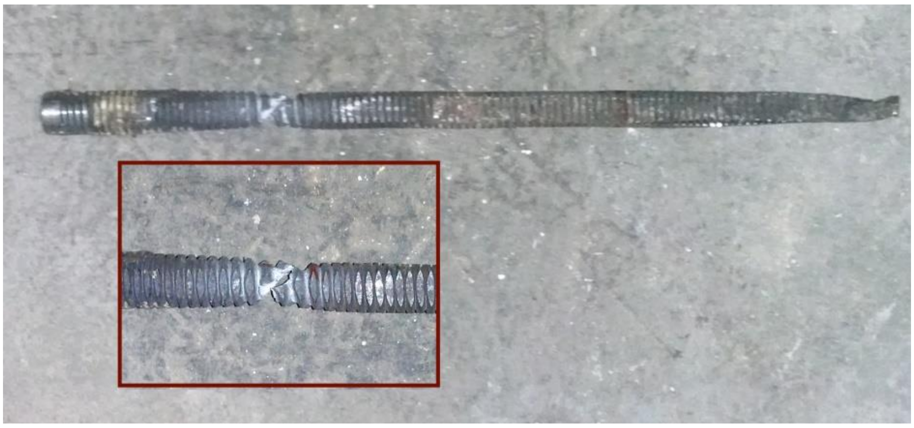


Figure 4.77 Failure type of the 9.45 mm*9.60 mm cross-sectioned nail

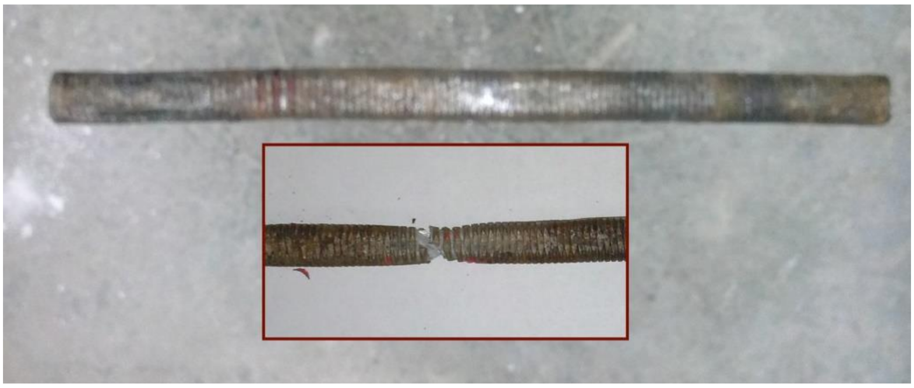


Figure 4.78 Failure type of the 18.40 mm*19.10 mm cross-sectioned nail

4.4 Analytical Modelling of the Buzlupınar Bridge in Accordance with the Reconstruction Project Prepared in August 2014

As previously mentioned, an analytical model of the Buzlupınar Bridge was prepared with SAP2000. As the bridge will be rebuild, the model geometry was based on the dimensions given in the reconstruction project of the bridge, which was prepared in August 2014 and shared with the authors.

The model was formed with frame, shell and solid objects and the material properties were assigned in compliance with the results of the material tests. Following the formation of the model geometry, the design live load and wind load values were assigned to model and a response spectrum analysis were made in order to see the seismic behaviour of the bridge.

4.4.1 Formation of the Model Geometry

When modelling, special care must be given to understand the structural system and form the model so as to show its structural behaviour realistically. The first step of the modelling process of the Buzlupınar Bridge was to examine the original structural features of the bridge in detail with the help of the field survey and previously taken photos; then to decide the formation of the model geometry in accordance with the reconstruction project.

Three different objects were used to model the bridge; frames, which are straight lines connecting 2 nodes together and used for modelling the beams, columns, braces and trusses; shells, which are area objects with at least 3 nodes and used to model the structural elements which show membrane or plate-bending behaviours such as floors, walls, 3D surfaces and decks; and solids which has 6 surfaces and 8 nodes at the edges of the surfaces and used to model three dimensioned structural systems.

The cantilever beams, main beams, posts, horizontal beams, roof elements and transversal elements were modelled using frame elements. The timber deck plates, roof piling sheets, L shaped elements and side covering sheets were modelled with shell elements. When modelling the ground, two different assumptions were made; in one of them the soil was modelled using rigid supported solid elements (Figure 4.79) whereas the second model was formed without solid elements, using rigid supports at ends and specified points of the frame elements (Figure 4.80).

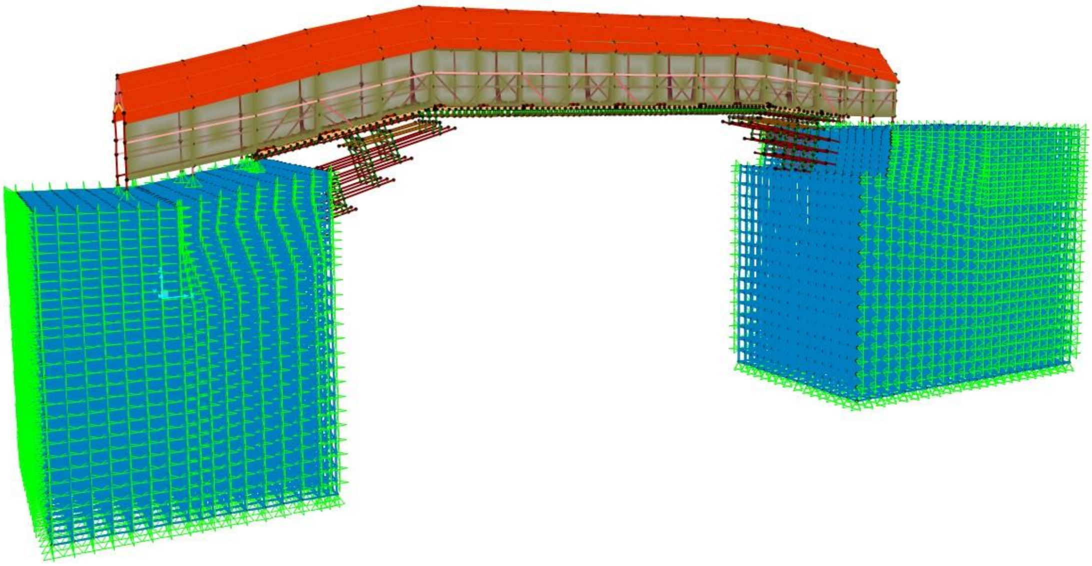


Figure 4.79 The ground modelled with rigid supported solid elements

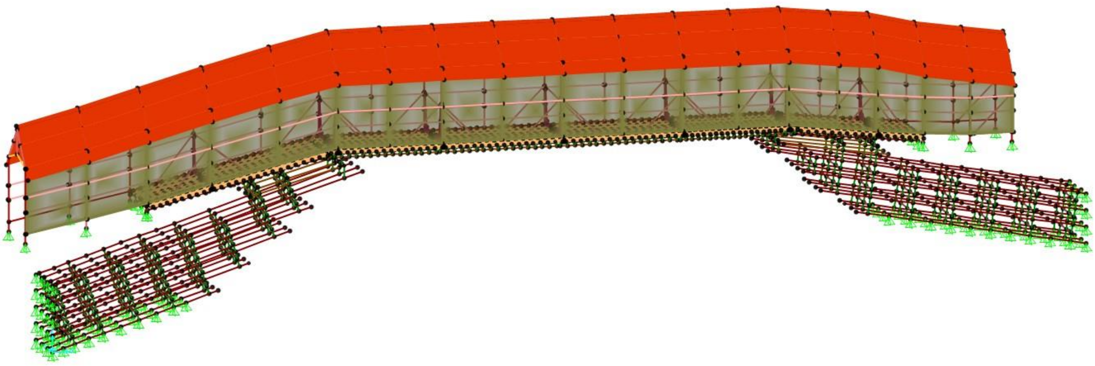


Figure 4.80 The soil-structure interaction formed with rigid supports

The distances between the bridge elements were measured from the centre of each element. The connections between the elements were made with rigid links whereas the nails and interlacing parts of the elements were not modelled in particular.

4.4.2 Mechanical Material Parameters Used in the Model

When modelling with SAP2000, the material properties which consist of the weight per unit volume, modulus of elasticity, Poisson's ratio, coefficient of thermal expansion, and shear modulus values of the materials should be defined. The modulus of elasticity and weight per unit volume of the timber has been estimated in accordance with the results of the material tests. The other properties were gathered from different sources. The material parameters used in modelling are given in Table 4.11.

Table 4.11 Mechanical Properties of the Materials Used in SAP2000 Model

Material	Weight per Unit Volume (kN/m^3)	Modulus of Elasticity (E) (kN/m^2)	Poisson's Ratio (U)	Coefficient of Thermal Expansion (A) (m/m K)	Shear Modulus (G) (kN/m^2)
Timber	5	5701551	0,04 [20]	3,0 E-6 [21]	2741130,3
Timber Side Covering Sheets	5	5701	0,04 [22]	3,0 E-6 [23]	2740,8
Soil	19,50 [24]	100000 [25]	0,25 [26]	1,0 E-5	40000

[20] Wood Handbook, p.5-2, (United States Department of Agriculture Forest Service, 2010)

[21] The data was gathered from; http://www.engineeringtoolbox.com/linear-expansion-coefficients-d_95.html, retrieved in 23.09.2014.

[22] Wood Handbook, p.5-2, (United States Department of Agriculture Forest Service, 2010)

[23] The data was gathered from; http://www.engineeringtoolbox.com/linear-expansion-coefficients-d_95.html, retrieved in 23.09.2014.

[24] TS 500 : Design Loads for Buildings

[25] The data was gathered from; ftp://www.clrp.cornell.edu/CDOT/Handouts/4c%20%20Materials%20Table.pdf, retrieved in 03.01.2015.

[26] The data was gathered from; https://support.prokon.com/portal/kb/articles/2-elastic-properties-of-soils, retrieved in 03.01.2015.

All the timber elements of the bridge have same material properties except the side covering sheets. The modulus of elasticity value for these elements was reduced at a rate of one per thousand since they are not load bearing elements and their width is quite narrow relatively.

Another material defined for the modelling was the rigid link which assigned to the rigid links used for connecting together the structural elements of the bridge. The material was defined weightless since the rigid links are fictitious/imaginary elements whose only task was to ensure a stiff connection between the structural elements.

4.4.3 Loading Data

When the model was formed completely, the design live loads and wind loads were assigned to the bridge in order to investigate the structural behaviour of the structure as a whole or each element separately in case of being exposed to these loads. In addition to the live load and wind load, a response spectrum function was defined and modal analysis was made to find out the seismic behaviour of the bridge.

All of the analyses were repeated both with the model in which the ground was modelled with solid elements and the model in which the soil-structure interaction was modelled with rigid supports.

4.4.3.1 Live Load

As mentioned in section 3.1 and section 4.2.1, the Buzlupınar Bridge was used for pedestrian, animal load transportation, and the design load for the structural analysis studies was taken as 5 kN/m^2 as defined for the areas where people could concentrate, in TS498 : Design Loads for Buildings.

The live load was assigned to the deck plates to the mid 1.85 m of the bridge width, in global Z direction, as shown in Figure 4.81.

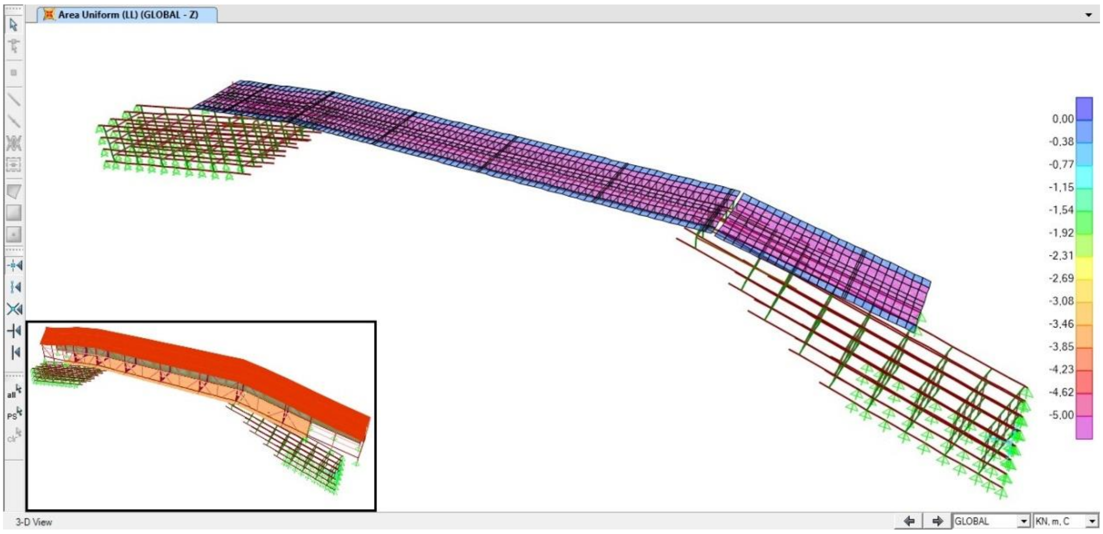


Figure 4.81 Live loads assigned to the SAP2000 model of the Buzlupınar Bridge

4.4.3.2 Wind Loading Data

The design wind loads were previously defined in section 4.2.3. The wind loads were assigned to the analytical model as shown in Figure 4.82 and Figure 4.83; as estimated for the one side covered form of the bridge with gaps between the deck plates and side covering plates, which is the most realistic situation when the geometrical form of the bridge is considered.

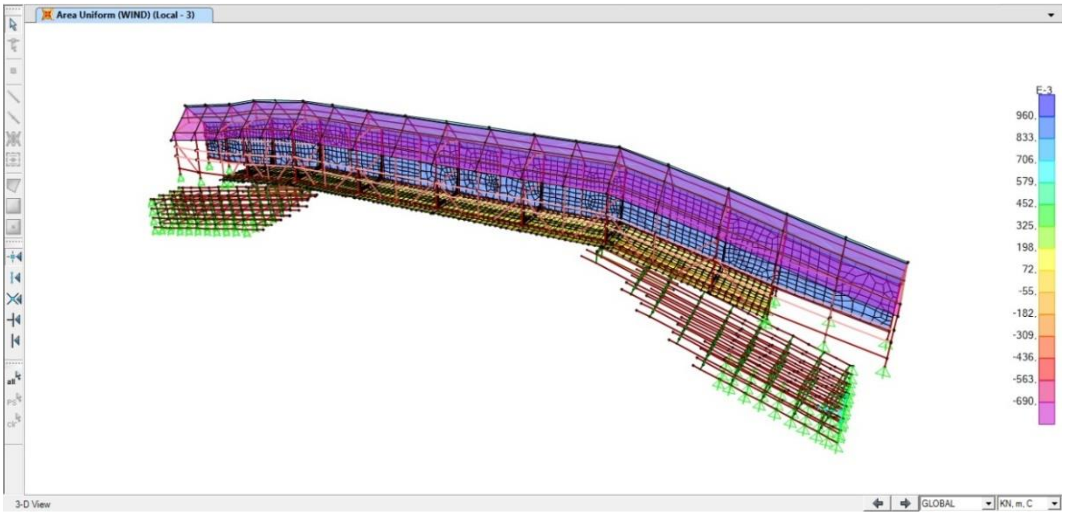


Figure 4.82 Wind loads assigned to the SAP2000 model of the bridge

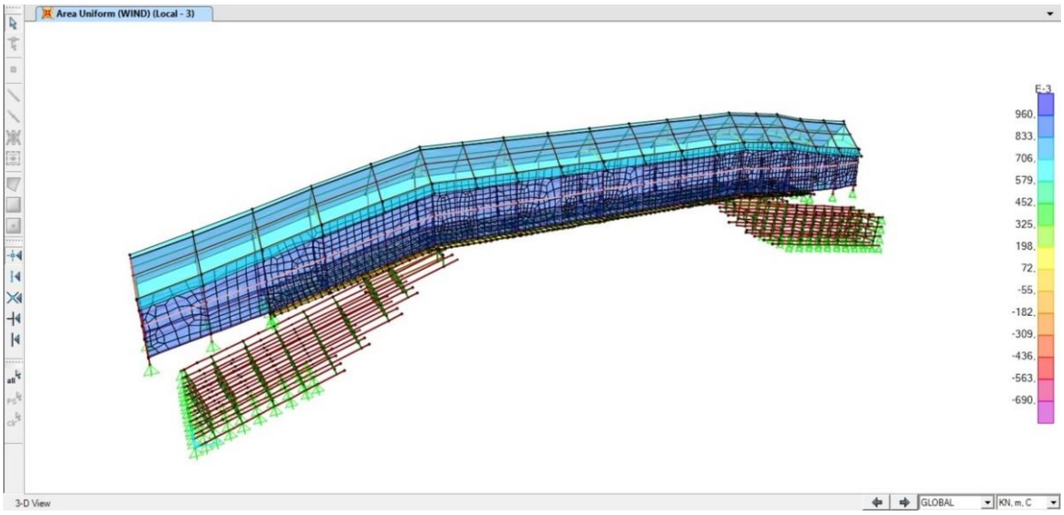


Figure 4.83 Wind loads assigned to the SAP2000 model of the bridge

4.4.3.3 Earthquake Loading Data

Rize is defined as the 4th degree earthquake zone in Earthquake Zones Map of Turkey within 5 levels as shown in Figure 4.84. The earthquake acceleration decreases from 1 to 5. As for the information gathered from the Prime Ministry Disaster and Emergency Management Authority[27] any major earthquake was not recorded in Rize since 1976 and Rize is relatively less risky region in terms of occurrence of destructive earthquake, yet a response spectrum analysis was made with SAP2000 in an attempt to investigate the behaviour of the bridge when exposed to seismic forces. The estimations were done in accordance with the parameters and formulas given in the Specification for Structures to be Built in Disaster Areas (2007). The local soil class of the ground was defined as Z2. Other parameters used in earthquake analysis are given in Table 4.2.

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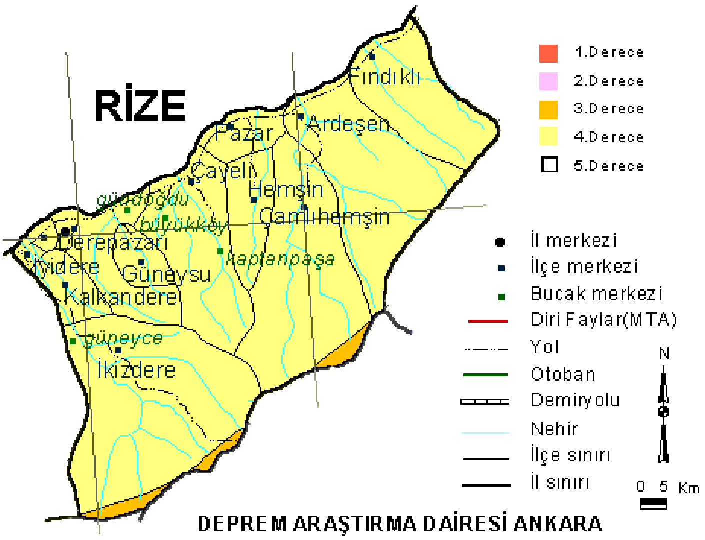


Figure 4.84 Rize earthquake zones map (Retrieved from http://www.deprem.gov.tr/sarbis/depbolge/rize.gif, 05.08.2015)

[27] Information gathered from http://kyh.deprem.gov.tr/buyukdeprem.htm, retrieved in 13.06.2015.

Table 4.12 Coefficients and characteristic periods

Spectrum characteristic period (T_A)	0,15
Spectrum characteristic period (T_B)	0,4
Effective ground acceleration coefficient (A_o)	0,1
Building importance coefficient (I)	1

The spectrum coefficient, period and scale factor for the earthquake analysis were calculated using the equations (4.15), (4.16), (4.17), (4.18), (4.19), (4.20), and (4.21). The spectrum graph obtained with the calculations is given in Figure 4.85.

$$V_t = W * \frac{A_o * I * S(T)}{R_a * (T)} = m * g * A_o * I * \frac{S(T)}{R_a * (T)} \tag{4.15}$$

$$S(T) = 1 + 1.5 * \frac{T}{T_A} \qquad 0 \leq T \leq T_A \tag{4.16}$$

$$S(T) = 2.5 \qquad T_A < T \leq T_B \tag{4.17}$$

$$S(T) = 2.5 \left(\frac{T_B}{T}\right)^{0.8} \qquad T_B < T \tag{4.18}$$

$$R_a(T) = 1.5 + (R - 1.5) * \frac{T}{T_A} \qquad 0 \leq T \leq T_A \tag{4.19}$$

$$R_a(T) = R \qquad T_A < T \tag{4.20}$$

$$\text{Scale Factor} = g * A_o * I = 9{,}81 * 0{,}1 * 1 = 0{,}981 \tag{4.21}$$

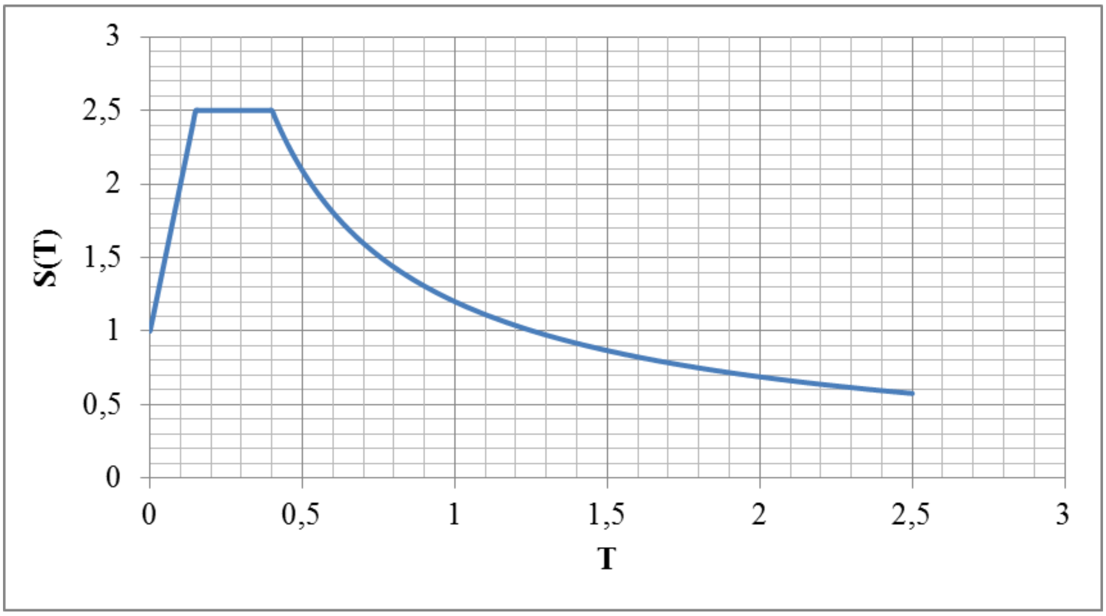


Figure 4.85 Response spectrum graph

The response spectrum analysis was performed both for the model in which the soil was modelled using rigid supported solid elements and for the other one, in which the soil formed without solid elements, using rigid supports at ends and specified points of the frame elements. In both models, the modal participation mass ratio was obtained over 90% at mod 41.

4.4.4 Comparison and Evaluation of the Analyses Results

In this section, the results of the analysis performed with SAP2000 were evaluated comparing the forces generated on the critical connections with the maximum loads that the nails on the connections can resist.

As it is understood from the photographs which were taken right after the collapse of the bridge and learned from the interviews with the locals, the collapse of the bridge was occurred as illustrated in Figure 4.86. With the effect of the wind forces, the nails between the deck plates and main beams were pulled up and the cover of the bridge started to rotate on the direction of the wind; finally it fell down to the river together with one of the three main beams.

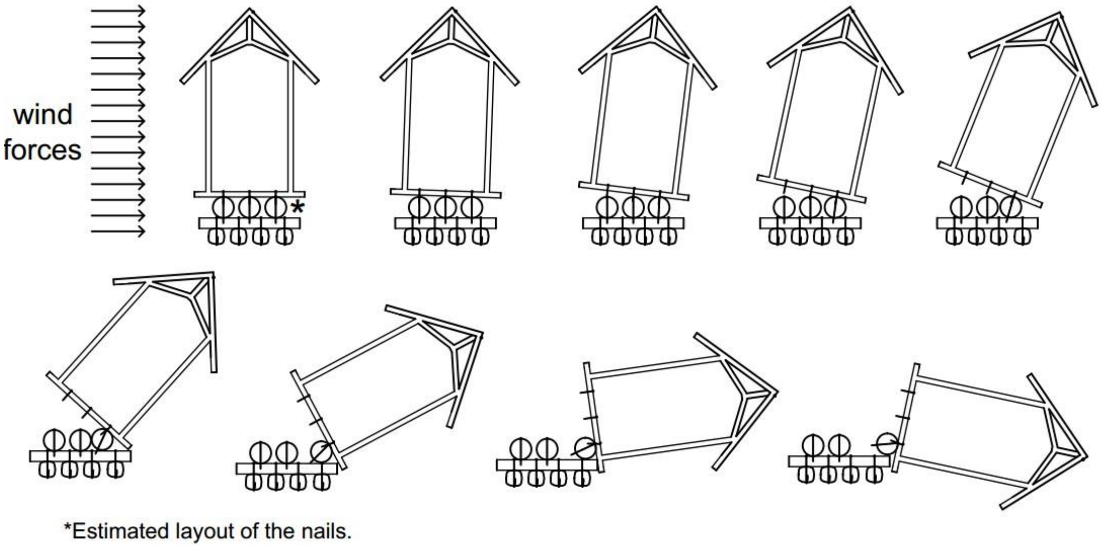


Figure 4.86 Collapse of the bridge

Since the bridge was thought to be collapsed as described in Figure 4.86, special care was given to evaluation of the forces generated on the connections between the deck plates and main beams; and main beams, cantilever beams, and the horizontal beams between them; which are shown in Figure 4.87 and Figure 4.88. The hand calculations showed that the bridge is not safe against the wind forces since the overturning forces generated on the bridge are bigger than the resisting forces when it is exposed to wind forces. However, on the hand calculations the contribution of the nails to the resisting forces at connection points were ignored, yet in reality the nails resist against the overturning forces too. Therefore, in this part of the study the calculations done considering the strength of the nails. The connections were checked for the forces generated under the live loads, wind loads and seismic loads.

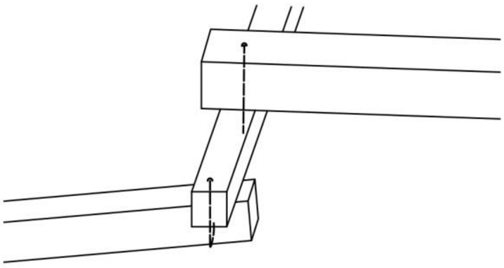


Figure 4.87 Connection between the main beams and the cantilever beams

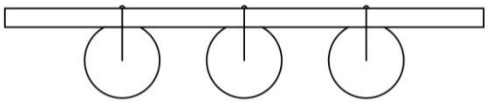


Figure 4.88 Connection between the deck plates and main beams

As given in Eurocode 5, for the nailed connections subjected to combined laterally and axially loads, the expression given in equation **(4.22)** should be satisfied. Here F_a is the combination of axial force generated on the connection, F_v is the combination of shear force generated on the connection, F_{ad} is the design load carrying capacity of the nail in tension and F_{vd} is the design load carrying capacity of the nail in shear.

$$\left(\frac{F_a}{F_{ad}}\right)^2 + \left(\frac{F_v}{F_{vd}}\right)^2 \leq 1 \qquad \textbf{(4.22)}$$

Some of the required values were determined with the material tests mentioned in section 4.3 which are the maximum loads that the 18.40*19.10 mm cross-sectioned nail (which will be mentioned as nail A thereinafter) can resist in shear and tension; and the maximum load that the 9.45*9.60 mm cross-sectioned nail (which will be mentioned as nail B thereinafter) can resist in shear. Together with these values the maximum load that the nail B can resist in tension was required for the calculations, which was not tested; it was calculated using the information obtained from the nail A tension tests. The maximum load that the nail A can resist in tension was determined as 20 kN in epoxy unapplied case and 45 kN in epoxy applied case. As shown in Figure 4.89, a line was drawn overlapping with the load-displacement graph of the tension test of the epoxy unapplied case of nail A. The equation of the line is F=ax+b in which b is approximately 5 kN which is the force applied to the nail by the log and x is the length of the nail inside the log. At the beginning of the testing x was equal to 20 cm and at that point F was approximately 20 kN, so a is equal to 15 kN which is also equal to the net force acting on the nail. The shear stress applied to the nail by the log is determined as shown in equation (4.23). Here, it was assumed the nail A has a rectangular cross-section 2*2 cm so its perimeter was taken as 8 cm.

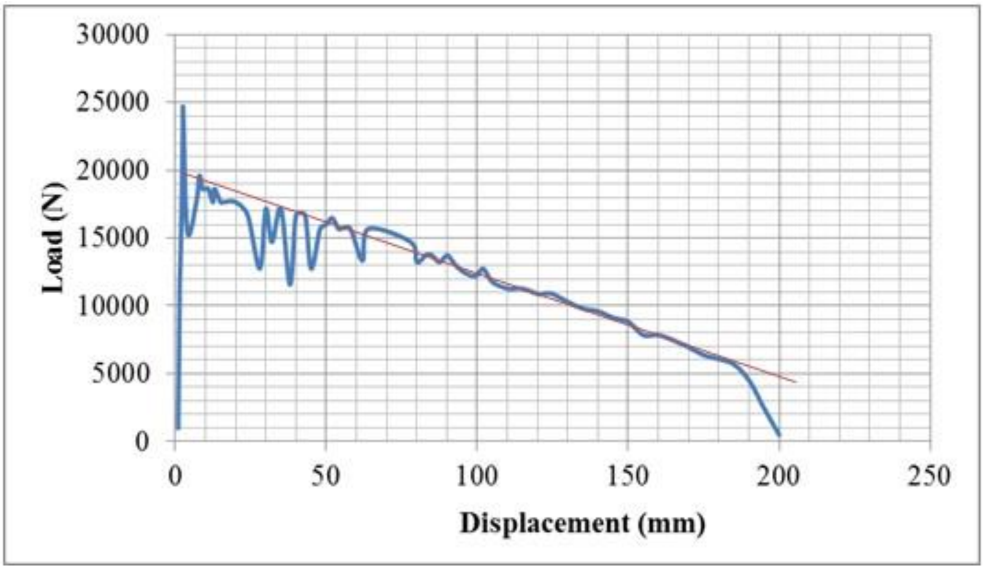


Figure 4.89 Load-displacement graph of the two logs nailed tension test

$$\tau = \frac{P}{A} = \frac{15}{8 * 20} \cong 0.09 \text{ kN/cm}^2 \qquad \textbf{(4.23)}$$

The same calculation was repeated for the epoxy applied case, shown in equation (4.24) in which the maximum load that the nail can resist was determined as 45 kN.

$$\tau = \frac{P}{A} = \frac{45}{8 * 20} \cong 0.28 \text{ kN/cm}^2 \qquad \textbf{(4.24)}$$

Using these values and equation (4.25), the maximum load that nail B can resist are given in equations (4.26) and (4.27). Here it was assumed the nail B has squared cross section of 1*1 cm, so its perimeter is equal to 4 cm and the length of the nail inside the log was taken as 9 cm.

$$P = \tau * A \quad \textbf{(4.25)}$$

$$P = 0.09 * 9 * 4 = 3.24 \text{ kN} \quad \textbf{(4.26)}$$

$$P = 0.28 * 9 * 4 = 10.08 \text{ kN} \quad \textbf{(4.27)}$$

Consequently, the maximum load that the nail B can resist in tension were estimated as 3.24 kN for epoxy unapplied case and 10.08 kN for epoxy applied case.

Table 4.13 Load carrying capacities of nail A and nail B with and without epoxy application

	Without epoxy		With Epoxy Application	
	F_{ad}	F_{vd}	F_{ad}	F_{vd}
Nail A (18.4mm*19.1mm)	20 kN	50 kN	45 kN	75 kN [28]
Nail B (9.45 mm*9.60 mm)	3.24 kN [29]	10 kN	10.08 kN [30]	15 kN [31]

[28] The shear capacity of the epoxy applied nails was not tested but it was assumed the shear capacity will increase 1.5 times in epoxy applied case

[29] Calculated value of design load carrying capacity of nail B in tension.

[30] Calculated value of design load carrying capacity of epoxy applied nail B in tension.

[31] [31] The shear capacity of the epoxy applied nails was not tested but it was assumed the shear capacity will increase 1.5 times in epoxy applied case

4.4.4.1 Evaluation of the Analysis Results under Dead Load and Live Loads

Under the live loads, on the connection between the main beams and the cantilever beam, the shear forces generated on connections are +0.34 kN in 2-2 direction and 0 kN in 3-3 direction; the axial forces are in compression that will not cause a critical situation for the structural safety of the bridge so it was taken as 0 to be on the safe side. Using this values and the expression given in equation (4.22), the result is found near-zero as given in equation (4.28), which satisfy the safety criteria given in Eurocode 5.

$$\left(\frac{0.34}{50}\right)^2 + (0)^2 \approx 0 \leq 1 \tag{4.28}$$

The maximum shear forces generated on the most critical connections between the deck plates and main beam are equal to ±51 kN in 2-2 direction, 0 kN in 3-3 direction and the maximum axial force is equal to +15 kN. These values are generated on the mid part of the bridge through its span. The maximum shear and axial forces occur on the connections between the deck plates at each side of the tie beam which is in the middle of the span. These forces are the greatest values obtained on the connections so it is recommended to use 6 nails on these connections to provide the safety requirements (Connections between the deck plates at each side of the tie beams and main beams). As previously defined, the tensile capacity of 6 nails is equal to 18 kN and shear capacity is 60 kN. Using these values, the obtained result is equal to 1.41 as given in equation (4.29).

$$\left(\frac{51}{6x10}\right)^2 + \left(\frac{15}{6x3}\right)^2 = 1.41 \geq 1 \tag{4.29}$$

By this result, it is seen the specified size and numbers of nails are not sufficient to prevent the overturning of the bridge by the live loads. Herein, it is important to notice that the shear capacity of nails shows differences with the epoxy application as determined with the nail tests. The calculation was repeated using the capacity values of the epoxy applied nail. If epoxy applied nails are used on the connection between the deck plates and main beams, the capacity of the nails are expected to be increased. Using the capacity values for epoxy applied nails given in Table 4.13 the result was obtained as 0.38 as given in equation (4.30).

$$\left(\frac{51}{6x15}\right)^2 + \left(\frac{15}{6x10}\right)^2 = 0.38 \leq 1 \quad \textbf{(4.30)}$$

4.4.4.2 Evaluation of the Analysis Results under Dead Load and Wind Loads

At the connection between the main beams and cantilever beams, the mean shear force generated on each connection is 0 in 2-2 direction whereas the shear force in the direction of 3-3 is -10 kN and the maximum axial force is 27 kN. The nails can resist to maximum 20 kN in tension and 50 kN in shear, using this values the result was calculated as 1.86 as given in equation (4.31).

$$\left(\frac{10}{50}\right)^2 + \left(\frac{27}{20}\right)^2 = 1.86 \geq 1 \quad \textbf{(4.31)}$$

Since the result is greater than 1, the calculation was repeated using the tensile capacity of the epoxy applied nails. In this case, the result was obtained as 0.38 as given in equation (4.32).

$$\left(\frac{10}{75}\right)^2 + \left(\frac{27}{45}\right)^2 = 0.38 \leq 1 \quad \textbf{(4.32)}$$

Another critical connection which has to be controlled is the one between the deck plates and main beams. The maximum shear forces generated on this connection in the direction of 2-2 is +14 kN and in the direction of 3-3 is 10 kN; and the maximum axial force is 6 kN. These maximum forces are generated on the connections which are located at first 2.5 m from the edges of the mid span. As calculated in section 4.2.1 and shown in Figure 4.6, the connection of a deck plate to each main beam will be made with 6 nails on that part, so the shear resistance of each connection is equal to 60 kN and the tension force is 18 kN; the result obtained using these values is 0.40 as given in equation (4.33), which satisfy the safety requirements.

$$\left(\frac{\sqrt{14^2+10^2}}{6x10}\right)^2+\left(\frac{6}{6x3}\right)^2=0.40\leq 1 \qquad \textbf{(4.33)}$$

4.4.4.3 Evaluation of the Analysis Results under Dead Load and Seismic Loads

Under seismic loads, the shear forces generated on the connections between the main beams and cantilever beams is equal to +2.87 kN in 2-2 direction and -2.9 kN in 3-3 direction. The axial forces generated on each connection are compression forces so the axial force value was taken as 0 to be on the safe side. The result obtained using these forces and capacity values of the nails given in Table 4.13 is near-zero, which means the safety requirement is satisfied.

$$\left(\frac{\sqrt{2.87^2+2.9^2}}{50}\right)^2+(0)^2\approx 0\leq 1 \qquad \textbf{(4.34)}$$

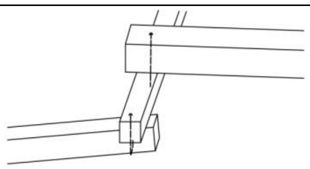
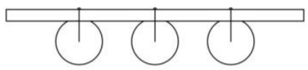
For the connection between deck plates and main beams, under seismic loads, the maximum shear force in 2-2 direction is -22 kN and ±10 kN in 3-3 direction; the maximum axial force is +4 kN. These loads were generated on the connections between the deck plates and main beams which are placed at each side of the tie beams and at the first 2.5 m of the main span from each side. As previously defined in section

4.2.1, 6 nails are proposed to use on these connections. Result obtained from the calculations is 0.21 as (4.35), so the safety requirement is satisfied.

$$\left(\frac{\sqrt{22^2+10^2}}{6x10}\right)^2+\left(\frac{4}{6x3}\right)^2=0.21\leq 1 \tag{4.35}$$

The results of the safety checks on the critical connections under live loads, wind loads and earthquake loads are given in Table 4.14.

Table 4.14 The results of the safety checks on the critical connections under live loads, wind loads and earthquake loads

	 Connection between cantilever beam and main beam	 Connection between deck plates and main beams
DL+LL	$\left(\frac{0.34}{50}\right)^2+(0)^2\approx 0\leq 1$	$\left(\frac{51}{60}\right)^2+\left(\frac{15}{18}\right)^2=1.41\geq 1$
DL+LL	-	$\left(\frac{51}{90}\right)^2+\left(\frac{15}{60}\right)^2=0.38\leq 1$
DL+WIND	$\left(\frac{10}{50}\right)^2+\left(\frac{27}{20}\right)^2=1.86\geq 1$	$\left(\frac{\sqrt{14^2+10^2}}{60}\right)^2+\left(\frac{6}{18}\right)^2=0.40\leq 1$
DL+WIND	$\left(\frac{10}{75}\right)^2+\left(\frac{27}{45}\right)^2=0.38\leq 1$	-
DL+EQ	$\left(\frac{\sqrt{2.87^2+2.9^2}}{50}\right)^2+(0)^2\approx 0\leq 1$	$\left(\frac{\sqrt{22^2+10^2}}{60}\right)^2+\left(\frac{4}{18}\right)^2=0.21\leq 1$
DL+EQ	-	-

These results shows that in two cases the safety requirements are satisfied with epoxy applied nails. However, there are doubts about long term performance of the epoxy; for this reason it is proposed to do not use epoxy applied nails on the reconstruction project. Instead, longer nails could be used on the connections and the ends of the nails could be bended as it was originally done on some connections of the bridge.

Together with the connections between deck plates and main beams and cantilever beams and main beams given above, the critical forces generated on the connections between tie beams and main beams were determined and structural safety of them were checked too. On the most critical connections the maximum shear force was determined as 32.3 kN in 2-2 direction and 9 kN in 3-3 direction and the maximum value of axial force was determined as 6.6 kN. Using equation (4.22) the results obtained as 1 for the nails without epoxy application as given in equation (4.36) and as 0.34 for epoxy applied nails as given in equation (4.37).

$$\left(\frac{\sqrt{32.3^2 + 9^2}}{4x10}\right)^2 + \left(\frac{6.6}{4x3}\right)^2 = 1 \leq 1 \tag{4.36}$$

$$\left(\frac{\sqrt{32.3^2 + 9^2}}{4x15}\right)^2 + \left(\frac{6.6}{4x10}\right)^2 = 0.34 \leq 1 \tag{4.37}$$

4.4.4.4 Evaluation of the Stresses Generated on Tie Beams and L Shaped Bracings

The L shaped bracings connects together the posts and deck plates, the forces are transferred from the post to the deck plates via L shaped elements. Therefore the stresses generated on L shaped bracings and moments generated on tie beams were determined; and the stresses were compared with the capacity of the timber element.

The maximum moment generated on tie beams were determined as 28 kNm/m (M22) under live loads. Since the width of the tie beams are 10 cm, the moment is equal to 2.8 kN.m and the stress is 7.45 MPa as given in equation (4.38).

$$\mathrm{M} = 0.1 * 28 = 2.8\ \mathrm{kN.m} \quad \textbf{(4.38)}$$

$$\mathrm{I} = \frac{\mathrm{bxh}^3}{12} = \frac{0.1\mathrm{x}0.15^3}{12} = 2.81\mathrm{x}10^{-5}\ \mathrm{m}^4 \quad \textbf{(4.39)}$$

$$\sigma = \frac{\mathrm{M.y}}{\mathrm{I}} = \frac{2.8\mathrm{x}0.075}{2.81\mathrm{x}10^{-5}} = 7466.67\ \mathrm{kN/m^2} = 7.45\ \mathrm{MPa} \quad \textbf{(4.40)}$$

The maximum stress generated on L shaped bracings was determined as 5921 kN/m2 = 5.92 MPa caused by the wind loads. This results show that the maximum stresses generated on the tie beams and L shaped bracings are less than the design bending capacity of the timber which is 25.50 MPa.

4.4.4.5 MAC Analysis of Two Models of the Bridge

As defined in section 4.4.1, two assumptions were made when modelling the soil-structure interaction; in one of them, the ground was modelled with rigid supported solid elements and in the other one rigid supports were used at the end of the cantilever beams and specified points of the cantilever elements. All of the analyses were run for both cases; the ratios of the results obtained from these two models are in range of 0.9 - 1.2 between each other. The results of the response spectrum analyses of both cases were compared using Modal Assurance Criterion (MAC), which measures the correlation between two modal vectors (Rigner, 1998). The MAC value is 1 if two vectors are the same or scaled with a constant value and 0 if the vectors are perpendicular and not correlated to each other. To investigate the similarity of the modes obtained from the response spectrum analyses of two models, 70 identical

points were selected in both models so as to represent the whole bridge. The selected points are shown in Figure 4.90. The joint displacement values of these points obtained from two analyses were compared with each other and a great similarity was observed between the mode vectors of two models as shown in
Figure 4.91.

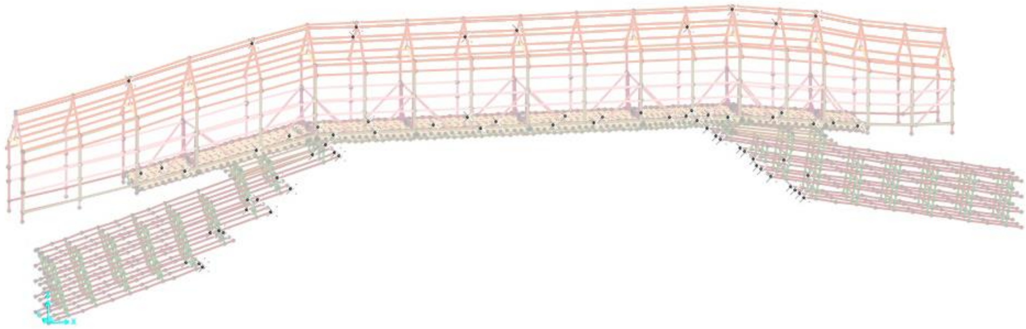


Figure 4.90 The selected points for MAC analysis

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
1	1,00	0,00	0,01	0,00	0,15	0,00	0,43	0,00	0,00	0,02	0,00	0,00	0,00	0,00	0,03	0,00	0,00	0,03	0,00	0,00
2	0,00	1,00	0,00	0,01	0,00	0,08	0,00	0,32	0,11	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,01
3	0,00	0,00	1,00	0,00	0,16	0,00	0,04	0,00	0,00	0,00	0,00	0,04	0,05	0,00	0,09	0,01	0,00	0,01	0,00	0,00
4	0,00	0,01	0,00	1,00	0,00	0,00	0,00	0,02	0,00	0,00	0,02	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00
5	0,14	0,00	0,14	0,00	0,99	0,00	0,02	0,00	0,00	0,02	0,00	0,00	0,15	0,00	0,14	0,01	0,00	0,11	0,00	0,00
6	0,00	0,08	0,00	0,00	0,00	1,00	0,00	0,03	0,01	0,00	0,00	0,00	0,00	0,01	0,01	0,00	0,19	0,00	0,00	0,05
7	0,38	0,00	0,05	0,00	0,02	0,00	0,98	0,00	0,00	0,00	0,00	0,04	0,00	0,00	0,01	0,00	0,00	0,16	0,00	0,00
8	0,00	0,33	0,00	0,02	0,00	0,03	0,00	1,00	0,06	0,00	0,01	0,00	0,00	0,00	0,00	0,00	0,02	0,00	0,02	0,00
9	0,04	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,96	0,00	0,00	0,01	0,00	0,04	0,00	0,00	0,04	0,00	0,00
10	0,00	0,09	0,00	0,00	0,00	0,01	0,00	0,04	1,00	0,00	0,03	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,01
11	0,00	0,00	0,00	0,01	0,00	0,00	0,01	0,02	0,01	0,00	1,00	0,00	0,00	0,00	0,00	0,00	0,01	0,01	0,13	0,13
12	0,01	0,00	0,05	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,89	0,00	0,00	0,09	0,00	0,00	0,00	0,00	0,00
13	0,00	0,00	0,03	0,00	0,16	0,00	0,00	0,00	0,00	0,00	0,00	0,18	0,98	0,00	0,00	0,01	0,00	0,01	0,00	0,00
14	0,00	0,00	0,00	0,00	0,00	0,01	0,00	0,00	0,00	0,00	0,00	0,00	0,00	1,00	0,00	0,00	0,00	0,00	0,00	0,01
15	0,02	0,00	0,09	0,00	0,14	0,01	0,02	0,00	0,00	0,02	0,00	0,12	0,00	0,00	0,98	0,00	0,00	0,25	0,00	0,00
16	0,01	0,00	0,03	0,00	0,05	0,00	0,00	0,00	0,00	0,01	0,00	0,01	0,02	0,00	0,12	0,94	0,00	0,04	0,00	0,00
17	0,00	0,00	0,00	0,00	0,00	0,19	0,00	0,02	0,00	0,00	0,01	0,00	0,00	0,00	0,00	0,00	1,00	0,01	0,01	0,01
18	0,03	0,00	0,01	0,00	0,13	0,00	0,16	0,00	0,00	0,03	0,01	0,00	0,01	0,00	0,27	0,01	0,01	1,00	0,00	0,00
19	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,02	0,00	0,00	0,13	0,00	0,00	0,00	0,00	0,00	0,01	0,00	1,00	0,01
20	0,00	0,01	0,00	0,00	0,00	0,04	0,00	0,00	0,00	0,00	0,12	0,00	0,00	0,00	0,00	0,00	0,01	0,00	0,01	1,00
21	0,00	0,00	0,52	0,00	0,08	0,00	0,01	0,00	0,00	0,00	0,00	0,00	0,14	0,00	0,04	0,01	0,00	0,00	0,00	0,00
22	0,00	0,01	0,00	0,01	0,00	0,01	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,01	0,02
23	0,00	0,01	0,00	0,21	0,00	0,00	0,00	0,01	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,01	0,00	0,02	0,00
24	0,01	0,00	0,00	0,00	0,01	0,01	0,01	0,00	0,00	0,00	0,00	0,01	0,01	0,00	0,00	0,00	0,01	0,00	0,01	0,01
25	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,03	0,06	0,00	0,18	0,00	0,00	0,00	0,00	0,00	0,04	0,00	0,02	0,20
26	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,07	0,01	0,00	0,13	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,05	0,10
27	0,01	0,00	0,00	0,00	0,00	0,00	0,01	0,00	0,00	0,02	0,02	0,18	0,16	0,00	0,12	0,00	0,00	0,15	0,00	0,00
28	0,00	0,00	0,00	0,00	0,02	0,01	0,00	0,00	0,00	0,00	0,01	0,01	0,03	0,00	0,00	0,01	0,00	0,00	0,00	0,00
29	0,01	0,00	0,00	0,00	0,05	0,00	0,00	0,00	0,00	0,01	0,00	0,02	0,18	0,00	0,00	0,01	0,00	0,00	0,00	0,00
30	0,00	0,00	0,02	0,00	0,01	0,00	0,01	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,06	0,00	0,00	0,01	0,00	0,00
31	0,00	0,00	0,00	0,00	0,00	0,05	0,00	0,01	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,08	0,00	0,00	0,04
32	0,00	0,00	0,00	0,00	0,00	0,16	0,00	0,03	0,02	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,27	0,01	0,01	0,05
33	0,00	0,00	0,00	0,00	0,03	0,01	0,00	0,00	0,00	0,00	0,02	0,02	0,04	0,00	0,00	0,00	0,00	0,02	0,00	0,02
34	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,01	0,00
35	0,01	0,00	0,07	0,00	0,01	0,00	0,00	0,00	0,00	0,00	0,00	0,04	0,01	0,00	0,00	0,00	0,00	0,00	0,00	0,00
36	0,00	0,00	0,01	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,04	0,00	0,00	0,00	0,01	0,00	0,00	0,00	0,00
37	0,00	0,00	0,00	0,00	0,01	0,00	0,00	0,00	0,00	0,13	0,00	0,00	0,00	0,00	0,00	0,46	0,00	0,00	0,00	0,00
38	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,01	0,01	0,00	0,20	0,00	0,00	0,00	0,00	0,00	0,00	0,01	0,21	0,23
39	0,00	0,00	0,00	0,00	0,00	0,05	0,00	0,01	0,00	0,00	0,01	0,00	0,00	0,01	0,00	0,00	0,04	0,00	0,01	0,19
40	0,00	0,00	0,00	0,00	0,01	0,00	0,00	0,00	0,00	0,01	0,01	0,00	0,03	0,00	0,00	0,05	0,03	0,00	0,00	0,00

	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
1	0,00	0,00	0,00	0,01	0,00	0,00	0,01	0,00	0,01	0,00	0,00	0,00	0,00	0,00	0,01	0,00	0,00	0,00	0,00	0,00
2	0,00	0,01	0,01	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00
3	0,52	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,01	0,00	0,00	0,05	0,02	0,00	0,00	0,00	0,00
4	0,00	0,01	0,21	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00
5	0,07	0,00	0,00	0,01	0,00	0,00	0,00	0,03	0,04	0,00	0,00	0,00	0,03	0,00	0,01	0,00	0,02	0,00	0,00	0,01
6	0,00	0,01	0,00	0,01	0,00	0,00	0,01	0,00	0,00	0,05	0,16	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,05	0,00
7	0,01	0,00	0,00	0,00	0,00	0,00	0,01	0,00	0,00	0,00	0,00	0,01	0,01	0,00	0,00	0,00	0,00	0,00	0,00	0,00
8	0,00	0,00	0,01	0,00	0,03	0,07	0,00	0,00	0,00	0,01	0,03	0,00	0,00	0,00	0,00	0,00	0,00	0,01	0,00	0,00
9	0,00	0,00	0,00	0,00	0,00	0,00	0,01	0,00	0,06	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,10	0,00	0,00	0,00
10	0,00	0,00	0,00	0,00	0,06	0,01	0,00	0,00	0,00	0,00	0,02	0,00	0,00	0,00	0,00	0,00	0,00	0,01	0,00	0,00
11	0,00	0,00	0,00	0,00	0,18	0,13	0,02	0,00	0,00	0,00	0,00	0,00	0,02	0,00	0,00	0,00	0,00	0,20	0,02	0,02
12	0,03	0,00	0,00	0,01	0,00	0,00	0,07	0,09	0,00	0,00	0,01	0,10	0,00	0,00	0,05	0,02	0,00	0,00	0,00	0,00
13	0,12	0,00	0,00	0,01	0,00	0,00	0,23	0,00	0,15	0,00	0,00	0,00	0,04	0,00	0,02	0,00	0,00	0,00	0,00	0,03
14	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,01	0,00
15	0,04	0,00	0,00	0,00	0,00	0,00	0,11	0,05	0,01	0,00	0,00	0,03	0,00	0,00	0,00	0,00	0,02	0,00	0,00	0,00
16	0,03	0,00	0,00	0,00	0,00	0,00	0,00	0,02	0,00	0,00	0,00	0,01	0,00	0,00	0,00	0,01	0,38	0,00	0,00	0,02
17	0,00	0,00	0,01	0,01	0,04	0,00	0,00	0,00	0,00	0,08	0,27	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,04	0,04
18	0,00	0,00	0,00	0,00	0,00	0,00	0,13	0,03	0,00	0,00	0,01	0,01	0,02	0,00	0,00	0,00	0,00	0,01	0,00	0,00
19	0,00	0,01	0,01	0,01	0,02	0,05	0,00	0,00	0,00	0,00	0,01	0,00	0,00	0,00	0,00	0,00	0,00	0,21	0,02	0,01
20	0,00	0,01	0,00	0,01	0,19	0,10	0,00	0,00	0,00	0,04	0,05	0,00	0,02	0,00	0,00	0,00	0,00	0,20	0,22	0,00
21	1,00	0,00	0,00	0,01	0,00	0,00	0,01	0,13	0,04	0,00	0,00	0,02	0,00	0,00	0,03	0,01	0,00	0,01	0,00	0,01
22	0,00	1,00	0,00	0,00	0,02	0,02	0,00	0,00	0,00	0,04	0,01	0,00	0,00	0,03	0,00	0,00	0,00	0,00	0,02	0,00
23	0,00	0,00	1,00	0,00	0,03	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,01	0,00
24	0,01	0,00	0,00	1,00	0,02	0,02	0,02	0,13	0,00	0,00	0,01	0,02	0,01	0,00	0,00	0,00	0,00	0,00	0,00	0,03
25	0,00	0,02	0,03	0,02	1,00	0,25	0,00	0,01	0,00	0,00	0,02	0,01	0,03	0,00	0,02	0,00	0,00	0,07	0,01	0,00
26	0,00	0,02	0,00	0,02	0,25	1,00	0,00	0,01	0,00	0,01	0,00	0,01	0,01	0,00	0,01	0,00	0,00	0,06	0,00	0,02
27	0,00	0,00	0,00	0,00	0,00	0,00	0,92	0,13	0,01	0,00	0,01	0,00	0,04	0,00	0,00	0,00	0,01	0,00	0,00	0,00
28	0,06	0,00	0,00	0,23	0,00	0,01	0,08	0,71	0,00	0,00	0,00	0,16	0,01	0,00	0,00	0,00	0,01	0,00	0,00	0,02
29	0,03	0,00	0,00	0,00	0,00	0,00	0,10	0,00	0,96	0,01	0,00	0,02	0,00	0,05	0,00	0,00	0,10	0,00	0,00	0,02
30	0,10	0,00	0,00	0,02	0,00	0,00	0,02	0,05	0,01	0,00	0,00	0,80	0,01	0,00	0,00	0,02	0,00	0,00	0,01	0,00
31	0,00	0,04	0,00	0,00	0,00	0,01	0,00	0,01	0,01	1,00	0,22	0,00	0,02	0,02	0,00	0,00	0,00	0,01	0,01	0,00
32	0,00	0,01	0,00	0,01	0,02	0,00	0,01	0,00	0,00	0,21	1,00	0,00	0,03	0,00	0,01	0,00	0,00	0,00	0,01	0,01
33	0,00	0,00	0,00	0,02	0,03	0,02	0,03	0,01	0,00	0,02	0,04	0,12	0,95	0,00	0,12	0,02	0,02	0,00	0,00	0,00
34	0,00	0,03	0,00	0,00	0,00	0,00	0,00	0,00	0,07	0,02	0,00	0,00	0,00	1,00	0,01	0,00	0,03	0,00	0,00	0,03
35	0,04	0,00	0,01	0,00	0,01	0,00	0,00	0,01	0,00	0,00	0,01	0,00	0,16	0,01	0,97	0,00	0,00	0,00	0,00	0,00
36	0,01	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,03	0,00	0,09	0,98	0,00	0,01	0,02	0,02
37	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,11	0,00	0,00	0,00	0,02	0,03	0,00	0,00	1,00	0,00	0,00	0,03
38	0,00	0,00	0,00	0,00	0,06	0,05	0,00	0,00	0,00	0,01	0,00	0,00	0,00	0,00	0,00	0,01	0,00	0,99	0,06	0,01
39	0,00	0,03	0,01	0,00	0,01	0,00	0,00	0,00	0,00	0,01	0,01	0,01	0,00	0,00	0,01	0,02	0,00	0,02	0,99	0,00
40	0,01	0,01	0,00	0,02	0,00	0,01	0,00	0,01	0,04	0,00	0,01	0,00	0,00	0,10	0,00	0,01	0,10	0,00	0,00	0,87

Figure 4.91 MAC analysis of the joint displacements of the selected points in SAP2000 models (ground formed with rigid supported solid elements vs. ground formed with rigid supports at the end of the cantilever beams and specified points of the cantilever elements)

CHAPTER 5

CONCLUSIONS

This thesis covers the investigation of original form of collapsed Historic Timber Buzlupınar Bridge located in Rize, Turkey, as well as structural & architectural characteristics, structural evaluation, and forensic analysis of the bridge collapse. Recommendations for the Buzlupınar Bridge reconstruction project in accordance with conservation principles are also provided.

The studies were started with field survey in October 2013, the bridge was partially collapsed, deteriorated, and in a derelict condition back then and continued with the examination of the previously made measured surveys and documentation. With the help of the measured surveys and old photographs of the bridge, its original architectural and structural characteristics were determined. Material tests of the original bridge remains done by the General Directorate of Highways and Forest Industrial Engineering Department of KTÜ have also showed that the existing timber elements of the bridge have substantially lost their strength.

The Buzlupınar Bridge is a rare example within the bridges in Black Sea Region with its structural and architectural features. It is a significant structure in its neighborhoods and locals have immaterial connection with the bridge and would like to have it standing as a landmark for their society. The bridge was decided to be rebuild in accordance with the original architectural and structural features but using new chestnut timber material. Hereby, it is important to note that the reconstruction project should be in line with the conservation principles and before any process it is proposed to test all of the remaining timber elements and reuse and/or present structurally healthy ones on the reconstruction.

In conservation of the historic structures, if possible, it should be aimed to conserve the authenticity of the structure; protect, consolidate and retain the original elements and materials as an evidence of the original form of the structure. To this respect, first of all it is recommended to investigate the physical and mechanical condition of the existing timber before any interventions; if needed, to take samples from the existing timber elements, test them and determine their strength properties. If any of them ensures the design strength criteria, it should be retained and reused on the reconstruction process taking due precautions. The causes of damage on the materials should be determined. The disassembled elements should be documented. The reconstruction should be based on the original documents. The wood used in reconstruction should be at least the same quality with the original one. New structural elements, nails and secondary elements should be produced in the original form and dimensions, using the same craftsmanship and technique. Changes in dimensions can be done if it is inevitable for the structural safety.

The structural analysis and evaluation studies were started with materials tests to determine the mechanical parameters of the timber that is going to be used on the reconstruction project. The material tests were conducted out in the Civil Engineering Department Materials Lab in METU. The bending strength of the timber was obtained as 47.34 N/mm^2, mean compression strength parallel to grains was obtained as 33.23 N/mm^2, and mean tension strength parallel to grains was obtained as 49.73 N/mm^2. Furthermore, nail tests were carried out to obtain the shear and tension capacities between timber blocks. The maximum shear forces that the ϕ24mm and ϕ12mm threaded bars (yielding strengths are 318 MPa and 270 MPa, respectively), which were deformed into square cross section and used to connect two timber blocks, was found to be 50 kN and 10 kN, respectively. The tensile capacities are a function of the friction between the timber and nail, which was depreciated by the initial drilling into the timber. Some of the nail tension failures were governed by the nail head moving into the drilled hole while other failures were governed by the friction between the timber and nail, where epoxy glue was initially used (the specimens were prepared and shipped to METU by the restoration company). The tension tests were done in two stages for the ϕ24mm; first the upper log was pulled while the other was standing firm, when the first one was finished the upper log was taken apart and the nail was pulled from the lower log. The nail resisted maximum of 20 kN and 45 kN for this two test, respectively. The capacity of the ϕ12mm nail was not tested in tension but calculated using information obtained from the ϕ24mm tension tests and were expected to be as 3.24 kN for epoxy unapplied case and 10.08 kN for epoxy applied case.

The hand calculations for the bending of the main span and deck plates were carried out to calculate the capacity versus demand. The bending stresses generated on the main beams under live loads were obtained as 4.92 MPa in compression at top and 6.79 MPa tension at the bottom of the beam, in case the deck plates and main beams work compositely by application of nails. The shear flow between the main beams and 0.10 m thick covering plates were obtained as 96.19 kN/m to be transferred by the nailed connection. The number of required ϕ12mm nails to be used on each connection between the deck plates and main beams were determined as six. Critical failure modes were also checked for overturning of the bridge under wind load and the bridge was found to be vulnerable against wind forces. Recommendations include the use anchor wires against overturning or use of bolted rods for improved load transfer between the main beams and the cantilever support beams.

The Finite Element Model (FEM) of the bridge was constructed using SAP2000 with detailed meshing using frame, shell, and solid elements. The critical forces between connections were obtained for Dead Load (DL), Live Load (LL), Wind Load (WIND) and Earthquake Load (EQ); nail designs were checked considering the combined forces (DL+LL, DL+WIND, DL+EQ) at the connections and nail capacity in combined tension & shear. The safety of the critical connections were checked using the load carrying capacity values of the nails; the result obtained under DL+LL (1.41 on the connection between the deck plates and main beams) and under DL+WIND (1.86 on the connection between the main beams and cantilever beams) are found to be not acceptable. These calculations were repeated using the maximum loads that the epoxy applied nails can resist and the results were decreased to 0.38 for both cases; which are admissible. However, long term performance of the epoxy glued nail connections is not known and it should be separately investigated. Instead of using epoxy applied nails, it is proposed to use longer nails and on the connections between deck plates and main beams and main beams cantilever beams and bend the end of the nails as it was originally done on some connections of the bridge as shown in shown in Figure 5.1 and Figure 5.2. Recommendations include use of anchor wires against overturning or bolts could be used at the edges of the nails since the nails have a screwed form originally.

Figure 5.1 Detail of the original connection between horizontal beams and cantilever beams (E.Çabuk, 2013)

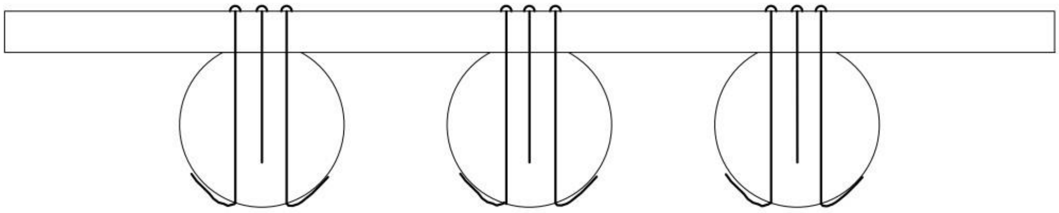


Figure 5.2 It is proposed to bend the ends of the nails on the connection between the deck plates and tie beams

The lengths of the cantilever beams (which consists of 5 rows) inside the soil are proposed to be extended about 1m. As a result of the hand calculations for the shear flow between the main beams and deck plates the required numbers of nails to be used on connections between these elements were determined. It is recommended to use 6 nails on each connection between the deck plates and main beams on the first 2.5 m of the span on both sides. The connections between 2.5 m to 5 m are recommended to be made with 4 nails, for the other connections 2 nails will be adequate. The first 4 deck plates at each side of the tie beam which is in the middle of the span should be connected to the main beams with 6 nails. It is also recommended to use 6 nails at each connection between main beams and one each of deck plates at each side of the tie beams. For the connections between the tie beams and main beams 4 nails are adequate (Figure 4.6).

As determined using the expressions given in Eurocode 5, the deck plates should be pre drilled and the diameter of the pre drilled holes should not exceed 0.8 cm; the minimum spacings between two nails should be 5 cm in the row parallel to grains and 4 in the row perpendicular to grains. Also the minimum values for end distance and edge distance were determined as 12 cm and 7 cm, respectively.

The long time performance of the wood is not investigated within the scope of this study but the aging and deterioration of wood would cause lose in strength over the time. The natural factors like the effect of sun light, water, moisture or fungal attack could reduce the strength of the timber too and this will possibly reduce the service life of the bridge.

In addition, the climate change is another issue which has to be considered for the future strength of the wood. In Black Sea Region where the bridge is located, the temperature and humidity are expected to be increase in the upcoming years (Bozkurt, Şen, Göktürk, Dündar, & Altürk, 2012). For this reason, it is also proposed to do further studies about the climate change and its possible effects on the condition and strength of the timber elements.

Proper monitoring and maintenance studies of the bridge should be done for a longer service life after the reconstruction studies. Monitoring is to record the condition of the materials and structural elements periodically, in an attempt to determine if there is damage or any changes in the materials and geometric properties of the structure. It must be followed by a proper maintenance which includes the activities required to extend the lifetime of a structure (Freas, 1982). After the reconstruction, regular monitoring and maintenance of the bridge is necessary to extend its service life. The joints and nailed connections, the parts between the structural elements such as the deck plates and main beams and the parts which could be damaged easily should be checked and examined carefully and proper maintenance activities and strengthening should be done if needed. In addition, any of the seismic activities, high winds, heavy rains or heavy snow loads that the bridge is exposed should be recorded, if the structure is survived once it does not mean it will survive again in such a situation. All of the monitoring and maintenance activities prolong the life of the bridge; also reduce the frequency and cost of the repairs and improve safety (Freas, 1982).

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