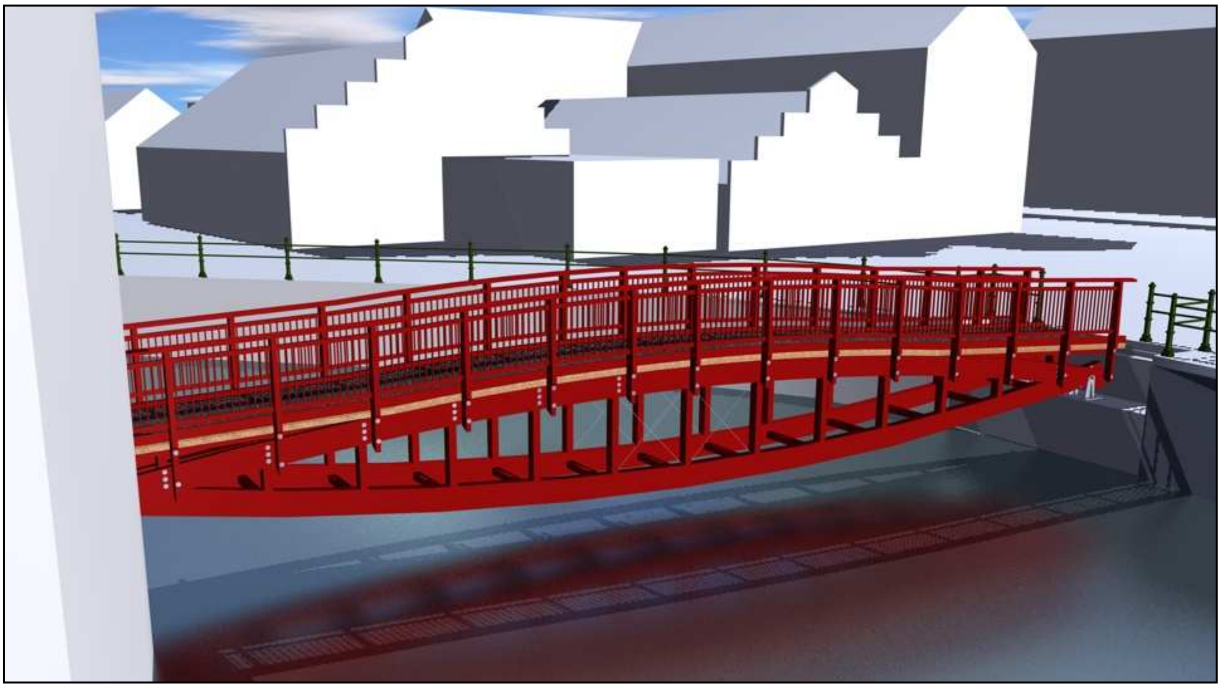
CHALMERS



From a conceptual design to a structural solution and erection of a timber bridge – a case study

Master of Science Thesis in the Master's Programme Structural Engineering and Building Performance Design

MATTIA TOSI

Department of Civil and Environmental Engineering
Division of Structural Engineering
Steel and Timber Structures
CHALMERS UNIVERSITY OF TECHNOLOGY
Göteborg, Sweden 2010
Master's Thesis 2010:78

MASTER'S THESIS 2010:78

From a conceptual design to a structural solution and erection of a timber bridge – a case study

Master of Science Thesis in the Master's Programme in Structural Engineering and Building Performance Design

MATTIA TOSI

Department of Civil and Environmental Engineering
Division of Structural Engineering
Steel and Timber Structures
CHALMERS UNIVERSITY OF TECHNOLOGY

Göteborg, Sweden 2010

From a conceptual design to a structural solution and erection of a timber bridge –
a case study

Master of Science Thesis in the Master's Programme in Structural Engineering and Building Performance Design

MATTIA TOSI

Examensarbete / Institutionen för bygg- och miljöteknik,
Chalmers tekniska högskola 2010:78

Department of Civil and Environmental Engineering
Structural Engineering
Chalmers University of Technology
SE-412 96 Göteborg
Sweden
Telephone: + 46 (0)31-772 1000

Cover:
Rendering of the Fish Belly Bridge in Borås

Department of Civil and Environmental Engineering, Göteborg, Sweden 2010

From a conceptual design to a structural solution and erection of a timber bridge – a case study
Master of Science Thesis in the Master's Programme in Structural Engineering and Building Performance Design
MATTIA TOSI
Department of Civil and Environmental Engineering
Division of Structural Engineering
Steel and Timber Structures
Chalmers University of Technology

ABSTRACT

In recent years, timber as construction material for bridges and footbridges has had a great development both in urban contexts and in natural environments since it allows to perform lightweight, economic and aesthetically pleasing as well as durable and easy to implement structures. Improvement and refinement of manufacturing processes of the "artificial" wood based products such as glued laminated timber (glulam) and Laminated Veneer Lumber (LVL), considerable progress over the last twenty years in the production of more and more efficient connecting devices, in making marketing of effective and safe products for wood protection and in the evolution of building systems have contributed decisively to this new trend in the structural field. This Thesis, inspired by the Timber Bridge Competition that took place at Chalmers University of Technology in February 2010, analyzes the whole design process of the 22 m long glulam timber footbridge to be built in Borås, which has been the winner project.

All the different steps that have followed during the development of the project are particularly analyzed: from the first ideas sketched on paper during the conceptual design phase, through the analysis of the two models in scale 1:20 built in wood and tested to failure during the Competition, up to the precise definition of all the constructive details and structural solutions needed to actually build the footbridge and ensure it a good durability and its effective construction.

The Thesis has been articulated as follows: in the beginning, along with an attempt to clarify the role that the bridge has always played in the European culture, the theoretical basis of the conceptual design process have been described. A wide overview of the most important examples of timber bridges built in history (from Julius Caesar to the present day) made to clarify the background that has laid the introductions for the current developments of this type of structures and an analysis of the advantages and disadvantages associated with the use of timber in bridges conclude the introductive section.

In the second part the description and the analysis of the most interesting project among the five ones participating to the Competition and of its structural characteristics are carried on.

In the third section the analysis focuses only on the winning project and its gradual translation from the idea to an actually feasible structural solution on the basis of building technologies, features and details to ensure durability and on the verifications in ULS and SLS based on EC5. In the fourth part the SCP, the Maintenance Plan and the Building Booklet are presented in order to describe how to build and to maintain the footbridge.

Key words: timber footbridge, conceptual design, durability, glulam timber, LVL timber products, constructive details, building site.

From a conceptual design to a structural solution and erection of a timber bridge – a case study
Master of Science Thesis in the Master's Programme in Structural Engineering and Building Performance Design
MATTIA TOSI
Department of Civil and Environmental Engineering
Division of Structural Engineering
Steel and Timber Structures
Chalmers University of Technology

ABSTRACT

Negli ultimi anni il legno come materiale da costruzione per ponti e passerelle pedonali ha avuto un grande sviluppo sia in ambito urbano che in contesti naturali considerato che esso permette di realizzare strutture leggere, economiche, esteticamente accattivanti oltre che durature e di semplice realizzazione. A questa nuova tendenza in campo strutturale hanno contribuito in maniera determinante il miglioramento e il perfezionamento dei processi produttivi dei materiali "artificiali" a base di legno quali il legno lamellare e il Laminated Veneer Lumber (LVL) e i notevoli progressi compiuti negli ultimi venti anni nella produzione di elementi di connessione sempre più efficienti, nella messa in commercio di prodotti per la protezione del legno efficaci e sicuri e nella evoluzione dei sistemi costruttivi.

La Tesi, prendendo spunto dalla Timber Bridge Competition che si è tenuta presso la Chalmers University of Technology nel Febbraio 2010, analizza nella sua interezza il processo di progettazione della passerella pedonale in legno lamellare di 22 m da realizzare a Boras che è risultata vincitrice. Vengono in particolare analizzati tutti i diversi step che si sono susseguiti durante lo sviluppo del progetto: dalle prime idee schizzate sul foglio di carta durante la fase di conceptual design, passando attraverso l'analisi dei due modelli in scala 1:20 costruiti in legno e testati a rottura durante la Competition, fino ad arrivare alla definizione precisa di tutti i dettagli costruttivi e le soluzioni strutturali necessari per poter costruire realmente il ponte e garantirne una buona durabilità nel tempo.

La Tesi è stata articolata nel modo seguente: nella parte iniziale, oltre a cercare di chiarire il significato che il ponte ha sempre avuto nella cultura europea, sono state spiegate le basi teoriche del processo di conceptual design. Una panoramica dei più importanti esempi di ponti in legno realizzati nella storia (da Giulio Cesare a giorni nostri) fatta per chiarire il background che ha posto le premesse per gli sviluppi attuali di questo tipo di strutture e un'analisi dei vantaggi e svantaggi legati all'impiego dei ponti in legno concludono la parte introduttiva.
Nella seconda parte si procede alla descrizione e all'analisi delle caratteristiche strutturali e del comportamento statico del progetto più interessante tra i cinque altri partecipanti alla Competition.
Nella terza parte infine l'analisi si concentra unicamente sul progetto vincitore e sulla sua graduale traduzione da idea a soluzione strutturale effettivamente realizzabile sulla base di tecnologie costruttive, dettagli per garantire funzionalità e durabilità e le verifiche agli Stati Limite Ultimi e Stati Limite di Servizio sulla base dell'EC5.

Contents

Preface

This Master Thesis work has been developed from January 2101 to June 2010 at the Division of Structural Engineering, Department of Civil and Environmental Engineering at Chalmers University of Technology in Göteborg, Sweden and then finished at the University of Trento, Italy during September and October 2010.

First of all I like to thank Prof. Maurizio Costantini, Professor of Building Production at the Faculty of Engineering, University of Trento, and Prof. Maurizio Piazza, Professor of Timber Engineering at the Faculty of Engineering, University of Trento for accepting to be my supervising professors for this Thesis.

My heartfelt thanks then dutifully goes to my supervisor Eng. Roberto Crocetti, Professor at the Chalmers University of Technology, who also played the role of co-supervising professor, for having proposed me the theme of this Thesis, for giving me a patient and constant help and also for having supported me in difficult times and giving me the opportunity to do such a work and experiences that otherwise I would have not be able to do in Italy.

This Thesis has been the opportunity to face in person the design of a wooden structure and has greatly enriched my cultural background, not to mention the benefits of having developed the work in English and having worked closely with students in Sweden: I have felt pride and a "Chalmerist” in effect.

I must also thank Prof. Karl-Gunnar Olsson who, though not officially involved in this Thesis, and despite being constantly busy, has always given me his time and desire to recommend me interesting cues to be added in my work and kindness to lend me books of his personal library to be included in my bibliography.
It should not be forgotten that the software *PointSketch* used in Chapter 4 is one of his creations.

I would express my gratitude to Prof. Robert Kliger, examiner of this thesis at Chalmers University of Technology, for his interest in me.

I would also like to thank Edwin Ogbeide, my opponent, for his help and his comments on my Thesis, Georgi Nedkov Nedev and Simon Johansson for the beautiful experience lived together and their collaboration, and Thomas Kruglowa, researcher and teaching assistant in the course of Timber Engineering at the Chalmers University, for his kindness.

My special thanks and infinite gratitude goes then dutifully to my parents, who have always supported and listened to me throughout my university career, but particularly during my six not always easy “Swedish” months.

Verona, October 2010

Mattia Tosi

Notations

Roman upper case letters

A	Cross-sectional area
C_e	Exposure coefficient
C_t	Thermal coefficient
$E_{0,05}$	Fifth percentile value of modulus of elasticity
$E_{0,mean}$	Mean value of modulus of elasticity
$F_{ax,Rk}$	Characteristic axial withdrawal capacity of the fastener
$F_{v,Rk}$	Characteristic load-carrying capacity per shear plane per fastener
$F_{w,d}$	Wind horizontal force
G_{mean}	Mean value of shear modulus
J	Moment of inertia
M	Total mass of bridge
M_d	Design bending moment
$M_{y,Rk}$	Characteristic fastener yield moment
N	Axial force
W	Section modulus
X_d	Design value of a strength property
X_k	Characteristic value of a strength property

Roman lower case letters

$a_{vert,1}$	Vertical acceleration from one person crossing the bridge
$a_{vert,n}$	Vertical acceleration from several people crossing the bridge
d	Fastener diameter
$f_{c,0,d}$	Design compressive strength along the grain
$f_{c,90,d}$	Design compressive strength perpendicular to the grain
$f_{c,0,k}$	Characteristic compressive strength along the grain
$f_{c,90,k}$	Characteristic compressive strength perpendicular to the grain
$f_{m,d}$	Design bending strength
$f_{m,k}$	Characteristic bending strength
$f_{t,0,d}$	Design tensile strength along the grain
$f_{t,90,d}$	Design tensile strength perpendicular to the grain
$f_{t,0,k}$	Characteristic tensile strength along the grain
$f_{t,90,k}$	Characteristic tensile strength perpendicular to the grain
$f_{v,d}$	Design shear strength
$f_{v,k}$	Characteristic shear strength

f_{vert}	Fundamental natural frequency of vertical vibrations
g_k	Characteristic value of uniformly distributed load (selfweight)
k_{mod}	Modification factor for duration of load and moisture content
$k_{c,z}$	Instability factor
k_{crit}	Factor used for lateral buckling
k_{def}	Deformation factor
k_m	Factor considering redistribution of bending stresses in a cross-section
k_{vert}	Coefficient
l	Length
m	Mass per unit length
n	Number of pedestrians
n_{ef}	Effective number of fasteners in line parallel to the grain
q_d	Design value of uniformly distributed load
q_{snow}	Characteristic value of uniformly distributed load (snow)
q_{vk}	Characteristic value of uniformly distributed load (live load)
s_k	Characteristic value of snow on the ground
t_i	Timber thickness depth
u_{fin}	Final deformation
u_{inst}	Instantaneous deformation
$u_{inst,G}$	Instantaneous deformation for a permanent action G
$u_{inst,Q}$	Instantaneous deformation for a variable action Q
$u_{net,fin}$	Net final deflection

Greek lower case letters

β_c	Straightness factor
$\gamma_{asphalt}$	Weight per unit volume of asphalt
γ_G	Partial safety factor for permanent loads
γ_{GL24h}	Weight per unit volume of glulam timber GL24h
γ_{KertoQ}	Weight per unit volume of KertoQ
γ_M	Partial factor for material properties
γ_Q	Partial safety factor for variable loads
γ_{solid}	Weight per unit volume of solid timber
ζ	Modal damping ratio
$\lambda_{rel,y}$	Relative slenderness ratio corresponding to bending about the y-axis
$\lambda_{rel,z}$	Relative slenderness ratio corresponding to bending about the z-axis
λ_z	Slenderness ratio corresponding to bending about the z-axis
μ_i	Shape coefficient

ρ_k	Characteristic density
$\sigma_{c,0,d}$	Design compressive stress along the grain
$\sigma_{m,d}$	Design bending stress
$\sigma_{t,0,d}$	Design tensile stress along the grain
χ	Shear factor
ψ_0	Factor for combination value of a variable action
ψ_2	Factor for quasi-permanent value of a variable action

1 General introduction

1.1 Problem description

1.1.1 Background

In recent years, timber as construction material for bridges and footbridges has had a great development both in urban contexts and in natural environments since it allows to perform lightweight, economic and aesthetically pleasing as well as durable and easy to implement structures. Improvement and refinement of the manufacturing processes of the "artificial" wood based products such as glued laminated timber (glulam) and Laminated Veneer Lumber (LVL), considerable progress over the last twenty years in the production of more and more efficient connecting devices, in making marketing of effective and safe products for wood protection and in the evolution of building systems have decisively contributed to this new trend in the structural field.

The theme of this Thesis has been the analysis of the entire process of designing a footbridge in order to understand in detail the dynamics and everything that happens throughout the *iter*: in fact, one usually focuses on some specific aspects, such as structural calculations. In this way, the Author believes he has made an interesting overview of the complexity of the work of a designer, treating both the architectural and structural aspects and not forgetting the most practical one, that is the effective execution of the project.

1.1.2 Aims of the thesis and limitations

The theme of this Thesis is a complete analysis of the design process of a wooden footbridge.
From the embryonic stage, that is the moment when the designer decides, starting from the needs perceived by the client and the environmental conditions of the site and other types of constraints, such as the static scheme and the structural principle to adopt, through the development of technological solutions and constructive details in order to translate into reality what he thought, to go then for checking the strength of the various elements on the basis of the Eurocodes and then finishing with the analysis of the strategies, machineries and phases by which actually build the bridge.
It is extremely interesting to analyze the process in its entirety to understand what and how many variables are involved at each step of the design of the structure, which obviously go to significantly influence the following stages, dramatically changing the decisions that have to be taken.

Obviously, since due to the vast scope of the investigation it has been necessary in some cases, however, to simplify or streamline the analysis in order not to lose sight of the ultimate goal of the work that aims precisely to provide a comprehensive and as much exhaustive as possible overview of the entire process.
Since the design of the structure has been quite empirical (the competition provided the building of scale models without any preliminary calculation) so the definition of the static model to be used has had a simplified management and in-depth analysis on

the possible alternatives have been not conducted. Finally, after having seen that the model built to scale had a satisfactory response, it has been decided to go further.

Concerning the definition of the loads acting on the footbridge, being the structure located between a building and the bank of the canal, the actions of any emergency or cleaning vehicle transiting on the deck have been disregarded, considering in the end only the actions of wind, snow and crowd (as provided by the Eurocodes).
To conclude in the chapter dealing with the installation and maintenance of the bridge the regulations have been complied with as regards the substance but from the formal point of view they have been adapted to the purposes and the need for discussion of the topic: in particular, the sheets concerning the use of all machineries and equipment used in the different working phases have been left out and the Maintenance Plan has been partially merged with the Building Booklet.

1.1.3 Outlines

This Thesis work is essentially divided into four parts. The first part is from Chapter 1 to Chapter 3 and serves to clarify and present the discipline in which we will work, to introduce properly discussion of the subsequent topics, or the wooden bridge designed by the Author.
Chapter 1 is the very *incipit* of the work: in addition to a general description of the problem underlying the thesis, it describes what is the symbolic meaning and social value that the bridge has generally in the collective imagination and its importance to levels not only practical but also philosophical. The second paragraph presents a concise treatment of logical and mental processes that govern the conceptual design phase of any building.
In Chapter 2 the focus shifts to the technological aspects related to the use of wooden bridges: the benefits that they bring to those of reinforced concrete and steel and at the same time the related problems are analyzed, both from the point of view of the manufacturing and of the durability (element to be designed carefully in the timber structures field).
Chapter 3 presents instead a historical analysis of the evolution of wooden bridges over the centuries: from the first of which we have a complete description, that is Julius Caesar's bridge over the Rhine, up to present days underlining the constants and differences and focusing on structural, constructive and aesthetic aspects.

The second part (Chapters 4 and 5) enter the substance of the main topic of the thesis, namely the design of a footbridge made of glulam timber. Since the beginning of the process has been the Bridge Competition that took place at Chalmers University in Goteborg, in Chapter 4, the course of the competition and the most interesting results produced by the participants have been illustrated and then the discussion has focused, in Chapter 5, only on the winning project, developed by the Author in co-operation with two other students. Initially it is described the whole design process undertaken to define the shape of the bridge and then the two structural solutions for the bridge are analyzed, in the form of scale models, based on the strengths and weaknesses found during the load tests. It continues with a description of the peculiarities and technologies used to produce the materials that make up the structure (glulam timber, KertoQ®), the analysis of static behavior of the footbridge conducted with the help of SAP2000® software (finite element analysis) and then it concludes the chapter with the definition of all main structural connections and analysis of solutions used to ensure the longest possible service life to the bridge.

The third part, consisting of Chapter 6, includes verifications of the bridge based on the Eurocodes, in particular 5. Design values for the strentgth of the materials, loads acting on the bridge, load cases, stress verifications in the ULS and the design in the SLS are analyzed.
All calculations have been performed by hand, basing on the results obtained from FE analysis developed by SAP2000®.

The fourth and last part, or Chapter 7, finally, is the most practical on the bridge: it deals with its construction and maintenance. In fact, based on the Italian law, the PSC have been developed (Plan and Coordination of Safety) which defines the steps necessary to prepare the building site and to assemble the bridge, the risks and safety measures to be taken to grant the health of workers, and the building area layouts that graphically represent the building site and its organization. Finally the Maintenance Plan and the Building Booklet have been prepared: they are the documents to be used during maintenance to ensure an adequately long service life to the construction and to define security measures to be taken to carry out maintenance operations without risks to workers' health.

1.2 About the bridge

The bridge, one of the most ancient archetype of the architecture, suggests two ways of movement: that of the river flowing beneath and that of the road running above: two movements which form a cross, two movements that become possible (though the one seems to prevent the other) through the rising of the road ribbon that shirks the binding condition of the crossing, the bridge replaces the ford, it is a partial and ephemeral victory of the man on the obstacle of water.
The victory becomes lasting when the road has changed into a bridge: it has jumped over the obstacle, has joined the two opposite "banks" into one thing; a sight continuity that passes over the fracture and makes it forgotten.
Using an intense poetic image Giorgio De Chirico attaches the Romans' passion for the arch to the desire of infinite that hits the man observing the sky vault: "A Rome le sens du présage est quelque chose de plus vaste. Une sensation de grandeur infinie et lointaine, la même sensation que le constructeur romain fixa dans le sentiment de l'arcade reflet du spasme d'infini que la couche célestielle produit quelquefois sur l'homme. Souvent le présage y était terrible come l'hurlement d'un dieu qui meurt. Des nuages noirs devaient s'approcher jusque sur les tours de la ville".
Through the bridge the river reveals its mobile nature, its flowing towards a constant direction. When a man looks out and sees the stream, his sight is something like dragged away, his imagination supposes the presence of a spring and of a mouth far away. So, he feels steady in the middle, in the very heart of what moves, of what continuously changes.
The water hit by his eyes is after a moment no longer the same. So the bridge is the archetype of the fight between being and becoming. The bridge has an anthropomorphal matrix because our body can be a bridge any time, like when we jump across a little creek included between the pair of compasses formed by our legs,

it is changed into a bridge when two hands shaking each other create a symbolic union between two living beings.
A godness with arched body symbolized in ancient Egypt the sky vault. The terminology of the bridge expresses its anthropomorphal matrix; the bridge has its shoulders, its brain, and its back. Every bridge has its parapet that enables us to lean out on the hollow, a wall with the right height to keep us back, that needs and gratifies the contact with our body. The symbolic fascination of the bridge had seduced even Goethe, who in his Maerchen in 1795 tells the story of a snake finally transformed into a wonderful bridge. “Do honour the memory of the snake, said the man, you owe him the life , tanks to him the two banks are now one village full of life.”
Goethe saw in the snake mainly the sign of friendship and of dialogue, but the image of the “transparent and bright snake” transformed into a bridge , like the one Palladio imagined for Rialto, contains in itself , besides the mystic architectural union between Palladio and Bernini, also the intuition of a deep and mutual analogy. Bridge and snake share the linearity, the dynamism, the repeated structure, the double movement in two directions, the making one thing out of two.. In the tale the snake winds the remote in one circle, thus recalling the ouroburos, symbol of eternity. But in this Goethe sees something else, an allegory of a happy conjunction that the man can make true with love, confidence and friendship.
The most clever and ingenious scientist of morphology, D'Arcy Wentworth Thompson, tells how a Swiss engineer became aware of the analogies between the bones’ structure and a giant crane he was designing. Prof. Culmann of Zurich learnt from his friend, the anatomy scientist, that the bone fibers arrangement is no more no less that the diagram of the isostatic lines, directions of compression and tension of the loaded structure: shortly, nature makes the bone stronger exactly in the mechanically necessary way.
In the arm of a crane the internal side on which the loaded end protrudes is the compressed part, the external one is the tense part: the compressed isostatic lines, starting from the loaded surface, join together in the direction of the resulting pressure, until they form a narrow beam that runs along the compressed side of the arm., while the tense isostatic lines running along the opposite side of the arm, spread in the end crossing the compressed lines at right angle.
The head of thigh-bone (femur) is a little bit more complicated in its shape and a little less symmetric of the schematic crane of Culmann [...]. but we have no difficulty to see that in a few words the mechanical disposition of the structure meets exactly the theoric diagram of the crane.
Taking into account of the different situations , we can draw the same disposition in any other bone bearing a load and subject to flexion. [...].Observing animals in captivity, we see that bone structure much differs from free wild animals.
We can learn many things from the examination of the birds’ bones, where the small bones corresponding to those of the hand have a heavy task to comply, since they support the long feathers of the wings and form the stiff rod of the final part of wings. The simple tubular structure, good for the long and thin bones of the arm, is no longer sufficient, because here a higher stiffness is requested. No other anatomy part is more beautiful that the metacarpal bones of the vulture.
The engineer can see here a perfect Warren scaffolding like that of the beams of aircrafts.

The analogy between the Warren beam and the vulture's metacarpal bones introduces the question of the truss spatial structures and the great nature scientist emphasizes the complex relationship with the sinews and muscles system.
He compares the skeleton of a horse with the Firth of Forth bridge, one of the masterpieces of the 17th century engineering. In a horse or an ox it is obvious that the two piers of the bridge, i.e. the fore and rear legs, do not bear (as in the a.m. bridge) independent loads, but the entire system makes up one structure.
The calculation is a little more complicated, but we can simplify the problem if we consider the skeleton as two shelves (fore and rear legs) and later consider that they are not independent but connected to each other in one system of forces and in one building design.
The book of D'Arcy Thompson was published in 1917 and influenced in some way the discussion that prepared the terrain to the architectural rationalism developed by the historical avantgardes in the years around the first world war.
It is asserted by Robert Maxwell in 1978, when he presented the works of Robert Venturi: "the book "On Growth of Form" shows that the forms of natural living structures are in harmony with a series of building principles and of geometric proportionality. Nature seems to act as if its forms were the result of a time saving economy and need of functionality. When these principles are applied to engineering structures like bridges, one discovers that they behave the same way and produce similar or almost similar forms. This discovery seemed to suggest that the general design principles were common to all natural or artificial constructions and that in them there were nothing specifically human, hence merely human conventions become useless and would be rather obstacles to a natural process that can attain to perfection if inspired by the laws of evolution".

1.3 Conceptual design

Underlying the construction of any structure there is always a perceived need by someone (and the term someone can be referred both to an individual and a community of individuals) and thus the spontaneous request for a solution that satisfies it was born.
The process of constructing a building or infrastructure, as it can be a bridge, is the result of the close interaction between a large number of figures involved with various functions and at different stages of the design in order to be able to translate into practice what initially was perceived as a need. The design of the structure is divided into phases in which the various professionals give their contribution relating to their capacities and the task they are requested to perform.
To understand the extent, the methods of implementation and the organization in carrying out any building the project life cycle should be described; it is a scheme of the typical stages in a process that follows any project to be transformed from a simple sketch on a sheet of paper to a real object: a design phase is followed by a construction phase which then ends to make room for the service life of the building where the structure, after being completed, is ready to be used by those who have expressed the need to make this happen. At each stage there are of course some inputs to elaborate and some objectives to achieve in order to get to the end of this phase with certain results then have to be used in subsequent processing.

As it is possible to see from Figure 1.1 the design phase is subdivided into three subcategories: conceptual design, preliminary design and detailed design. This thesis only deepen the discussion about the conceptual design, referring to other texts for discussion of other topics.

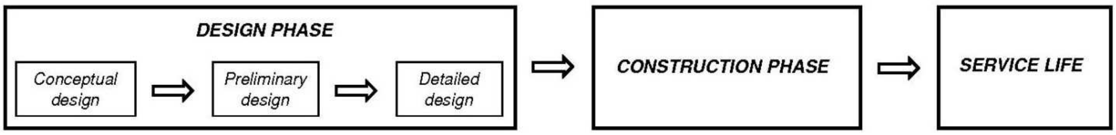


Figure 1.1   The project life cycle

Conceptual design is the starting point for the implementation of any project: it can be defined as the founding moment of the entire design process, as during this phase something completely new is going to be created on to satisfy the needs perceived by those who requested the project. The ideas that emerge during the analytical and preparation work lead to the implementation of different solutions that try to satisfy as fully as possible the needs (which are the real booster of the whole design process).
The options processed (usually three, but even more are possible) are compared with each other trying to find the one which optimizes the factors involved and which could be the best answer to all requirements, so that in the conclusion of this stage a final conceptual solution that will be later further developed is chosen.
It is important to emphasize that this phase is extremely important to the economy of the entire process of design and construction of the structure, because in this moment the foundations are laid and major decisions are taken that will have great influence on later stages and on the entire work .
The very first step to take is that of analyzing and identifying in a precise and complete way the needs which give rise to the request of a new structure: this will lay the foundations for an effective design that will be even much better the more clearly the needs are understood. In this way the design requirements are also focused, so that all the professionals involved can understand what their work and what their operational role will be.
During the identification of the needs it is essential to understand what the real needs are because very often clients who express this state of necessity are not experts and therefore they are not able to put questions in technical terms. Here, therefore, the analysis of the needs makes the designers able to understand the difference between perceived problems and real need, or understand clearly what are the client's needs rather than what he thinks he needs (Kroll, 2001).
During the needs analysis it is also important to recognize and avoid referring to bias in the definition of the need itself: the designer must be able to ensure that potentially suitable alternatives are not excluded "a priori" because of the influence of solutions already taken or ideas related to the everyday practice.
The best way to produce viable solutions is to succeed in focusing only on current issues without referencing to ideas already used: doing this way there will be the certainty that the final result will be of higher quality and we will be sure to have produced something truly innovative.
At the end of this stage there will be a sufficient number of design requirements.
To be able to get a good assortment of solutions to satisfy the needs it is helpful to see possible solutions as a black box with inputs and outputs: the solution will be acceptable if it enables the designer to connect any requirement "ingoing" with one of

the “outgoing” project objectives. The task of the designer will be to understand and clearly define all the elements required to make this black box work (Kroll, 2001).

Once a number of alternatives have been worked out they must be compared with the constraints of the real world. Imagining the set of all potential solutions as a virtual space of infinite dimensions it gets restricted by some boundaries that reduce their potential viability (see Figure 1.2). The constraints are such if their violation makes the final product not suitable to fulfil the initial needs.

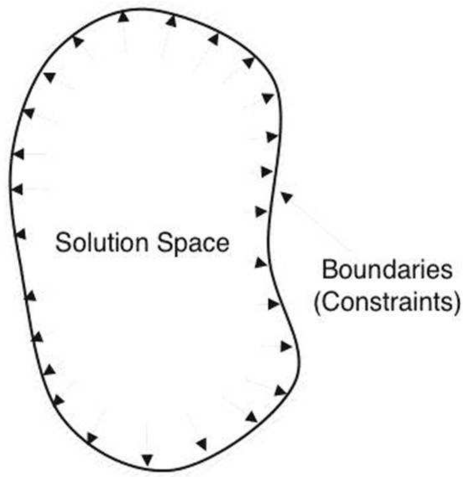


Figure 1.2 Schematic of a solution space bounded by constraints. Task of need analysis is to maximize the size of the solution space (Kroll, 2001)

The list of these constraints must obviously include all the elements that really act as such in order to better focus attention solely on practical solutions; attention must then be paid to the artificial constraints that might reduce and limit the space of action for no reason.

The constraints (that can be regulations, specific requests of the client, design requirements for previous similar situations, experience, knowledge) can be classified into explicit or implicit. The former are the easiest to identify because they are contained into the initial design task or directly derived from it, they must be approved by the client in case of their change. The implicit ones are more complex because they are not known a priori before the beginning of the design process and may be the result of initial calculations, analysis of the project site or simply as a product of the needs analysis.

An intuitive way to find out what the constraints are is to classify them according to a list of five broad categories that allow the designer to have clear the boundary situation avoiding accidentally omitting some elements and to carry on the understanding of project tasks. These five categories, as it is also possible to see in Figure 1.3, are: performance, value, size, safety and special (Kroll, 2001).

1. Performance
2. Value
3. Size
4. Safety
5. Special

Figure 1.3 The five general categories of constraints (Kroll, 2001)

Once clearly defined the size of the space of potential solutions, the degree of freedom granted by the boundary conditions and all design requirements which have arisen from the need analysis (which must be chosen with care to avoid an inappropriate choice to go affecting the proposed alternatives), it is necessary to identify the key parameters that become the *sine qua non* criterion to say whether a solution can take the next step or not. Like all other steps of conceptual design also the definition of key parameters is extremely important and needs attention: for example, it is absolutely useless act schematic or repetitive because each project has its own peculiarities and characteristics and there are no ready-made solutions and it is extremely dangerous the endeavor to adapt solutions already used in similar cases because this attitude can lead to problems of considerable magnitude below the design process.

In order to point out as much precisely as possible all the key parameters it could be useful to bear clear in mind which are the requirements and which the objectives the project is going to face. The parameter could be a factor, an issue, an information or a concept but it has not to be a dimension or a physical property in order to avoid misunderstandings that can compromise the proper conduct of the operation. A set of well chosen parameters needs knowledge, experience, mental flexibility and also good qualities of inventiveness (Goral, 2007).

The conceptual design process concludes with a final conceptual solution which will be detailed later in the preliminary design phase.

It is worth underlining how the various project phases of a structure are not among them watertight and how it is always possible to "go back" to change and better define the design requirements but the goal of conceptual design is to define precisely at the best all basic information to be able to avoid problems, misunderstandings or conflicts of roles during later stages.

An effective scheme (Figure 1.4) was developed by Engström and Lierud (2006) based on the theories of Knoll to exemplify the process of choosing a suitable alternative during conceptual design phase and it seems appropriate to bring it:

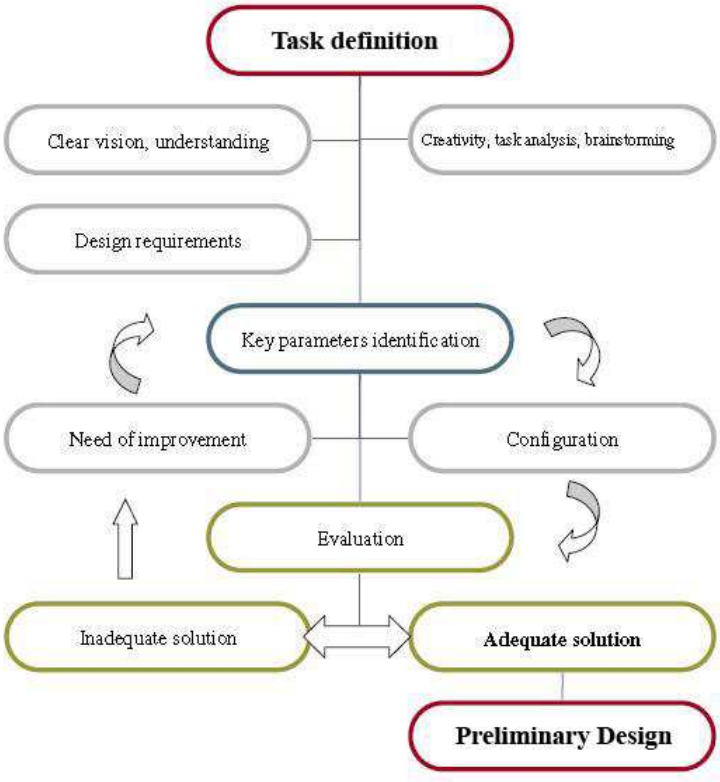


Figure 1.4 Five-step methodology to choose the best alternative (Goral, 2007)

Until now there has been talking about conceptual design in terms of generic structures but it can be useful to propose a reading that gives the general lines in the case of designing a footbridge.

Being facilities for pedestrian traffic only this type of bridges have more freedom than the road and railway bridges: their path is not simply to connect point A with point B in the shortest possible time and with less waste of resources but rather to highlight and promote territorial development around them; they have then the opportunity to free themselves from the bond to be a simple straight line and can become free spatial forms that offer to those who pass through experiences that transcends the simple moving across an obstacle.

Their full usability and their human scale size obviously make them objects of direct experimentation by pedestrians who stop on it, look out and if possible sit on it: ultimately they live it.

The design of a footbridge becomes then more complex than that of other bridges due to this great variability of factors that go to determine what form and structure it should have.

In particular concerning the needs of the customers, who are the users, designers must take into account and pay attention to:

- control of vibrations during the service life of the structure in relation to the kind of users

- adequate sizing of the carriageway in relation to the expected traffic flows in order to allow easy passage even in case of overcrowding, not to mention that many times too narrow pathway produces a feeling of insecurity

- possibility of including benches or resting places where environmental conditions allow it and are deserving

- particular forms (for example spirals) or too small dimensions of the staircases can cause difficulties to elderly, disabled or people with physical impairments

- inclination of the ramps must, wherever possible, permit the transit of wheelchairs; if the boundary conditions do not permit it the placement of a lift must be taken in consideration

- access as easy and comfortable as possible at the entrances of the bridge to ensure in all conditions; especially in case of very narrow access areas viable alternatives should be considered including linking to adjacent buildings

- if the footbridge is placed in a densely populated environment and it passes through residential building areas it should be taken into account the fact that it is not pleasant to feel observed, for both residents and passersby, and adjusting accordingly plant, height or shape of the structure.

Also regarding the choice of materials and construction technology to build a footbridge, the variety is very wide; since the loads are usually modest and limited deformation takes place it is possible for the designer to experiment with a variety of different materials such as steel, concrete, timber, masonry, aluminum alloy and plastic composites even. This way you can effectively investigate new forms of expression and new structural solutions, being also easier to convince the client due to the usually small size of the structures. This freedom of choice of materials must still not obscure that the national or European regulations have to be fulfilled however, that the relationship with the urban context in which the bridge will be put has to be protected and that the materials chosen have to be suitable for the kind of users that will transit on it.

As already highlighted footbridges are structures that have the ability to become social condensers, symbols of a real intention to renew and in this way carriers of values and meanings. Understanding this valence is essential and therefore to obtain a final result that is fully satisfactory, and indeed goes beyond the expectations of the client (also referred to a community), it is desirable architects and engineers to work together in the implementation of the project to give both aesthetic and structural aspects. The design work should be carried on since the early steps so that the two contributions come together organically and harmoniously, avoiding harmful and unnecessary adjustments during the work. It often happens that only engineers design the bridge, implementing the structure with an eye only on cost and structural efficiency, and then the architect is left with the task of trying to give formal contents to a standard solution. There is also the case in which the project is initially developed by architects who, based solely on aesthetic considerations, develop solutions structurally unacceptable then engineers must try to fulfill the minimum requirements: in this way it often leads to a radical change of the form and an increase in costs, making it useless de facto the whole process of conceptual design (FIB, 2005).

2 Aspects of timber bridges

2.1 Advantages of timber bridges

Bridges have always played an extremely important role in the lives of human beings. Every bridge that is built brings human beings into contact with one another, and this – over and above its purely technical character – makes the bridge both a symbol and part of our culture. As structures, bridges are not only a part of human history, but also a measure of the planning expertise and technical skill we have attained (Wittfoht, 1984).

The age of wood spans human history and it was probably the first material used by humans to construct a bridge. The stone, iron, and bronze ages were only interims in human progress, but wood - a renewable resource - has always been at hand. One reason why wood has remained indispensable for constructing bridges for thousands of years is that it is natural: as a building material, wood is abundant, versatile, and easily obtainable. Without it, civilization as we know it would have been impossible.

The first bridges were made of simple, unhewn tree-trunks. To this days, the basic types of load-bearing structures – namely the beam and the arch – have not changed since human beings first attempted to make them. Until about the 1970's not great changes occurred in the use of the material despite a great number of different structural solutions: wood has never been changed in its nature but was adapted in the form that was necessary from time to time. It is only with the relatively recent developments of new wooden materials with large dimensions, a high load-bearing capacity and improved connections that wood has again attained novel and independent status in the engineering field of bridge construction. This has resulted in new footbridges and road bridges, characterized by original, high-quality design and impressive dimensions.

In most countries where timber bridges have been always used, an evolution, divided in three phases, can be observed: the situation in Finland may serve as an example. The first generation timber bridges owned by the Finnish Road Administration were built before the 1970's, made of logs and sawn timber, without any preservative treatment. In the middle of the 70's, the second generation, the glulam bridges took over. Now the third generation, in Finland represented by the wood-concrete composite bridge, is developing. Many of the first generation timber bridges were not properly designed and some were not even treated with chemical preservatives. In many other European countries, the second generation is almost completely lacking (NTC, 1997).

A period of more than half a century of bridge construction was dominated by concrete and steel: the situation with the discarding of older functionally obsolete or structurally deficient timber bridges, and replacing them with concrete or steel bridges, has lead to a decline of timber as bridge construction material.

Moreover, people often expect only half the service life for timber bridges compared to concrete and steel bridges, even though it is a well-known fact that concrete and steel bridges have often required substantial repairs after a short time of use.

There are many examples of very old and well-kept timber bridges: in Switzerland, for example, several covered timber bridges with superstructures that date back a couple of hundred years ago are still in use. In spite of the fact that nearly 50% of the timber bridges in the United States are assessed as structurally deficient, states and counties continue to build bridges out of wood as it has numerous characteristics that makes it a desirable material for transportation structures (NTC, 1997).

Until recent years, the worldwide fall out of favor of timber was characteristic: it was the result of the fact that older timber bridge designs do not fulfill the requirements for modern roadways. Poor detailing on many of the structures also combined with low durability timber, resulted in the perception of timber bridges being short term solutions to bridging problems. This perception is still held by many engineers practicing today.
Concrete in particular was perceived as providing long term maintenance-free structures and remarkable advances in steel technology provided opportunities for longer span structures with increased load capacities.
The limited knowledge and experience of timber of most design engineers has lead to a generally negative attitude towards the use of timber for bridges. The result was stagnation in the advancement of technologies which would enhance the use of wood in transport infrastructure projects. Timber bridges were delegated to a solution for small pedestrian bridges and low load capacity vehicle structures on the likes of golf courses and private rural farm properties.

A gradual revival of timber bridge building in Europe started during the 1980's. A circumstance leading to the current increase in the use of wood as the construction material for bridges is some loss of image of the materials competing with wood, particularly the increasing awareness that concrete is by far not the everlasting material it was expected to be. New timber bridge designs are also more competitive than the old ones.
The third generation timber bridges, represented, for example, by stress-laminated bridges, are providing better load distribution and a solid base for the moisture barrier and the wearing surface.
The competitiveness of timber bridges depends on costs for construction and maintenance as well as expected service life, in comparison with other materials. In many cases, aesthetical factors are also important for the choice of construction material. Ultimately, the timber strength is based on the balance between function, cost-effectiveness, technology and aesthetic considerations.
In Finland, a study of construction costs of timber bridges compared to concrete bridges, pre-stressed, precast concrete bridges and steel bridges has been carried out. This study shows that the cheapest alternative for the shortest bridges is concrete bridges cast in situ, but timber bridges provide the cheapest alternative for bridges with a span of about 14 meters up to the maximum span for timber bridges (NTC, 1997).
Two Swedish studies which have been also carried out report on attitudes and reasons for decisions taken when building bridges: these studies show that the cost often is a decisive factor. The timber bridge is preferred because it is the cheaper alternative compared to concrete or steel bridges. In addition, timber bridges usually offer aesthetical advantages along with short construction time and a simple construction process (NTC, 1997).
Depending on its special qualities, timber is totally different from other construction materials and it offers a potentially low-cost alternative to materials such as steel and concrete.
The mechanical performance of timber is closely related to the natural origin of the material and to the functions that this material has in nature: it in fact has the duty to support the foliage, acting as a cantilevering structure. The morphology of the cells of wood and their conformation ensure high resistance values with low dead loads.
The cellular organization of wood is the foundation of a strong anisotropy of mechanical properties and this leads to a marked difference in values of strength and

stiffness depending on the direction of the grain: timber is in fact more stiff for stresses oriented along the direction of the fibers and, conversely, is much less efficient for ones orthogonal to the direction of the fibers (especially in tension).
In the case of stresses parallel to the grain, timber has a high structural efficiency compared to other construction materials. One possible criterion for defining this efficiency is the ratio between a resistance parameter f of the material (for example to compression) and its density ρ: the resulting value is very similar to that of steel and it is about five times higher than of reinforced concrete. These values show that it is possible therefore, with wooden elements, considerably lighten the structure, with many advantages, not least in the seismic design (Piazza, 2007).
Equally important as structural parameter is the ratio between the modulus of elasticity E and the parameter f, which takes values equal to one third of those of reinforced concrete and equal to those of steel.
Even if timber life is not particularly long, it may be improved if one takes appropriate measures: in fact building with timber always requires comparing tight with the issue of a constant exposure to weather conditions to which the structure is subjected, due to the natural tendency of material to biological degradation. This trend must be prevented or at least delayed as long as long is the lifespan required for the considered element and it must be considered that bridge protection is also crucial to maintenance costs: in fact bridges designed with exposed supporting structures, e.g. trough bridges, show more damage and higher costs than ones designed with good wood protection by design.
Although timber, among construction materials, is the most sensitive to degradation, it is at the same time the one which can provide a significant durability to the structure, as long as it is properly protected: a recent research indicates that timber bridges may be more durable than those constructed from other materials, particularly in cold climates where salts and other deicing agents are frequently used (NTC, 1997).
It is therefore clear that it needs a proper planning and execution of the work so that it can fulfill not only the aesthetic, architectural, structural, economic and functional requirements, but also those related to durability and to the eventual efficient and effective maintenance of the whole structure.
In conclusion, wood protection is crucial to the function and life of a timber bridge, and must be given the same attention as the strength. Accordingly, protecting the wood against deterioration in the first place relies on limiting the moisture content through the type of construction: the timber components that are exposed to the weather must be designed so that the water drains away as quickly as possible and just as important as a fast water run-off is the rapid drying out of the water absorbed by the wood through effective ventilation of the respective component.
Where it is not possible to limit the moisture content or to avoid a quick drying out of water, resistant species of wood can ensure a durable construction: it is important to choose a species of wood that is resistant to the particular action and a wood-based product class to suit the respective application.
In terms of detailing, protection for the wood begins with cutting relieving grooves in logs or squared logs to avoid uncontrolled splitting; and fissures are always an entry point for insects or water. Glued laminated timber and wood-based products are less at risk because they have a lower tendency to split. To ensure that the effects of shrinkage and swelling do not cause any damage, small material cross-sections and small surfaces are preferred for components exposed to the weather in particular.
Only in places where neither the detailing nor a resistant species of wood can be used to provide protection is it necessary to use chemicals.

In conclusion, the advantages in using timber as a bridge construction material can be summarized as follows:

- it is a renewable resource and it suits the cycle in nature and should, in a good way, manage a life cycle analysis and environment declaration compared to other bridge construction materials

- due to timber low dead weight, timber bridges are very light and could be almost totally be erected in the factory and lifted in place with minimal workforce, so that the time to completely erect the whole structure is much shorter

- a timber bridge that has been built and maintained in the proper right way, should not have a substantially shorter service life or higher maintenance costs than other bridge alternatives.

- it is naturally resistant to the effects of deicing agents (so timber bridges can be particularly suitable for cold climates)

- often timber is the cheapest alternative all in all on a first-cost analysis and shows great advantages when life cycle costs are compared

- timber bridges allow easy assistance for maintenance and repair problems

- timber bridges present a natural and aesthetically pleasing appearance and due to their non-traditional applications lead to a wide range of construction practices and design concepts unique to timber alone.

2.2 General problems connected to timber bridges

2.2.1 Manufacturing issues

Up until the recent past, the chances of building wooden structures were limited by the fact that it was necessary to adapt to the possibilities offered by the size of the elements found in nature in the form of logs and sawn timber: ultimately the maximum lengths reachable were about 20 m and cross-sections seldom exceeded 150x450mm.
Nowadays, thanks to technological innovations introduced in the field of timber constructions, if one needs large items it is possible to create composite elements consisting of multiple parts joined together by adhesive (just think about glulam timber). In this way two advantages will then be obtained: firstly, structural elements are produced with sizes and shapes that would be difficult to find in nature, and secondly building materials with easily verifiable physical properties are available. Timber in fact, being a natural material, has a number of defects, which affect its reliability, but splitting it into smaller pieces and rearranging them, one can minimize or distribute these "non-compliances" in the new material.
The so-called wood-based products (glulam timber, plywood, OSB, LVL and others) are made of veneers, flakes, chips and fibers (Polastri, 2006).

During the production of these materials great attention should be paid to all the different stages of the process, from the choice and quality of raw materials, through continuous monitoring of subsequent phases to finally arrive at the finished product.
In the factory it is also necessary to carefully control internal temperature and humidity since timber tends naturally to reach balance with hygrothermal conditions of the environment in which it is located. To prevent such phenomena it is advisable to thoroughly dry the wood before processing.
A recurring problem in the field of timber constructions is connecting together different components to obtain complex structures: hence it becomes crucial in properly designing the connections so you can get both a structural solidarity between the various elements and an appropriate management of the dimensional changes that timber undergoes as a result of climate variations. Connections must then:

- Make the joints between elements as easy as possible
- Create linear paths for the involved forces in order to prevent building of unexpected stresses
- Ensure that forces are always possibly directed in a direction parallel to the grain
- Respect minimum spacing required by law

The design of the supports needs much attention because they have the task of transferring the forces to the bearing structures or foundations. They, theoretically speaking, should allow translations and movements of timber elements due to the variation of moisture content, in reality, they must remain as close as possible to the static model developed in the design stage.
In case of steel-to-timber connections (the most frequent ones) it is necessary to check the strength of both metal fasteners and timber elements, in particular with regard to normal and shear stresses in the most unfavorable combination (Polastri, 2006). Not to forget the checks on stresses perpendicular to the grain.
Very important are finally the checks on glued connections that are increasingly being used in the field of timber constructions: this kind of adhesives tends to create a union as if the structural material were still intact.

2.2.2 Durability issues

A very important aspect of the design of timber structures concerns the durability of the timber elements, which naturally tend to deteriorate, particularly if exposed to direct contact with weather agents. The variation of moisture content within the timber causes dimensional variations and in the long run leads to a loss of mechanical strength.
Timber is also subject to attacks by microorganisms such as fungi and molds that threaten its integrity.
For these reasons, it is necessary to plan, beside the use of protective coatings and elements of "active protection", a scheduled maintenance in order to achieve a service life as long as possible.

If timber is in direct contact with water (in liquid form or in the air) it swells: the important thing is to allow this increase in volume with appropriate planning arrangements (otherwise you get the risk of developing dangerous internal actions) and above all to enable water to evaporate quickly.

The critical areas which need to be focused on are the connections in which the structural discontinuity and the different behaviour of the materials can produce cracks, which gradually get deeper due to further water introduction and to the potential growth of microorganisms harmful for timber.

Also the sun radiations have negative effects on timber structures because electromagnetic waves building them affect lignine, an organic binder, which being soluble, is then washed away by water in its various forms.

Particularly dangerous for glulam timber are finally the frequent moisture variations for the so-called risk of “glue line delamination”: delamination takes place between the lamellas and so the strength of the material decreases esponentially, especially concerning the shear resistance (Polastri, 2006).

3 Historical evolution of timber bridges: structure, aesthetics, construction

This Chapter will analyze the most important wooden bridges built during various historical periods in order to be able to understand the changes that have occurred and the constants that these structures have maintained over time. The works that will be described have structural features and qualities that have permitted them to emerge and to become models and examples to follow in the field of timber constructions.
It is very important to understand the development of technologies and of structural solutions that have occurred over the centuries to get a better comprehension the of background for this type of wooden structures and in order to succeed, in the design stage, in managing the creative process, being aware of what has been done before us by others.
The bridges analyzed cover a span of more than 2000 years and include the Caesar's and Trajan's bridge, the bridges of Palladio, Hans Ulrich Grubenmann's masterpiece, North American timber bridges, the bridge over the Canal Grande in Venice by Miozzi (one of first glulam timber bridges), and two contemporary works extremely innovative due to technologies and static systems adopted: the Essing bridge by Dietrich and the Traversina footbridge by Conzett

3.1 Caesar's Bridge over Rhine

In ancient Rome the bridge had always had a particular importance since it had meanings that were at the same time both symbolic and functional.
Suffice it to think that bridge Sublicio, the first bridge in Rome, was made entirely of wood and its connections were completely devoid of metal fasteners, and for this was rebuilt and restored several times, always using the same technique and numerous public ceremonies took place on it (Maggi, 2002).
Culture of ancient Romans placed man and his skills at the center of the universe and therefore in this sense the tremendous efforts made by Roman engineers in creating bold civil engineering works including roads, bridges and aqueducts can be fully understood. Managing to achieve these constructions allowed to assert the superiority of human intellect against the power and the limitations imposed by the forces of nature.
The bridge in particular was considered the most important of the infrastructures as it could overcome natural obstacles such as rivers, which with their length defined and imposed borders often impassable.
Much more than the stone ones, perceived as definitive and stable, timber bridges (especially those built during military operations) have left deep marks in the collective memory and have been subjected to detailed descriptions on their structure. They were indeed held in high esteem due to their complex construction because, besides having to be able to combat and subdue the natural elements, they also should be constructed rapidly and quickly destroyed or rendered unusable in case of need (Maggi, 2002).
Particular importance in history has played the Caesar's bridge over the Rhine built in 55 BC during the war against the Germans: his detailed description, made by Caesar himself, in fact constitutes the first evidence of a construction manual applied to wooden bridges and formed from the Renaissance onwards the starting point for the

study and design of any wooden bridge, not to mention also that the difficulties related to the width and depth of the Rhine had urged the adoption of unconventional technical solutions (Maggi, 2002). The bridge solution was also, in Caesar's opinion, certainly more worthy than a bridge on boats.

Description given by Caesar in his *De Bello Gallico* (Book IV, XVII-XVIII), necessarily brief, reveals a project in which the static and functional roles of the different elements of the construction are clear: the integrated ensemble pier-foundation, the additional stiffeners, primary and secondary structures of the deck, not to forget the works of passive defense.

Caesar, indicating that it was completed in ten days, accurately describes the work of engineering but in the text there are no few ambiguities and uncertainties, especially related to the meaning of some technical terms.

Just these black spots have generated great interest and produced a series of constructive assumptions by many scholars and architects on the possible shape of the bridge.

Based on the original text we can deduce the general structure of the work: piers were made by wooden stakes driven into the riverbed through mechanical devices specifically designed or simply manually operated by workers on board vessels. Stakes were set not vertically but with a certain inclination: in the direction of the current upstream and in the opposite direction downstream. Each pier consisted of two piles and a transversal beam, that engaged the pier located on the opposite side, was inserted in the room between them. The transverse structure that was thus created resembled so much like the cover of a roof and worked in traction rather than in compression (Maggi, 2002).

To complete the structure additional piles at an angle were placed downstream close to the piers and groups of piles were placed upstream in order to protect the piers against the impact of material transported by the stream. The deck consisted of planks nailed on top of longitudinal beams that had a span equal to the distance between two non-consecutive piers.

The greater uncertainty in the Latin text turns around the term "fibulae" used to indicate the means of union between the piles making up the piers and the transversal beams and generally called the connection to the top of two pieces of wood and many interpretations of this term were different.

The first representation ever of Caesar's bridge over the Rhine, and for this the most valuable, has been the effort to create a graphic model, as shown in figure 3.1, by an anonymous designer based on the translation from Latin into ancient Italian by Pier Candido Decembrio but it is so rough that does not allow a real evaluation of the chosen solution (Maggi, 2002).

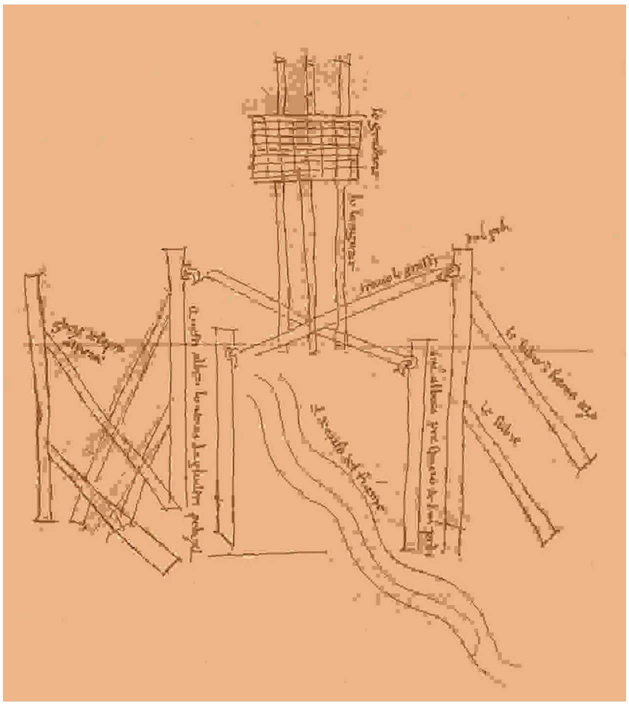


Figure 3.1 The first known representation of Caesar’s bridge by an anonymous drawer (Maggi, 2002).

Fra Giocondo in the sixteenth century suggested two oblique elements on each side between the transversal beam and the pier (Figure 3.2) that it has not a clearly understandable utility considering that in Caesar’s text there is no mention of such a solution. Moreover two diagonals partially immersed in water are then added, which is quite doubtful effectiveness.

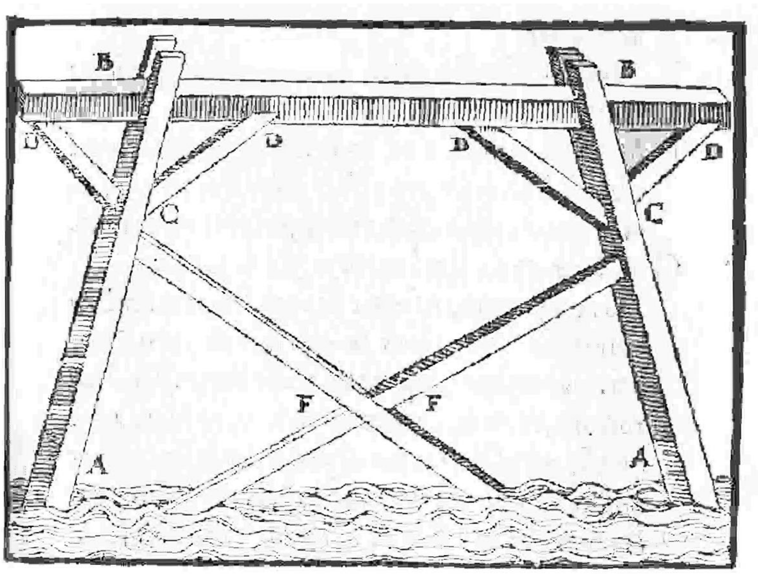


Figure 3.2 The detail of a pier of Caesar’s bridge according to Fra Giocondo (Maggi, 2002).

Leon Battista Alberti in his version of Caesar's bridge (Figure 3.3) represented probably hemp ropes which tightly tied the beam with the pier: this solution seems unlikely, given the low efficiency of ropes in the presence of water.

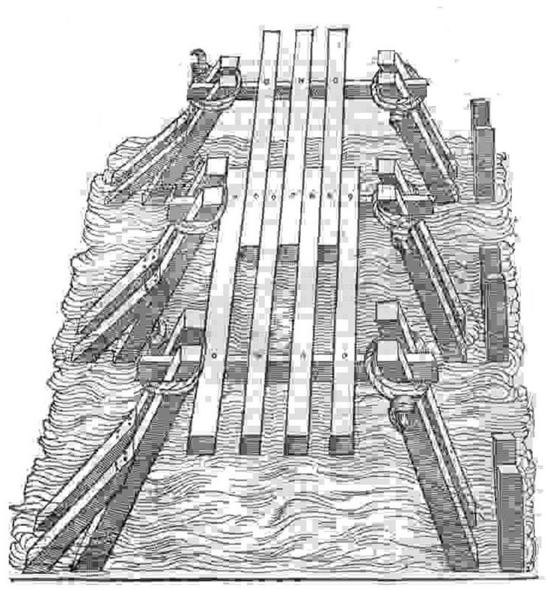


Figure 3.3 The solution proposed by Leon Battista Alberti in the XVI century (Maggi, 2002).

The most reliable proposal (Figure 3.4) from the structural point of view is undoubtedly that of Palladio which draws a wood bracket with notches: doing this way it was possible to eliminate holes and ropes that are not suitable to a structure placed in water, also considering the significant forces, but mainly it fastens the two piles together and keeps a constant distance at the same time; it also allowed the structure to be stabilized with increasing vertical loads.

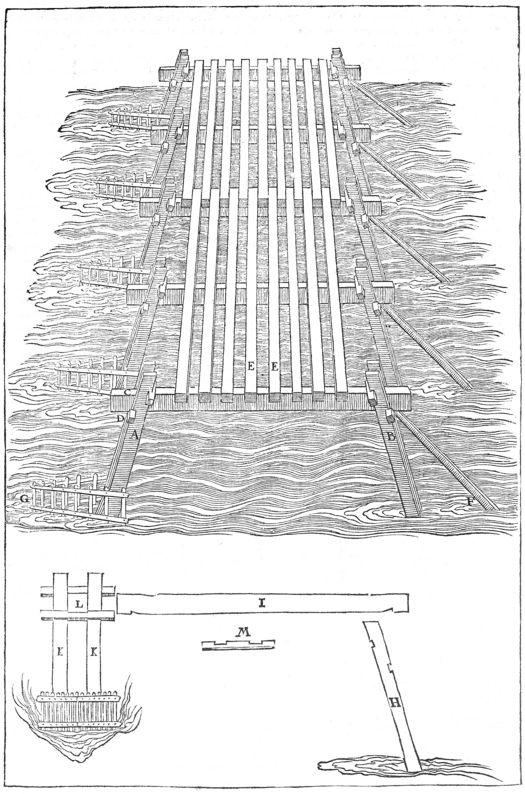


Figure 3.4 The Palladio's hypothesis of Caesar's bridge (Puppi, 1999).

The reconstruction of Sir John Soane, the famous English architect, is also noteworthy and it can be seen in Figure 3.5, who in addition to place a parapet that it is not mentioned in the Latin text, performed the connection between the transversal beams and the piers through metallic elements and set elements between the two piles forming a pier in order to space them. His solution provided also internal diagonals placed next to the piers, very similar to what has been done by Fra Giocondo (Maggi, 2002).

Figure 3.5 The impressive reconstruction of the Caesar's bridge prepared by Soane for his Lecture Diagrams (Maggi, 2002).

3.2 Trajan’s bridge over Danubius

The Trajan’s bridge over the Danube at Drobeta in *Dacia Inferior* still remains one of the biggest and most daring feats of engineering ever built by the Romans.
After a prolonged and violent military campaign in 101 AD Emperor Trajan succeeded in defeating and subduing the Dacian tribes commanded by General Decebal. The region of Dacia, geographical classification which covered a wide territory of central Europe corresponding to the current areas of central Romania and Moldova, was thus transformed into a Roman province and many settlers from all over the empire moved there in order to pacify it permanently, introducing the Roman’s language, customs and traditions.
In the peaceful period immediately following these events the bridge at Drobeta was built (103-105 AD), strongly backed by Trajan in order to be able to move quickly and safely troops across the Danube and create a reliable supply route, without resorting to risky temporary bridges of boats and above whatever meteorological condition (crossing the river was extremely dangerous with the winter frosts).
The responsible figure for achieving this noticeable building was Apollodorus of Damascus, the official architect of the Emperor, author of, among other things, the Trajan’s Column, who designed an extremely complex structure in the incredible reduced period of one year, choosing the location of the bridge just near Drobeta, given the presence of a natural ford which gave a narrowing of the riverbed (800 meters) and at the same time an outcropping of land: the bridge reached the considerable length of 1135 meters and for more a thousand years was the longest bridge ever built. Its most innovative feature was specifically that the arches were not made of single wood trunks, but they were polygonal that is formed by a series of pieces with relatively small dimensions: in this way the entire process of construction became easier and faster.

The quality and accuracy of the extant sources is quite low and rough concerning the description of the bridge, but fortunately the sculptures from the Column of Trajan (Figure 3.6), in the scene where the victory of Trajan over the Dacians is celebrated, show that it was made up of twenty timber arches resting on twenty masonry piers. At each edge were two fortified towers that controlled the passage.

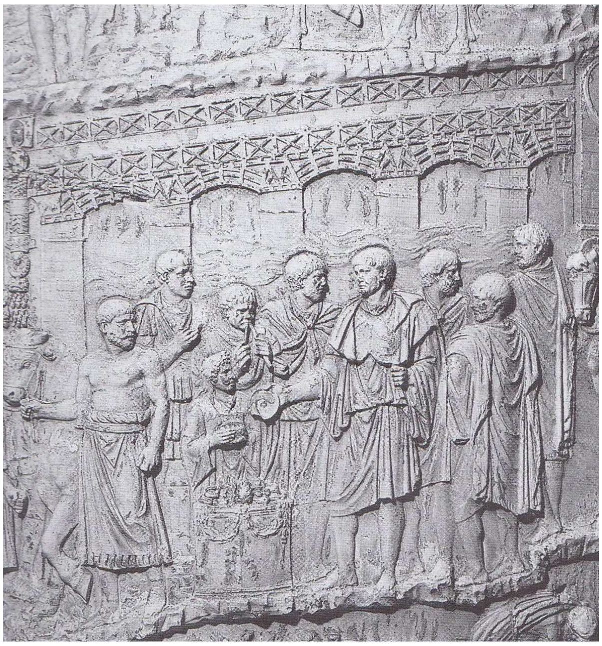


Figure 3.6 The scene from the Trajan's Column: on the background one can see the wooden bridge built in Dacia (Maggi, 2002).

Information that can be derived about the kinds of carpentry joints is very limited but from the sculptures may be understood as the arches had an average span of about 30 meters each and they were made of six curved timber beams parallel one to each other, which in turn consisted of three arches concentric mutually interconnected and simultaneously joined with the upper deck. The connection between the arches and the horizontal part was performed by some inclined element positioned in the radial direction and whose length variable depending on their location. On the piers, in the meeting point between two adjacent arches, there were triangular structures to balance the horizontal pushes provided by the arches (Paolillo, 1999).

There is no certainty either on the types of connections used to build timber structures but it seems likely that on the top of the studs, where they met the longitudinal beams, there were nailed joints; nodes where the arrows were going to engage on the arches were rather more complex, as in the same place there was the presence of transversal beams that served to stiffen the six arches and most likely the connection was performed through the combined use of nails and metal strips.

Each arch was also segmented into different straight bars and the connection between segments occurred at the studs: the structural continuity was ensured through the use of structural carpentry joints (tenon and mortise) and with the addition of nails passing through both the stud and the two overlapping bars (Paolillo, 1999).

The deck was 12 meters wide and consisted of the main longitudinal beams on the top of those the secondary warping of beams was placed, positioned in the transversal direction with a spacing of about 80 cm. A simple nailed plank rested on it.

Longitudinal beams behaved as simple supported beams since they were placed in the room between one stud and the other and connections were performed similarly to what has been achieved between the different segments of an arch and the corresponding studs: also the edges of the beams presented notches in order to perform a tenon-mortise joint and then there were nails passing through to integrate the two parts with the corresponding arrow. The parapet was modular, stiffened by

diagonals, connected by ropes and nails to the transversal beams that supported the deck.
Concerning the piers, they were clearly oversized in relation to the relatively low weight of the timber structures above them: this is because Trajan probably had wanted the arches made of timber in order to use the bridge as quickly as possible due to the ongoing military conflict, referring perhaps later the final realization of the stone arches, but never realized. They were realized using the usual Roman technology: the core was composed of stone fragments and rounded river stones held together by mortar (a typical *opus caementicium*) and the outer sides were covered with bricks (*opus latericium*) at the bottom and with stone plates on the higher level.
A possible reconstruction of the bridge was proposed by Rondelet in his Treatise (edited in 1832) in the section on wooden bridges, as shown in Figure 3.7.

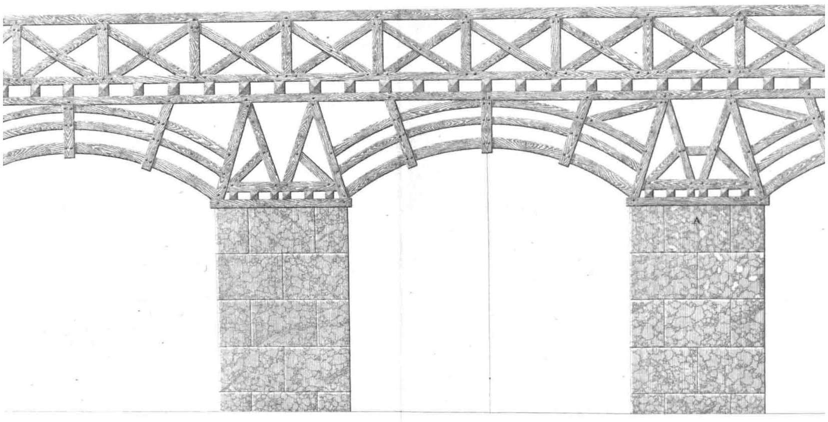


Figure 3.7 The Rondelet's solution concerning the Trajan's bridge (Maggi, 2002).

Regarding the building aspects of the bridge and the solutions adopted in the construction site, after underwater inspections it has found out that formworks for piers were made up of oak trunks and they formed a double layer sealed by the same method used for boats, that is with pitch. The room between the two formworks was drained from the water using buckets and filled with pressed clay and other materials. Each pier was based on a palisade of oak posts whose ends were covered with mortar and formed the base for piers themselves. (Paolillo, 1999).
The issue regarding the installation of the timber arches was rather different, which was implemented for a real process of prefabrication: the wooden arches already formed and grafted with the studs, after being assembled on the ground, were put on boats and brought near the piers, on the top of which were installed cranes that provided to hoist structural components and place them in the correct position.

3.3 Palladio's bridges (bridge over Cismon river and first invention)

Although during the Renaissance timber bridges were considered to be less "noble" than the stone ones, nevertheless Palladio was a smart designer as he thought they had the advantage of being nice and cheap. Nice since timber texture was elegant, and cheap because timber had a lower price, both for the material itself and for the

setting up. Moreover this kind of works were lighter and more flexible than the stone ones.
Palladio's timber bridges have a great importance due to the fact that they give us the idea both of his static perceptions and the terrific success they have had, not only at the time of their setting up , but also and above all in the iconography and in the later literature, thus becoming a model in the 18th century England.
The architect in the third of his *Four Books of Architecture*, not only analyses and hypothesizes the aspect of the Caesar's Bridge over the Rhine, but also proposes the projects for five timber bridges. Apart from the Bassano Bridge, there are four others, only one of which has been actually erected (the one over the Cismon), while the remaining three ones are just "inventions" in order to be able to build bridges "without posts in the river". According to these solutions every element is strictly essential and functional, and the structure is simple and minimal, in a way that even the smallest piece can't be taken away without making the whole bridge to collapse. Unlike the masonry bridges, there are no decorations and the bearing system coincides precisely with the visible architecture.
The models carried out by the Author are really similar to the modern truss beams, with a much more lower height of the beam: these bridges behave like a typical truss work because they are made up of relatively small size webs, simply tensed or compressed, of hinges and with point loads on the nodes. Moreover Palladio drew very clear structures showing the kind of timber joints to be performed and he not only indicated the whole span of the bridge , but also the ratio between the height and the width of the beams, i.e. 4:3 (Funis, 2000).
One of Palladio's most important achievements, besides the fundamental one concerning the extraordinary distance between the banks, has been the constructive simplification of the bridge structure and the innovations related to the building site organization. These improvements have been obtained making it easier to assembly the webs thanks to the offsite timber being cut following a list of pieces, the lifting in order (absolutely rational) in the right altitude, the easy positioning on the beams (which working also as spacers, determined a constant spacing between the elements), and speed and precision in performing the connections through the "*arpesi*", rejecting the traditional complex carpentry joints in use at that time. The transversals had predrilled holes in the edges in order to make the *arpesi* come through and the first of them, measured and drilled with precision, acted as a ruler for the next ones, so ensuring the whole structure a high quality assembly and a geometrical regularity and thus avoiding any instability risks (Tampone, 2000).
Therefore the *arpesi* and the particular building site organization have made it possible to achieve an improvement in the structure prefabrication, already inherent in timber construction. At the same time the process contrived and the devices in use to carry it out, did significantly cut both building time and costs .
It may seem hard to carry on an aesthetic analysis of the proposed structures , due to their extreme essentiality, but it is indeed their main feature which allows a better understanding of the modernity and actuality of their designer, who presents his bridges not as arid technical handworks but as architecture pieces with their own dignity and with aesthetic, functional and structural features.
Palladian bridges are pure structure, immediately intelligible. Their beauty derives from the expressiveness of the shape compared to the assigned function, the slenderness of the elements and the proportions between the parts and the transparency of the warping.

3.3.1 Bridge over the Cismon river

Palladio, quite similar to those roof structures he designed for many of his villas (Villa Emo above all), proposed in the bridge over the Cismon a genuine timber truss working as it is possible to see in Figure 3.8.

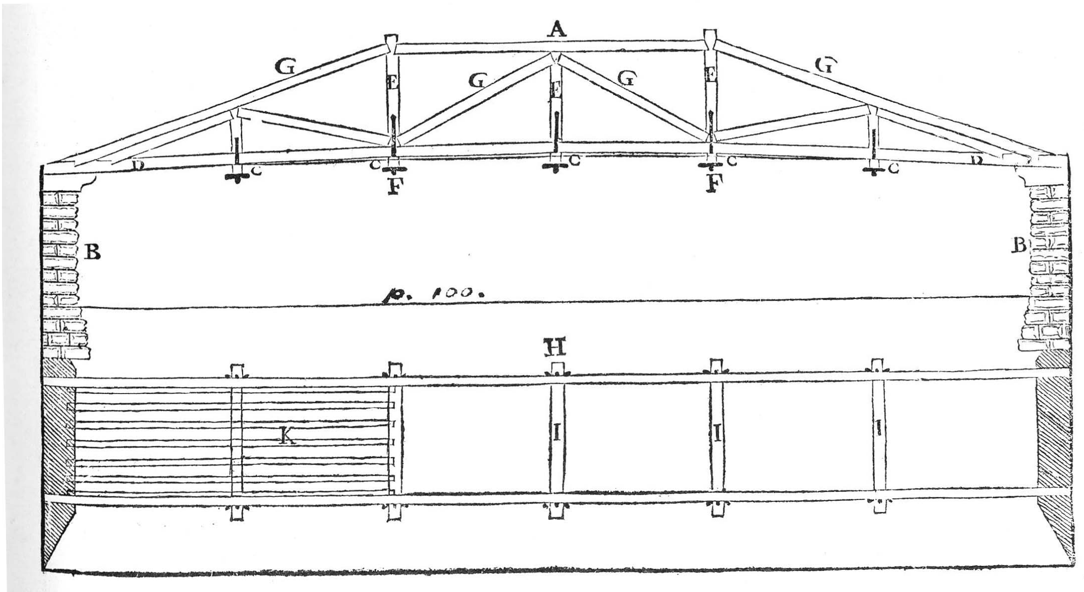


Figure 3.8 The Bridge over the Cismon river (Puppi, 1999).

In this bridge the carriageway on the lower part and the variable loads are applied, due to construction, on the lower nodes and the global behaviour presents a great static and functional clearness and furthermore it allows an excellent load distribution.
The architect, without intermediate supports in the riverbed, designed a very daring bridge of 36 meters span, size really hard to be spanned by any other kind of timber structure. Which is why the issue arose of devising a system membering, in which the elements, despite being much more shorter than the distance to span, were assembled in order to form a single beam (Tampone, 2000).
All the webs work exclusively in tension or compression, while only the longitudinal beams and the rafters, supporting the deck, work in bending, transferring the loads to the bearing structure through the junctions. The bridge ultimately works as a simple beam, whose elements are simply tensed or compressed, depending on their position: the upper and the inclined ones work in compression, whereas the lower and the vertical ones in tension.
The bridge was composed of two big truss works, with a height varying from 1/7 to 1/12 of their length , placed at a certain distance between them. The pins were joined on the bottom, in correspondence with the vertical studs, to tough transversals. Then rafters were put on and finally the carriageway was placed upon these elements so the two truss beams acted also as parapet. From the plan inside the *Four Books* it can be seen that the deck is 4,37 m wide.
Each truss beam was divided into six equal bays by vertical studs, the *colonnelli*, resting on the bottom chord and anchored to it through metal fasteners, the *arpesi.* In order to close the triangular section there were the upper chord and the inclined pins. Inclined pins, the longer ones, disposition is very important in a composite structure since in the event they join the *colonnello* in the lower part they work in tension, while if they join in the upper part, as in the Palladian bridges, they work in

compression. The drawback in this case is that the compressed pins are the longest, that is the inclined ones, and it is thus necessary to oversize them in order to prevent buckling (Funis, 2000).
Single elements were connected between them, as already stated, with metal fasteners: the *arpesi* (see Figure 3.9). Palladio invented these fasteners as special devices strictly functional in his bridge structure: they were expressly created to the composability of the bridge and they had the advantage of being able to be manufactured elsewhere and then assembled directly on site during construction.

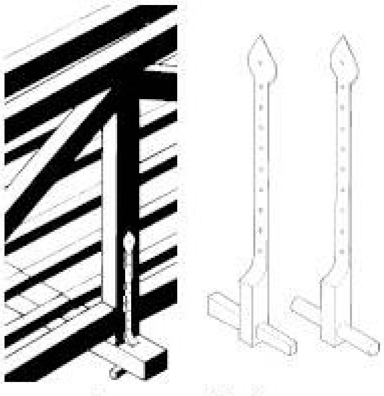


Figure 3.9   Detail of the connection between a colonnello, the longitudinal and transversal beam by an arpese (Funis, 2000).

The use of *arpesi* is particularly important because it reveals which were, according to Palladio, the limits of wood: in joints subjected to tension he put auxiliary metal implants, which were used to fill defects or limitations of timber. Even if timber resists well to tension, the difficulty was to create a T-shaped link, which resisted well to tension: this is the joint between the *colonnelli* and the transversals, which simultaneously serves as a graft also between the longitudinal beams and the transversals.
The difficulty of achieving the joints between the different elements and their limited resistance are therefore, according to Palladio, the limits of timber and the reasons why, in these areas of 'weakness', he considered appropriate for metallic accessories that they had good resistance to tension.
The *arpesi* were so artfully placed in the connection among the *colonnelli*, the longitudinal beams and the transversals, in order to hold together both *colonnelli* with longitudinal beams, and to suspend the carriageway to the bearing structure.
Analyzing the original print a further construction detail will be also noticed : a line under the longitudinal beams that is nothing but the prospect of the rafters that support the floor system. Indeed, they do not follow the inclination of the bridge, but they are flat. The lower part of the truss work does not coincide with the carriageway. The truss structure and the deck are kept in this distanced position by the inclusion of some wedges that, placed between the longitudinal beams and the transversals, are held together to the whole structure by the *arpesi*. They are of increasing thickness from the banks towards the center, in order to always fill the gap between the arch and the rope (Tampone, 1999).
Palladio thus distinguished the bearing structure, i.e. the two trusses, from the borne one, the carriageway, which was suspended from the bearing structure. Through this device the deck, for the convenience of passers-by, was plan, without thereby

compromising the efficiency of the whole structure, which, for structural reasons, could maintain the arched bottom.

About the organization of the construction site of the bridge over the Cismone, following the description (but not complete in this regard) provided by Palladio (who, after having divided the width of the river in six equal parts, laid the tranversals) it may be assumed that the structure was assembled on a rib or through the creation of five temporary vertical supports in the riverbed; The other hypothesis, less plausible however, is that the entire structure was built on the ground and then placed at a height using ropes and trestles.
The structure of the bridge over the Cismone, as shown on the Four Books, has anyway a weakness, that is, being deprived of a bracing. This needed to make the structure behaving as a box and also resistant to the force of the wind, could be added later, perhaps even as a coverage, both for the convenience of pedestrians, and to provide protection to the deck and structural elements.

3.3.2 The first invention

Unlike the Cismon Bridge, this project has never been carried out; however this invention is important for the role it has played, along with the other two, in the history of trusses and due to the originality of constructive solutions. As to the bridge over Cismon, the truss beams have a similar structural organization and they are quite high in order to fill the gap without big inflections, and burnishings are defined by the same nomenclature (see Figure 3.10).

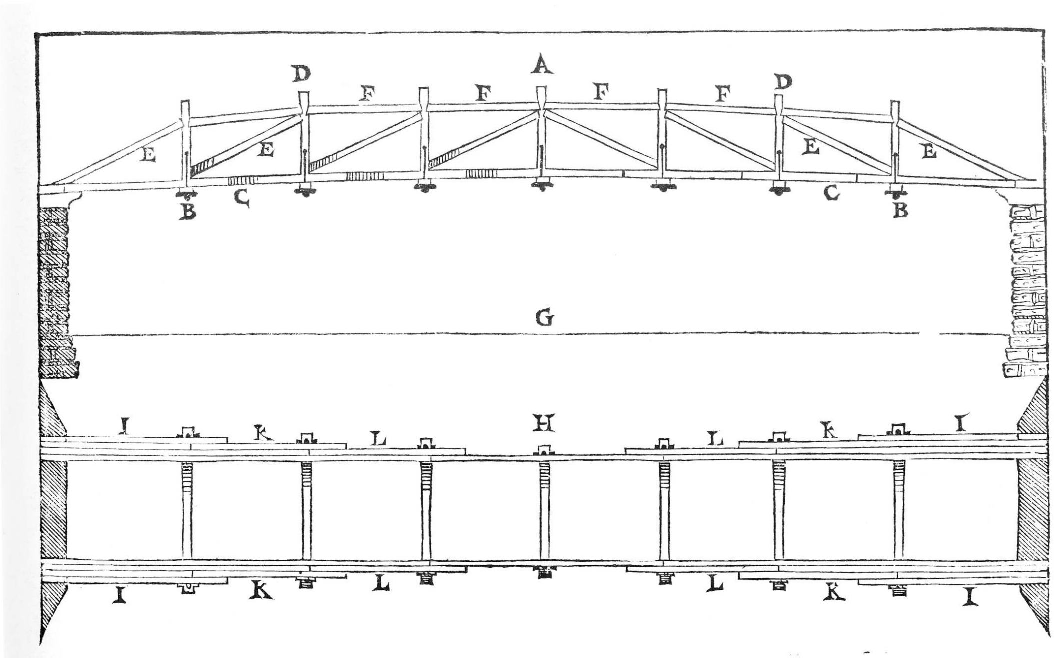


Figure 3.10 The first invention, or the bridge with a variable section (Puppi, 1999).

The structure is composed of eight bays, the bottom of the bridge is formed by juxtaposed beams, each sticking out in relation to the more central one. These beams are arranged in such a way that the bridge at the bottom, from support to the mid-length, has a variable section decreasing by four beams up to one.

According to some theorists this bridge was not leaning on the banks but clamped to them. According to this hypothesis the variable section of the lower beam is not therefore representing a case. Were it a wedged beam, it would be then appropriate to thicken the beams near the joints where the bending moment is higher.

Most likely the variable section of Palladio's first invention has to be sought in constructive motives rather than in an equoresistant section. This is an application of construction techniques for gradual steps. In this technique, the construction, which is carried out through ropes and stands, starts from the longitudinal beams of the shorter length, the most external ones. These, already linked to *colonnelli*, are put in place and retained cantilevering by mean of fixed cables. The construction proceeds by putting in place the beams a little longer, also already connected to the *colonnelli*, and kept cantilevering through another system of ropes. This new cantilevering system is then connected to the previous one by the inclined and upper beams. In this way the structure proceed gradually from the two sides towards the middle. The last beam to be put is the central one, which is composed of one piece. At the time that the two parts join together in the centerline, through the last beam, the bridge does not need cables or stands anymore and it behaves as a single unit, supported at both ends.
The construction of the first invention was then carried out moving one bay of the bridge after another, and provides indications of the more general case in the construction of infrastructure, which occurs when the bed of the river is impassable, and in this the invention is placed as an alternative to that on Cismon where precisely it is supposed to be possible to put an intermediate temporary support.

3.4 Bridge over Rhine in Schaffhausen by Hans Ulrich Grubenmann

There were many works of Hans Ulrich Grubenmann that, due to their effectiveness and their daring, made his name known, not only within the borders of the small Appenzell canton where he was born, but even in Switzerland and later, thanks to the travel reports of many foreign scholars attracted by the enthusiastic descriptions of his work, throughout Europe. But why Grubenmann will be remembered are undoubtedly his ingeniously designed bridges of considerable length, the most famous of them are certainly the bridge over the Rhine at Schaffhausen and the Wettingen bridge on the Limmat.
Their extraordinary singularity unfortunately did not allow them to be saved: the French troops destroyed them, along with 8 other of his bridges, in 1799 during the war against Austria to secure the retreat. So now there are only two Grubenmann's bridges: the Kubel and Tobel bridges over Umasch near Herisau (Killer, 1985).
Fortunately, however, the sketches of plants and sections in the travel notebooks of many travelers, first of all, the English architect John Soane, allow to keep their memory alive and give us the opportunity to study them and still admire their beauty.
The wonderful works of Grubenmann were based on a synthesis of knowledge, experience and personal research, embracing a unique insight and an acute sense of observation, honed by experience and supported by a vast construction expertise, as result of a secular tradition. He was in fact the descendant of a family of great carpenters and then consolidated tradition in working with wood combined with direct experience in building roofs for churches with his brother Jakob, based on the

principles of carpentry to cover considerable spans, has allowed Grubenmann having the tools necessary to create structures never seen before.
From a structural point of view in fact, the analysis and the construction of roof structures with average spans of 15 m was a unique way to understand the strength of materials and the behaviour of wooden structures by means of practical experience.
Not to mention that in addition to the creation of ingenious solutions to construction problems, he also experimented with new types of structures and some of these were adopted later as structural systems and witness his remarkable insight and understanding of statics.
His structures are an exception in the panorama of bridge engineering of that time because he was the first to build bridges of such span sizing the structures only through empirical methods and static insights: from the beginning of the nineteenth century the engineering approach begins to take over, in which the design of structural elements was done according to the mathematical calculations of stresses.
Grubenmann’s bridges, due to their high experimental, were characterized by an abundance of static principals that exist simultaneously: their hyperstaticity allowed the structure to be more resistant thanks to the positive cooperation of all the parts building it, not to mention that in case of failure of one of them, the bridge was able nevertheless to keep its functionality through the others.
The bridge, shown in Figure 3.11, built by Hans Ulrich over Rhine at Schaffhausen (1756-58) was one the most important (if not the most) results achieved at that time in the engineering field of wooden bridges.

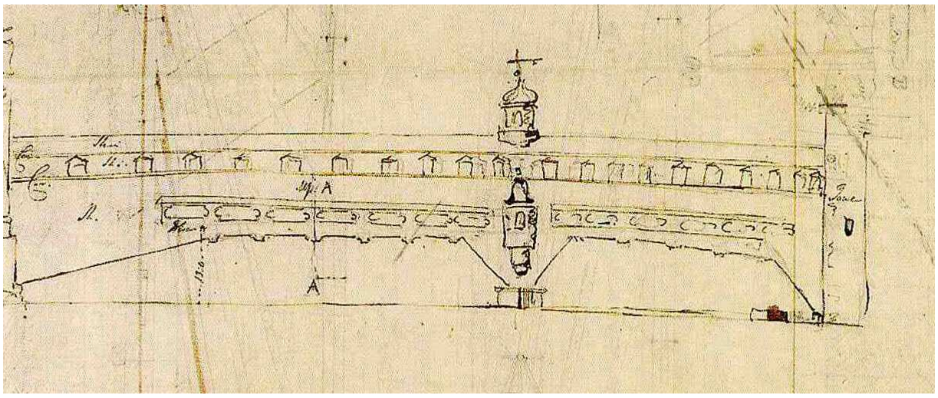


Figure 3.11 The Grubenmann’s Bridge sketched by John Soane (Maggi, 2003).

The city of Schaffhausen had a stone bridge crossing the Rhine that was overwhelmed by a flood in 1754. The City Council decided to invite tenders for the construction of a new wooden bridge: the notice also included the construction of scale models that were then subjected to "elementary experimental tests" (they were proving useful because they gave an idea about the relations that there were between the kind of structure selected and the resistance of the system).
Grubenmann proposed a project with which he thought to build a bridge with a single span of 119 m across the river without intermediate supports: to prove the effectiveness of his ideas he also sat on the scale model. But the Council did not trust the solution provided and ordered the builder that the bridge had to use the central masonry pier of the previous collapsed bridge as support.
The new design significantly scaled the boldness of the previous one but it still repeated the same structural principles: It was therefore a structure divided into two spans, the longest of which measured 58,8 m, and remained however a bridge out of

the ordinary by the standards of the period. If the first project had been approved, with its single span of 119 meters it would be the longest timber bridge ever built before.
The static capacity and the skilled construction shown in the draft project of the first bridge and the completion of the second witness to the author's considerable knowledge of the strength and stiffness of structures of great span and of the details appropriate to perform them through the use of iron in addition to wood for structural members and connecting elements.
The bridge of Schaffhausen was the first ever "*Sprengundhaengewerkbrucke*": it gathered itself together in fact two different design principles (Figure 3.12). The *Sprengwerk* consists of a static system where the deck is supported by struts, while in the *Haengewerk* principle the deck is suspended through the uprights to a structure formed by two chords, an upper and a lower, and two diagonal struts.

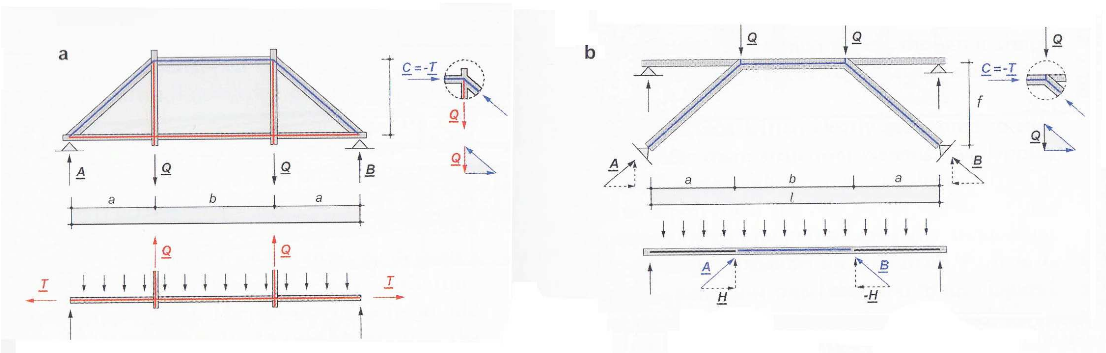


Figure 3.12 Load carrying principles: a) the Haengewerk system (or queen-post truss), b) the Sprengwerk system (frame with inclined struts) (Maggi, 2003).

The bridge was the synthesis of all Grubenmann's knowledge and ability: in fact the combination of many elementary structural systems, required both remarkable design qualities, to manage the complex connections between the different parts of the structural system, and a deep, intuitive knowledge of behaviour of structures.
In the first project Grubenmann (Figure 3.13) tried to support the bridge with three different main structural systems, making them work together in their functionality without giving them a clear hierarchy.

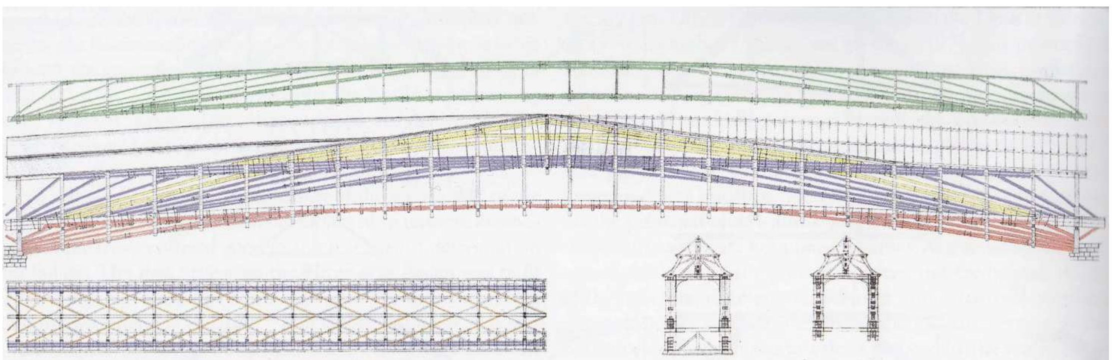


Figure 3.13 Grubenmann's first project for the Schaffhausen bridge: individuation of the principle structures (Maggi, 2003).

The bearing structure of the bridge consists of a polygonal arch (yellow lines), a fan-shaped frame formed by inclined struts and centered on the abutments placed under the bridge carriageway (red lines) and a multiple overlapped queen-post truss (blue lines). The tensile stresses in the beams of the deck as a result from their action as bottom chord in the blue system are partially thwarted by the compressive forces induced by their role as straining beams in the red system. The major height of the bridge in the center is that the greater rise of the arch in the middle reduces horizontal thrusts, compression stresses, and the required cross sections of the wooden elements (Maggi, 2003).

In addition to the three principal structural systems, Grubenmann added a further structure (green lines), a multiple overlapped trusswork similar to the blue one, located within the roof space parallel to the axis of the deck and supported by its portals. This structure had two roles: the first was to drive part of the weight of the roof directly to the abutments of the bridge located on the riverbanks, reducing thus the stresses in the other three main structural systems, and the second was obtained by connecting its bottom chord, working in tension, with the top bracing system (brown lines) to reduce the compression stresses in the upper chord of the main truss (blue lines) (Maggi, 2003).

Grubenmann probably wanted this last truss to participate in supporting the deck, but to make this happen it was necessary to connect it somehow with the other three systems, which, looking at the material we have available, it seems he has not done it.

As shown in the 3D model (Figure 3.14) of the bridge actually made the structural system is very similar to that of the first project.

Figure 3.14 The 3D model of the second Grubenmann’s project: individuation of the different structures (Maggi, 2003).

Each of the two spans of the new bridge had a frame with a series of inclined struts centered in the abutments (red lines) and an overlapped multiple queen-post truss (blue lines) (Maggi, 2003).

Similarly, the longitudinal beams composing the deck act as straining beams for the lower frame working in compression and as bottom chords of the upper truss working in tension. Even here there was a unique polygonal arch (in yellow) crossing the whole width of the river and standing on the riverbanks. The height of the arch was less than the one of first proposal: the stresses in the structural elements and the horizontal forces, resulting from the typical behaviour of the arch, would therefore have been higher (Maggi, 2003).

Ultimately it is possible to see the strong similarity between the two projects and the Grubenmann’s extreme projectual flexibility that, despite not being able to realize his bold ideas, he still succeeded in offering advanced technological solutions borrowed from the first project.

Much has been discussed about the role actually played by the masonry pile in ensuring the stability of the structure: there were those who argued that the bridge was initially suspended and then because of the creep phenomena would be supported and those who said that the support from the pier was absolutely essential (Killer, 1985).
However it seems highly unlikely that such high stresses generated by the system could be borne by so low arches that also had an inadequate cross section due to the excessive slenderness. This hypothesis is also observing that the axis of the bridge is not perfectly straight but rather inclined to a horizontal angle of about 8% to the central stack. This hypothesis is supported by observing that the axis of the bridge is not perfectly straight but rather inclined to a horizontal angle of about 8% towards the central pier. Inclining the deck this way were generated some horizontal deviation forces due to the thrusts of the two spans that the structural organization as actually realized does not seem able to withstand.
One more element that confirms the fact that the central pier, despite the legends, was a cooperative part of the system derives from observing that the inclined struts (in red) relied entirely on the support. If the pier would not have been used the portals would have had the task of transferring somehow the forces to the polygonal arch but his does not seem possible, given both the manner in which they were performed and the high point loads concentrated in the arch crown (Maggi, 2003).
It can be thus concluded that the function of the polygonal arches was not being the main bearing structure but essentially they contributed to a better distribution of the loads, reducing stresses in the two other systems and providing an additional safety device in case of failure of one of them.
Finally the bridge of Schaffhausen has been one of the most intelligent and efficient structural systems that have exploited the structural redundancy to cross large spans.

3.5 American timber bridges

The great experiment results and subsequent knowledge achievements that took place in Europe, and particularly in Switzerland, during the eighteenth century had a slowdown towards the end of the century.
In the U.S. the expanding population and the systematic colonization of the immense territory between the two oceans required that road and infrastructure network was completed as quickly as possible in order to be able to link the various parts of the country.
The geographical features of the country (rivers of considerable size and valleys and ravines of big extent) demanded the realization of bridges of remarkable length, not only for the big boom of the railroad, whose development went becoming more intensive, but also for the highway use. The fact that the exigent circumstances required the construction of works that were economic, quickly and easily achievable and rebuildable if necessary in a few years, they produced an enormous development of wooden bridges. In fact, they were much cheaper than both iron and concrete and had the undoubted advantage of being extremely easier to implement. Not to mention that since there was no American tradition of building with stone, wood was used because there was plenty of it.
The United States then in the late eighteenth and early nineteenth century became the scene of the experimentation of new forms and innovative structural solutions regarding the wooden bridges. The elementary structures which the American engineers and builders focused on to develop countless variations (all duly patented)

were essentially the truss and the arch: just think that, between 1797 and 1860, 51 patents were registered relating to structural solutions for wooden bridges (Ritter, 2005).

The peculiarity of the American spirit in dealing with the construction of a bridge, unlike the European mentality, was the purpose not only to be able to produce an efficient and effective structure that could satisfy as best as possible a need perceived by the community, but above all to create something totally new and better than anything else seen before.

Through this we witness the blooming of extremely attractive structures and a continuous testing on the optimal use of the material, on the arrangement of structural elements and on possible combinations of wood and steel to produce bolder and stronger structures.

Great attention was then put by American engineers on the protection and study of details that ensured a longer durability to the structures, especially in road bridges, and then recurrent became the use of shingles on upper and lateral sides that also produced the undoubted advantage to make structures stiffer, thanks to a "box-behaviour ": hence the peculiarly American kind of covered bridge arose, partly borrowed from the Swiss ones, but however with own specific characteristics.

The two crucial figures who have given impetus to the development of wooden bridges engineering in the first half of the nineteenth century were undoubtedly Wernwag and Burr: they have been called the “*American carpenter bridge builders*” due to their largely intuitive and practical approach, rather than mathematical, in the bridge construction. They were the inventors of a new typology of structures that combined the two primary structural systems of the arch and the truss: the results were the so-called arch truss bridges that could cover significant spans that other structural systems were not able to do.

Lewis Wernwag was a renowned German manufacturer immigrated to Pennsylvania; in his career he has built 29 bridges scattered among six countries of the East Coast of the United States. His works had as distinctive feature that they were able to integrate the arch and truss in a composite efficient structure. His most famous bridge was undoubtedly the Colossus Bridge on the Schuylkill River: it was composed of four arched trusses parallel and interconnected to form an additional truss with a square cross section inside which the carriageway passed: in this way the truss simultaneously acted as bracing system and the vertical elements were much larger than the diagonals, which were rather thin because they were iron tie rods. The Colossus Bridge deserves special attention because, besides being the longest American wooden bridge at that time (90 m), it was also the first great span wooden bridge to use iron elements. It was destroyed by fire in 1838 (Ryall, 2000).

The other leading figure in American engineering was Theodore Burr: he was the first with his Hudson River bridge (52 m), shown in Figure 3.15, who almost succedeed in constructing the first truss bridge ever built, but the structure he designed, which consisted in a combination of trusses with parallel chords, proved highly unstable (especially under the action of moving loads) and therefore he had to stabilize it with two reinforcing arches that were set on the abutments at a level lower than the deck one. It should be underlined that in this structure it were the arches to be added later to the truss and not vice-versa and in this feature lies its peculiarity. This type of bridge, given the ease of installation and the very low implementation costs, became for a while the most popular and famous in the USA and known as the Burr truss (Ryall, 2005).

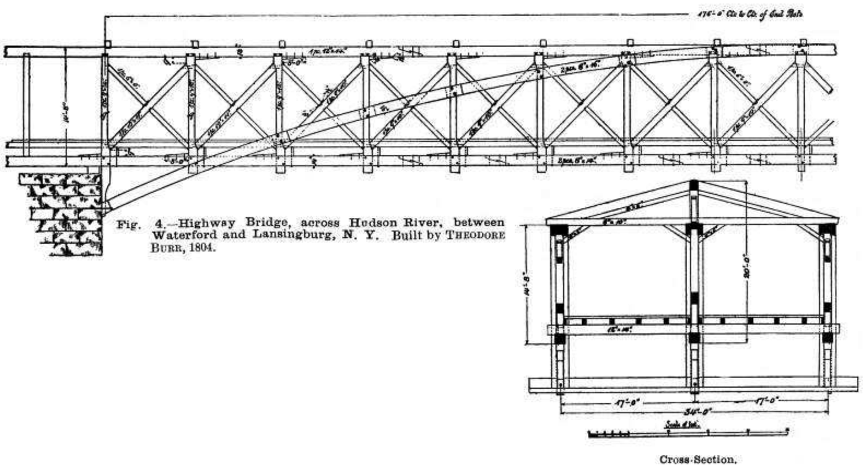


Figure 3.15 Burr bridge built in 1804 over the Hudson River between Waterford and Lansingburgh, New York (Ritter, 2005).

After these remarkable works there was not any particular development and all possible variations achievable from Wernwag and Burr models were tested: only in 1820, thanks to Ithiel Town there was the appearance of the first real truss bridge that had no supporting arch and that was not solicited by any horizontal thrust. It was patented as Town Lattice (Figure 3.16) and was extremely light, quick to build and cheap: it could be erected in a few days since elements of solid wood with uniform cross sections were used. The various trusses composing the structure were formed of wooden planks, criss-crossed with an angle of 45 ° and connected at the intersections with wooden pins (Ritter, 2005).

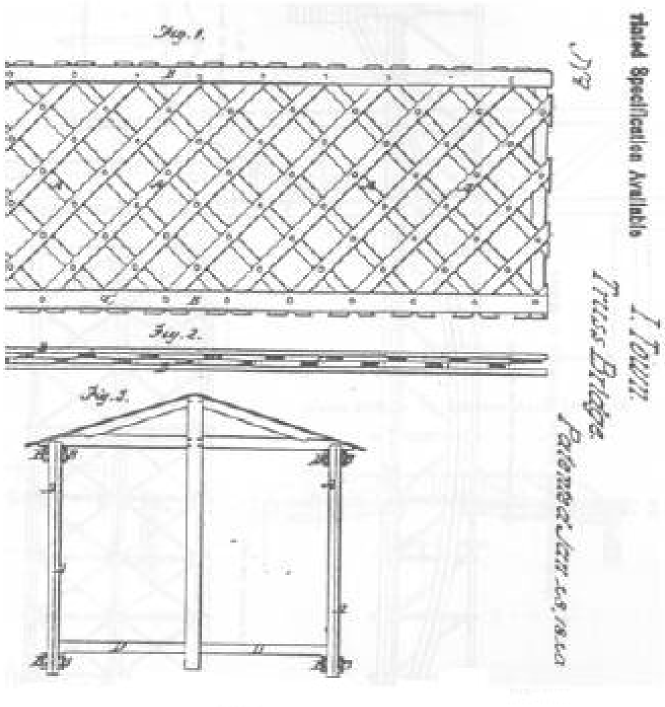


Figure 3.16 The Town lattice patented in 1860 (Ryall, 2005).

With the demand for bridges that were capable of supporting the rail traffic, structures entirely made of wood or with small and simple iron fasteners could no longer meet the requirements, so that here in 1840 Howe patented the first truss in which steel structural parts were inserted: the Howe truss (Figure 3.17) consisted of two parallel wooden chords, wooden diagonal braces working in compression and tense vertical iron rods. This was also the first patent in which the internal forces in the elements were calculated with mathematical procedures.

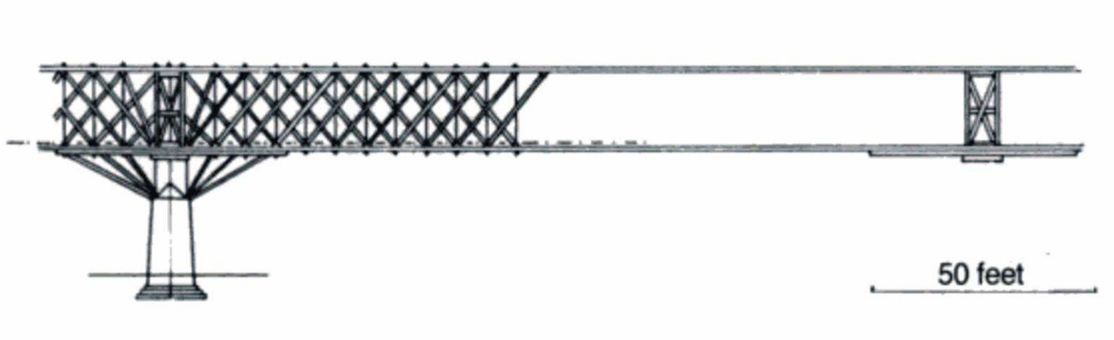


Figure 3.17 The Howe truss (Ritter, 2005).

In 1844 then also the Pratt brothers patented their truss in which the wooden vertical elements were compressed while the iron diagonals were tense: in this type of system, as shown in Figure 3.18, the advantage was that the wood was used to the fullest, without any particular inclinations and consequently with simpler manufacturing, but the disadvantage was that the overall cost was much higher due to the large quantities of iron used and especially because details for the anchorage of the diagonals had to be closely studied.

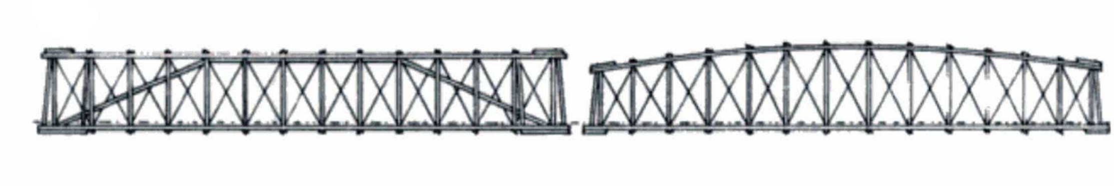


Figure 3.18 The Pratt truss (Ryall, 2005).

Towards the end of 1800 the growing use of iron and the introduction of steel for the construction of bridges involved an irreversible decline for wood, which was still used but to a lesser extent and no longer for wide span bridges, supplanted by the new building materials.

3.6 Accademia Bridge in Venice by Eugenio Miozzi

Building timber bridges in Venice has always been a challenge due to the difficult environmental conditions, especially since the material near to the salt water of the lagoon is subjected to very high degrees of humidity and to constant changes in water level leading to full or alternate immersion in relation to the tide. The wooden piles that indicate the channels of the lagoon and those who, in many cases, support girders of the bridges are an example of how long can the wood last in such bad situations.

Well preserved when immersed in the mud, always acceptable when wet, the piles get deteriorated, as known in the area of "wet wipes".
The use of wood in Venice is secular and it has accompanied the urban development of the city, in the form of foundation piles, floors, roofs and bridges above all.
The uncertain consistency of the soils outlined essential rules and methods of construction, morphology of the environment suggested distributive precepts for building and for the whole urban organization, imposed by traffic both by land and water. In this context, bridges were essential: the use of wood was dictated (and is still dictated) by the needs of lightness (due to reduced volumic mass of the material) in order not to weigh too much on the channels bed and on the banks, of speed of construction (and reconstruction) and of lower costs compared to masonry structures.
An important timber bridge to analyze is without any doubt the Acccademia Bridge designed by the engineer Eugenio Miozzi.
Already in the fifteenth century it was thought to build a second bridge over the Canal Grande, as well as the Rialto, then still made of wood, but the Accademia bridge had to wait until the nineteenth century.
Indeed, the proposal came only in 1852 by Alfred Neville, engineer and owner of foundries for melting iron: his metal truss bridge was inaugurated in November 1854, and it had a span of about 50 m. During the thirties of the twentieth century became apparent some problems due to corrosion, to the excessive slimness of the compressed webs of the trusses and to the neglected dynamic action of the crowd the steel bridge could no longer be used (Barizza, 1986).
The city authorities, waiting for the results of a national competition sponsored by the municipality of Venice, decided to entrust the implementation of a temporary bridge, which would also guarantee a long service life, to Miozzi, the main engineer of the municipal technical office, who designed a wooden truss structure. The engineer designed the structure so that the replacement of damaged elements could be performed without interrupting the transit and he even predicted the possibility to replace the entire bridge (Miozzi, 1933). It is also very important as it has been one of the first glulam timber constructions.
In December 1932 the Accademia steel bridge was dismantled and replaced by the new structure. The new bridge, for pedestrians only, consisted of a single truss arch structure (shape chosen to minimize the visual clutter) formed by two ribs of laminated wood held together by metal plates and bolts on which the deck has laid. It was one of the largest wooden bridges at time, reaching a free span of 48 m (Populin, 1998).
In Figure 3.19 it is possible to see how the work was structurally organized: the two ribs had a constant width of 1 m and a variable height from 2 m at the imposts up to 1 m in the midspan, tapering toward the centre the bridge guaranteed a clearance of 8,25 m from average water level for the passage of boats. In addition, as already said, there was a second lattice structure that was overlapping the arches and that was to form the carriageway.

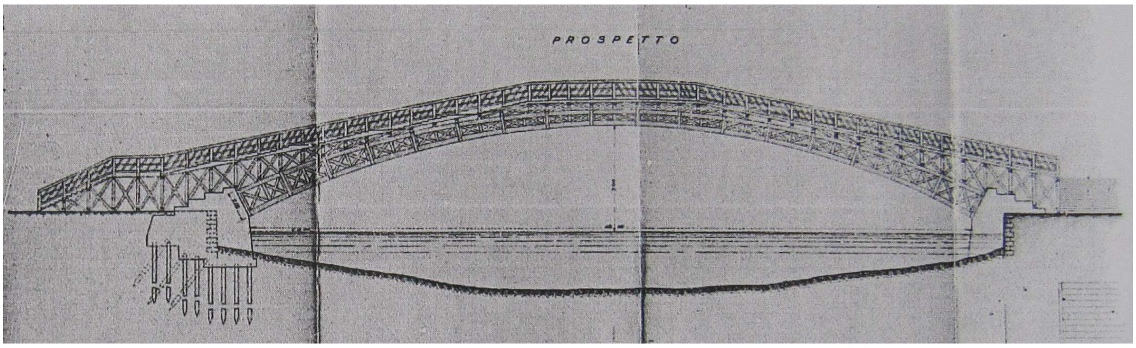


Figure 3.19 front of the project for a wooden bridge over the Canal Grande (Populin, 1998).

Also in Figure 3.20 one can see that each rib was formed by two chords connected to each other: each one was 30 cm wide 1 m high and made of larch wood boards. Between the two chords (upper and lower) of each arch uprights and braces were placed and there were also metal components put perpendicular to the direction of laminations: evidence indicating the experimental phase of the technology. The elements were not glued but kept locked by two metal plates 5 mm thick welded together to ensure the structural solidarity. To allow for a greater stiffness to horizontal thrust of the wind and other fluctuations, the ribs were not parallel but had the imposts retracted: the overall width varied from 5 m in the middle up to 7 m at the imposts (Miozzi, 1933).

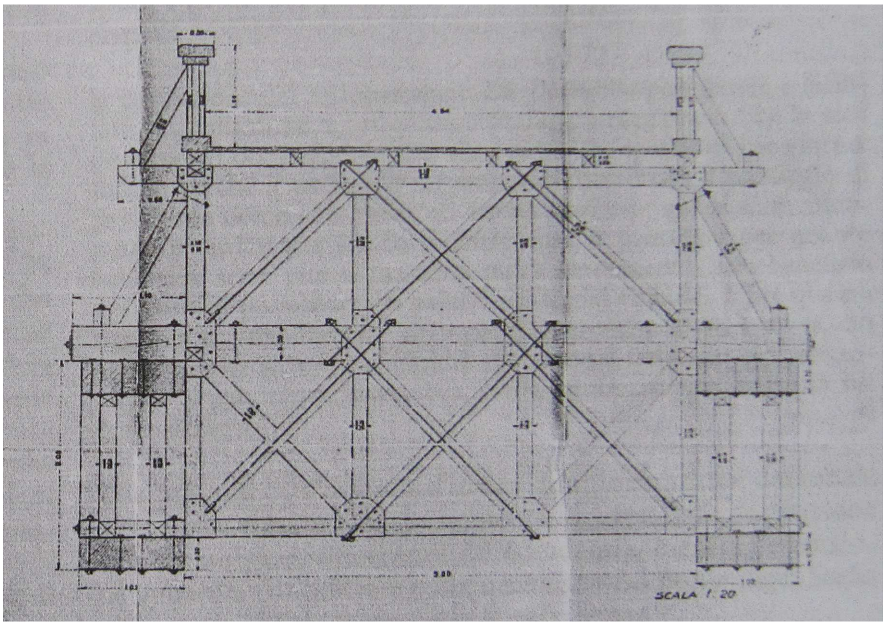


Figure 3.20 The cross section of the bridge near the abutments (Populin. 1998).

Miozzi had also planned a coverage for the Accademia bridge, being conscious of the necessity to protect the material from deterioration caused by stagnation of rain water, but then for lack of funds it was not realized.
The whole structure was built in just over a month (from December 1932 to January 1933): the glulam arches were assembled on the ground in four pieces each and then they were mounted by means of scaffoldings, as shown in Figure 3.21.

Figure 3.21 Construction phases of the arches in glulam timber (Populin, 1999).

The bridge was tested in February 1933 with 2000 sandbags, to simulate an overload of 400 kg/m^2 (Miozzi had also considered 140 kg/m^2 for lateral wind load), and the deflection in the middle was only of 9,5 mm.
As a result of dimensional variations due to connections and to viscous behaviour of wood there would have been a significant lowering of the structure in its entirety; there was then the necessity to avoid this phenomenon in anticipation of future additional dimensional variations due to the drying of wood. Miozzi specified so that, by some oak wedges placed in the middle of the arches, this issue could be prevented: the wedges deeply hammered provoked the disarmament of the bridge so that the scaffolding that supported the arches could be taken away without any effort. Practically Miozzi achieved a state of internal coactions to improve the stability of the arches (Populin, 1998).
In fact it is to be emphasized that a timber truss structure with such a high number of joints could not be regarded as a flexible structure and it would be very deformed under the action of forces acting on it before reaching the equilibrium position. Hammering the wedges in the top part of the arches enough length was returned gradually to them that was then absorbed by the creep phenomena, making it possible to disarm without significant deformation of the primitive shape; the shape of the bridge was however designed and built in dependency of the loads.
As a temporary bridge just five years after its opening it had to be subjected to a series of refurbishment works (such as the verification of the tightness of bolts and surface treatment with linseed oil) because of a poor maintenance.
After the end of World War II the Accademia Bridge was in precarious static conditions and it was decided to restore it through the use of four steel arches coated with wood panels and connected by elements in solid and glulam wood: it was a bridge which resembled the previous one by Miozzi but it became a steel-wood composite structures: in this way one of the first and most interesting examples of glulam timber bridge disappeared.

3.7 Essing Bridge by Richard Dietrich

The Essing Bridge, by Richard J. Dietrich, is a structure that for its unconventional structural solutions and the originality of the design has become one of the most famous contemporary wooden bridges. It is located in the charming natural landscape of Altmuehl River Valley and was designed to cross the Rhein-Main-Donau canal, which connects the North Sea with the Black Sea. Due to the beauty of the natural environment, both for the building owner and the designer himself, it was important to capture the genius loci and to preserve the atmosphere of the place: therefore, Dietrich designed a bridge that used the traditional local materials and reflected the harmonic-curving, hilly landscape (Figure 3.22). As a result the owner decided that the bridge had to be wooden made.

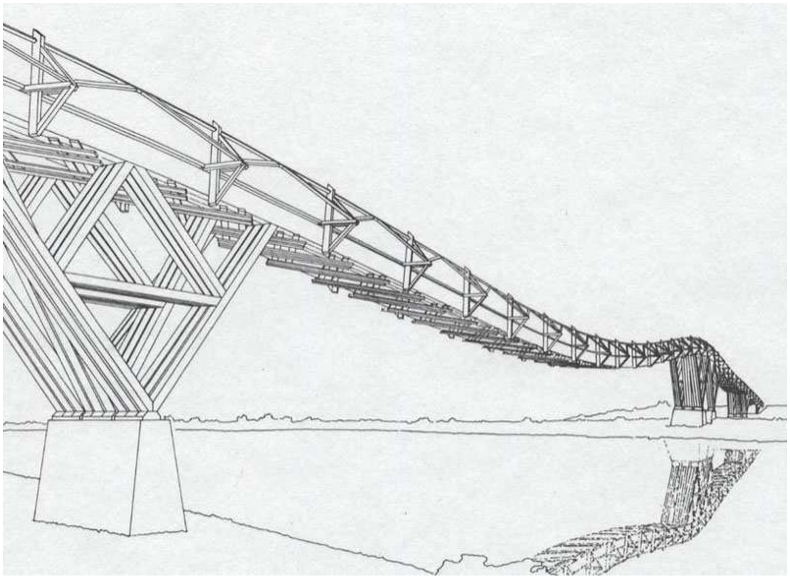


Figure 3.22 Perspective of the bridge crossing the channel (Dietrich, 1998).

The local conditions needed an approximately 200 m long pedestrian and cycle bridge to be designed that crossed the 73 meters wide Main-Danube canal, one main road and two rural roads. Besides a clear carriageway width of 3,2 m were also required a clear height of 6,7 m above the water and of 4,7 m above the roads. Another requirement was to make the bridge not only for the use of pedestrians and cyclists, but in case of need, emergency vehicles should be able to pass through the bridge so that a live load of 5 kN/m² has been adopted during the design stage.

Experiments with standard forms of construction and usual structural principles, such as trusses or arch bridges, have proved to be not effective, as they have, especially in order to cross a 73 m free span, too chunky dimensions. Dietrich therefore considered the age-old principle of free-hanging rope bridge, but found that suspended cable structures did not work in that specific case because of the high piers required; apart from that the main structure steel should have been made of steel, but wood was required.

The designer then tried to develop a possible filigree wood construction, which all claims, the design and static, would be fair: these considerations led to the idea of trying a wooden tensile structure. However, a tensile wooden structure with such dimensions have been never built, so it took a seven-year planning period with

continuous cooperation of all figures involved in the project, also due to a lack of computational and design tools.
The final result has been a so called “stressed ribbon bridge” or “tension bridge” because it has been designed as a tension structure following a cable line behaviour and thus it is able to transform some 90% of the vertical loads into tension forces, with only 10% acting as bending moment (Brueninghoff, 1993).
The bearing structure, the ribbon, is made up of nine spruce glulam beams (Figure 3.23) with a thin cross section in relation to the entire span of the bridge; they are joined in three groups of three beams each.

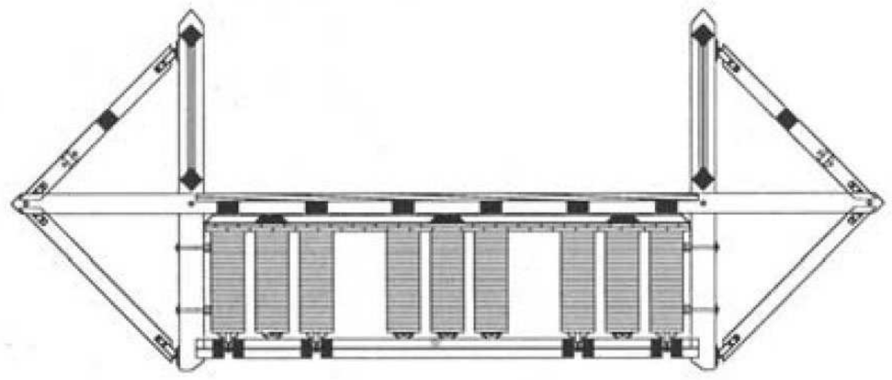


Figure 3.23 Cross section of the deck of the bridge with the nine glulam beams (Brueninghoff, 1993).

Dietrich designed the ribbon as a series of 42 equal elements with a length of 4.6 m. This means that all the elements used were carefully planned and prefabricated, and they only had to be put in place. It also resulted in an assembly accuracy of a few millimeters difference.
The normal forces produce deformations which lead to bending stresses in the beams: in order to keep the bending moment small it proved effective to use beams of small depths relating to the width and therefore minimal bending stresses. Moreover the small dimensions (22x65 cm) of the beams is appropriate to transfer the loads in the form of tension forces, on the order of 4000 KN, directly to the abutments (Brueninghoff, 1993).
The piers, which carry the ribbon, as shown in Figure 3.24, have a composed structure similar to a truss; the pillars composing each piers are divided in two groups: the “external” one consisting of three couples of piles with a 22x22 cm cross section, and an "inner" one of three couples of piles with the same size. The inner group has a triangular disposal and there is a hinge at the base of every pier; the connections at the joints of the piers have been carried out with nailed plates. The pillars are used almost solely in support of the tense band.

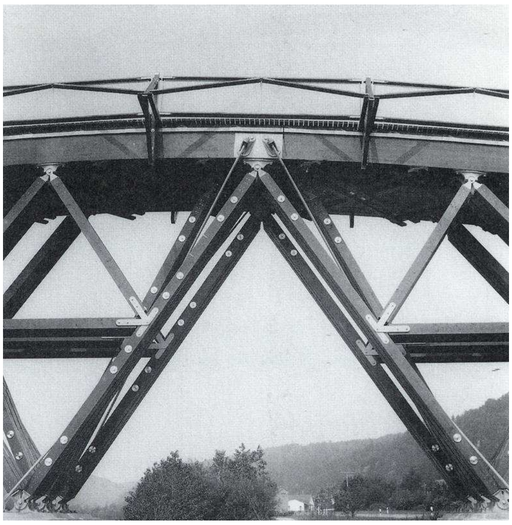


Figure 3.24 Detail of the triangular piers of the bridge (Dietrich, 1998).

Each band is gathered by coupling elements near the upper and lower support points and connected to the articulated ribbon up and on the reinforced concrete base down.
Dietrich’s original idea was abutments and base in stone, but he had to change material because of the high forces involved. Instead, they were executed in reinforced concrete. The base was established on the basis of the poorly viable soil on large diameter drilled piles and permanent ground anchors.
Bracing system is accomplished by two cladding layers nailed crosswise to the top of the beams and diagonally to the bottom.
Wind forces, walking impulses and vibrations are absorbed by the railing design, thus preventing the bridge from bouncing in the critical area. In addition to the parapet design support even steel struts set diagonally through the pillars provide more stiffness to the whole structure.
A long service life has been granted by covering with a waterproof sheet of titanium zinc the whole substructure just under the carriageway; all structural elements were designed so that no water can accumulate and the deck has been performed with a waterproof tropical hardwood.

Attention on the most rational construction implementation and a high degree of factory pre-assembly have been achieved at the building site and most elements could be assembled without expensive auxiliary scaffoldings. Thanks to the power of the firms involved a high degree of perfection has been performed during the execution and so almost all the elements could be factory-produced and had only to be brought to the building site and assembled (Dietrich, 1998).
Already in the planning phase Dietrich has focused on appropriate design and simple solutions, especially concerning the detailing. All fasteners are made of cast steel and the same piece was used for every detail: this means that, for example, for all joints only a negative had to be prepared, and then a corresponding number of copies was made from it.
All connections, joints, drive shafts, coupling elements and nailed plates have been made in cast steel and were to be mounted on site in just a few steps.
The construction work started with the casting of the reinforced concrete abutments and plinths in situ. The plinths were equipped with bearing plates on which the hinged joints of the piers were then bolted.

The pillars were already pre-assembled in the factory so they have been brought to the site, put in the right position with a crane and with the aid of scaffoldings and finally connected to the joints by nailed plates.
Then the glulam beams, which were already manufactured in the factory and provided in parts with a maximum length of 45 m, have been delivered to the erection site by trucks and placed by a crane; the separate sections were joined in situ by finger joints and the appropriate climate conditions to realized them were granted by using a tent and a movable gluing and pressing equipment (Figure 3.25). Moreover every finger joint, after its completion and hardening, has been tested with loads up to 150% of those assumed during the design stage (Brueninghoff, 1993).

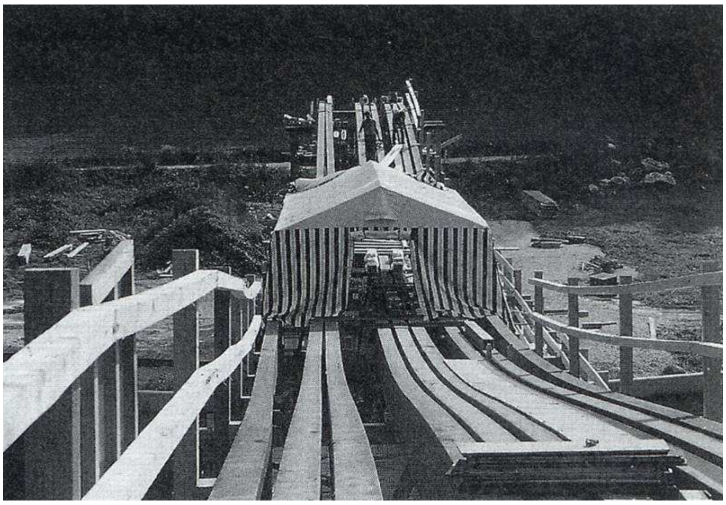


Figure 3.25 The in site process of finger jointing (Dietrich, 1998).

After the completion of the bracing system on the full width of the carriageway sheets of zinc were placed to protect the bearing structure from rainfall; then the balustrade has been assembled and the last load tests were carried out in the middle of the largest span through water-filled containers in order to prove the carrying capacity of the bridge under full load (Dietrich, 1998).

3.8 Traversina footbridge by Jurg Conzett

Traversina footbridge, designed by the Swiss engineer Jurg Conzett, is undoubtedly a structure that deserves to be described due to the manner how its designer has been able to cope with, and resolve brilliantly a number of difficulties (mainly related to the natural environment and economic issues) and succeeding at the same time in making a simple, innovative and aesthetically strong impacting product. To give an idea of its effectiveness it is enough to think that, though the bridge was destroyed by an avalanche in 1999 just two years after its completion and was rebuilt again by Conzett with a completely different shape, however, it is still remembered and is considered, rightly, as one of the best examples of contemporary bridge engineering.
The simplicity of the structural solutions combined with effectiveness of performance and optimum use of material resources is admirable and should serve as an example and inspiration to any designer at the drafting stage of a project.

The environmental context in which the bridge had to be inserted (Figure 3.26) was quite complex: within the Viamala, one of the largest canyons in Grisons, were traced the remains of an ancient Roman path, now very difficult to recognize. This trail most likely involved a wide path leading down into the ravine and reached the opposite end through a ford. The aim of the contracting authority (the local mountaineering club) was to make the route (path) viable again.

Figure 3.26 The footbridge crossing the ravine (Conzett, 1997).

Being very complex and expensive to repeat the same route as the path to the original because of the eroded and saggy soil and of situations of extreme variability of stream flow in the gorge, the choice led therefore to the construction of a bridge.

The difficulties that the designer had to face were mainly those related to find the position of the site: it presented extremely difficult ways of access, the usable space to set up the shipyard to build up the structure was virtually nonexistent, and also the wind forces were remarkable because the airflows channeling in the ravine increase their speed. Beyond that there were also the issues of limited economical resources and very short time to achieve.

Furthermore, considering the spectacular beauty of the alpine scenery, the use of such materials was required that were not going to affect the perception of the observer and were as much as possible "environment friendly".

Based on these limitations the design and construction techniques for the bridge were taken to the limit and, given the exceptional situation and the fact that there were no previous similar cases, unconventional methods were adopted: the designer decided to use a helicopter to transport the partially prefabricated structure and all the necessary equipment in situ.

Consequently the dead weight of the structure had to be equal to the maximum load-carrying capacity of the most powerful helicopters available, i.e. 4,3 tons.

The materials chosen therefore to build the bridge were fir timber (given its good ratio between strength and weight) and stainless steel cables.

The distance to span between the two sides of the ravine was 47 m at an inclination of 6%. The abutments for the bridge, given the favourable geological conditions of the soil were made in reinforced concrete on previous roman stone articles (Conzett, 1997).

In order to optimize the total dead weight the structure was divided into two parts (Figure 3.27): the substructure and a superstructure slightly heavier with the task of protecting and making more stable and stiff the substructure.

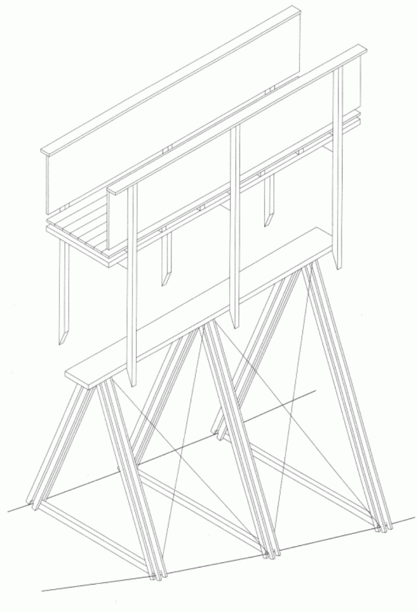


Figure 3.27 Diagram showing the composition of the footbridge: superstructure and substructure (Conzett, 1997).

Ultimately the bridge consisted of two structural systems completely different but cooperative and complementary, in which the designer was inspired by the work of his illustrious compatriot Hans Ulrich Grubenmann (especially concerning the bridge over the Rhine at Schaffhausen, see Paragraph 3.4), where the overall behaviour of the bridge was simultaneously under two different construction principles.

In the Traversina footbridge the substructure is ultimately a very light space truss system: it is a three-chord truss girder with two parabolic wire ropes as bottom chords. This solution was chosen after an evaluation of different types of trusses as it was by far the lightest of all. This type of conformation of the truss also has the advantage that under the action of permanent loads the diagonals appear to be minimally stressed with the undoubted advantage of being replaced if broken without major problems, considering also the difficult management of resources and equipment on site.

The static behaviour is clear: the lower steel wires worked in tension while the upper chord (glulam beam) was compressed.

The substructure acted as a simple supported beam and, also for the particular parabolic shape that reproduces the trend of the bending moment, therefore requires a bracing system that provides to ensure torsional stability out of plan. The superstructure acted as a stabilizing element and was made up of two three-plies stiffening girders, which went together to form the parapet, and a glulam beam placed under the carriageway. The entire superstructure was connected to the trusses below by the vertical elements that were going to put inside the diagonal struts; they were composed of four wooden poles so as to simultaneously maintain the steel wires tense and screw the uprights of the superstructure.

As already mentioned, in this project the wind was a crucial factor in defining the shape and peculiarities of the structure: in fact the bridge was placed transversely to the direction of Foehn (an alpine periodical wind) and during the design stage values

for the wind forces were taken equal to 1,3 KN/m. In the case of a strong gale the tense wires subjected to small normal forces behave as brittle components: in fact the chord on the upwind side may be have a loss of tension and consequently the chord on the downwind side becomes the fulcrum around which the whole structure will rotate. To ensure safety attention should therefore be given to the ratio between the wind force and the vertical load and to the geometric configuration of the truss (Conzett, 1997).
To withstand the horizontal loads due to wind, in addition to the resistance provided by the truss itself, a glulam beam has been added beneath the carriageway, whose horizontal stiffness significantly increases the resistance of the entire bridge.
Since the budget was very limited, the solutions adopted by the designer were all directed to save money and to ensure the longest possible lifetime: the three chords that maked up the substructure and constituting the main feature of the bridge were properly protected.
The two lower chords were of stainless steel while the upper one, which consists of a glulam beam, was protected against rainfall by another glulam beam inserted as stiff element against the wind, larger and protruding, placed above.
The diagonal struts were, as mentioned above, consisting of four elements each with a section of 30x80 mm and thanks to their small sizes, although exposed to the elements, were able to completely dry out very quickly.
No one of the wooden elements was impregnated but all joints were studied in order to run replacements as easily as possible: just think that with the adopted static system the steel diagonal tie-rods and the transverse beams were minimally loaded, and this facilitated an eventual removal, while for the elements directly exposed they were all easily accessible.

The process of erection of the bridge was certainly the crux of the whole operation: the weights of the various components were constantly monitored so as not to exceed the maximum load-carrying capacity of the helicopter, and during the assembly phase, which took place 500 m from the place where the bridge had to be built, all wooden elements were covered to prevent rain or moisture imbued them, thus increasing their weight.
First, the upper chord was raised to a certain height above the ground and below it triangular elements were set. Then the steel wires were attached. The complete substructure was transported by the helicopter (Figure 3.28) directly over the abutments and then with the help of ropes was positioned and secured to the concrete plinths. The whole operation lasted less than half an hour.

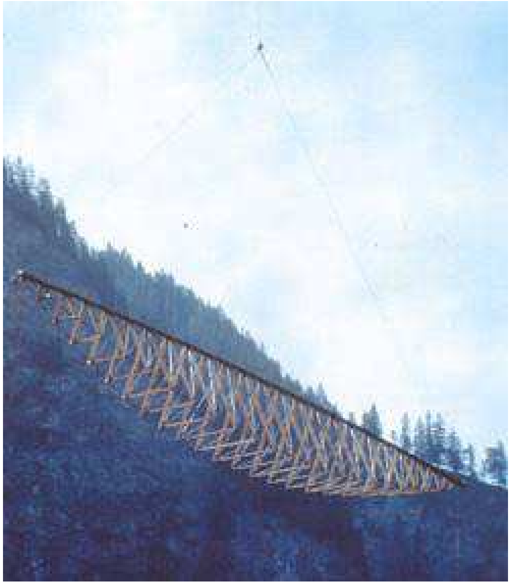


Figure 3.28 Transportation by helicopter of the whole substructure fully assembled (Conzett, 1997).

The elements of the superstructure (vertical struts, transverse beams, panels and carriageway) were then transported by a smaller helicopter and fixed progressively to the substructure until the bridge was practicable.

4 Analysis of one of the bridge projects taking part in the Competition at Chalmers University, Gothenburg

4.1 The Competition at Chalmers University of Technology

During February 2010 a Bridge Design Competition took place at Chalmers University of Technology in Gothenburg, Sweden. All Engineering and Architecture students registered at Chalmers, united in mixed groups with presence of members of both departments in each group, could really take part in it.
Promoter of the competition was the municipality of Borås, a city 60 km far from Gothenburg (see Figure 4.1), which requested a project for a glued laminated timber footbridge to connect in a direct way the new city culture center to the heart of the city, located on the opposite site of the river Viskan (see Figures 4.2 and 4.3).

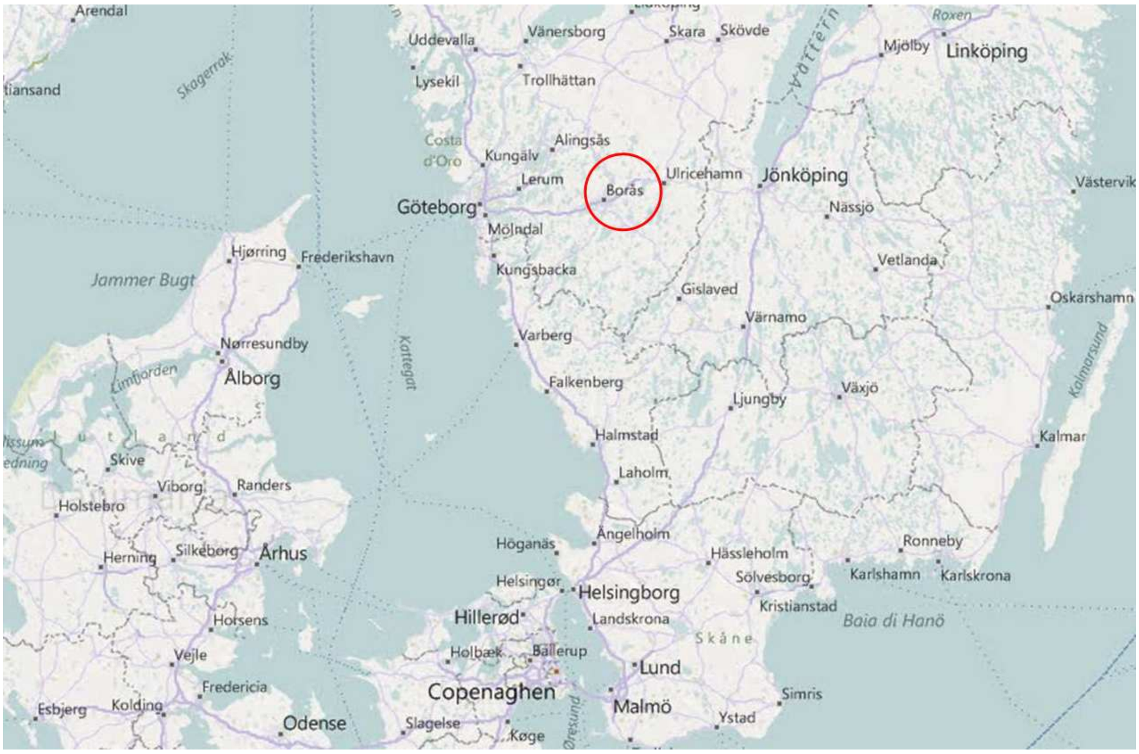


Figure 4.1 Map of Sweden, showing the city of Borås. One can easily see Gothenburg and Copenhagen (© Bing Maps).

Figure 4.2 Satellite orthophoto of the project location in Borås (© Google Maps).

Figure 4.3 Project location in Borås (© Google Maps).

Being evident the aim of the municipality to promote and create an interest by the citizenship to the new exposition center, the most possible complete proposal was requested: the requirements that were posed had to be satisfied not only, as obvious, from the structural point of view, but there was also an explicit request concerning the formal content of the work.

The bridge had to be finally an urban object, which beside the strictly function task to cross a natural obstacle could give an added value to the context in which it was put in, and to be able, in a certain way, to create an impulse to promotion. To give importance again to an area of the city presently rather out of reach it was necessary

to stimulate citizenship to visit it more often thanks to an object of interest that could draw the attention.
Another item to focus on was the economic issue: being the budget for the work non clearly defined but certainly small, the project solutions had to be as cheap as possible by taking into account even the cost of labor for the erection, in order to optimize resources and to get a fully satisfactory final result under all points of view.
Not less important was then to search a design skill with attention to the needs of durability so that the smallest repair and the longest possible life was granted to the structure, based on accurate design solutions.

The aim of the competition was therefore to give the accent on the entire design process of a timber bridge involving all aspects necessary to make the work able to satisfy the requirements of the client and in a peculiar way:

- clearly defined static behavior of the whole structure
- strength and stiffness of the system with regard to the safety
- serviceability criteria fulfillments
- detailing taking into account durability issues
- reflections over building issues and economical problems connected to the chosen structural solutions

The competition included a very "operational" design planning: thus it took place in two steps and in both the evaluation of the proposals presented by the various design groups was made on the basis of load trials of models, and beside that, it was checked whether and how far the offered solutions fulfilled the project requirements.
The six competing groups were asked to produce for the two further steps two 1:20 wood models of their own bridge, The restrictions posed by the judging commission regarding the peculiarities of the model were as follows:

- free span of the model of 100 cm (it means a real span of 20 m)
- the 100 cm span could not be gapped with unique elements but with interconnected pieces with a maximum length of 70 cm
- the width of the carriageway should not be larger than 15 cm
- cross sections of the available sticks to use: 5 x 5 mm, 5 x 7,5 mm and/or 5 x 10 mm
- maximum weight of 200 grams
- only wires and glue to perform the joints
- the bridge should be designed and behave like a simple supported beam, with a pin and a roller at the edges.

During the first step all models have been loaded with two point loads put at one third of the span each and then tested to failure, by adding increasingly sand in order to understand whether and to which extent the designed static system would actually

meet the requirements and to understand the weak points of the chosen structural solution.
After this moment every group had to decide whether to continue and develop the same model, and to improve it on the basis of what appeared during the collapse trial, or otherwise to basically change the shape and the static organization, also taking advantage from the experimental evidences and from the comments expressed by the judging committee, but always respecting the limitations imposed by the competition regulations.
In the second step all models were loaded again in the same way as the first time, but not up to failure, but with a load of 27 kg, applied all at once in order to verify the strength and the stiffness.
After this second load test the jury has decided the winner of the competition.

As already said the groups taking part in the competition were six and each of those produced two models, in some case similar one to another in other completely different, depending on the conclusions drained from the first load test.

In this Chapter the most interesting project among the other five competitor projects will be carried out in order to explain its original features, also using the software PointSketch to help illustrate the behaviour of the structure. The other five proposals are quickly presented just to give a general overview of the outcomes of the Competition. The analysis of the winner project will be carried out in Chapter 5.

4.2 Analysis of the "Reverse Arch Bridge"

4.2.1 First model

The first project to be presented during the first stage of the Competition was, as called by its own designers, the "Reverse Arch Bridge".
The model was extremely intelligent and original in its initial idea because it proposed an unconventional and, theoretically, just as simple and functional solution.
The starting point for the reflections of the design team was to observe one of the simplest and most common static schemes currently used to build a bridge with a span not excessively long, or the arch with a tie rod. In this static scheme the arch works in compression and, under the action of the loads suspended beneath it, it tries to lower and widen and transmitting then horizontal thrusts to the abutments and these forces will be higher the larger the radius of curvature of the arch itself is.
The role of the underlying tie rod is therefore to oppose and absorb these horizontal forces, working in tension accordingly, and ensuring the entire structure to transmit only vertical reactions to the supports, simplifying and making the abutments less stressed, with obvious economic, structural and design benefits.
The idea of the design team was to literally overturn the traditional structure trying to reverse the roles of the elements.
This first model, as shown in Figure 4.4, had two simple beams, with the role of tie rods, and the outer side of each two polygonal arches with the concave side down were placed, whose edges were not going to directly engage the longitudinal beams but they extended upwards.

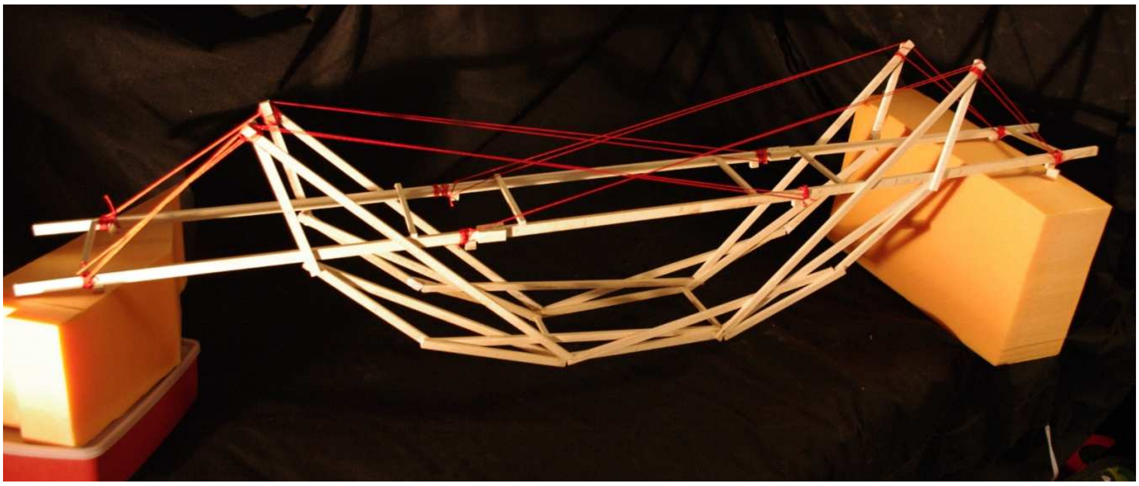


Figure 4.4 The first model of the “Reverse arch bridge”.

In order to manufacture the longitudinal beams, as the wooden slats were up to 70 cm long and the span to cover was 1 m, the designers have glued two pieces with different lengths along the longitudinal section, adding then two additional pieces both sides to make the connection stronger.

Concerning the arches instead they were composed of two different layers: the main part consisted of seven straight line segments with the edges overlapping, forming the curved profile of the arch, and then other six ones have been added to increase the stiffness and stabilize the system.

The intentions of the designers were the structure to behave as follows the point loads applied directly to the deck should have made the longitudinal beams work in compression, which, deflecting, would have transferred some stresses to the underlying arches in order to make them work in tension. The tense arches due to their shape, would tend to close, sending then back to the beams horizontal thrusts directed towards the center of the bridge and subjecting them to compressive stresses that would have added to the previous state of compression and bending.

Moreover from the top of each arch have been stretched wires which were linked to the final part of the deck on one side and to the point of application of the load on the other side in order to be able to redistribute the stresses within the various structural components and at the same time to prevent the arches to close too much, preventing the edges from approaching in a dangerous manner.

If one observes Figure 4.5 it is possible to see how the design team has not been able to completely and effectively translate their ideas in the real structural model: it is immediately obvious that the edges of the beams near the supports are the least stressed and a similar story applies to the bottom of the arch which is completely unstressed. Even the cables, which had to collaborate actively in the transmission of stresses "work in compression”, which it means that they are completely discharged and therefore completely useless. The compressive stresses are concentrated in the central part of the structure between the two points of application of the loads: in that area the bracing system is virtually non-existing and therefore the longitudinal beams are exposed to a high risk of buckling.

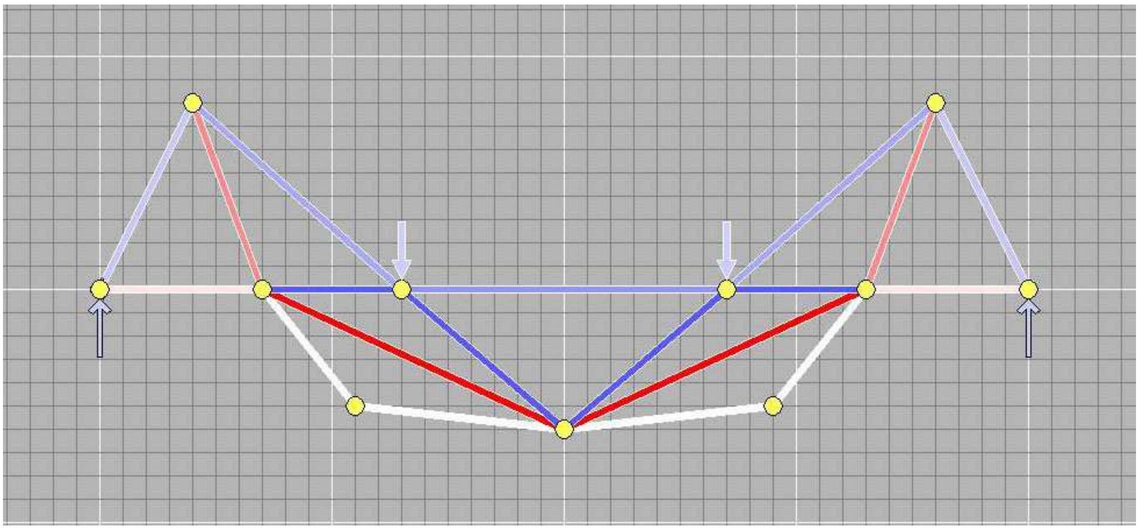


Figure 4.5 The static behaviour of the first model realized in PointSketch: tensile stresses are shown in red while the compressive ones are in blue.

Observing the structure of the bridge before the load test, despite the originality of the solution, by a purely qualitative analysis it first appeared that the cross-sections of the longitudinal beams were insufficient to perform the role that had been entrusted to them and it also appeared that polygonal arches were manufactured approximately and they did not seem stiff enough. Not to mention that even the general stiffness of the whole structure seemed to be very low, especially in the transverse direction.
The behaviour of the model during the load test was not very good in the sense that the two structural systems could not interact in transferring the loads: in fact the connections between the longitudinal beams and the two arches were too small (in practice arches were glued on the beams in two points) and they did not allow the two static systems to work together. Ultimately the arches were the least stressed and almost the entire load was carried by the longitudinal beams, working in simple bending.
The bearing structure of the bridge was reduced to be made up then by two simple supported beams loaded by two point loads and, as mentioned above, their cross section was too small: in fact the maximum load before the failure was one of the lowest.
In Figure 4.6 it is possible to see how the longitudinal beams, just before failure, are extremely deformed while the arches are nearly discharged (just see the wire hanging on the left).

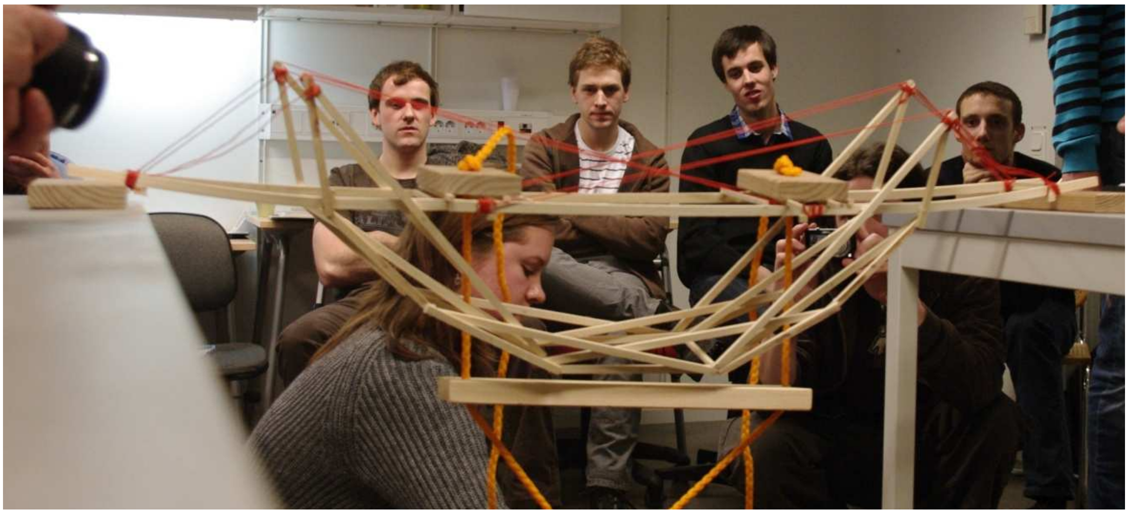


Figure 4.6 The model just before the failure: it is possible to see the deformed longitudinal beams due to excessive bending, especially near the edges.

Not to mention that the bracing system, always required for compressed elements, was insufficient and structurally inefficient (only six sticks arranged transversally between the longitudinal beams and two between the arches) and in fact the break has occurred due to buckling.

4.2.2 Second model

The second model presented in the final phase of the Competition consisted, as shown in Figure 4.7, in an evolution of the previous project but with significant improvements, and according to the Author's opinion, was the most valid solution along with the winning project.

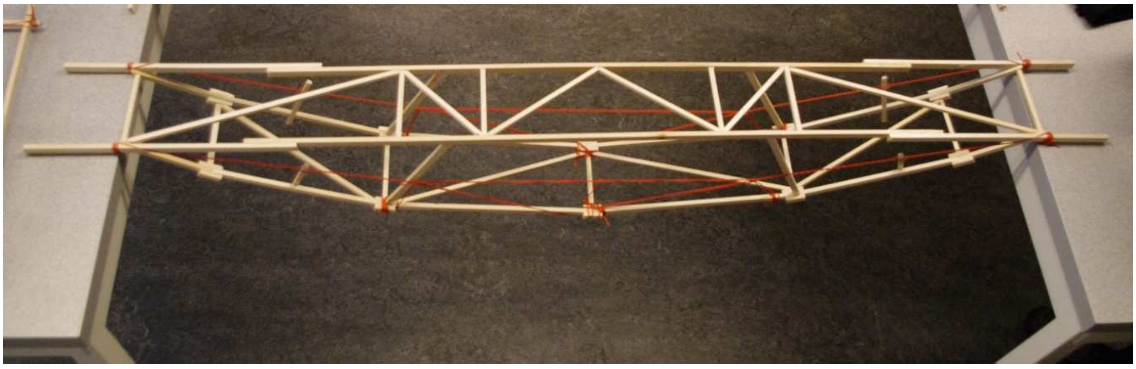


Figure 4.7 The second model presented during the final stage of the Competition.

Compared to the previous one this second model presented new features (see Figure 4.55: first of all the radius of curvature of the arches has been increased making them lower and the edges have been connected near to the supports of the bridge; also the connection between the two arches and the longitudinal beams has been made thus effective allowing an efficient transfer of stresses and a real collaboration between the structural elements: the edges of the arches have superimposed on the internal side of

the girders and then a transversal element has been put so as to act at the same time both as bracing system and as locking joint.
The profile of the arches was still composed of straight line segments (six) but the connection between them has been improved by gluing the various pieces on the "heads" and putting then two more pieces to the sides to improve the structural solidarity.
The longitudinal beams, composed of three pieces, have been made more efficient and connections were performed by overlapping the edges and gluing them.
To stiffen the structure two bracing systems have been performed, one between the girders and one between the two arches, together both diagonal and vertical elements, thickened in the proximity of the point of application of the loads.
The static scheme seemed to be generally remained the same: there were still the arches and the beams working in compression but with the new model the designers have managed to carry on a more rational usage of material going to focus on weaknesses identified by load test on the first model, but above all they have implemented a completely new static behaviour.
As it is possible to see in Figure 4.8 the main novelty of this project was the introduction of a pre-tensioned cable system that has completely revolutionized the structural layout of the bridge: in fact the cables subjected to a prior state of tension (that is before the application of the loads) cause compressive stresses both in the longitudinal beams and in the lower arch creating a situation of internal coactions that provides stability to the system.

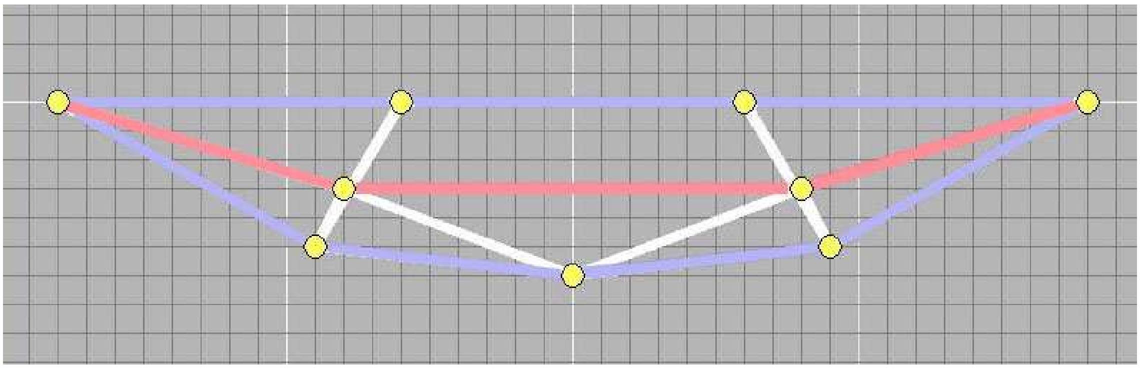


Figure 4.8 The static behaviour of the second model without any load realized in PointSketch: tensile stresses are shown in red while the compressive ones are in blue.

The great insight and the very interesting feature of this structure is that it presents a reverse arch not working in tension as many would say at first glance, but rather subjected to compression. Once the loads are then applied (see Figure 4.9), the new stress situation by external loads produces makes it happen that compressive stresses in the longitudinal beams increase while in the arches the tension induced by the point loads gets balance with the preexistent compressive stresses and then in the end both the arches are less loaded than before.

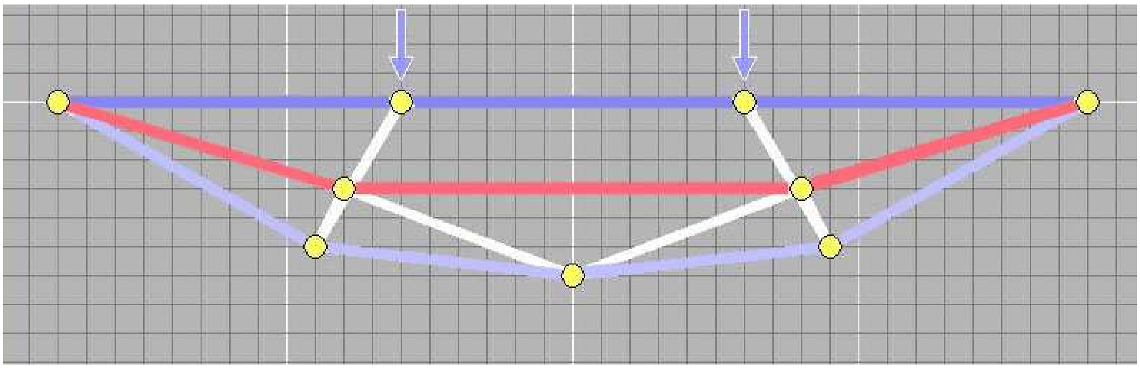


Figure 4.9 The static behaviour of the second model after load application in PointSketch: tensile stresses are shown in red while the compressive ones are in blue.

During the load test with the instant application of 27 kgs model behaved very well getting virtually no deformations and it proved thus really effective , as can be seen in Figure 4.10.

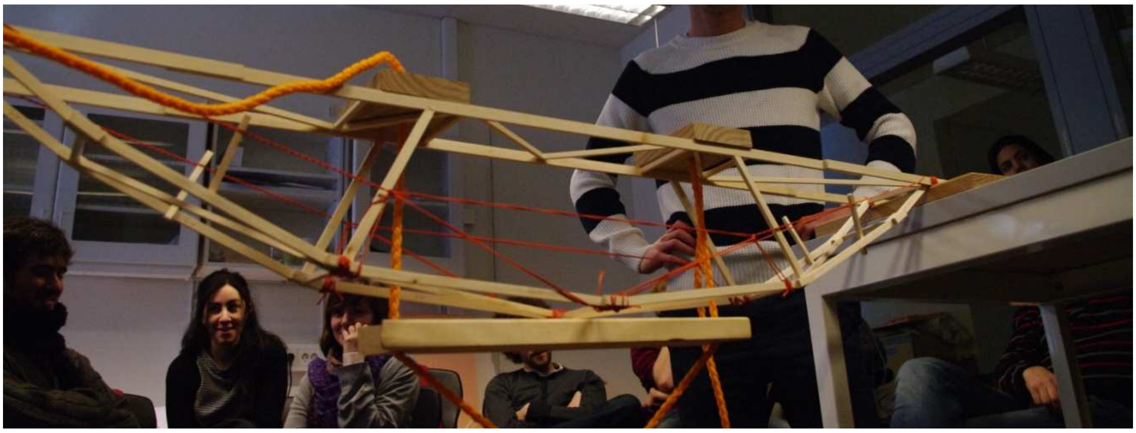


Figure 4.10 The model during the load test: it is possible to see that the deformations are almost nil.

The only criticism that can be moved to the project is the following one: the structure has been optimized for the loading conditions adopted during the competition (loads applied at one third each) but one wonders if it would be equally effective under different load conditions.

4.3 Second team proposals

4.3.1 First model

The proposal of the second design team participating in the Competition provided a solution structurally interesting, although the aesthetic appeal was quite poor (see Figure 4.11).

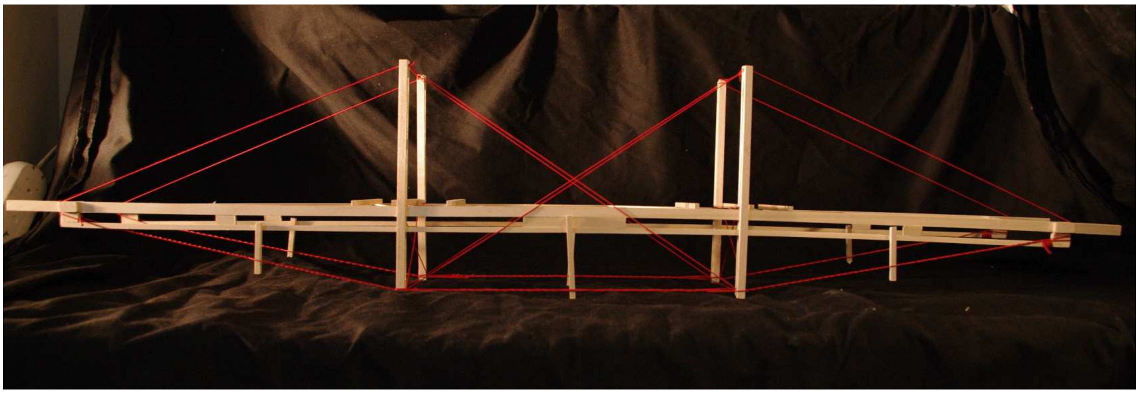


Figure 4.11 The model presented by the second design team during the first phase of the Competition.

The idea behind this concept is borrowed from the prestressed concrete technology: in fact prestressing with artificial compression stresses the concrete elements an improvement in resistance and a decrease in deformation due to application of external loads are obtained.
Similarly in this project the designers wanted to create a state of compression in the longitudinal beams in order to substantially improve the strength and at the same time to have small deflections. To do so a lower wire has been pre-tense so that the elements of the bearing structure were longitudinally compressed before the applying of the loads. The other two cables would be used to redistribute the efforts induced by point loads and to send them in part to the supports at the edges.
To increase the transversal section of the beams the designers have adopted a system similar to that of Vierendeel beam, resulting in a greater height without increasing excessively the weight (also considering the limitations imposed by the Committee).
The bracing system was pretty poor though it was deepened near the point of application of loads.
During load test, despite the prestressed state, it was clearly noted that the deflection was too much even with low loads (see Figure 4.12): this is because the width of the beam appeared inadequate compared to the height and because the junction between the upper and lower chords was performed in a not adequate manner, thus creating a point of weakness.

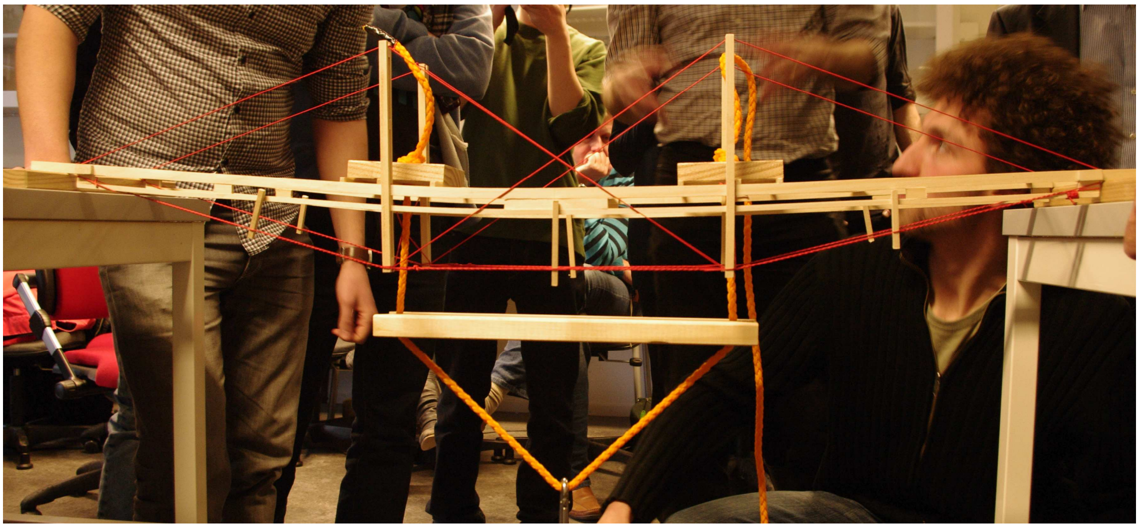


Figure 4.12 The model during the load test: one can easily see the deflection of the longitudinal beams.

In the end, the beams were too slender and in fact the collapse of the structure occurred for torsional buckling that caused the detachment between upper and lower chords of one of the girders near the support, as shown in Figure 4.13.

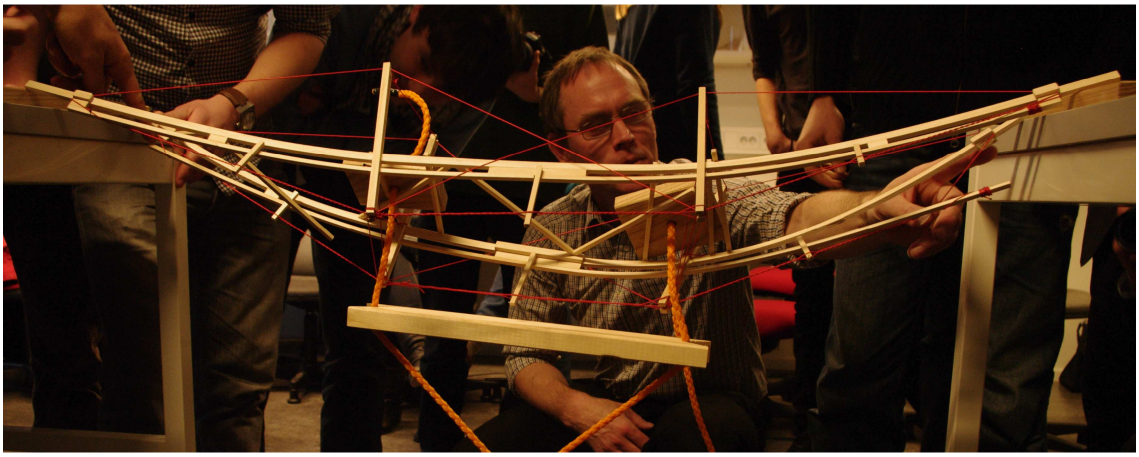


Figure 4.13 The failure of the structure due to torsional buckling.

4.3.2 Second model

The second model (see Figure 4.14) has presented evidence of both continuity and innovation compared to the previous solution: in fact, designers, noting the weakness of the longitudinal beams due to an imprecise manufacturing, have improved them and made them more stiff thanks to a more careful connection between the two chords and especially stronger close to the supports.

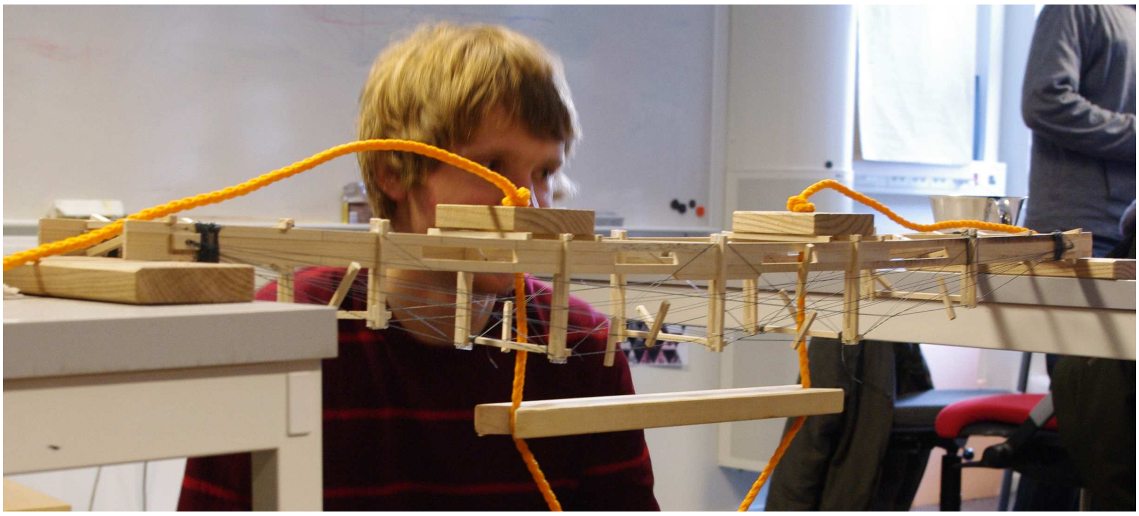


Figure 4.14 The model presented at the final stage of the Competition.

The new element in this model has been the use of the static model of the Fink truss, where the load is distributed and directed to the supports through a system of ropes (in this case in metal although against the rules) where each rope, stretched between the abutments, passes through each stud.
During the load test structure showed a good response despite the obvious and inevitable deformation due to the static scheme adopted.

4.4 Third team proposals

4.4.1 First model

The first model presented by the third design team provided for a structural solution usually employed to build wooden roofing: it was ultimately a truss.
The model contained neither particular architectural contents nor special considerations concerning the static behavior: the inclined elements are compressed while the tie rod is tense. The structure under load has borne, like the winning project, about 27 kgs without relevant problems, also because it has been made with special care (see Figure 4.15).

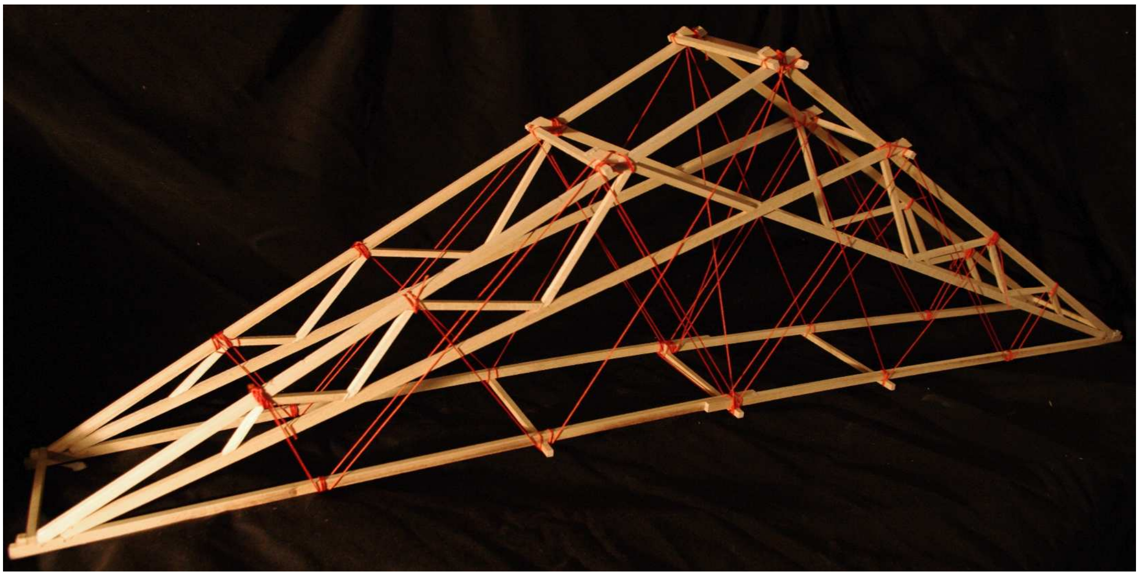


Figure 4.15 The first model developed by the third design team.

4.4.2 Second model

The second proposal of the team (Figure 4.16), considering the excellent response of the first model during loading, remained virtually unchanged except for some minor detail irrelevant to the overall behaviour of the structure.

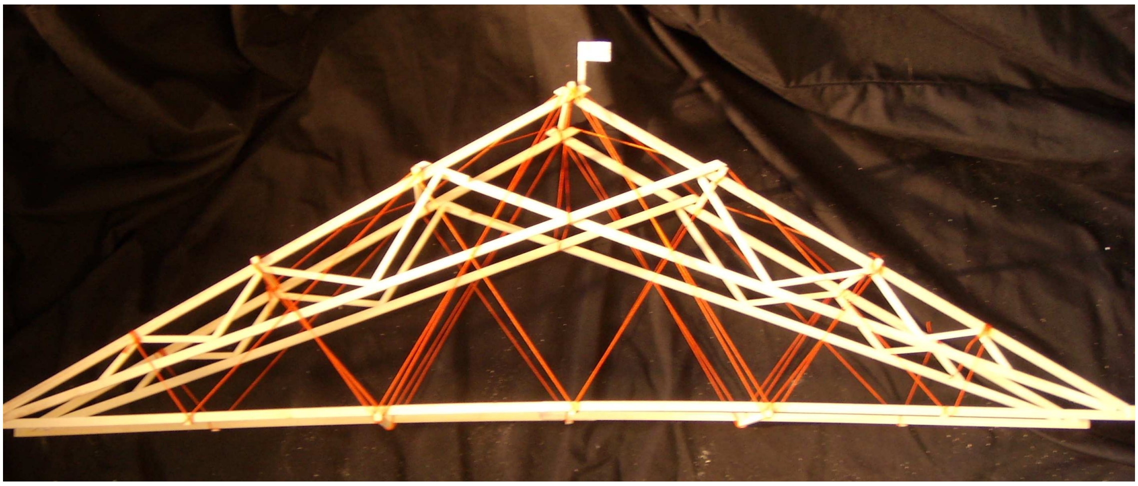


Figure 4.16 The second model presented at the final stage, identical to the first one.

4.5 Fourth team proposals

4.5.1 First model

The first solution proposed by the third design team has become noted for its originality: it was an explicit reference to Japanese architecture. The intent of the group was to create a structure whose nodes were made to joint (Figure 4.17): the result has been a hybrid between an arch and two cantilevering beams at the ends whose global behaviour seemed immediately very precarious.
The transversal section of the arch was in fact too thin and the arch itself was not properly braced, the cable tense along on the bottom to counteract the horizontal thrusts did not seem an appropriate solution and also the supports of the two abutments also seemed very small.

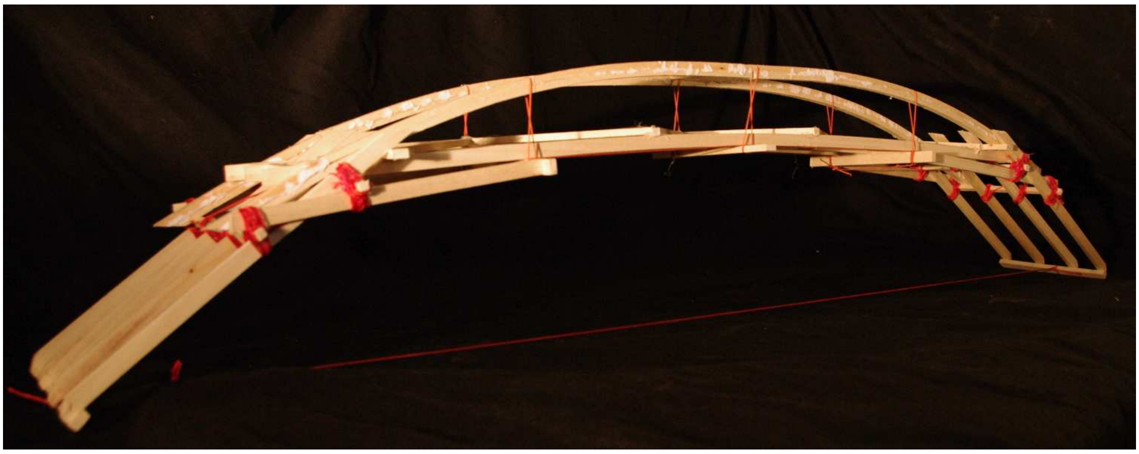


Figure 4.17 The first solution proposed by the design team.

In fact, during the load test the problems related to instability of compressed elements raised: the arch was not stiff enough and also the transversal elements necessary to maintain the rope tense were too thin to be able to withstand the high stresses caused by the application of the external loads (Figure 4.18).

Figure 4.18 The first model just before the failure: the instability problems are clear.

4.5.2 Second model

The second proposal, as shown in Figure 4.19, has simplified the static scheme while maintaining the same "philosophy" of Japanese mortising: the team focused on the arch eliminating the cantilevering elements: the curve was not made by one single piece but by five separate elements that created a splitted line.

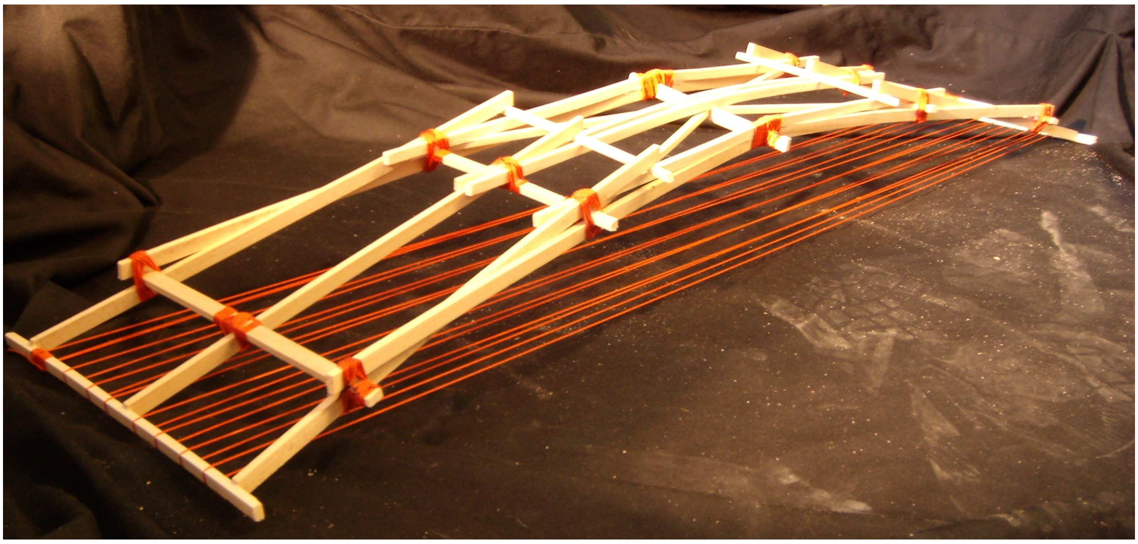


Figure 4.19 The second model presented at the final stage of the Competition.

In plan the bridge was tapered toward the center to offer a higher torsional stiffness, and, though modified, the solution of tense cables to counteract the horizontal thrust of the arch remained.

Also this solution was not structurally satisfactory: in fact during the load test it has been the only model that has not held the 27 kgs sand bucket (Figure 4.20) because of the persistent slenderness of the transversal elements used to keep the wires tense, and because of the weakness and not moment-resistant of the middle joint between the two halves of the bridge.

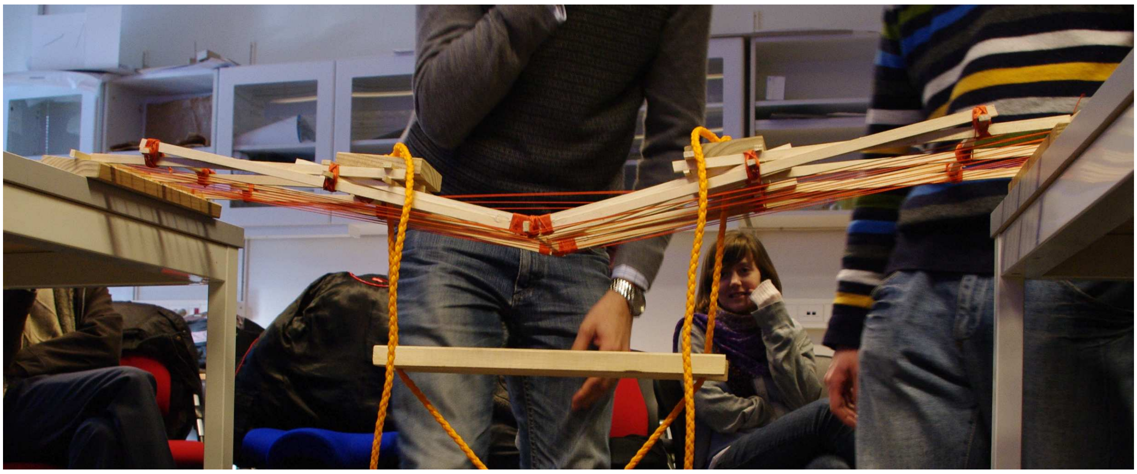


Figure 4.20 The failure of the second model also: the joint in the middle was not moment-resistant.

4.6 Fifth team proposal

4.6.1 First model

The model presented by the fifth design team at the first stage was based on a simple attempt to make the different materials work under different stress states: the intent of the designers was to give the wire the shape of the bending moment (and for that pyramid-shaped elements have been prepared) so that it worked in simple tension and so that the longitudinal girders were only compressed (see Figure 4.21).

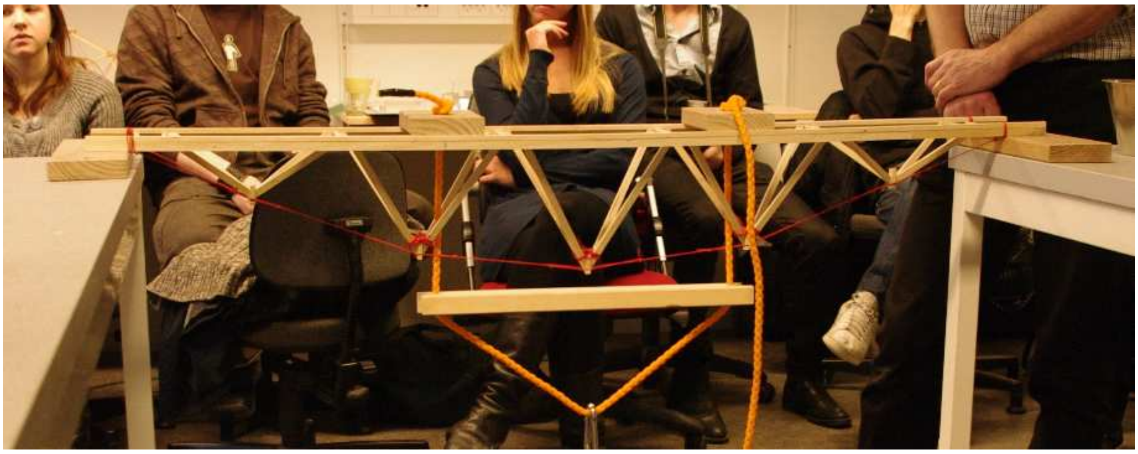


Figure 4.21 The first model presented by the fifth design team.

Anyway, during the load test, it was proved that the wire was practically discharged and the whole stress went to concentrate in the girders, making the entire structure work as a simply supported beam: in fact, the failure occurred in the lower tense part of one of the beams near the support due to the lack of adherence between the two sticks that formed the composite member, as can be seen in Figure 4.22.

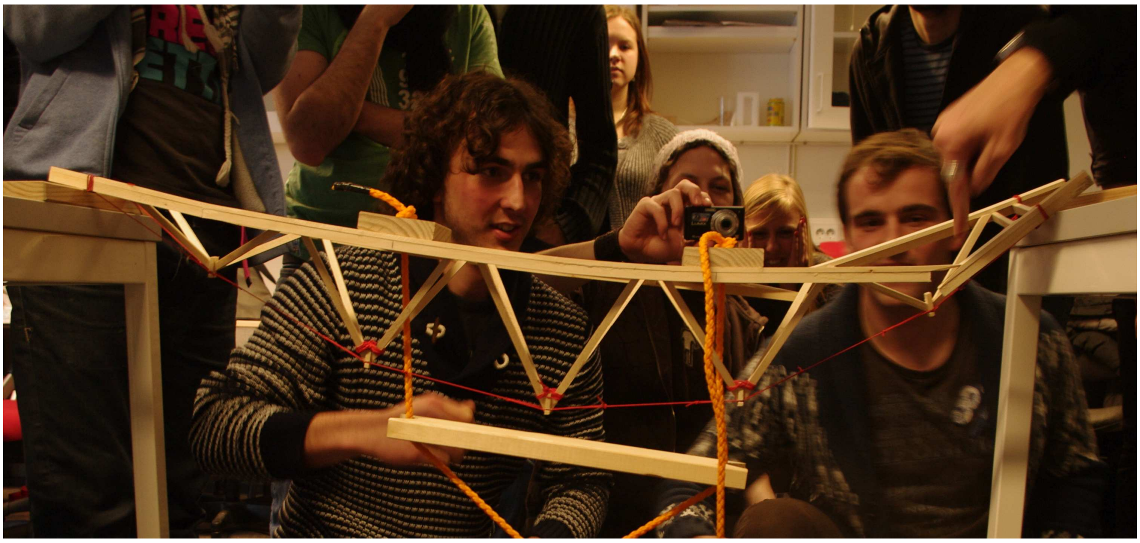


Figure 4.22 The failure of the structure.

4.6.2 Second model

The second proposal made by the team has borrowed from the second team the idea of the pre-tensioning of the wire to establish a state of pre-compression in the longitudinal beams, beside the principle of the Vierendeel beam to increase the cross section without adding too much weight. The pyramid elements in the lower part have remained to give the wires a curved shape. Again, however, designers were not able to make the two systems effectively interact and the result was that the structure still acted like a simply supported beam, with a contribution close to zero by the wire (Figure 4.23).

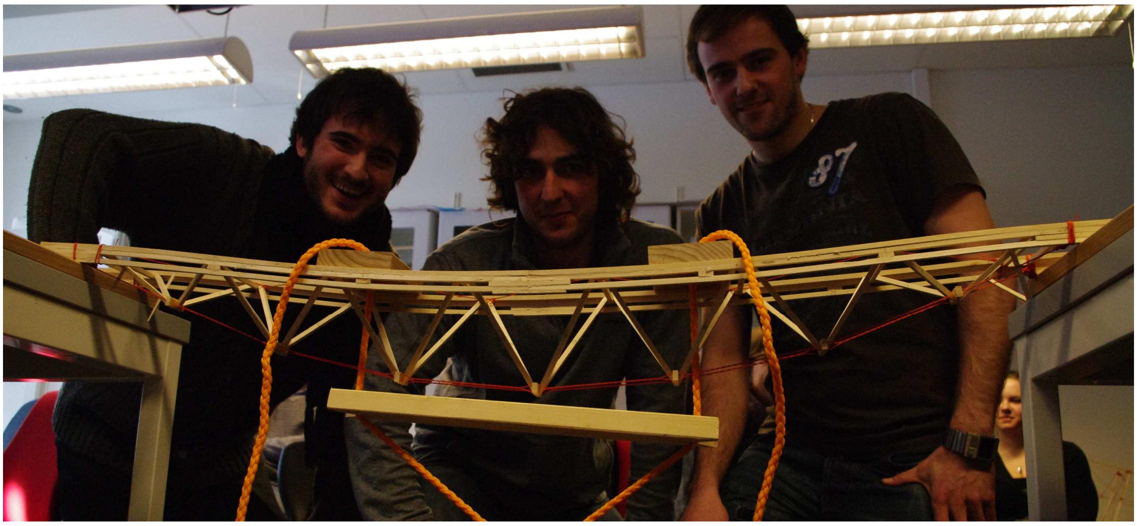


Figure 4.22 The deflection of the second model under the action of the two point loads.

5 Analysis and developments of the winner project (Fish Belly Bridge)

5.1 Analysis of the design process of the models participating in the Competition

In Chapter 4 the projects which took part in the competition have been analyzed in order to show how the different groups of competitors interpreted and translated into physical models the limitations imposed by the judging commission.
After having commented all the various options it’s time to discuss the winner project of the competition and this analysis will be carried on in this chapter.
The attention in fact will be drawn now to this project, hereafter called Fish Belly Bridge due to its distinguishing feature consisting in the shape of the bearing structure, and this thesis work will go into detail of all the structural and design solutions necessary to be able to actually implement a simple idea initially sketched on a sheet of paper and passed through a first refinement by the building and implementation of the two scale models.

In the next paragraph the whole process of design of the bridge, from the concept designed and discussed among the three components of the design group until the prizegiving of the model during the second step of the competition, will be described and the evolution of the concept accurately reported to demonstrate how the intrinsic merits of a project proposal allows that the design choices taken since the beginning remain unchanged, maintaining their strength, even after a series of inevitable downsizing and comparisons with reality of construction.

The design group consisted of three people: the Author, Georgi Nedkov Nedev, a Bulgarian Civil Engineering student, and Simon Johansson, a Swedish Architectural Engineering student (see Figure 5.1).

Figure 5.1 The logo used during the Competition by the design team.

For each of them this was the first experience of joint working in such a mixed group.
After a briefing, discussing together, it showed that for all group members the main themes to focus on to achieve a competitive project were the structural simplicity combined with a captivating shape capable to catalyze the attention of the observer.

The initial ideas had immediately placed the emphasis on structural behaviour, by essentially trying to find a simple structural solution that allowed to obtain an aesthetic effect of impact.
The first idea that had begun to be developed was a structure consisting of three related trusses that form a triangular cross section, as shown in Figure 5.2. The concept offered an interesting solution not to mention that the behaviour of the whole structure was extremely linear.
The major obstacles to the further development of this idea appeared to be first the no little difficulty in performing the connections between the three beams at the edges, and complexity of the structure seemed particularly excessive for a span of only 20 m The high number of items would indeed have first an impact on the cost and secondly on the weight of the bridge (very strict limitation of the competition): it would far exceed the 200 grams placed as limit.

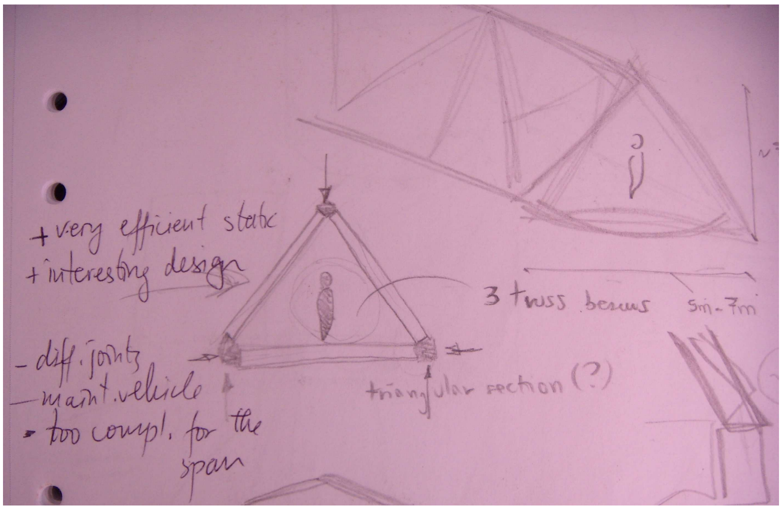


Figure 5.2 The first idea for the bridge sketched on a paper.

The next step was to think of an option diametrically opposite to that of the truss, which was an extremely essential solution that did not include a large number of pieces, but unlike used a primary system as static scheme.
The second proposal (see Figure 5.3) provided a structure that used a glued laminated timber arch: simple, lightweight, cheap and effective. To comply with the rules of the competition of course the arch had to be made in two parts in order to cover the span of the river.

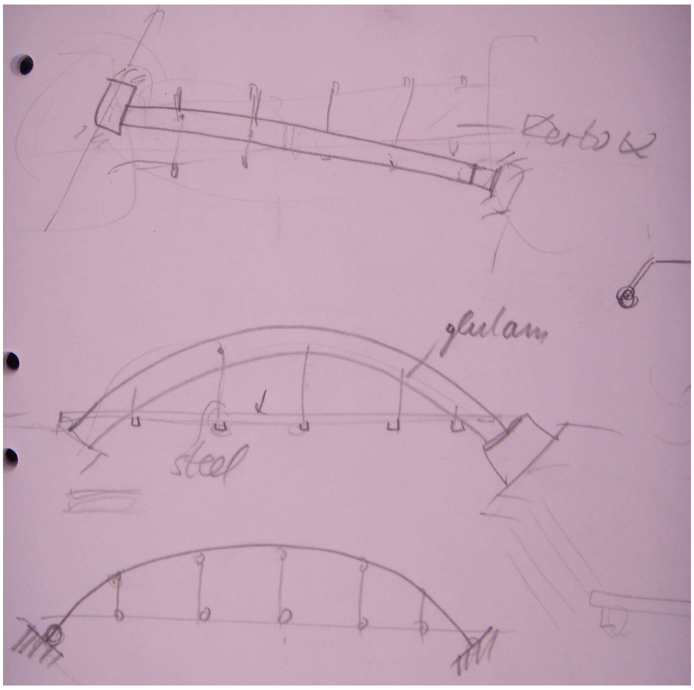


Figure 5.3 The second proposal for the timber bridge.

Various options were explored with the arch placed in different positions and with different conformations of the same. Most of the solutions were, however, although reliable and efficient, unfortunately very commonplace for the excessive simplicity.
One idea that seemed to be having some intrinsic quality to be developed was that of a deck suspended by steel cables to a single arch working in compression and set at an angle from the axis of the carriageway. But the doubts that had arisen within the group concerned the stability of the entire structure out of plan and in particular the stability of the arch, since two clamps at the base did not seem sufficient (steel as building material would be ideal) and the positioning of additional elements as bracing system made the structure too complex and aesthetically ungainly.

After the initial ideas and reflections on the theme we had realized that the best way to get a project that met the criteria was to use in an intelligent way the form of the arch, perhaps combining more than one, in order to get a concept whose static behaviour was clearly understandable but did not pose too many problems in its realization.
The inspiration for the shape of the bridge was found finally in a context that the bridges had nothing to do: looking at the system of wooden beams supporting the roof in an industrial building in St. Martino Buon Albergo (Italy) has been noticed how they performed the function similar to a bridge as it is possible to see in Figure 5.4.

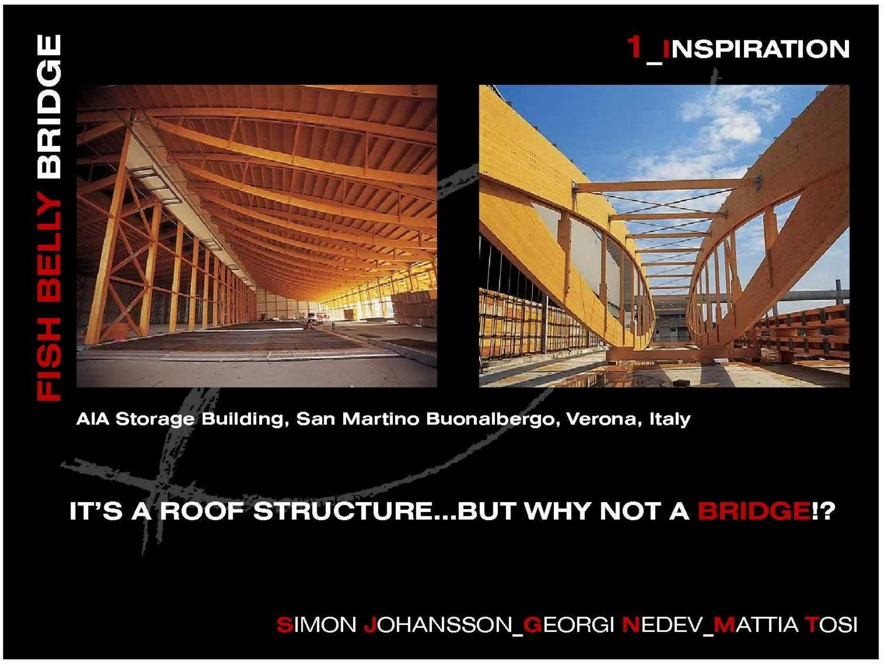


Figure 5.4 *The roof structure of an industrial building in Verona realized with lenticular glulam timber beams.*

Indeed, they crossed the span between two columns and behaved as simple support beams: ultimately the same boundary conditions that must be satisfied for the competition. It was therefore decided to extrapolate the concept and adapt it to the current situation (see Figure 5.5)

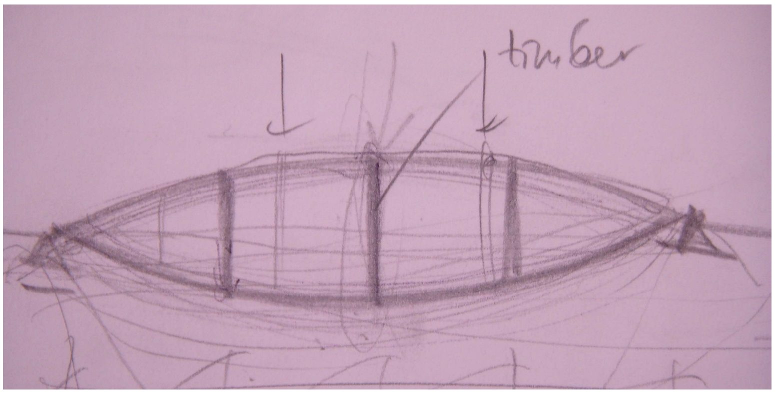


Figure 5.5 *The idea to use the lenticular beam as bearing structure for the footbridge.*

The roof beams were lenticular, and each of them consisted of two arches with the same curvature connected at the edges and with vertical struts which maintained the distance between the two halves.
The discussions were aimed now on how to use these lenticular beams in order to realize a footbridge.
The first option was to place the deck exactly in the middle between the two arches: this proposal, however, meant to resolve the not simple issue of supporting the carriageway since performing a simple supported beam made unnecessary the use of

lenticular beams; not to mention that the problem of the parapet had to be solved: making the rise of the arches too low so that they might act as handrails meant diminished them and instead making them too high meant the introduction of transversal elements to prevent instability out of plan that would have hampered not just the circulation of pedestrians.
The hypothesis with the carriageway on the underside of the beams was rejected since the persistence of the problem of bracing system, necessary to prevent the upper compressed arches from buckling; also because it seemed inappropriate to make excessively high the lenticular beams to achieve a result not too disproportionate in relation to the context where the footbridge had to be put in.

The final solution was to place the deck on top of the lenticular beams. In this way the static behaviour of the whole structure was clearly defined and structural clarity goes perfectly with the formal linearity and efficiency, without accessories nor redundant elements (see Figure 5.6).

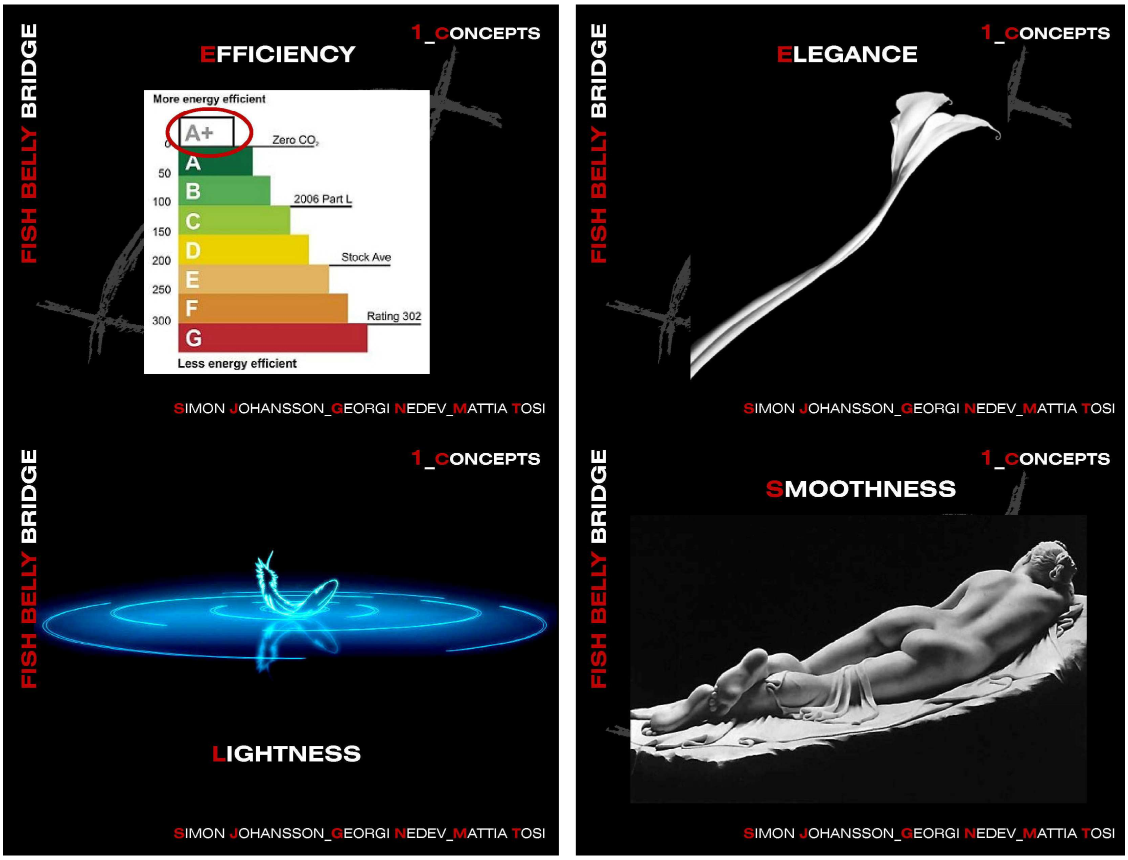


Figure 5.6 The concepts that the Fish Belly Bridge carries with it.

The static behaviour of the model is clear (Figure 5.7): the upper arch of the lenticular beam, that is directly loaded by the weight of the deck and of the crowd transiting, works in compression, so do the struts that transfer the stresses to the lower arch that is so tense.

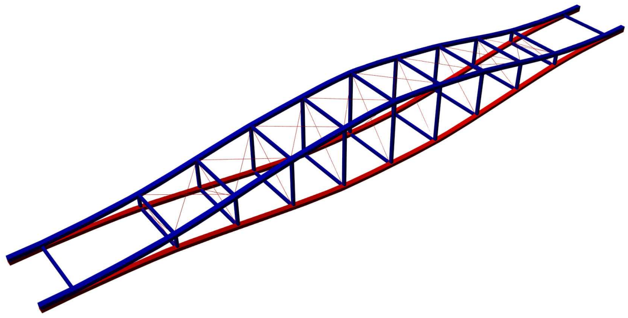


Figure 5.7 Structural behaviour of the scale model of the bridge: the red elements are working in tension and the blue ones in compression.

Practically the lenticular beam behaves exactly like an arch with a tie rod, where the role of the rod is played in this case by the lower arch that absorbs horizontal thrusts provided by the upper loaded arch.

Determined the shape and the structural conformation of the concept the group has passed to the operational phase that was the construction of the physical scale model.
The immediate problem obviously concerned the implementation of two curved beams and how to perform the connection between them.
As a first hypothesis it has been thought to build the arch through the model of isostatic three hinges arch: the intention was to connect the two parts of each arch in the middle with a nail and do the same at the edges of the two halves in order to get an isostatic ring and to make it stiff enough by inserting vertical struts. Unfortunately, the correctness of theoretical assumptions clashed with the harsh reality of the facts as nails provided freedom of rotate to the joints but unfortunately they could not guarantee a perfect solidarity between the parties: the tail end of the nail in fact, despite having been bent, not locked up enough the two overlapping edges. Ultimately the resulting structure was not stiff at all and it does not keep its shape.
It was decided at this point to operate differently: in order to realize the upper arch two wooden pieces with a cross section of 5x10 mm (the largest ones) were used, they were glued on the edges, with one another, to obtain a perfect material continuity, and to make the union stiffer other two pieces of wood were glued at the sides. The joint thus created gave the possibility of having a unique piece 140 cm long that could be bent as needed. The same was done for the lower arch.
Subsequently, the central vertical element was glued between the two halves to have a reference for the distance.
Once the glue has dried it has proceeded in bending the two pieces at the edges thus creating a sort of pre-bending process that established prior stresses in the bars.
To maintain the correct curvature of the two arches the ends were glued and secured with clamps and even more connected with wires. After gluing the two halves of each arch other struts with height decreasing from center to ends were included to create the desired slope.
The struts were seven for each beam, the ones placed near the points of application of the loads had the biggest cross-section, while all others had the smallest (5x5 mm). To make sure that under the action of the loads vertical elements would not slip away due to excessive compression, small wooden wedges were glued near the ends of the elements.

Once the two beams were constructed, they were connected together with horizontal elements simply glued (with a cross section 5x5 mm) placed near the ends of the struts, thus creating a box section with variable dimensions depending on the position, and two others near the edges the connection between the two halves of the beam. The upper transversals had notched edges in order to perform a better connection with the beams.
To stiffen the whole structure a bracing system consisting of wires stretched between the beams and also among the transversals was finally implemented.
The weight of the finished model (see Figure 5.8) was 201 grams.

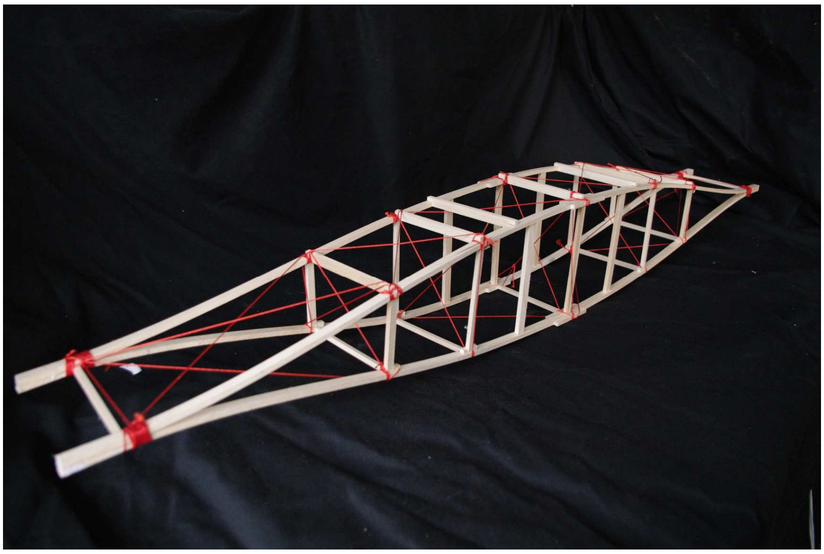


Figure 5.8 The completed model just before the load test.

During the load test the model supported a load of more than 27 kg and showed, along with another, to be the strongest. The failure occurred due to excessive deflection of the edge of one compressed arch (Figure 5.9). In fact the distance between the connection of the two arches and the nearest strut was too great and due to the high compression stresses induced in the upper arches by the applied loads above the auction buckled too much and the whole structure collapsed.

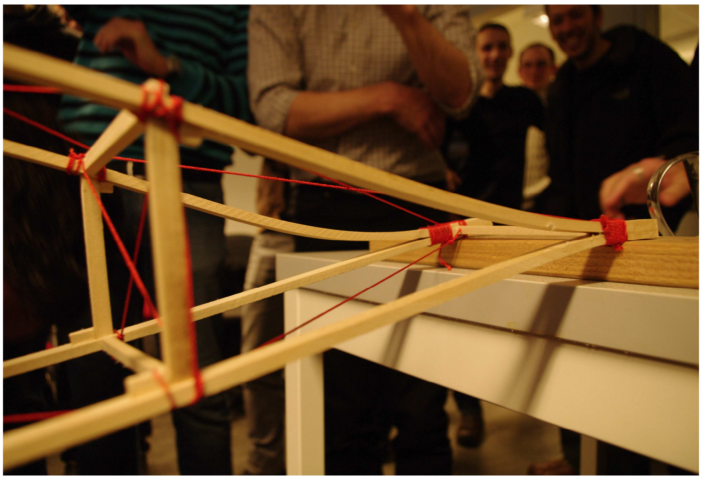


Figure 5.9 The particular of the excessive deflection of one of the compressed chords of the lenticular beams .

It must be said that the phenomenon of buckling has been compounded also by a state of internal pre-existing stress due to the process of pre-bending implemented during the construction of the model.

After observing the excellent performance of the model (which few would have bet on), despite a design objectively a bit empirical and a fragile appearance, for the second stage of the competition it was decided to develop it further, trying to optimize the use of material and especially the arrangement of structural elements in order to try to fix the deficiencies identified during the load test.
To build the second model, the means and the techniques adopted were roughly the same used in the first. Inferring from the observation of failure modes it was clear that the major problems for lenticular beams were related to buckling (a phenomenon that occurs only for elements subjected to compression) and to the design of the details of the upper arch, it was decided to reduce the cross section of the lower arch as absolutely immune from such problems, and succeeding in this way in reducing the weight significantly.
The number of struts was increased bringing them from seven to nine (one every 10 cm) and thus reducing the free length of inflection that proved the weakness point of the previous model. The sections of the struts were then varied according to the position: 5x7.5 mm for the three central ones and 5x5 mm for all the others. In addition, wedges for each have been added in advance to prevent them from slipping (though observing the behaviour of the model under load conditions no such problems associated with struts had experienced).
Regarding the bracing system made with wires it has been implemented, performing saltires in the upper part and among all cross sections.
The weight of the second finished model was 188 grams. Changing the cross section of the lower arches produced thus a significant weight reduction (12 grams approximately).

At the second stage of the competition another load test took place but this time the load of 27 kg was applied all at once: the behaviour of the model showed a significant general improvement over the previous one but still remained clearly visible (Figure 5.10), even if substantially reduced, the inflection of the compressed part of the lenticular beams next to the supports, despite the precautions taken; also because of the way of realization of the connection through pre-bending that previously stressed the bars.

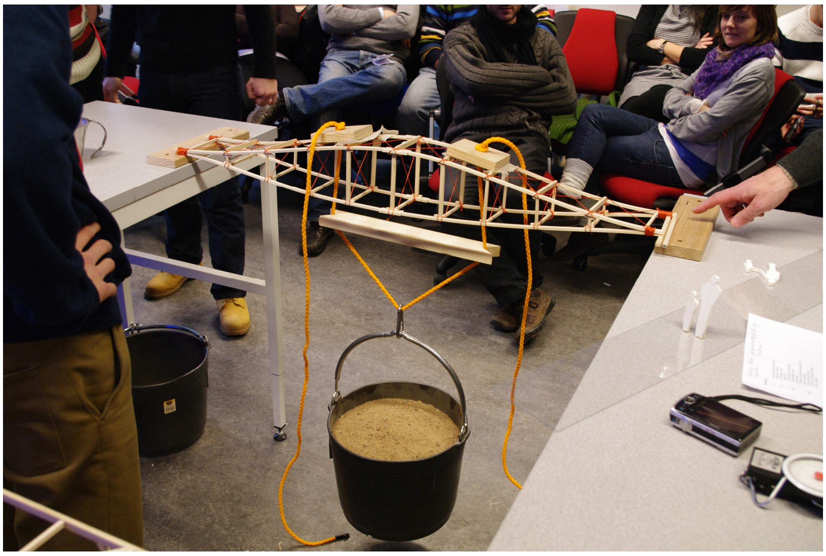


Figure 5.10 The second model during the load test: it is possible to see the deflection of the beams near the support on the right.

5.2 Materials composing the bridge

Until a few years ago the exploitation of forest resources has not been particularly encoded and forest product producing countries along with forest product industries have made extensive use of old growth timber, the best concerning the fiber quality, in order to obtain large quantities of solid sawn timber.

The current trend indicates a growing world population which is associated with a disproportionate increase in demand for structural wood products. Despite attempts to regulate global use and sustainable management of forest resources, the difference between demand and availability of raw materials continues to be unprofitable: existing resources are insufficient to satisfy the needs.

Not to mention the realization by the society that finally understood the importance of forests globally as part of an ecosystem critical to the natural balance of the planet. Forests in fact as well as being a natural reserve of carbon dioxide, produce oxygen, provide food for many animals and greatly influence global climate change.

It is a well known fact that the old growth timber is the best for structural use, having regard to the optimal shape of the fibers but it is also true, as already pointed out, that its availability is extremely limited. Thus the only way of using the forest resources in a sustainable way is to improve the use of wood fibers, avoiding waste and maximizing usage of wood in all its aspects through the development of new technologies. From this perspective it becomes important to make the most of lower quality woods, the exploitation of wood species hitherto neglected and the use of scraps (Thelandersson, 2003).

Technological innovation plays a key role in developing new products that be able to ensure a sustainable use of the resources at our disposal: research conducted in industry in recent years has led to the introduction of the so-called Engineered Wood Products (EWP), which can be used as a building material instead of solid wood. They are mostly made from waste of wood processing and thus they optimize the use of the material: wood in fact is reduced into pieces of smaller size (as needed) and then reassembled with the aid of adhesives (Thelandersson, 2003).

The EWPs consist of veneers, strands, flakes or planks that are suitably arranged and assembled as needed and then bonded with structural adhesives, and using pressure and heat, to obtain panels or structural elements of the desired shape.
An important advantage of these artificial structural materials, beside a more rational exploitation of natural resources, that is, precisely due to the production process, their behaviour is predictable in a much more accurate way than can be done for solid wood. Moreover, a large number of macroscopic defects, such as knots or other defects that may cause a reduction of strength, are eliminated or dispersed within the new material: in this way it gives a more uniform and reliable material where uniformity leads to a more efficient utilization of the fiber resources.
It is now a fact that, through experimentation and research, the structural composite lumber products have achieved high levels of reliability and they are beginning to replace concrete and steel in many applications and in the next future, with the reduced availability of solid sawn timber due to environmental limitations, EWPs will become increasingly crucial and important as building material (Thelandersson, 2003).

5.2.1 Glued laminated timber beams

The term glued laminated timber, abbreviated glulam, means a structural timber product composed of overlapping wooden planks, usually 33 mm or 40 thick (in Sweden usually 45 mm), which are glued together along their cross sections ("heads") by means of adhesive to form a section that behaves similarly to an element of solid wood. The fibers are also oriented in the same direction. Softwoods are usually used because it easier to work and direction of the grain is sufficiently straight.
Glulam technology makes it possible to manufacture structural elements with variable sections and allows therefore an optimization of the raw material and more flexibility during the design phase, depending on project needs and situations.
As already anticipated, a remarkable advantage of glulam is being an artificial material, and it is hence possible to make on it a more accurate control of reliable performance. While in a solid wood beam knots and other defects, which have a decisive influence on the strength and behaviour in service of the element cannot be removed or even detected if they're within the cross section, during the manufacturing process of glulam instead it is just sufficient to remove the weaker parts or, at worst, to discard the entire plank: in this way the influence of single potential failure areas is reduced (Figure 5.11).

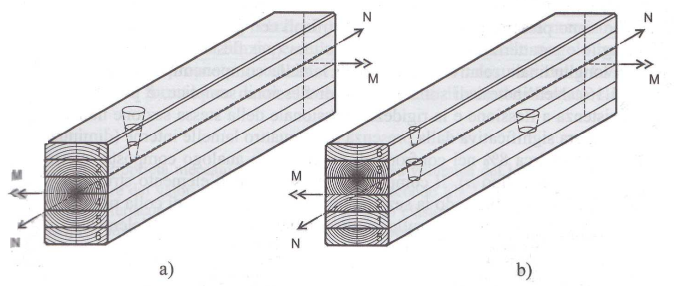


Figure 5.11 Wooden beam element with defect (a) and after a random reconstruction (b) (Piazza, 2007).

The length of the strips is not significant and those obtained from other processes remains may be also used, not to mention that the quality of the strips can then be chosen basing on the prevailing loading situation and to the level of stress, placing lower quality wood in the less stressed areas and the most valuable one where it is really needed: for example in a member working in bending the higher strength class laminations will be positioned in the outer parts. This is the so called combined glulam (Piazza, 2007).
The final result is a material that can guarantee better static performance than elements of solid timber, having a behaviour similar to that of ideal wood, that is defect-free.
Another benefit of this technology is that it completely eliminates the uncertainties due to moisture content of the material: it is in fact much easier (and faster and cheaper) to dry planks of small dimensions rather than a solid wood element of considerable size. The various lamellas, which will then form the glulam element, are of similar size and it becomes then easier to control moisture contents and eventually to adapt them depending on the environment in which the structural component will be placed, reducing the risk of shrinkage and inducing lower internal stresses.
The structural features of the finished product depends primarily on the strength characteristics of individual planks but also on the proper implementation of joints at the edges and on gluing of the several overlapping laminations. In particular the norm EN1194:1999 provides four strength classes (Table 5.1): each one of them is then doubled according to whether it is homogeneous glulam (GL xx h) or combined (GL xx c), where GL stands for glulam and the number indicates the characteristic bending strength (Piazza, 2007).

Table 5.1 The five strength classes for glued laminated timber according to EN1194:1999 (Piazza, 2007).

	GL24h	GL24c	GL28h	GL28c	GL32h	GL32c	GL36h	GL36c
(MPa)								
$f_{m,g,k}$	24		28		32		36	
$f_{t,0,g,k}$	16,5	14,0	19,5	16,5	22,5	19,5	26	22,5
$f_{t,90,g,k}$	0,40	0,35	0,45	0,40	0,50	0,45	0,60	0,50
$f_{c,0,g,k}$	24,0	21,0	26,5	24,0	29,0	26,5	31,0	29,0
$f_{c,90,g,k}$	2,7	2,4	3,0	2,7	3,3	3,0	3,6	3,3
$f_{v,g,k}$	2,7	2,2	3,2	2,7	3,8	3,2	4,3	3,8
(GPa)								
$E_{0,g,mean}$	11,6	11,6	12,6	12,6	13,7	13,7	14,7	14,7
$E_{0,g,05}$	9,4	9,4	10,2	10,2	11,1	11,1	11,9	11,9
$E_{90,g,mean}$	0,39	0,32	0,42	0,39	0,46	0,42	0,49	0,46
$G_{g,mean}$	0,72	0,59	0,78	0,72	0,85	0,78	0,91	0,85
(kg/m³)								
$\rho_{g,k}$	380	350	410	380	430	410	450	430

The production process, as shown in Figure 5.12, begins with the preparation of the planks in order to obtain the dimensional stability and a perfect bonding: laminations are artificially dried in a kiln and then stored for some time in air-conditioned rooms to prevent from having further dimensional changes due to moisture variations.

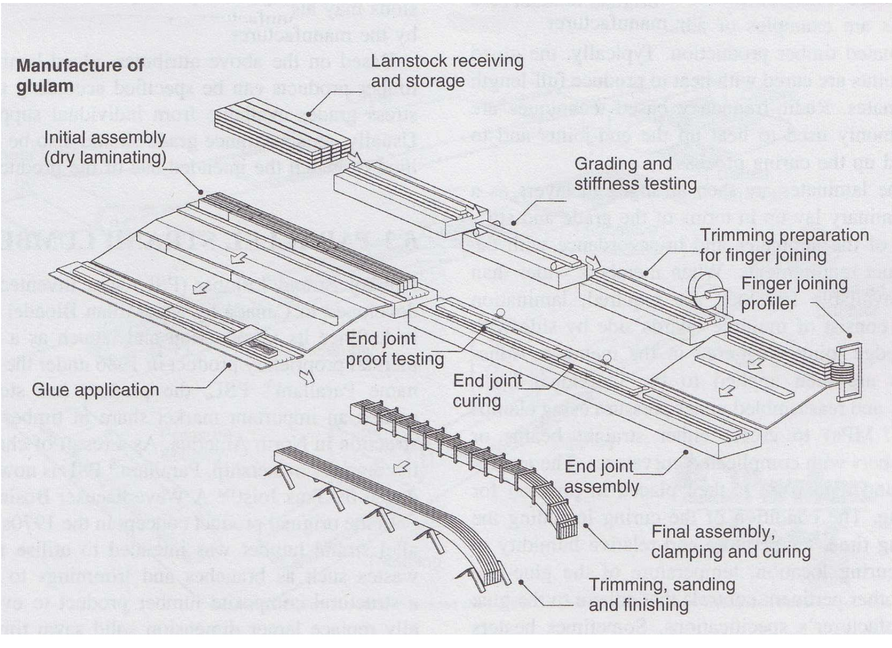


Figure 5.12 Manufacturing process of glued-laminated timber (Thelandersson, 2003).

After a planing on all four sides a selection of the strips is carried on to remove defects and deformities. After the completion of this phase, planks consist of pieces of equal width and thickness but with different lengths due to the elimination of defective areas.
The planks to be joined with one another need to be worked at the edges with a profiler that produces the so called "finger joint" (Figure 5.13), which is then coated with a layer of adhesive.

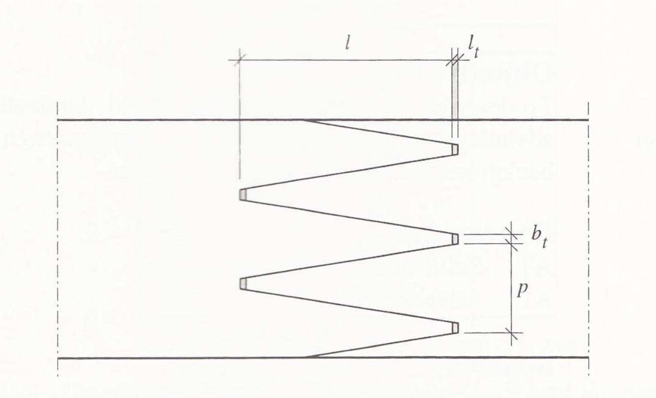


Figure 5.13 The finger joint: l = finger length, p = pitch, b_t = tip width, l_t = tip gap (Blass, 1995).

The tightening is done immediately to prevent air dry the glue and inactive the curing process: during the process of gluing the adhesive flows across the contact surfaces and forms a continuous film.
The adhesives used to manufacture glulam elements are of the MUF kind (melamine-urea formaldehyde) and have the advantage of creating a "clear glueline"; they must fill the gaps between the edges of the elements and must perform a connection that not only guarantees the solidarity between the parts, but they have also to reproduce the cohesive strength existing in the wood before cutting (Blass, 1995). These glues are not attackable by insects, have a good resistance to heat and direct exposure to fire and do not emit dangerous vapors.
The continuous sections, after being cut into laminations of the desired length, are planed again to achieve a smooth surface for subsequent bonding and matching.
The stack of laminations intended to form the element is first formed in dry condition and then on one side of each lamination a layer of glue is spread; afterwards the final assembly takes place within clamps in order to complete the gluing (Figure 5.14).

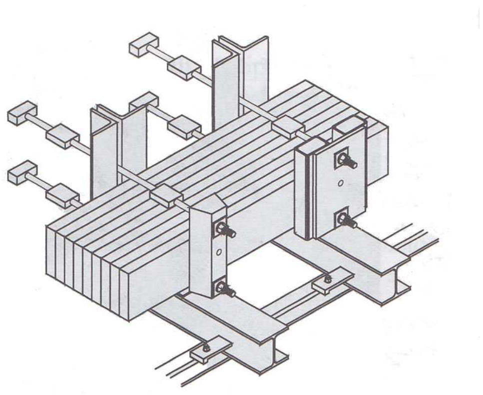


Figure 5.14 Clamping device for the realization of elements in glulam timber (Piazza, 2007).

In order to manufacture curved or non-straight elements a curving of the laminations and the adaption of clamping devices are required.
The operation of planing surfaces serves to remove the adhesive spilled out during pressing, to correct the deficiencies and give the items their final appearance.

5.2.2 LVL KertoQ

To perform the deck of the bridge a LVL (Laminated Veneer Lumber) two panels of KertoQ® 63 mm thick have been used.
The idea underlying LVL concept is to increase the strength (especially the tensile one) of wood reassembling the sawn timber into a new artificial and more homogenous material.
This material is a product similar to plywood but with the difference that most veneers are parallel: it is a layered composite of wood veneers and adhesive. Kerto is produced in two variants, D and Q: in the first case laid with the grain of the layers running exclusively parallel in the longitudinal direction of the panel while in the second some of the panels (20%) are also laid cross-grain. In the case of KertoQ the presence of panels arranged in both directions provides a behaviour similar to a slab, with a good stiffness also in the weak direction (Table 5.2).

Table 5.2 Disposition and number of veneers for the different thicknesses of KertoQ panels (© Finnforest Corporation –Kerto brochure).

Kerto-Q: Veneer structure

Thick-ness (mm)	Z qty	X qty	Veneer structure
27	7	2	II-III-II
33	9	9	II-IIII-II
39	10	3	II-III-III-II
45	12	3	II-IIII-IIII-II
51	14	3	II-IIIII-IIIII-II
57	15	4	II-III-IIIII-III-II
63	16	5	II-III-III-III-III-II
69	23	5	II-IIII-III-III-IIII-II

z = veneer running longitudinally to main panel direction I
x = veneer running crosswise to main panel direction -

The wood species used are mainly pine and spruce.
These panels are made from continuous wooden sheets (obtained using techniques similar to those used to produce plywood) 3,2 mm thick (which become 3 mm thick after compression). The plies are dried and glued and stacked after a selection. The pressing phase follows (Figure 5.15).

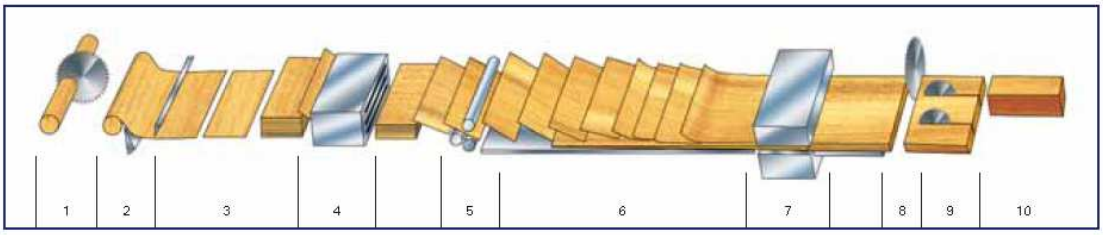


Figure 5.15 *Manufacturing process for KertoQ panels (© Finnforest Corporation – Kerto brochure).*

The product obtained is a panel that has the same behaviour of solid wood and glulam concerning the rheological phenomena and the dependence of the mechanical properties on the service class and duration of load. It is also easy to see in Figure 5.16 how the elastic modulus E_{0mean} is comparable to that of C24 solid wood and to that of GL32 glulam but regarding the strength values they are much higher (about twice the C24 ones).

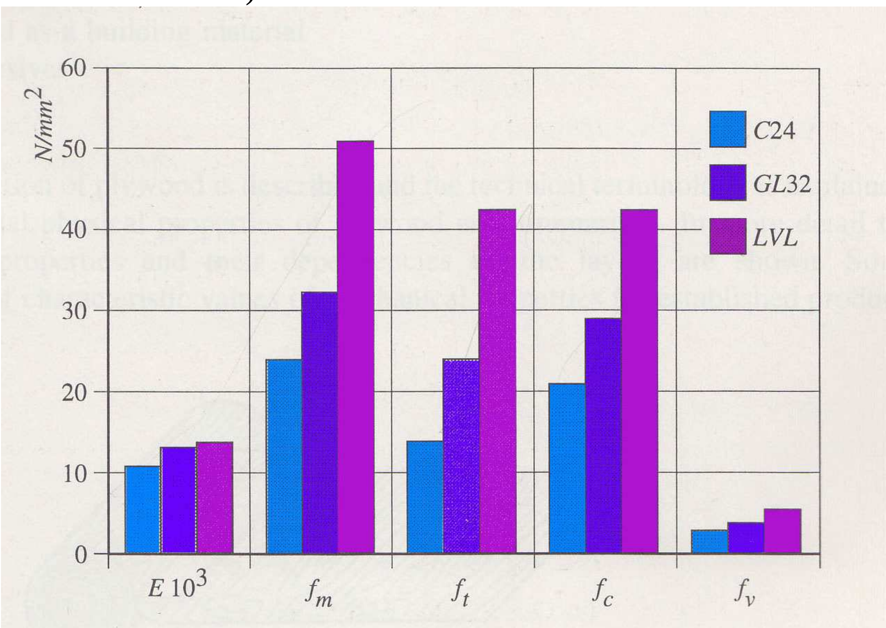


Figure 5.16 *Comparison of the characteristic values of sawn timber (C24), glulam timber (GL32) and LVL (Blass, 1995).*

Observing then the data in Table 5.3 provided by the manufacturer (Finnforest) it's possible to see how the dimensional changes (expansions/shrinkage) due to moisture variations are in the order of 0.01%.

Table 5.3 *Physical properties and moisture expansions/shrinkage of KertoQ (© Finnforest Corporation –Kerto brochure).*

Physical properties	
	Kerto-Q
Moisture content (when leaving the mill)	10 %
Dimensional variation coefficient *	
Thickness	0.0024
Width	0.0003
Length	0.0001
Density (kg/m^3)	510
Fire resistance, charring rate (mm/min.)	0,70
Reaction to fire	D-s1, d0

Ultimately this type of material guarantees high strength and dimensional stability.
Use of KertoQ is preferable in service classes 1 and 2 but it can also be used in service class 3 since the adhesives are water resistant and also, after autoclaving, it can be impregnated and then placed in direct contact with the weather or in moist conditions.

5.3 Description of the static behaviour of Fish Belly Bridge

In Chapter 4, the undertaking methods of the Competition in which the Fish Belly Bridge attended have been described: to better frame and understand the situation and the dynamics in which the design strategies have been carried out during the Competition also five models of other project participants have been analyzed in both their proposals (for the first and second phase of the Competition). In particular in Paragraph 5.1 the design process and choices made by the design team since the first sketch through the creation of two scale models to the choice of final solution have been described throughout.
In this section then the Fish Belly Bridge will be analyzed in its final form and in its translation into practice of all choices made during the earlier stages.

The Fish Belly Bridge is an extremely simple and effective footbridge consisting of two lenticular glulam beams assembled with two KertoQ panels that act as deck and as bracing system at the same time.
The actual length of the bridge is 22 m and the width of the carriageway is 2,5 m (that is the maximum available width for KertoQ panels).

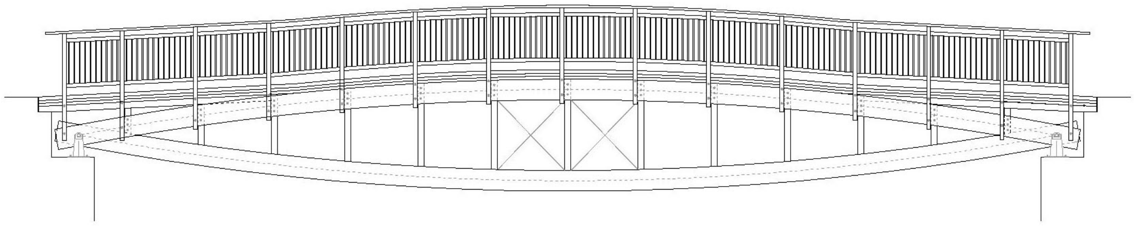


Figure 5.17 Longitudinal view of the Fish Belly Bridge.

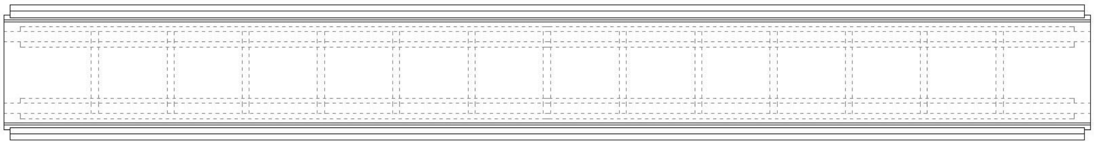


Figure 5.18 Plan of the bridge: the dotted lines are the beams and the studs below the deck.

Based on the limitations imposed by the Competition rules the static system of the entire bridge acts as a simple supported beam at each end with a hinge and a roller pin that allows the structure to have some horizontal displacements.

The main feature of the footbridge are undoubtedly the two lenticular beams (see Figure 5.19): they consist of two curved chords (one upper and one lower) spaced apart by glulam studs placed at a constant distance of 1,65 m. In fact, the upper chords are in turn composed of two parallel elements, with a cross section of 115x450 mm, and spaced at a distance of 215 mm. The fact that the upper chord is divided in two meets two requirements: the first was to be able to simplify the connection with the lower chord (which has a cross section of 215x450 mm) and the second was to make more effective and simpler the connection with the vertical studs, making them "pass through" the upper chord.

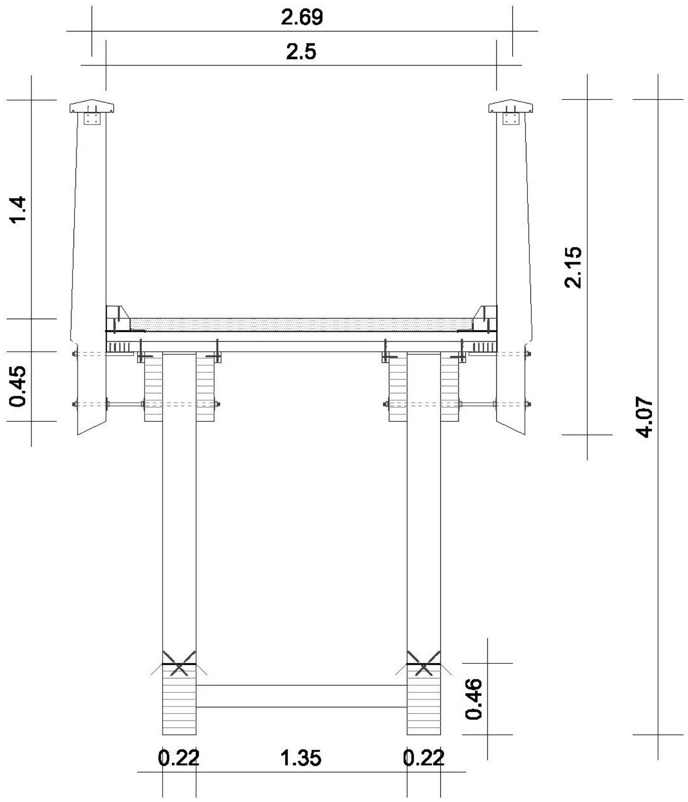


Figure 5.19 Cross section of the footbridge in the middle of the span.

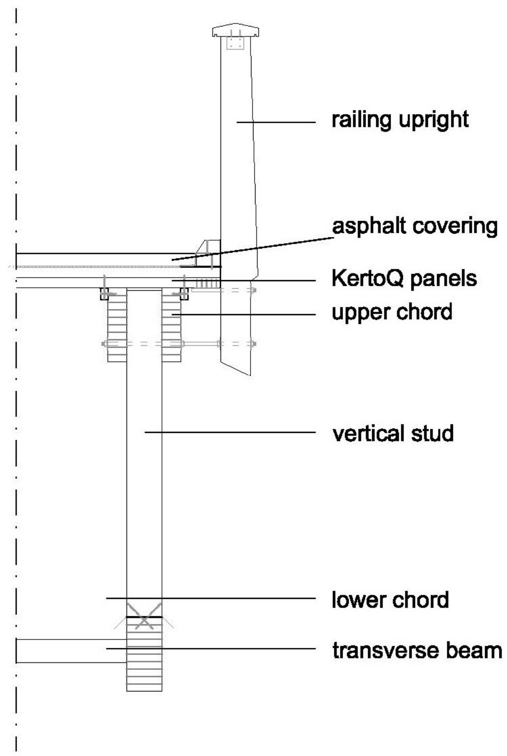


Figure 5.20 Structural elements composing the bridge.

The connection between the three elements of each lenticular beam (see Paragraph 5.4.1), in contrast to what had been assumed during the construction phase of models, where the halves were connected along their base surfaces, is performed as follows: on the edges of the beams, on the outer sides of the lower chord and on the inner sides of the two parts composing the upper one, steel plates nailed and drilled in the center are placed (see Figure 5.21). Through the hole a dowel with a diameter of 60 mm, which is in contact only with the steel plates, is passing (as the hole in the glulam elements is much wider than the diameter of the dowel itself). Since the dowel is 65 cm long, its edges are projecting compared to the profile of the lenticular beam: so it becomes both fastener and support for the entire structure.

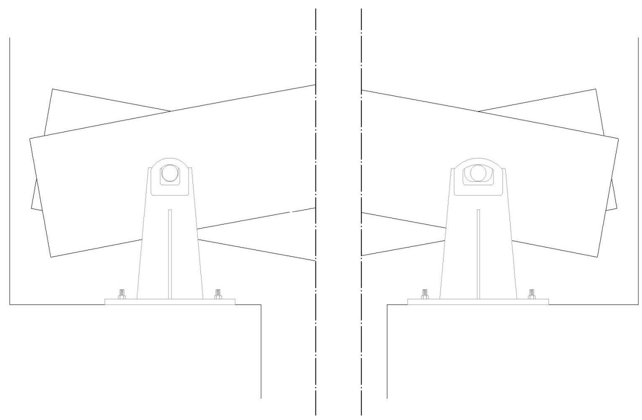


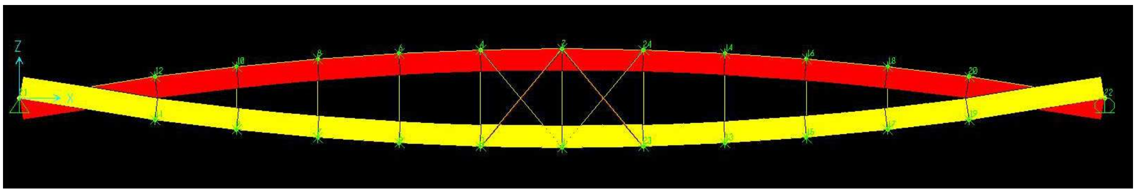
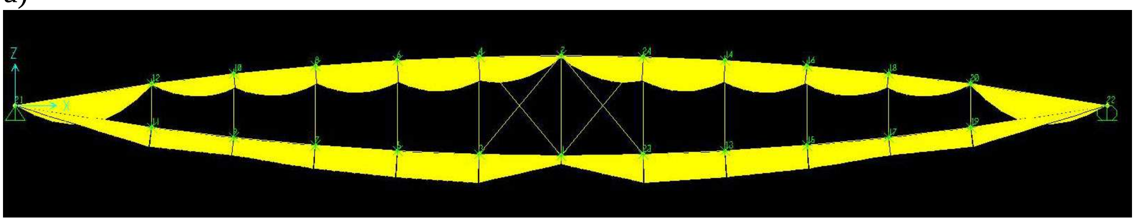
Figure 5.21 Detail of the two different kind of supports with the hinge on the left and the roller pin on the right.

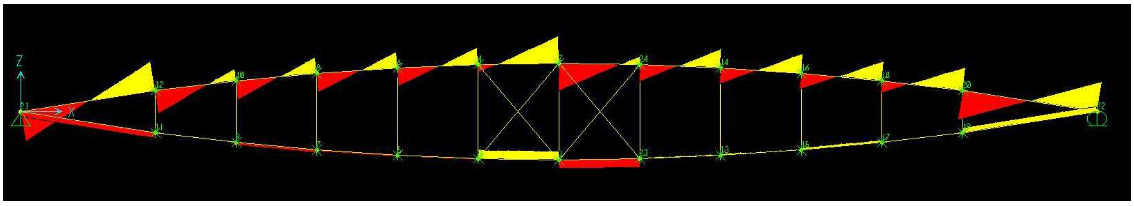
The edges are in fact housed in special steel supports fixed on the river banks: they are four (one for each dowel) and two of them only allow rotation while the other two also give the possibility to the dowel to have a horizontal displacement being the slot wider than the diameter of the dowel itself.
With regard to the studs they have a cross section of 215x140 mm so they can be placed directly upon the lower chord with the entire width without any further processing: in fact the connection is performed through the use of two screws set at an angle of 45 degrees (see Paragraph 5.4 concerning the construction details).
The deck consists of two KertoQ panels, 63 mm thick, glued together and connected to the lenticular beams by screws at the bottom to prevent water from accidentally penetrating the elements: KertoQ, given its composition which provides good resistance also in the weak direction (see Paragraph 5.2.2), has a behavior similar to a slab and then it works as a stabilizing element for instability out of plan and acts as carriageway at the same time reducing in this way weights and costs and greatly simplifying connections thus making the whole process of assembly and erection easier.

In order to maintain a slope of the carriageway corresponding to 1:20, one had to operate as follows: since the top surface of the lenticular beams is curved, it was not possible to meet the requirements for public facilities, then it has been decided that the KertoQ panels maintain the correct slope, separating them from the beams in the final parts towards the abutments. To support the raised edges studs were added to the ends of the lenticular beams close to the connection between the two chords and to prevent

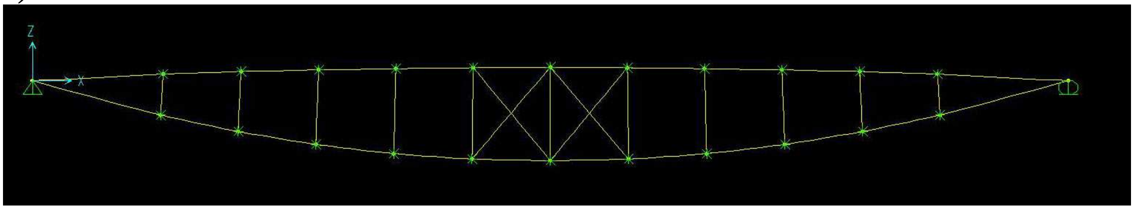
the studs from going to press too much locally over the two panels two beams on each side (with cross section 215x140 mm) have been added, which then enter the free space between the two components of the upper chord.
Above deck there is a layer of asphalt 80 mm thick which protects the timber bridge deck from traffic abrasion and moisture and at the same time provides a smooth and resistant walking surface.
At the bottom of the lenticular beams, in correspondence with the vertical studs, transverse beams are placed with a cross section of 140x140 mm, acting as bracing elements.
Moreover, at the studs are then placed uprights supporting the parapet: each vertical unit is connected both to the upper chord by a bolt passing through and to the bottom part of KertoQ panels by another bolt welded to a plate and then nailed to the deck.
Steel cables were also added in the two fields formed by the three central vertical studs to reduce deformation of the structure in case of asymmetrical loads.

Through the program SAP 2000 an analysis of the static behavior of the structure under the action of variously distributed loads has been carried on.
In the case of evenly distributed load over the entire length of the bridge (for example the situation where the whole crowd is present on the whole bridge) it is immediate to understand how the structure is extremely efficient. Thus, regarding the axial forces, the behavior of the bridge is very simple (Figure 5.22a): the upper chord works in compression while the lower one works in tension and for both the halves of the lenticular beam stresses are constant along the entire length of the elements and practically have equal values. The studs work in simple compression (being modeled like bars hinged at the ends) and the stresses in them are really small.
In practice the lens beam behaves like an arch with a tie-rod: the lower half of the beam act as tie-rod and the vertical studs are simply connecting factors between the two main parts that serve to guarantee the structural continuity but that actually are not stressed excessively (the compression stresses within the vertical bars are about 20 KN).
The diagrams of bending moment and shear are shown in Figures 5.22b-c.
Even the deformed shape of the structure (Figure 5.22d), as expected, is symmetric and regular, and does not create any particular problem.


a)

b)



c)

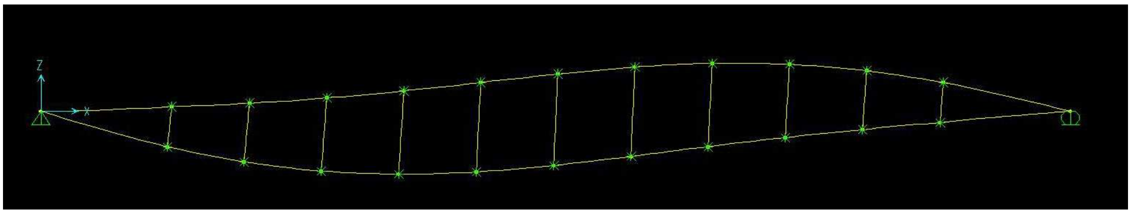


d)

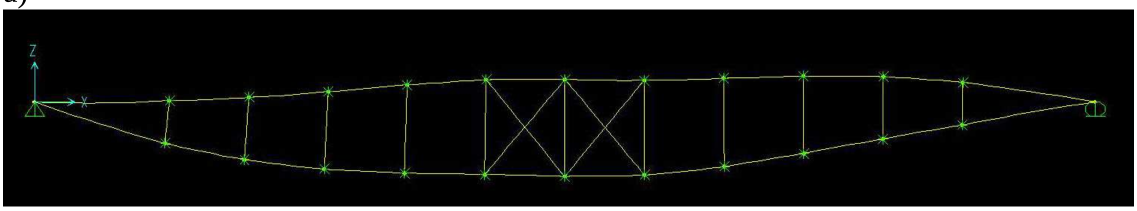
Figure 5.22 Axial forces diagram (a), bending moment diagram (b), shear (c) and deformed shape of the bridge(d) in case of uniformly distributed load on the whole span .

In the case of distributed loads, respectively, on a half and on a quarter of the length of the bridge (to simulate crowd in movement) the behaviour is still good (see Appendix in order to check all the diagrams for internal actions). Axial forces, which are of course higher towards the loaded side (with values equal to one half and one quarter of those obtained with the bridge fully loaded, despite the same load values) are always mirrored in the sense that the stresses in the lower and upper chords are equal and opposite. The studs work always in compression and where the load is not present are even unloaded.

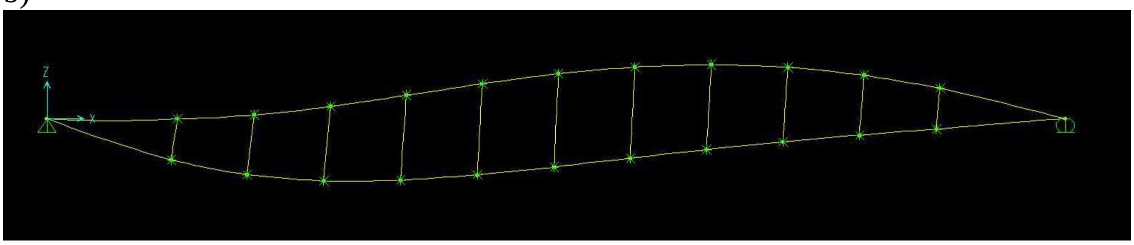
The steel cables work in tension and help to reduce the deformation of the structure, as it is possible to see in Figure 5.23a-d.



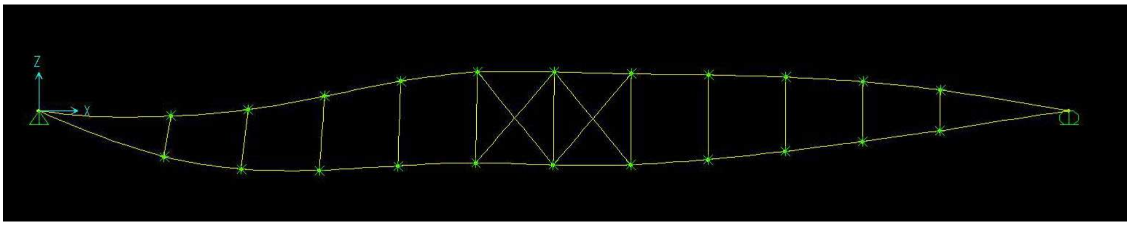
a)



b)



c)



d)

Figure 5.23 Deformed shape of the bridge loaded only on a half without (a) and with steel cables; deformed shape of the bridge loaded on a quarter without (c) and with steel cables (d).

In the final analysis conducted in this Paragraph the following conclusions may be drawn:

- The footbridge has a very good behaviour under the action of distributed loads (these are the typical loads that such bridges are subjected to);
- The fact that the axial forces in the elements are constant leads to a simplified design of the elements themselves in the sense that the cross sections necessary to support such actions need not to be changed and then the beams have constant dimensions;
- The instability phenomena related to the compressed elements are resolved on the basis of constructive choices. The upper chord in fact, being compressed, would present problems of instability both in the vertical plan and in the horizontal plan: in the horizontal direction KertoQ panels shall provide a bond stiff enough to prevent the structure from buckling, regarding the vertical direction, an increase in the external loads corresponds to an increase in the stress state within the lower chord, which, tending to rise with respect to its longitudinal axis, produces a thrust upwards through the studs which stabilizes the upper chord, which tends instead to drop and push down.
- Given the structural simplicity and regularity of internal actions (in some cases quite negligible), even the realization of the connections between the different elements composing the structure is very simple unlike the joints required to perform truss beams, often considerably more complex instead, due also to the inclination of the forces at play.

5.4 Structural details

In the previous Paragraph the structure of the bridge was described as it actually is and its behaviour under the action of external loads. All structural components of the work have been described and identified to understand how the initial idea was translated into reality on the basis of materials and technological solutions currently adopted in the field of timber constructions.

In any timber structure it is essential to design carefully the connections and systems linking the various parts in general due to the particular physical properties (decrease in resistance over time, different strengths depending on the direction of the load compared to the grain, etc ...) not to mention that it should be also paid particular attention to those components that work in tension, more complicated to realize than the one working in compression.
In this Paragraph it will be described as the details have been performed, or the connections between the main structural components that make up the footbridge and, in particular, those concerning: upper and lower chord of the lenticular beams, parapet and upper chords, upper deck and chords, vertical studs and upper chord and finally vertical studs and lower chord.

5.4.1 Connection between the two chords of the lenticular beams

As already pointed out the main feature of Fish Belly Bridge is the conformation of its structure: the two lenticular beams are composed of two curved and specular chords that join in the extremities.
The connection between the two parallel elements that make up the upper chords and the single element which is the lower chord appears quite complex: in fact in the node where the three beams must be connected (which corresponds to the geometric point of intersection of the longitudinal axes of the elements) both tensile and compressive stress parallel to the grain are present at the same time with values that are around 200 KN.
In order to effectively absorb the stresses it has been decided to use a dowel that passes through the three structural components to perform a real hinge that eliminates bending moments at the ends of the structure and allowing thus to have actions on the supports oriented only in the vertical and horizontal direction (even though in the loading cases studied horizontal forces at the supports are not present) and being then able to recreate the model of a simple supported beam.
The dowel is made of strength class S355 steel (with characteristic strength of 510 MPa) has a diameter of 60 mm and a length of 650 mm but there is a strong risk that the repeated horizontal stresses to which the fastener is subjected during the service life finally lead to embedment in timber elements with the consequent getting oval of the housing hole. To avoid this phenomenon it has therefore decided not to put the dowel in direct contact with glulam beams but rather to ensure that the stresses are transferred from timber elements to a steel plate placed in the middle and then to the dowel so as to achieve basically a steel to steel connection, definitely much more resistant.
The system adopted is as follows: the lower and upper chords are drilled holes with a diameter of 80 mm in order to avoid contact with the fastener. On both sides of the final extremity of lower chord (which has a cross section of 215x450 mm) and only on the inner sides of both halves of the upper chord are nailed square steel plates 5 mm thick of side 41 mm and with a central hole which has a diameter of 62 mm (or a tolerance of 1 mm around dowel). To make the plate more resistant and to increase its shear resistance near the hole is welded a second square plate of side 15 mm and 10 mm thick.
The total cross section of each lenticular beam close to the connection is 445x450 mm: to ensure that the dimensions of the section remain unchanged despite the

presence of the four steel plates, from both sides of the lower chord and from the two inner sides of the elements above 15 mm of timber have been taken away (Figure 5.24).

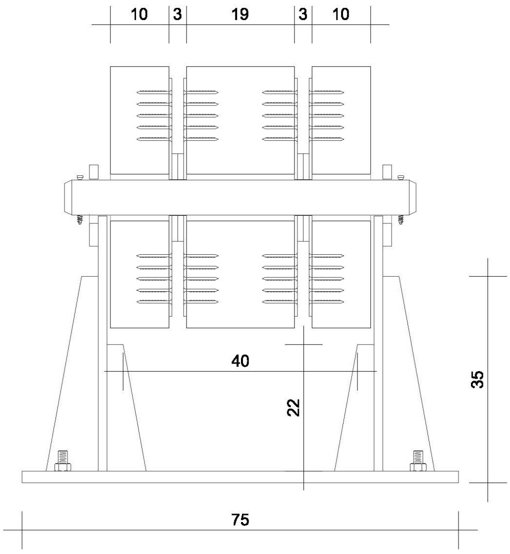


Figure 5.24 Detail of the connection between the two chords: one can see the dowel passing through the three elements, the internal nailed steel plates and the steel support.

The plates are connected to the glulam beams with 200 anker nails 4.0 x60 mm for each plate. Furthermore, on each side of the steel plates are added two screws with a diameter of 12 mm and 400 mm long to increase the shear strength of wood and to avoid the creation of cracks in the direction parallel to the grain (see Figure 5.25).

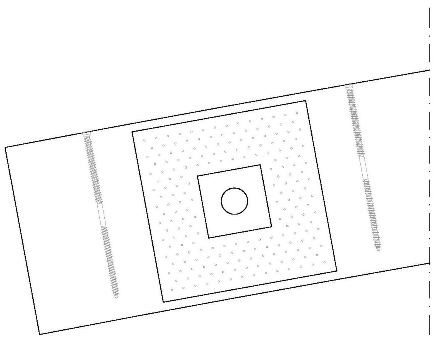


Figure 5.25 Detail of one side of a chord: it is possible to see the steel plate with the 200 nails and the screws each side to reinforce the shear strength of the glulam timber.

This kind of solution has two main advantages: first of all, as mentioned above, it establishes a steel to steel connection, certainly more durable and stable than a steel to timber one in relation to weather and moisture conditions, and secondly it has unquestionable aesthetic advantages since metal elements are hidden and an observer sees only the timber chords from the outside.

Not to forget also the question related to the durability of the connection (see also Paragraph 5.5): all the nails are placed on the inside of the beams and are therefore not directly exposed to the weather, and moreover, on the outside, since the hole in the wood is wider than the diameter of the dowel it remains a gap of about 1 cm each side that allows air to circulate and consequently the water, which might possibly be able to penetrate into the wood, to evaporate.
Being the dowel 650 mm long it is to have edges protruding outside the contour of the timber elements of about 100 mm on each side. In this way it is possible to perform the connection with supporting elements in a very simple and effective manner.
The idea behind the design detail of the attack on the ground is to create two constraints that reproduce the conditions necessary for a simple supported beam: two different types of support have been therefore carried out acting as a hinge and as a roller respectively.
The hinge consists of a steel plate 20 mm thick and 500x750 mm in size which welded to two vertical uprights 15 mm thick that have the top shaped in such a way so that it is possible to insert the dowel completely (the size of the housing in fact is 62 mm): thus the edges of the lenticular beam are placed inside the room created by the two uprights, as shown in Figure 5.26.

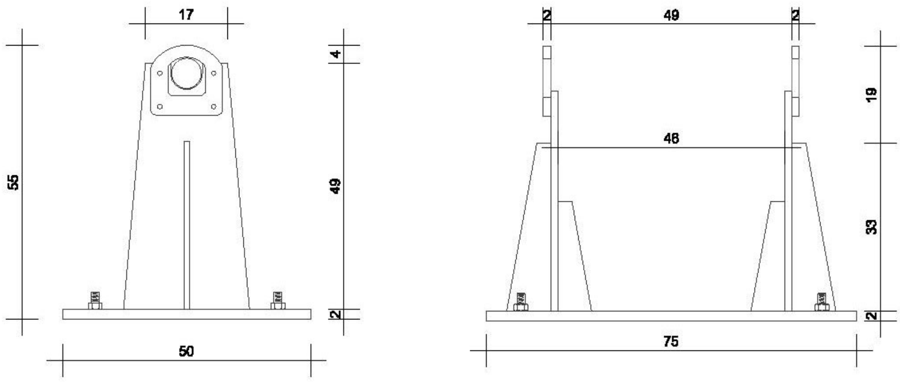


Figure 5.25 Different views of the hinged support.

To prevent a possible lifting of the dowel (and its subsequent exit from the support) on the external side of each upright two steel profiles have been welded with a thickness of 15 mm closing the housing preventing thereby upwards movements. To stiffen the uprights in the transversal direction four steel plates 10 mm thick more are present both on the outside and on the inside.
Slightly different are the supports that act as a roller pin (Figure 5.26): in fact in this case the width of the carving in the steel element is much greater than the dowel (10 cm) to leave free the connection to translate horizontally. Also here there are obviously the lateral stiffeners and the upper closures.

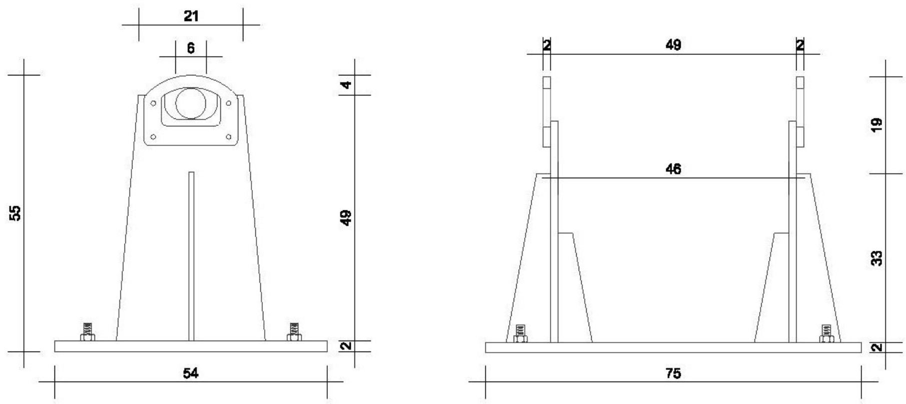


Figure 5.26 Different views of the roller pin support.

The steel supports are then four, two "hinges" at one edge of the bridge and two "roller pins" at the other edge; they are fixed to the concrete abutments by threaded rods, tightened then by hexagonal nuts.

5.4.2 Connection between parapet and lenticular beams

The second detail that deserves to be described is the connection performed between the parapet and the upper chord of the lenticular beams.
The railing is composed of wooden uprights with a cross section of 90x180 mm, on top of which a handrail is placed; in the free space that exists between a vertical element and the other a fence is placed that prevents people (particularly children) from falling outside.
The height of the railing over the walkway is 1,40 m and uprights that form it are placed at a constant distance of 1,65 m, except the last ones at the edges of the bridge near the abutments that have a spacing of 1,55 m because of the shape of the structure. In practice, they are placed at the position of the vertical studs of the lenticular beams, slightly off the axis center of gravity for not overlapping the connections between the studs themselves and the upper chord of the lenticular beams.
The length of the uprights is variable depending on the position they occupy: as the profile of the beams is curved and also being the deck spaced from the beams themselves at the ends of the bridge, the elements are long at least 1.85 m (near the middle) and a maximum of 2 m (the outer ones).
The connection of the upright with the bearing structure is very simple and the points where it is attached are two: one towards the lower end and the other just above where there is contact the with the two KertoQ panels forming the deck, as shown in Figure 5.27.

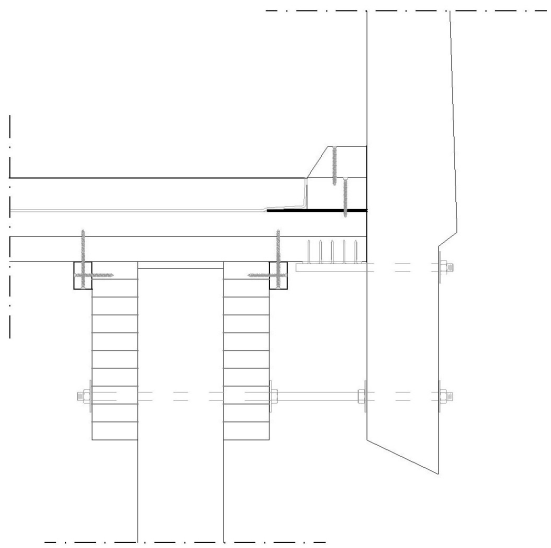


Figure 5.27 Detail of the connection between the railing upright and the upper chord of the lenticular beam.

Since the deck is projecting 25 cm over the underlying lenticular beams to connect the upright with the upper chord is necessary to use an M20 steel threaded rod 900 mm long and with a diameter of 20 mm that passes simultaneously through the upright itself and the two parallel glulam elements: near the outer surfaces of both components some hexagonal nuts are placed to ensure that the connection is as solidal as possible and to ensure the tensile strength in case of horizontal loads (1 KN usually) applied the handrail. Between each nut and timber steel washers should be added in order to spread better the stresses transmitted, preventing thus the nut from penetrating into the timber.

The connection with the deck is rather different: in this case, being direct the contact between the vertical element and the KertoQ panels, the threaded rod with the same diameter as the previous one and passing through the upright, in the inner part is welded to a drilled steel plate 5 mm thick which is then nailed to the bottom of the deck. A single nut in this case is placed on the outside of the vertical support.

To underline how the uprights, on the outer side where the holes for passage of threaded rods are drilled, have a recess prevents the weather directly affects the nuts and stainless steel elements, and also their lower end is inclined to prevent water from going up back.

The connection between the handrail and the uprights is performed through L-shaped steel plates 5 mm thick (two on each vertical) placed on the bottom of the handrail and then nailed (see Figure 5.28).

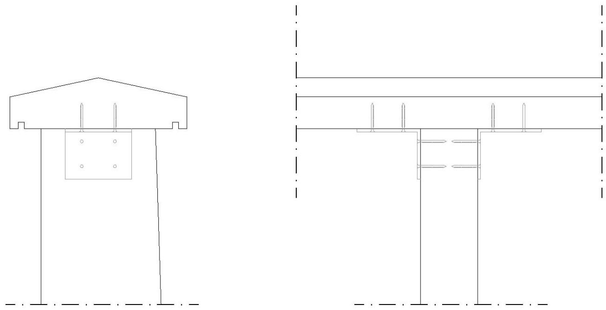


Figure 5.28 Two different views of the connection between the handrail and the upright.

5.4.3 Connection between deck and lenticular beams

The connection between the lenticular beams and the deck of the footbridge takes place at the top of the beams themselves.
Since the presence of the studs at regular intervals in the middle of the two parallel beams that form the upper chord, the connection with the two overlying KertoQ panels has been made obligatorily on the lateral surfaces of the beams.
To realize the connection has been made use of solid timber strips with a cross section of 45x70 mm: they form an "auxiliary" system linking the different elements as shown in Figure 5.29.

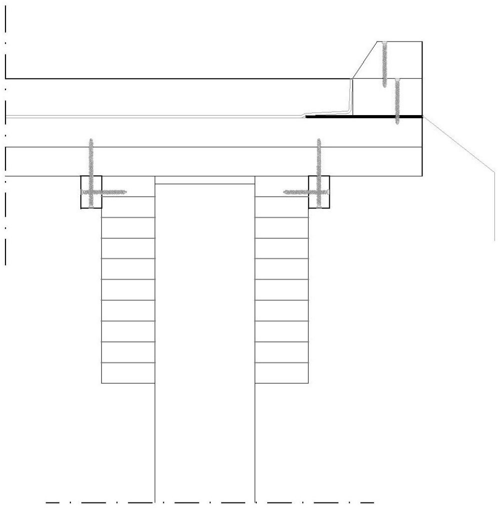


Figure 5.29 Connection between the KertoQ panel deck and the upper chord of the lenticular beams.

Within these intermediate elements are put two sets of screws with a spacing of 30 cm: the first set consists of screws with a diameter of 5 mm and 120 mm long arranged horizontally in order to connect the strip with the two halves of the upper chord, while the second set consists of screws with the same diameter but 200 mm long placed vertically and connecting the strips with the bottom of the double KertoQ panel.

The resulting union is protected (as fully covered), quick, easy to implement and effective.
On the upper side of the deck a welded bitumen mat is applied and on it is then spread a layer of asphalt 80 mm thick which creates a smooth and homogeneous walking surface and that behaves as a protective element for KertoQ panels.
On the lateral extremities of the carriageway coated steel plates with a thickness of 0,6 mm are placed, they are used to protect the lateral surfaces of the KertoQ panels and the top part of lenticular beams; between the plate and LVL, however, a rubber mat 5 mm thick and 250 mm wide is interposed to avoid direct contact between metal and timber. To fix the mat and the plate two boards screwed on top of them are use. They are in solid timber with a cross section of 80x150 mm and they are a kind of protection for the feet while acting as formwork for casting the layer of asphalt.

5.4.4 Connection between vertical stud and upper chord

The connection between the vertical studs and the double upper chord is performed in an extremely simple way considering the very small compressive stresses to which these items are submitted (about 20 KN).
When modeling with SAP2000 bonds at the ends of the studs were summarized as hinges so as to obtain only stresses due to axial forces.
To achieve the union are enough only three bolts with diameter of 16 mm and length of 500 mm passing through the two parallel glulam beams forming the upper chord and the stud inserted in the middle. At the edges of the bolts there are hexagonal nuts with washers (for the same reasons explained in Paragraph 5.4.2) as shown in Figure 5.30.

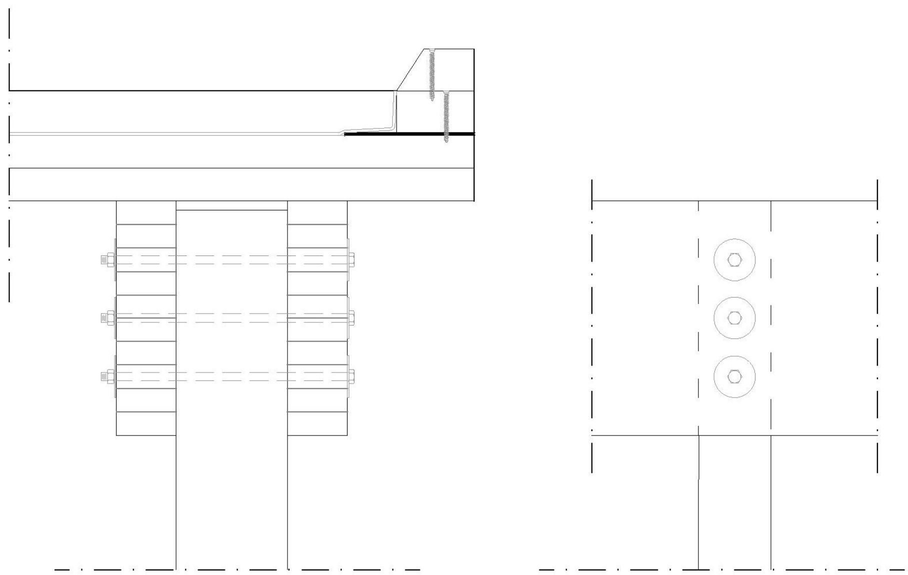


Figure 5.30 Connection between the vertical stud and the double element upper chord.

Again the connection is protected from direct water action as it is covered by the overhanging deck to which the coated steel plate is added, increasing in this way the "shadow" area.

5.4.5 Connection between vertical stud and lower chord

The connection between vertical stud and the glulam beam that forms the lower chord of lenticular beam is perhaps even simpler than the one implemented in the upper part. The stud is standing upon the glulam beams and to connect it, given the low stresses, it enough to use two screws with a diameter of 8 mm and 220 mm long set with an inclination of 45 degrees (see Figure 5.31).

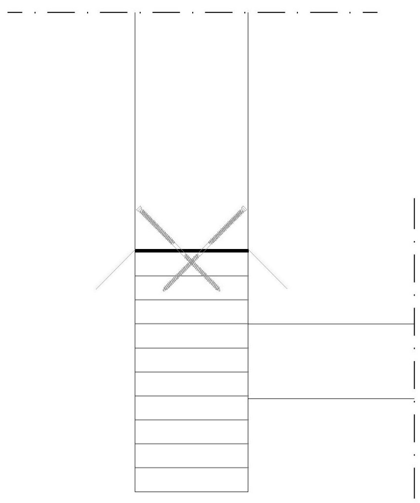


Figure 5.31 Connection between the vertical stud and the lower chord with two screws set with an inclination of 45°.

In the space between one stud and one other places a coated steel plate 0,6 mm thick covering the whole width of the beam and a rubber mat 2 mm thick are placed to avoid stagnation of water on timber.
Again the connection is simple, cheap and protected as the screws are fully inserted into the timber and it is enough to close the entrance hole with a stopper to ensure full protection to metal fasteners.

5.5 Maintenance and durability of the bridge

Each element that is part of a structure is constantly exposed to the weather conditions of the environment in which it is placed and it must withstands the actions and the effects from arising. This is a simple principle that remains valid irrespective of the material used, the type of construction or function of the element considered (bearing, non load-bearing, etc.). Unfortunately this principle, apparently very obvious, is often overlooked and not only for timber construction. Building with wood it always

requires a confrontation with these issues for the natural tendency of such material to the natural biological degradation. For the purposes of exploitation of wood for the production of long lasting works, the biological degradation must be therefore prevented or delayed in any case at least as long as long is the required service life of the product in question.
Each component has thus a demand for durability, or a service life dictated by the expectations of the customer and of the users of the construction.
Durability is determined by many factors, some related to the physical and biological properties of materials used, partly due to service conditions and partly due to design, maintenance and protection solutions applied to prevent deterioration of materials and structures.
With the general definition of protection measures means all necessary steps to maintain, secure and ensure the durability of the building.
Although wood is for sure the most sensitive to the degradation of the building materials is at the same time the one that, if properly used, can provide significant durability to structures over time, although it is difficult to quantify.
To obtain results that are satisfactory and long lasting there must be a proper planning and execution of the work and the designer's task is to design the structure in such a way that will satisfy not only the aesthetic, architectural, static, functional and economic needs, but also those related to durability and to efficient and effective maintenance of the building itself.
The consideration of the possible degradation of wood is therefore an indispensable aspect of planning and designing of a building. The description of the concept of protection of wood selected and implemented can then become an integral part of the designer's task and lead to what may be defined as the verification of wood protection: similarly to the verification of structural resistance in this case it has to be proved that the requirements concerning durability and assigned conditions are fulfilled.
Unfortunately currently this stage of the project is not required and it is therefore rare in practice, although for a designer that is still conscious and aware of the characteristics of the material he is working with, this issue should not increase the work.
The basis therefore for the durability of timber construction is the problem of degradation of woody material as a result of environmental conditions in which it is inserted. It will then briefly discussed below as the destruction of wood is caused by biotic agents (insects, fungi, etc.) while cyclical actions due to the weather cause a serious deterioration of the material. An exception in this context are the kinds of degradation due to fire, which are of significant importance but not covered in this Thesis.

Biotic attacks by insects and fungi (although in modern construction they represent a theoretical rather than a real danger) deserve a fast deal.
Regarding the development of fungal spores on wood surfaces, this can happen when the suitable climatic conditions take place (for example adequate temperature and humidity). The ideal temperature for the life processes of fungi varies between 18 and 30 °C, while below 5 °C there is no possibility of development of spores (Blass 1995).
From this it can be drawn that only during winter time and under conditions of very low temperature the material is safe from a possible degradation of wood due to fungal attack.

It is however more interesting to note that the critical condition for the development of these organisms on a wooden substrate is the presence of a water content in the wood higher than 20%. This condition can only be achieved with air humidity above 90% or so in the presence of stagnant liquid water: avoiding these service conditions for structural elements it is possible to limit the moisture content in the wood below to 20%, removing thereby virtually all the risks related to the degradation caused by fungal attack.
Climatic conditions in which wooden elements are normally used are always favorable to attacks by insects xylophages, which could occur over either seasoned wood or not. In reality, the danger for timber structures is rather remote mainly due to: application of layers of paint on surfaces for aesthetic and protective reasons, use of adhesives for production of glued laminated timber and frequent use of synthetic protective materials.
In conclusion it is possible to say that the principal danger of biological degradation for wood comes from xylophages fungi, thus limiting the durability of wooden structures in most cases. So durability is always guaranteed if there will be no service conditions in which the moisture content of wood can exceed the 20% value. This statement, apparently simplistic, is really simple and correct.

The biggest problem is always the action of weather and of water in particular.
Analyzing such an element of timber (solid or glulam it makes no difference) in service condition and in hygroscopic equilibrium with the surrounding environment, the effect of weather can be described in a simplified manner as a cyclical and irregular action of drying and wetting of the material.
The direct action of rain water on exposed surfaces leads to a water absorption in the area of the section immediately below the surface (usually limited to a few millimeters in depth): the result is a high gradient of moisture and, because of swelling of the surface region, the presence of internal actions that can lead to a local collapse of the material. After the cessation of precipitation, in presence of direct solar radiation, this surface area tends to return quickly in hygroscopic equilibrium with the environment, releasing water to the outside. The phenomenon of drying is accelerated by increased thermal conductivity of wet wood and this results in a greater shrinkage of the surface region compared to the inland areas one. Consequently in the surfaces, even in the presence of local collapses due to compression, the cracks are easily formed, even small, but permanent and irreversible (Piazza, 2007).
The surface cracks are still present even when the moisture content of wood goes back to its original condition, facilitating the penetration of water deeper in the section during the next rainfalls.
The cyclical degradation phenomenon just described therefore goes on and it erodes deeper and deeper layers of the section, aggravated by a gradual and inevitable increase of moisture in wood, since the water absorption is faster than drying. The water content can also easily exceed the 20% limit within the section, helping to create favorable conditions for the development of fungi and other organisms (Blass, 1995).
This phenomenon is common to all elements of large dimensions, directly exposed to the weather and whose cross section is so great that it does not allow moisture to return below the critical humidity value of 20%: therefore, in a very dangerous way, the material can degrade from the inside without showing any external symptom.
The elements of the building that make up the bearing structure are generally of considerable size, are essential for safety and functionality of the work and are usually

not replaceable or their eventual replacement is complicated and expensive. Considering the size of the section of those elements, in case of direct exposure to the elements, the phenomenon of decay of the material from the inner layers can easily take place.
Since the failure or deterioration of these parts determine the fact that the structure can no longer be used, they should be protected very carefully to ensure a service life as long as possible, or at least commensurate with the needs of users.
Regarding the elements that are thin or having reduced sections, they are not subjected to the phenomenon described above because their drying process is fairly rapid once the environmental conditions permit it. The degradation in these cases is spread more or less uniformly everywhere and it is visible on the section, and in any case starting from the outside. These are usually non-structural elements, but directly responsible for the durability of the structure, as they often act as constructive protection elements and they thus ensure the durability of structural bearing elements. These elements are usually directly accessible and easily replaceable, and their service life is not decisive or coincident with the service life of the whole structure. They can still be sealed to prevent the direct absorption of water by wood.
It should however be underlined that, regardless of the degree of protection or exposure, on horizontal surfaces of each element water has no chance to flow away and then it stagnates, favoring the absorption. The wood surface then often has irregularities or small cracks: the secure flow of water so it must be ensured by appropriate surface inclinations. Similarly then to stagnant water, constructive details may create areas where water can remain trapped, creating a dangerous increase in moisture that, even if it is localized, can become dangerous. These little puddles of water may occur in areas where there is not a perfect contact between different wooden elements or between wooden elements and metal parts in the connections.

In perfect analogy with the structural verifications the two components of the relationship can be identified in order to check if the verification of the durability will be fulfilled:

$$\text{Action of degradation} < \text{Resistance to degradation}$$

To influence the durability one can therefore act on both components, for example reducing the actions and increasing the resistance to biological attack. The first objective is achieved through constructive solutions such as a proper design of the structure or the addition of special items with the task of protecting timber, while the second is obtained with the treatments of wood, either on the surfaces or by impregnation.

The specific solutions adopted in the Fish Belly Bridge to obtain adequate durability of the structure will be analyze below.
First of all, as a measure of passive protection, all structural elements (lenticular beams, vertical studs, uprights of the parapet) are treated on surface with a protective varnish applied in several layers. Using this device wood becomes less attractive to insects, the wood surface is less sensitive to dirt and aesthetic qualities are also added to the structure as a whole. It must underlined that these paints do not provide a long lasting protection as they do not block water exchange with the environment and do not even prevent the formation of cracks but they are an effective complementary protection.

This surface treatment must be renewed periodically in order to be able to maintain its effectiveness.
As already said, to protect the upper parts of the lenticular beams KertoQ panels are projecting over the beams themselves, in order to provide an active protection.

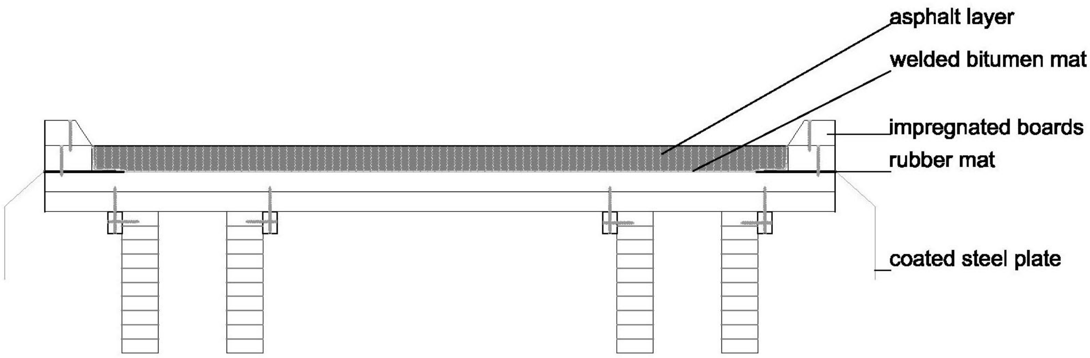


Figure 5.32 Devices used to protect the deck from the weather and the water.

In addition to this two sets of coated steel plates have been added to both edges of the carriageway so as to facilitate the water flowing and to direct its flux downward, avoiding at the same time that it comes into contact with the side faces of the LVL and also with the upper part of the beams below.
To fix the steel plates to the deck below two solid timber boards have been used which are then been screwed. These two overlapping boards are impregnated with preservatives that increase their resistance to biotic and abiotic attacks but they are not structural elements, not essential for the functioning of the structure and they can be easily replaced if necessary.
Even the layer of asphalt 80 mm thick will prolong the durability of the work but must be separated from wood by a welded bitumen mat (insulating mat) 5 mm thick that, being waterproof, protects the wooden elements of the deck from a direct contact with weather conditions and also the curved shape of the carriageway makes easy the removal of water towards the abutments.
A second set of coated steel plates is placed at the top of the lower chords to facilitate the drainage of rainwater and to avoid dangerous stagnations also considering the concave shape of the chords themselves. In the proximity of the studs steel plates are bent to cover the point of contact between the stud and the beam so that there is no chance for the water to penetrate into the connection. Moreover, the entry holes of the two 45° screws are sealed up with waterproof and elastic mastic that allows the shrinkage and swelling but it does makes it impossible for water to enter.
The phenomena of absorption and drying are faster in the direction parallel to the grain than in the transversal one; for these reasons, an increased risk of cracks opening takes usually place in the heads of the beams which are particularly vulnerable in case of exposure to weather or other sources of moisture. To prevent a premature degradation from beginning in these areas and then spreading throughout the beam coated steel plates have been placed on all the heads of the chords that form the lenticular beams.
It is worth underlining how in the connections between the two halves of each lenticular beam (Figure 5.33) all metal elements (steel nails and plates) are placed inside the timber elements and how on the external sides between timber and steel

dowel there is a distance of 1 cm in order to allow air circulation in case of water ingress.

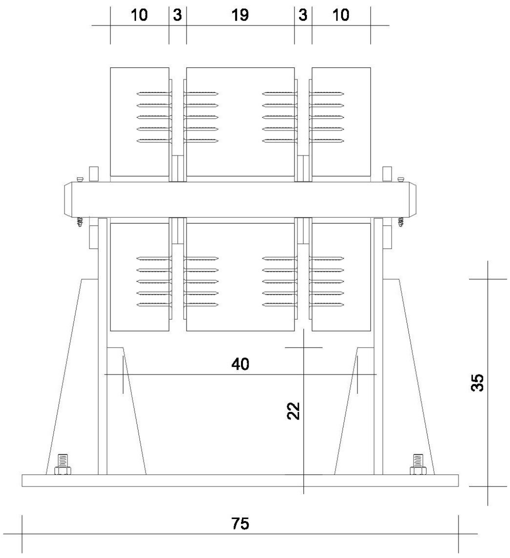


Figure 5.33 Detail concerning the connection between the two halves of the lenticular beam.

The structural elements which are more exposed in the structure are the uprights of the parapet: even though they are superficially treated with protective paint they still present constructive devices necessary to ensure their durability. First the lower edge of each upright has such a shape that does not allow water flowing over it to go back up and secondly the external surface has some carvings as shown in Figure 5.34. The nuts of the threaded rods in fact usually slow down the water flow and prevent the swelling of wood (creating then cracks): retreating the metal connectors resolves this problem.

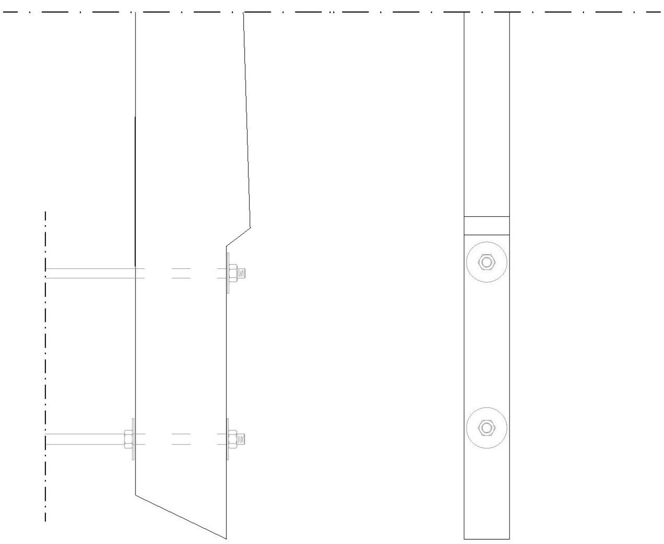


Figure 5.34 Detail of the lower edge of the upright: the carving prevents water from slowing down.

The handrail, shown in Figure 5.35, although it is not a structural element, requires however special attention in order to be effective for as long as possible: its top has a double slope and near the bottom part gutters are performed to remove the water quickly from wooden element. The connection with the upright are realized in the lower part with nailed L-shaped steel plates: this solution is more tricky but it is also risk free of rapid degradation.

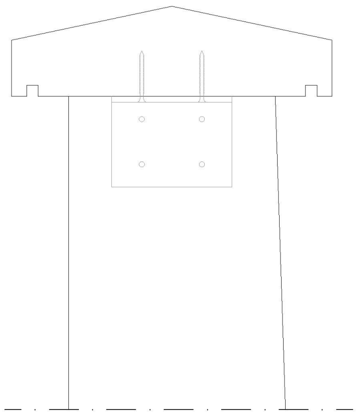


Figure 5.35 The handrail top has a double slope, gutters and the L-shaped steel plates are put below the element to be protected.

In case of contact between timber elements and elements made of other materials the easiest device to perform in order to ensure durability is physical separation: often in fact porous materials (like concrete) can transport water by capillarity and passing it then to timber. Where there is contact between the beams that support the deck at the edges and the concrete abutment the solution is the following one: a waterproof rubber mat 1,2 mm thick is spread over the concrete and close to the point of contact two rubber pads 20 mm thick are placed under the timber elements. In the room between the final part of the deck and the abutment a steel plate should be put in order to prevent water from flowing and on it a welded bitumen mat and the asphalt paving are laid.

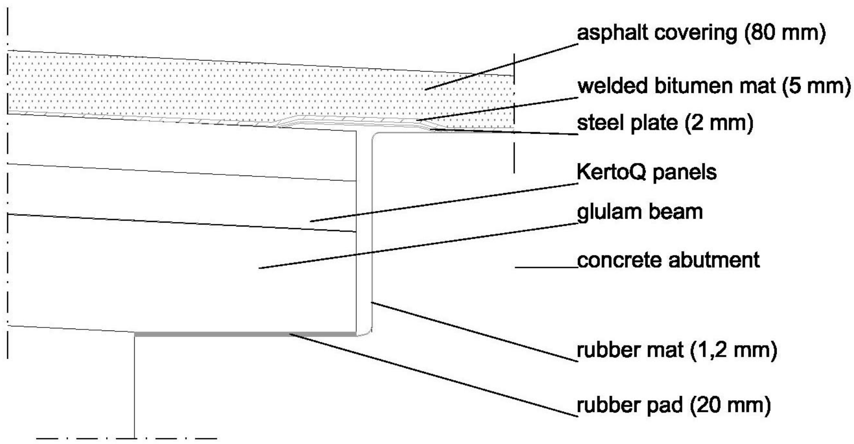


Figure 5.36 Detail concerning the point of contact between the timber deck and the concrete abutment.

To ensure a long lasting service life it is also necessary to provide a Maintenance Plan (see Paragraph 7.2) with certain intervals to check the efficiency of all parts of the structure.
The maintenance plan for the Fish Belly Bridge can be articulated with checks that take place annually, every five and every ten years.
During the annual checks:

- Abutments and intermediate supports shall be checked once in spring time and once in autumn time and the vegetation shall be removed.

- Railings shall be checked and possibly repaired, if they are damaged.

- Mechanical fasteners shall be checked and bolts shall be tightened. Tightening should occur between September and October.

- Wearing parts at the sides and at the ends of the bridge shall be checked. Holes or small openings where water and/or debris can enter upon, are not accepted.

During the quinquennal checks:

- Paving and waterproof layer shall be checked. Possible damage shall immediately be retrofitted
  .
- Moisture content (MC) shall be measured on 15 different points in the underside of the deck and its mean value of MC should be lower than 18%. If MC in one or more points is considerably larger than MC mean there is a risk for leakage; if the leakage has already occurred, the waterproof membrane and the asphalt paving should be removed around the damaged point (within a distance of 2 m from the damaged zone). Then the deck should be dried and the waterproof membrane and asphalt paving restored.

During the decennary checks:

- Surface treatment (coating) of the elements of the bridge shall be checked by professional staff, who will decide if retrofitting or simple repainting will be applied.

- Damaged coating on steel parts shall be steel brushed and improved by means of zinc-based coating.

Finally after 25 years the asphalt paving and waterproof membranes, bridge joints and wearing plates shall be removed and replaced with new ones.

5.6 Renders of the bridge

In this paragraph the Author has included some renders of the bridge to allow the reader to better understand the whole operation through a virtual simulation of what could be the final appearance of the structure placed in its context.

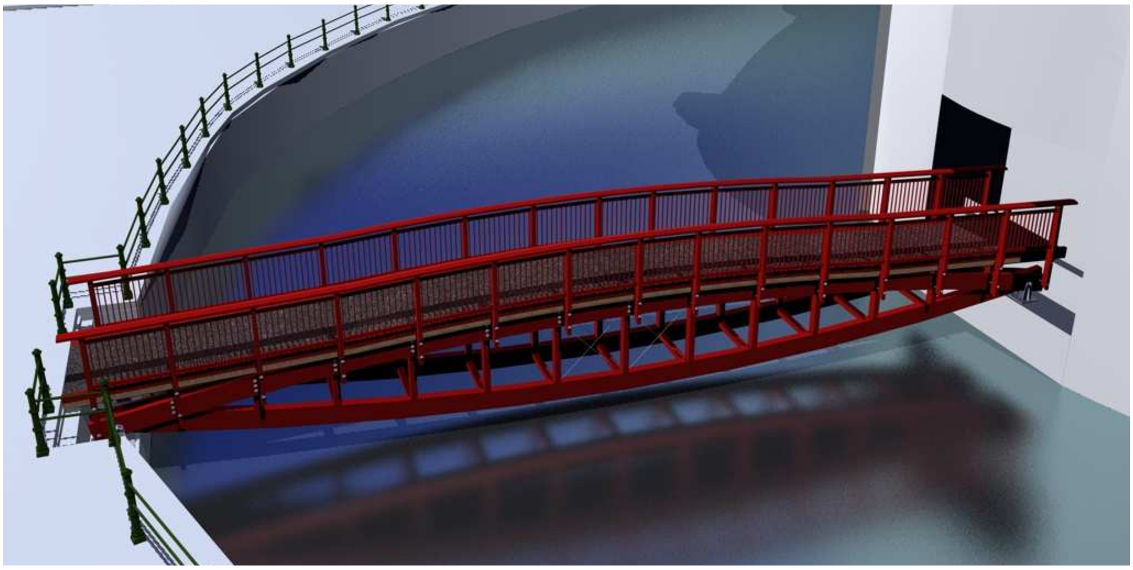


Figure 5.37 View of the bridge from north-east: on the right side of the Viskan there is the City Centre.

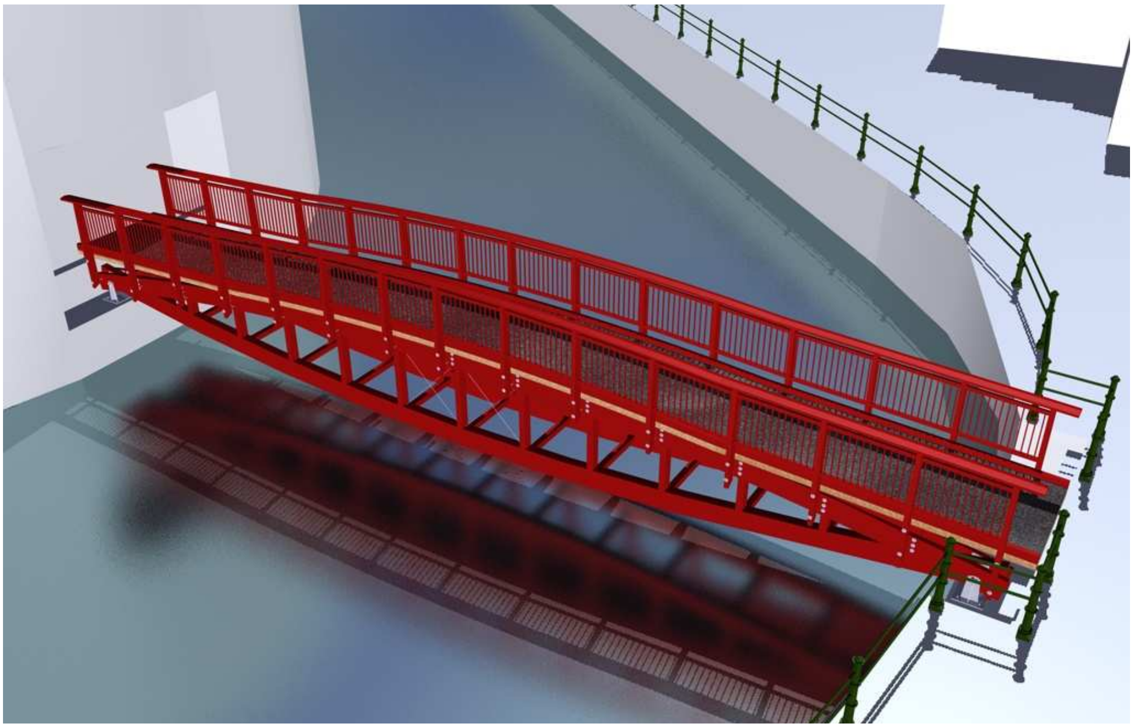


Figure 5.38 View of the bridge from south-west: on the left side of the Viskan there is the City Centre.

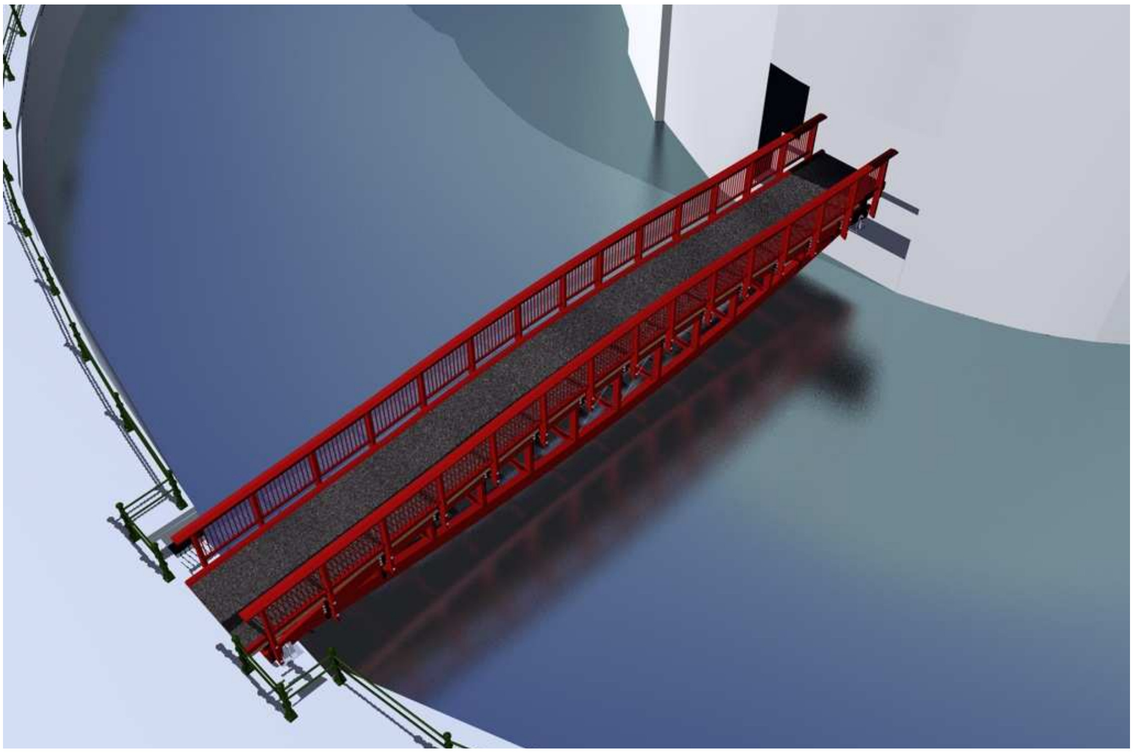


Figure 5.39 View of the bridge from above.

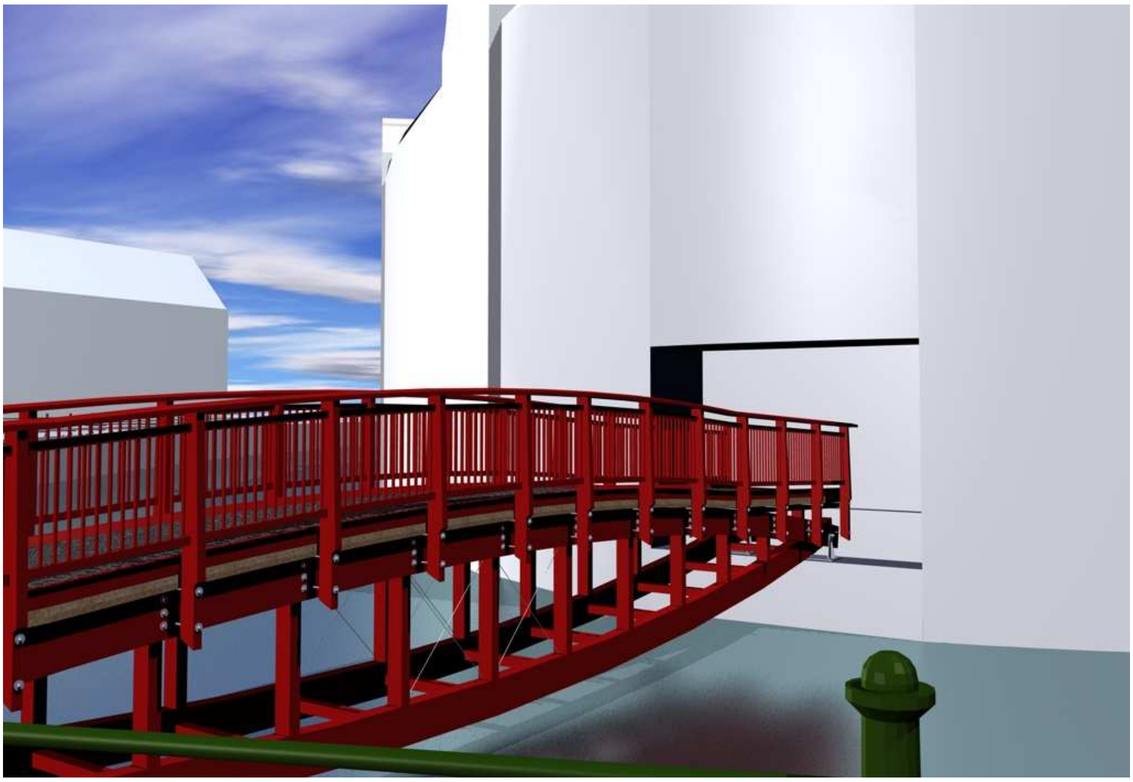


Figure 5.40 Detail of the connection between the bridge and the façade of the City Centre.

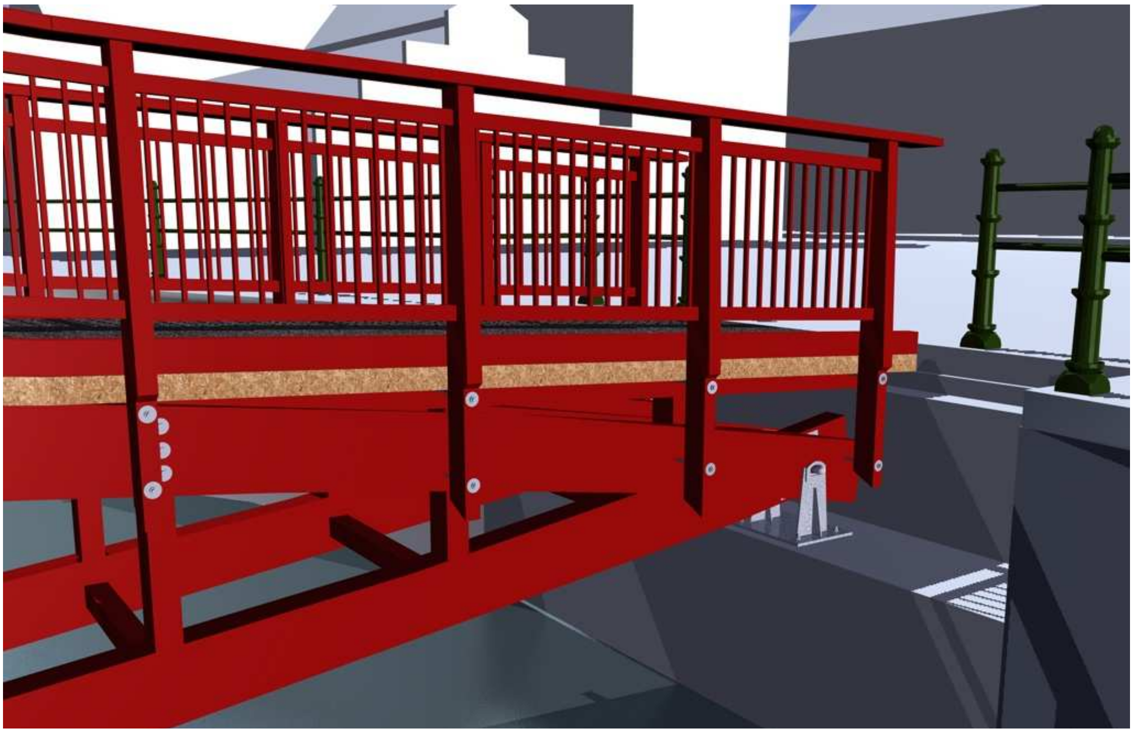


Figure 5.41 Detail of the steel supports placed on the left side of the Viskan.

Figure 5.42 Detail of the steel supports placed on the right side of the Viskan.

6 Verifications of the Fish Belly Bridge based on Eurocodes

The bridge analyzed in this Thesis work has been already presented in the previous Chapter 5. The information gathered and described in the first chapters together with the considerations formulated in Chapter 5 are useful also now that the design of the verifications are going to be made. The coefficients for the load combinations, the safety factors for the materials, the service classes, the duration of loads assigned in the Eurocode 5 (Anon. 2004) and the characteristic strength of materials assigned in UNI-EN 1194 (1993). All the components of the bridge are simplified as isotropic materials.

This Chapter analyses in details the design of the bridge based on EC5. It is divided in 6 parts: 1) definition of the design values for the strength of materials, 2) definition of the loads acting on the bridge, 3) definition of the load combination, 4) calculation of the design moment and the design shear, 5) verifications in the ULS: stress verification, 6) verifications in the SLS: deflection and vibration verification.

6.1 Design values for the strength of materials

For timber and wood-based materials the design strength in the ULS is calculated starting from the characteristic strength *Xk* in a simple way in order to achieve the design strength X_d:

$$X_d = k_{\mathrm{mod}} \frac{X_k}{\gamma_M} \qquad \text{(Eq. 6.1)}$$

where γ_M is the partial factor for material properties; for glued laminated timber γ_M is equal to 1,25 while for the LVL products is 1,2 (see EC5, part 1-1, point 2.4.1).
To take in account the real mechanical behavior of timber, when it is loaded by external forces, the introduction of the modification factor k_{mod}-parameter has been necessary.
This parameter depends on the "Service class" (or the environmental conditions where the elements are placed) and on the "Load duration class".
For the bridge studied it has assumed that the structure is in Service class 2 (that is characterized by a moisture content in the materials corresponding to a temperature of 20° C and the relative humidity of the surrounding air only exceeding 85% for a few weeks a year) and the duration of actions is the short one because the variable actions due to pedestrian traffic should be regarded as short term actions (Anon. b, 2004): these assumptions lead to a k_{mod} value equal to 0,9.

The design values for the GL24h glulam timber used to realize all the bearing load elements can be found in the Table 6.1:

Table 6.1 Design values of strength for GL24h glulam timber

$f_{m,k}$	24	MPa
$f_{t,0,k}$	16,5	MPa
$f_{t,90,k}$	0,4	MPa
$f_{c,0,k}$	24	MPa
$f_{c,90,k}$	2,7	MPa
$f_{v,k}$	2,7	MPa
$E_{0,mean}$	11,6	GPa
$E_{0,05}$	9,4	GPa
$E_{90,mean}$	0,39	GPa
G_{mean}	0,72	GPa
ρ_k	380	kg/m^3

$f_{m,d}$	17,28	MPa
$f_{t,0,d}$	13,07	MPa
$f_{t,90,d}$	0,29	MPa
$f_{c,0,d}$	17,28	MPa
$f_{c,90,d}$	1,94	MPa
$f_{v,d}$	1,94	MPa

And the design values for the KertoQ panels acting as deck for the bridge the design values are the ones shown in Table 6.2:

Table 6.2 Design values of strength for KertoQ panels

$f_{m,k}$	36	MPa
$f_{t,0,k}$	16,5	MPa
$f_{t,90,k}$	6	MPa
$f_{c,0,k}$	26	MPa
$f_{c,90,k}$	1,8	MPa
$f_{v,k}$	1,3	MPa
$E_{0,mean}$	10,5	GPa
$E_{0,05}$	8,8	GPa
$E_{90,mean}$	2,4	GPa
G_{mean}	6	GPa
ρ_k	480	kg/m^3

$f_{m,d}$	27,00	MPa
$f_{t,0,d}$	13,61	MPa
$f_{t,90,d}$	4,50	MPa
$f_{c,0,d}$	19,50	MPa
$f_{c,90,d}$	1,35	MPa
$f_{v,d}$	0,98	MPa

6.2 Loads acting on the bridge and loads combinations

The loads acting on the bridge are essentially three: the selfweight of the structural components, the live load given by the people that walk through the carriageway and the snow.

1. Selfweight

The structure is composed primarily of two lenticular glulam timber beams with a length of 22 m, two KertoQ panels 63 mm thick slightly longer (23 m), an 8 cm thick asphalt covering and then there are the wooden uprights that support the railing and the railing itself.

Assuming the foregoing values of weight per unit volume provided by EC1, Part 2-1, point 4.2 the data shown in Table 6.3 are obtained:

Table 6.3 Weights per unit volume of the structural elements composing the bridge

γ_{GL24h}	6	KN/m^3
γ_{KertoQ}	6	KN/m^3
$\gamma_{asphalt}$	23	kN/m^3
$\gamma_{solid\ timber}$	4	kN/m^3

Multiplying these values for the cross sections of the respective elements it is easy to get the characteristic weight per unit length for half of the bridge (considering an area of influence of half carriageway 1.25 m), that is:

$$g_k = 5{,}46 \ \frac{KN}{m}$$

2. Live load

The live load is a uniformly-distributed load that represents the people walking or staying upon the bridge. According to EC1, part 3, point 5.3.2.1 the magnitude of the uniformly distributed load is usually equal to 5 KN/m^2 but in case of pedestrian bridges over 10 meters long (the Fish Belly Bridge is 22 m long), unless otherwise specified, the following values must be taken:

$$2{,}5\ \frac{KN}{m^2} \leq q_{vk} = 2{,}0 + \frac{120}{L_{sj} + 30} \leq 5 \frac{KN}{m^2} \qquad \text{(Eq. 6.2)}$$

where L_{sj} is the length of the span.

Then applying Equation 6.2 and dividing the resultant by 1,25 m (width of the area of influence) we obtain the characteristic value per unit length of the crowd acting on a half of the structure:

$$q_{vk} = 5{,}38 \frac{KN}{m}$$

3. Snow load

On the basis of EC1, Part 2-3, point 5.1 is easy to see how the snow load can be calculated as follows:

$$s = \mu_i C_3 C_t s_k \qquad \text{(Eq. 6.3)}$$

where μ_i is the shape coefficient, C_e the exposure coefficient, C_t the thermal coefficient and s_k the characteristic value of snow on the ground.

The s_k value is easily achievable by EC1Appendix A.16 where one can see that it is equal to 2 KN/m^2 for Boras surroundings. The other coefficients are the ones suitable for monopitched roofs.
Finally the snow load acting on only one half of the bridge is:

$$q_{snow} = 2\frac{KN}{m}$$

As stated in EC1, part 3, point 5.3.2.3 the pedestrian bridge should also be designed to be able to carry the loads caused by a service vehicle used for several services, like maintenance, assistance for people (it could be an ambulance for example), or other kind of functions (for example a vehicle used to clean the roads from the snow) but the fact that the footbridge is going to fit inside a building makes the application of this load totally useless.

The load combinations that have to be adopted in order to carry on the verifications in the ULS is the fundamental combination:

$$\sum_{j>1}\gamma_{G,j}\cdot G_{k,j} + \gamma_P \cdot P + \gamma_Q \cdot Q_k + \sum_{i>1}\gamma_{Q,i}\cdot \psi_{0,i}\cdot Q_{k,i} \qquad \text{(Eq. 6.4)}$$

But, as specified in EC0/Amendement A1 Annex A2, point A2.2.3, the combination rules for footbridges say that the snow loads don't need to be combined with the other live loads (crowd in this case); so the load acting on the bridge is only due to the linear combination of selfweight and live load as follows:

$$q_d = \gamma_G g_k + \gamma_Q q_{vk} \qquad \text{(Eq. 6.5)}$$

where the partial factor γ_G is equal to 1,35 and the other one γ_Q is equal to 1,5.

The final design load acting then on only one half of the bridge is:

$$q_d = 15{,}45\frac{KN}{m}$$

6.3 Load cases

To design and to verify the structure in the ULS the load cases analyzed in the design of the Fish Belly Bridge have been three:

1. Design load uniformly distributed on the whole length of the bridge
2. Design load uniformly distributed on half length of the bridge
3. Design load uniformly distributed on a quarter of the length of the bridge.

The FEM software SAP 2000 has been used in order to gain the internal forces present in the structure depending on the different cases.

1. Load case 1

Load case 1 is shown in Figure 6.1:

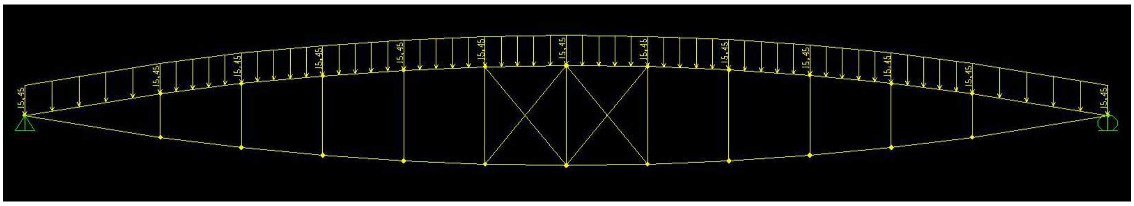


Figure 6.1 Load case 1 acting on the lenticular beam.

The internal actions that are produced by this load condition are shown in the figures below. In Figure 6.2 it is possible to see the axial forces diagram:

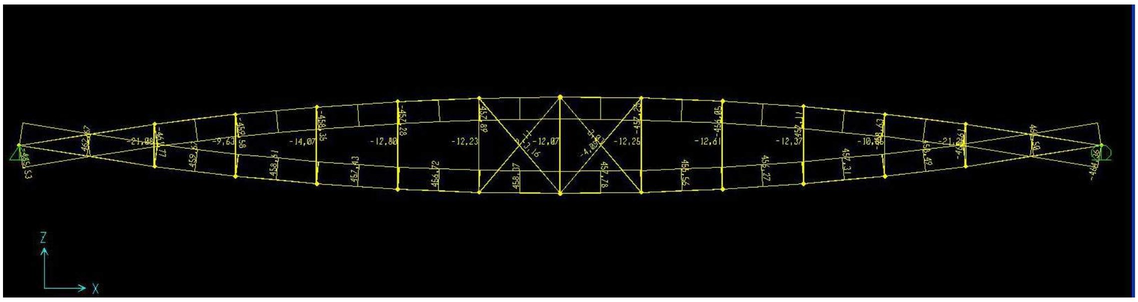


Figure 6.2 Axial forces produced in Load case 1.

As one can see the axial forces are approximately equal and constant along the length of the two chords that form the lenticular beam with maximum values of 465 KN in the lower one (tension) and 462 KN in the upper one (compression). The struts are minimally compressed and the maximum value is reported to be 21 KN for those closest to the supports. The steel cables are practically unstressed.
In Figure 6.3 it is shown the bending moment diagram:

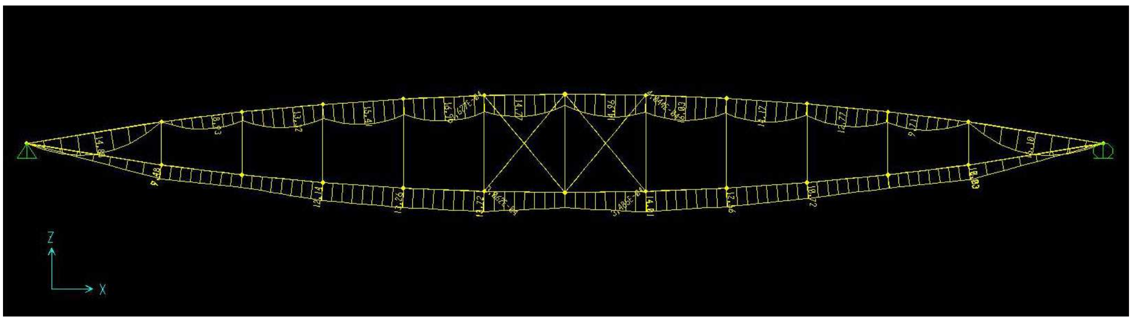


Figure 6.3 Bending moments produced by Load case 1.

Also concerning the results produced by SAP 2000 it is easy to see that the bending stresses are really low and almost constant. The maximum value in the lower chord is 14 KNm near the center of the bridge and in the upper one the highest value is 16 KNm. Both in the lower and in the upper chord bending moment are positive.
In Figure 6.4 it is possible to see the shear diagram:

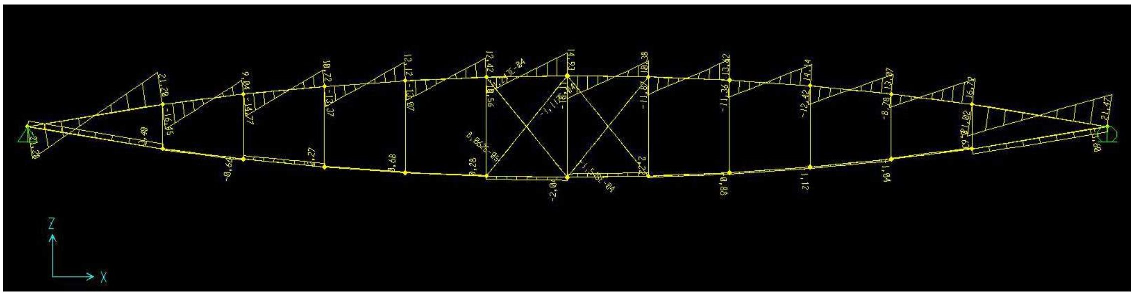


Figure 6.4 Shear stresses produced by Load case 1.

2. Load case 2

Load case 2 is shown in Figure 6.5 and it is possible to see how only half of the span of the bridge has been loaded, in order to simulate the crowd moving.

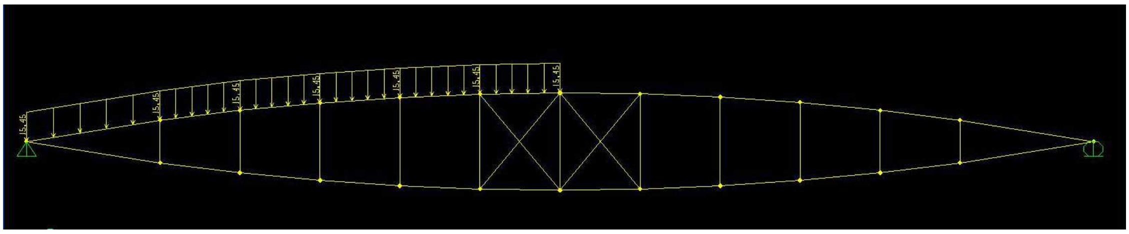


Figure 6.5 Load case 2 acting on the lenticular beam.

The loads in this situation produce different axial stresses depending on where one analyzes (Figure 6.6). The general behaviour remains unchanged with the upper chord subjected to compression and the lower on the contrary tense. The values are roughly symmetrical with the maximum compression value of 115 KN and maximum tension value equal to 112 KN. The struts on the right near the roller pin are unstressed and the higher compression value is in the strut near the hinge (20 KN); steel cables are solicited with tensile stresses of around 40 KN.

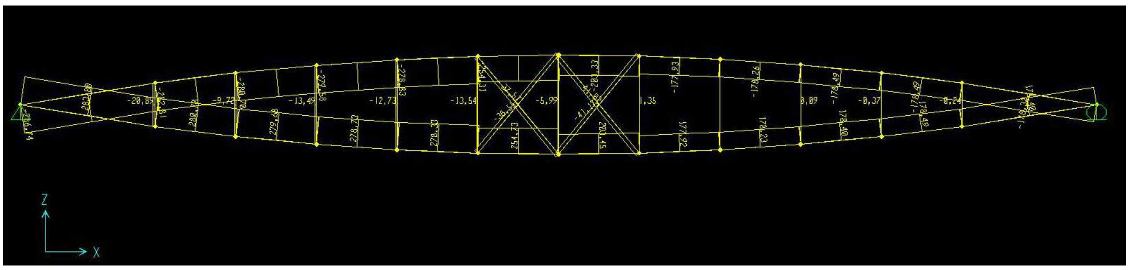


Figure 6.6 Axial forces produced in Load case 2.

The bending moment diagram can be seen in Figure 6.7. In this situation bending moments as well as being higher than in the previous case (the maximum value is around 37 KNm in both the chords) there are also changes in sign due to the asymmetry of loads: in the areas where there is this inversion there is the risk that tension perpendicular to the grain takes place and verifications to combined bending and axial compression for the upper chord and the opposite for the lower chord, or verification to combined bending and axial tension should be done (see Paragraph 6.4).

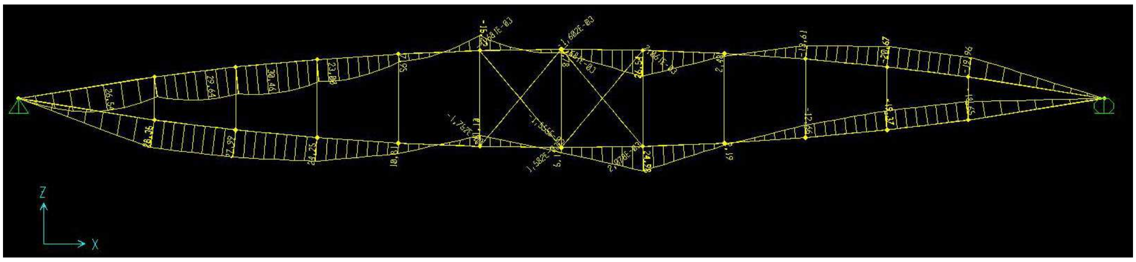


Figure 6.7 Bending moments produced by Load case 2.

In Figure 6.8 it is shown the shear diagram related to load case 2:

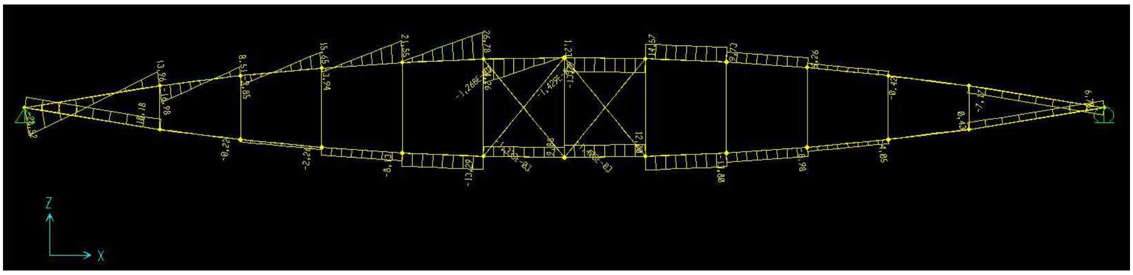


Figure 6.8 Shear stresses produced by Load case 2.

3. Load case 3

In Figure 6.9 it is shown the Load case 3 where only a quarter of the span of the bridge is loaded by the crowd.

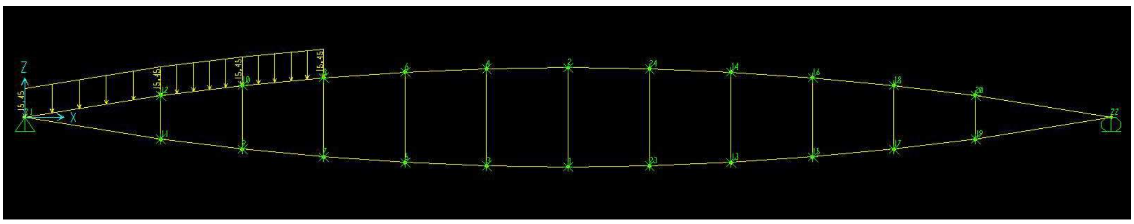


Figure 6.9 Load case 3 acting on the bridge.

It is possible to observe how the axial actions are always symmetrical about the longitudinal axis of the footbridge and how they obviously steadily diminish towards the not loaded edge. In the upper beams stresses have values of about 115 KN (both compression and tension) and also in this case the strut near the hinge is subjected to a compressive force of 20 KN while most of others do not even work. The steel cables work in tension and the most stressed bear a force of about 26 KN.

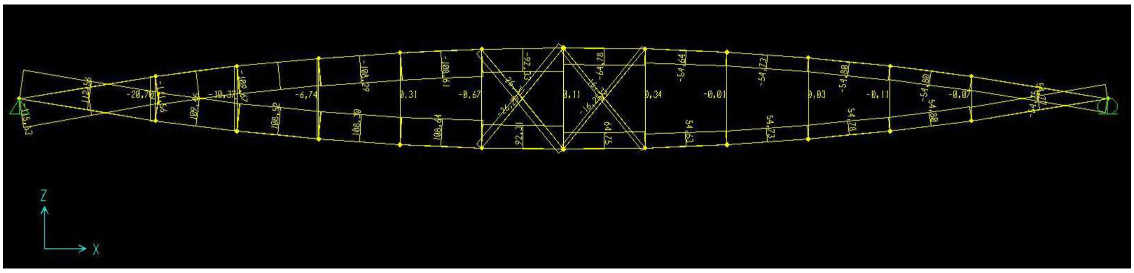


Figure 6.10 Axial forces produced in Load case 3.

Concerning the bending moment diagram (Figure 6.11) there is nothing special to emphasize except that the maximum values for the stresses are approximately equal to those obtained with the Load case 2. The other half of the bridge is really little stressed.

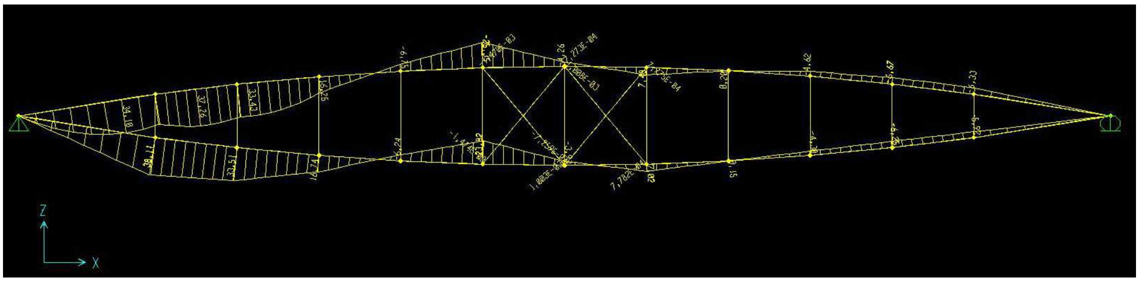


Figure 6.11 Bending moments produced by Load case 3.

In Figure 6.12 the shear diagram is shown:

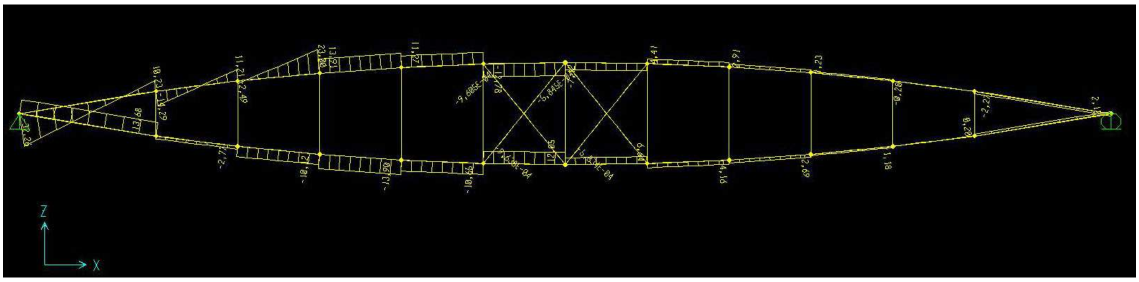


Figure 6.12 Shear stresses produced by Load case 3.

6.4 Stress verifications in the ULS

In this Paragraph verifications in the ULS will be carried out for the main structural elements, which are the upper and lower chord and the struts, to see if the cross

sections and the dimensions taken are enough to ensure the structural safety of the footbridge.
Here are therefore reported the verifications based on EC5 guidelines for the design of timber structures, and more precisely: tension and compression parallel to the grain for the lower and upper chord respectively, bending for both the chords, combined bending and axial tension for the lower chord, combined bending and axial compression for the upper chord, column stability for the studs and beam stability for the last segment of the upper chord.

In Table 6.4 cross sections of the various structural elements are given:

Table 6.4 Cross sections of the main structural elements composing the bridge

Upper chord	230 x 450	mm
Lower chord	215 x 450	mm
Studs	215x140	mm

Working the structure mainly in of tension and compression the first verifications to be done are obviously those involving the resistance of the elements in tension and compression parallel to the grain.

6.4.1 Compression and tension parallel to the grain

The axial force of compression to use for testing the upper chord is that derived from Load case 1, that is N_d = 461 KN.
The following expression should be satisfied:

$$\sigma_{c,0,d} \leq f_{c,0,d} \qquad \text{(Eq. 6.6)}$$

Where

$\sigma_{c,0,d} = 4{,}46$ MPa (design compressive stress along the grain)
$f_{c,0,d} = 17{,}28$ MPa (design compressive strength along the grain)

The check is fulfilled.

Concerning the tensile force to adopt in the verification of tension parallel to the grain it is obtained again from Load case 1, that is N_d = 464 KN.

The following expression should be satisfied:

$$\sigma_{t,0,d} \leq f_{t,0,d} \qquad \text{(Eq. 6.7)}$$

Where

$\sigma_{t,0,d}$ = 4,80 MPa (design tensile stress along the grain)
$f_{t,0,d}$ = 13,07 MPa (design tensile strength along the grain)

The check is fulfilled.

With regard to the studs instead the Load case that leads to the situation of greatest stress is the number 1 as the N_d = 21 KN for the elements on the edges near the supports. In this case:

$\sigma_{c,0,d}$ = 0,70 MPa
$f_{c,0,d}$ = 17,28 MPa

The check is fulfilled.

6.4.2 Bending

The design moments M_d to use in this situation are obtained from Load case 3 and they are equal to:

$M_{d,\text{upper chord}}$ = 34,31 KNm
$M_{d,\text{lower chord}}$ = 35,86 KNm

And the following expression should be satisfied:

$$k_m \frac{\sigma_{m,d}}{f_{m,d}} \leq 1 \qquad \text{(Eq. 6.8)}$$

Where $\sigma_{m,d} = \frac{M_d}{W}$ is the design bending stress and their values can be seen in Table 6.5:

Table 6.4 Design bending stresses and design bending strength for upper and lower chord.

	M_d (Nmm)	W (mm^3)	$\sigma_{m,d}$ (Mpa)	k_m	$f_{m,d}$ (Mpa)
Upper chord	34310000	7762500	4,42	0,7	17,28
Lower chord	35860000	7256250	4,94	0,7	17,28

And both the checks are fulfilled.

6.4.3 Combined bending and axial tension

This verification has to be carried out for the lower chord with the actions found in the Load case 1.
The following expression should be satisfied:

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,d}}{f_{m,d}} \leq 1 \qquad \text{(Eq. 6.9)}$$

Where:

Table 6.5 Design stresses and strengths for the lower chord in Load case 1.

$\sigma_{t,0,d}$	4,80	MPa
$f_{t,0,d}$	13,07	MPa
$\sigma_{m,d}$	2,02	MPa
$f_{m,d}$	17,28	MPa
k_m	0,7	

The check is fulfilled.

6.4.4 Combined bending and axial compression

This verification has to be carried out for the upper chord with the actions found in the Load case 1.
The following expression should be satisfied:

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m \frac{\sigma_{m,d}}{f_{m,d}} \leq 1 \qquad \text{(Eq. 6.10)}$$

Where:

Table 6.6 Design stresses and strengths for the upper chord in Load case 1.

$\sigma_{c,0,d}$	4,46	MPa
$f_{c,0,d}$	17,28	MPa
$\sigma_{m,d}$	2,18	MPa
$f_{m,d}$	17,28	MPa
k_m	0,7	

The check is fulfilled.

6.4.5 Stud subjected to compression

This verification will be carried on based on the compressive design force obtained from Load case 1 (see Paragraph 6.4.1). In this case $E_{0,05}$ should be used.

$$\lambda_{rel,y} = 0{,}12$$
$$\lambda_{rel,z} = 0{,}18$$

are the slenderness ratios corresponding to bending about the y and z-axis. Where they both are minor than 0,3 they should satisfy the expressions:

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 \leq 1 \qquad \text{(Eq. 6.11)}$$

And the check is fulfilled because furthermore it was already fulfilled in Paragraph 6.4.1.

6.4.6 Beam subjected to combined bending and compression

This verification has been done on the last segment of the upper beam near the supports, in Load case 1.
The length of the segment is 2,78 m.

Table 6.7 Values and coefficients in order to obtain $k_{c,z}$

$l_{0,z}$	2780	mm
i_z	66,40	mm
λ_z	41,87	
$\lambda_{rel,z}$	0,67	
β_c	0,1	
k_z	0,75	
$k_{c,z}$	0,94	

Once found out $k_{c,z}$ value the stresses should satisfy the following expression if there is a combination of moment M_d and compressive force N_d:

$$\left(\frac{\sigma_{m,d}}{k_{crit}\, f_{m,d}}\right)^2 + \frac{\sigma_{c,d}}{k_{c,z}\, f_{c,0,d}} \leq 1 \qquad \text{(Eq. 6.12)}$$

And with k_{crit}=1, the check is fulfilled.

6.5 Horizontal loads

The wind horizontal force (see Appendix C) is equal to:

Fw,d = 47,50 KN

and it can be applied 50% on the upper chord of the lenticular beam, where it will be opposed by the double KertoQ panel, and 50% on the lower chord, where it will be withstand by the transverse beams.
Taking as certain the stiffness of KertoQ panels and the adopted assembling system, we don't expect any need of checking the upper chord to the horizontal loads, and therefore only instability checks of the transverse beams will be carried out.

6.5.1 Tranverse beam subjected to compression

The cross section of the transverse beam is 140x140 mm and its length is 1,35 m. Each beam is subjected to a compressive force equal to 1,76 KN (the spacing between the elements is 1,65 m) so one can obtain:

$\sigma_{c,0,d}$ = 0,09 MPa

The relative slenderness ratios corresponding to bending about the y- and z-axis are respectively (they are actually identical because the cross section is square):

$$\lambda_{rel,y} = 0{,}54$$
$$\lambda_{rel,z} = 0{,}54$$

In both case $E_{0,05}$ should be used. The stresses should satisfy the expressions:

$$\frac{\sigma_{c,0,d}}{k_{c,y} \cdot f_{c,0,d}} \le 1 \qquad \text{(Eq. 6.13)}$$
$$\frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} \le 1 \qquad \text{(Eq. 6.14)}$$

where $k_{c,y} = k_{c,z}$ = 0,97. Both the checks are fulfilled.

6.6 Design in the SLS

6.6.1 Deflection of the lenticular beams

EC5, part 2 deals with only vibration problems concerning design in SLS for bridges. The aspects concerning deflection of the beams are treated in the general part 1-1 in sections 2.2.3 and 7.2.
In order to calculate the deflection of a timber beam the load combinations have to be different from how loads have been calculated in the ULS.
The instantaneous deformation u_{inst} should be calculated for the characteristic combination of action:

$$\sum_i G_{k,i} + Q_{k,i} + \sum_{j>1} \psi_{0,j} \cdot Q_{k,j} \qquad \text{(Eq. 6.15)}$$

while the final deformation u_{fin} should be calculated for the quasi-permanent combination of action:

$$\sum_i G_{k,i} + \sum_{j\geq 1} \psi_{2,j} \cdot Q_{k,j} \qquad \text{(Eq. 6.16)}$$

Where the partial coefficients γ_G and γ_Q are equal to 1 and $\psi_{2,j} = 0$ (Anon., 2002).

The expression to calculate the deformation for a simply supported and inflected beam, with a length of l, due to a load q is the following one:

$$u = \frac{5ql^4}{384E_{0,mean}J} + \chi\frac{ql^2}{8G_{mean}A} \qquad \text{(Eq. 6.17)}$$

Where it is clear that the mean values of the stiffness moduli $E_{0,mean}$ and G_{mean} should be used.

$E_{0,mean}$	11600	MPa
G_{mean}	720	MPa

To calculate the instantaneous deformation of one lenticular beam (supporting half bridge) practically the characteristic and the design loads are the same; so:

g_k	5,46	KN/m
q_k	5,38	KN/m

And the respective instantaneous deformations are:

$$u_{inst,G} = 10{,}49\,\text{mm}$$
$$u_{inst,Q} = 10{,}34\,\text{mm}$$

In order to calculate the final deformation u_{fin} instead, the mean values of the stiffness moduli have to be modified in the following way:

$$E_{mean,fin} = \frac{E_{mean}}{(1+k_{def})} \quad \text{and} \quad G_{mean,fin} = \frac{G_{mean}}{(1+k_{def})} \qquad \text{(Eq. 6.18)}$$

Where k_{def} is a coefficient dependent on the kind of timber and on the service class, in our case with glulam beams and in service class 2: $k_{def} = 0{,}8$.

Finally the final deformation u_{fin} can be calculated:

$$u_{fin,G} = 18{,}89\,\text{mm}$$

The recommended limiting values of deflections for beams are:

$u_{inst,Q}$	≤ l/300	73,33
$u_{fin,Q}$	≤ l/200	110,00
$u_{net,fin}$	≤ l/200	110,00

And the situations are verified.

6.6.2 Vibrations

Pedestrians' traffic represents a dynamic load that can cause vibrations of the bridge. If the eigen frequency of the bridge is close to the frequency of the dynamic load, movements in the structure are produced. The magnitude of these vertical displacements is increased by the effect of resonance, causing an unsafe and uncomforting feeling to the pedestrians. The bridge will not be considered a safe structure by the people that use it, even if the verifications in the ULS are largely satisfied. Hence, it is important to verify vibration.

In EC0, Annex A1it is specified that the acceleration for any part of the deck should not be higher than 0,7 m/s^2. Furthermore it is stated that a verification of the comfort criteria should be performed if the fundamental frequency of the deck is less than 5 Hz.
From the analysis of the bridge (see Appendix D) has resulted that the fundamental frequency (for the middle cross section) is equal to:

$$f_{vert} = \frac{\pi}{2l^2}\sqrt{\frac{EI}{m}} = 1{,}975 \text{ Hz} \qquad \text{(Eq. 6.19)}$$

So according to EC5 part 2, the acceleration from one person crossing the bridge is:

$$a_{vert,1} = 200/M\zeta = 2{,}63*10^{-6} \text{ m/s}^2$$

The acceleration from several person crossing the bridge:

$$a_{vert,n,distinctgroup} = 0{,}23 a_{vert,1} n k_{vert} = 6{,}30*10^{-6} \text{ m/s}^2$$
$$a_{vert,n,stream} = 0{,}23 a_{vert,1} n k_{vert} = 1{,}60*10^{-5} \text{ m/s}^2$$

And the acceleration from one person running on the bridge:

$$a_{vert,1} = 600/M\zeta = 7{,}90*10^{-6} \text{ m/s}^2.$$

7 Construction and maintenance of the Fish Belly Bridge

In this Chapter the aspects related to the construction site of the Fish Belly Bridge are analyzed, and the SCP (Safety and Coordination Plan, mandatory in Italy), the Maintenance Plan and the Building Booklet specifically written for the bridge designed are presented.
Essentially all of these scripts are required to ensure the maximum safety for all those people involved in the working stages both during the construction/installation of the bridge and during the subsequent maintenance phase of the bridge itself.
The following documents have been produced by Law no. 81 dated 9/4/2008 (Health and Safety at Work) which is in fact the law in force in Italy in the field of occupational safety.
It should be emphasized that in this Thesis work the main goal has been to give greater importance to the intrinsic contents required by Law rather than to the formal aspects, and therefore the directions given by the Legislator in relation to the preparation of that documentation have not been fully complied with but it has been individually adapted to the purposes and the scope which this work has as its final aim.

The SCP must be compulsorily completed by the Coordinator for the design phase and it must contain essentially: a description of the work and of the building site and its organization, the identification, analysis and evaluation of specific risks related to individual work phases that make up the workmanships required to implement the project, and the preventative and protective measures that have to be taken (Anon., 2008).
This document must be specific to each individual building site, operationally feasible and especially consistent with the design choices.
In this Thesis work the Author has left out the drafting of the forms concerning machineries and equipments to be used during the different phases as it is not of primary interest for the analysis conducted in this work.

The Maintenance Plan, as codified by the Presidential Decree 554 dated 21/12/1999 regarding the management of public works, is a complementary document to the executive project that foresees, plans, according to project documents that have actually been executed, the maintenance activity of the intervention in order to preserve the functionality, characteristics of quality, efficiency and economic value of the work (Anon., 1999).
It is divided into three sections: user manual, maintenance manual and provision plan. Specifically the last two documents have been drafted, planning the system of checks and interventions to be carried on, to set deadlines, in order to ensure the proper management of the footbridge and integrating it with the necessary information for appropriate maintenance of the main parts that make up the work.

Finally, there is the Building Booklet which is a document always drafted up by the Coordinator for the design phase in case of public works, closely related to the Maintenance Plan and that contains useful information about the prevention and protection from the risks that workers are exposed to during the course of subsequent workmanships that take place after the completion of the work. It accompanies the

building for its entire service life and must be produced during the design or at least during the execution stage of the work.
In this Thesis work, given the strong interdependence between the two documents, an integration of the Maintenance Plan and the Building Booklet has been carried on in order to make possible a more immediate understanding of the simultaneity of operations, risks and safety procedures.

7.1 Safety and Coordination Plan (SCP)

This document aims at safeguarding the health and safety of all workers who work in the building site. The contents of the paper with its annexes are the Safety and Coordination Plan as required by Law no. 81 dated 9/4/2008, art. no. 100.
This SCP, in order to be previously effective, has been composed to be:

- *specific*: that is thought specifically to carry out the work to which it relates. The specificity of the document is highlighted by the technical, design, architectural and technological choices and by the layouts of the site

- *readable/accessible*: that is written in an understandable way to be well received by the building contractors, the employees of the building contractors, the self-employed and the client as well.

This document should be used as a guide by all those subjects involved in the organizational system of security to apply at the best all the measures that have to be taken during the various workmanships in relation to the risk factors actually present.
Everybody will be held in full compliance with and enforcement of the safety measures listed in the following Plan.
The measures, the personal protective equipment and the safety measures are: strictly required, to be implemented properly and continuously, to be observed personally.

The analysis of the working phases needed to achieve the Fish Belly Bridge assembly is structured as follows:

1. arrangement of the building area
2. trench works
3. construction of the abutments in reinforced concrete
4. positioning of the footbridge complete with parapets and bracing system in the factory on the lorry with mobile crane
5. transport of the footbridge on site
6. launching with mobile crane and fixing of the footbridge to the supports
7. waterproofing and asphalting
8. dismantling and closing down of the building site.

NOTE: the pictures reproduced in the following paragraphs and used to better illustrate the concepts explained in words are not related to the Fish Belly Bridge of this case, but to several other wooden bridges built by Eng. Roberto Crocetti during his activity.

7.1.1 Arrangement of the building area

The present stage is divided in turn into four sub-phases namely: cleaning and fencing of the area, placement of adequate traffic signs, installation of the barracks and finally the connection facilities (water and electricity) for the works to be done on site and to serve the barracks.

WORKING PHASE	SHEET PHASE F1
	ARRANGEMENT OF THE BUILDING AREA
Description of works	The phase includes: cleaning and fencing of the area, placement of adequate traffic signs, installation of the barracks and finally the connection facilities (water and electricity) for the workmanships to be done on site and to serve the barracks.
Risks	Falls from height, cuts and abrasions, accident, injuries from manual handling of loads, interference machineries/ workers.
Security measures and coordination	Particular attention must be paid to properly signal the accesses to the building site to prevent entry of unauthorized personnel. Drivers of vehicles will be assisted by a person on the ground during maneuvers in tight spaces or when visibility is incomplete. The vehicles must move at low speed in the area, and to a crawl near the workplaces. Personnel on the ground must be outside the scope of the vehicles. It is not allowed to transport people on mechanical vehicles except for the driver. It is required to maintain, for each worker, personal protective equipment supplied in perfect conditions so that they can provide effective protection from specific risks present in the various construction phases of work performed (gloves, shoes, helmet, etc.).
Machineries and equipment	Truck, mobile crane, skid steer loader, wheelbarrow, hand ladders, scaffoldings, hand tools.
Risk assessment	The level of risk related is average.

7.1.2 Trench works

Following the preparation of the area one will proceed with the trench works, starting with the tracings. Great care must be taken during the execution of the tracings that are included in those topographic applications needed to identify and demarcate on the ground the concepts designed on the map: they are operations that must be completed with particular precision, such as to leave no doubt in the staking of points that contribute to the definitions of shapes and elements that might be useful. The footbridge, being in fact completely prefabricated, requires that the size and position of the abutments and supports are very precise so that it can be installed without a hitch.
Then the excavation by machine, the installation of supporting structures of the walls of the trench and finally the enclosure of the trench will take place.

WORKING PHASE	SHEET PHASE F2
	TRENCH WORKS
Description of works	the phase includes: tracings followed by the excavation done by machine, then it continues with the installation of supporting structures of the walls of the excavation. It concludes with the fence of the trench.
Risks	Falls from height, cuts and abrasions, acoustic lesions, material falling from above, investment, injuries from manual handling of loads, dust, interference machineries / workers.
Security measures and coordination	The arrangement of a wooden fence with a minimum height of 1 m around the perimeter of the area affected by the excavation is expected. Any storage of materials and equipment must be stable and positioned at a distance of at least 2.5 m from the edge of the excavation. The operational vehicles moving near the edge of the excavation must be kept a distance from the edge itself of at least 2.5 m. Personnel on the ground must be outside the scope of the vehicles. Prohibition of circulation in the area affected by mining operations for workers not involved.
Machineries and equipment	Truck, mechanical excavator, circular saw, hand tools
Risk assessment	The level of risk related is average.

7.1.3 Construction of the abutments in reinforced concrete

To carry out the abutments in reinforced concrete it is first necessary to create the wooden formwork, after which the application of releasing agents will take place inside them. The installation of reinforcement bars will follow. Once positioned the scaffolding it will be possible to cast the concrete with a mixer truck and to constipate it with an electrical vibrator and in the end, as it matures, the disarmament will take place.

Figure 7.1 Details concerning the abutments in reinforced concrete before the positioning of the footbridge (Crocetti, 2010).

WORKING PHASE	SHEET PHASE F3
	CONSTRUCTION OF ABUTMENTS IN REINFORCED CONCRETE
Description of works	The phase includes: construction of the wooden formworks, application of the releasing agents, the installation of reinforced bars, casting of the concrete with a mixer truck, constipation of the casting with electric vibrator and finally disarmament.
Risks	Knocks and bumps, cuts and abrasions, electrocution, material falling from above, injuries from manual handling of loads, castings and sketches, allergens, interference machineries/workers.
Security measures and coordination	Drivers of vehicles must be assisted by a person on the ground while maneuvering in reverse. All protruding reinforcement rods must be folded or protected by special caps. The formwork must be adequately shored, that it does not collapse. It is required to maintain, for each worker, personal protective equipment supplied in perfect conditions so that they can provide effective protection from specific risks present in the various construction phases of work performed.
Machineries and equipment	Mixer truck, mobile crane, hand pump for releasing agents, circular saw, hand tools, electric vibrator for concrete.
Risk assessment	The level of risk related is average.

7.1.4 Positioning of the footbridge complete with parapets and bracing system in the factory on the lorry with mobile crane

This step simply expects that the footbridge entirely prefabricated, just leaving the factory, is loaded on a lorry to be transported to the site where it will be mounted. To do this a mobile crane will be used.

Figure 7.2 Positioning of the footbridge over the lorry by means of a mobile crane (Crocetti, 2010).

WORKING PHASE	SHEET PHASE F4
	POSITIONING OF THE FOOTBRIDGE ON THE LORRY
Description of works	During this phase the positioning of the footbridge complete in all its parts (railings, bracing system) on the lorry takes place. Afterwards it will be moved to the building site.
Risks	Knocks and bumps, cuts and abrasions, materials falling from above, injuries from manual handling of loads, interference machineries/workers.
Security measures and coordination	The placement of the footbridge will be done through the use of a mobile crane. The area of operations must be properly demarcated and marked with warning signs and no one has to pass in it. Before use brakes, rotating beacon and acoustic devices should be checked. To avoid overturning the gradients, the presence of deep holes and lift must be verified. The passage of suspended loads over workers during lifting should be avoided: if this precaution cannot be observed it must report the ongoing operation to allow the evacuation of people in the affected area. It is not allowed to transport people on mechanical vehicles except for the driver. It is required to maintain, for each worker, personal protective equipment supplied in perfect conditions so that they can provide effective protection from specific risks present in the various construction phases of work performed. Operators on the ground must accompany the load with ropes during the lifting phase.
Machineries and equipment	Lorry, mobile crane, hand tools.
Risk assessment	Operation at high risk.

7.1.5 Transport of the footbridge on site

A crucial phase in which to pay particular attention is represented by the transport of the footbridge from the factory to the building site where it will be mounted.
Since the artifact is about 22 m long, it is classified as an exceptional transport, governed by Law no. 285 dated 30/04/1992, articles 10, 61 and 62 (New Highway Code) (Anon., 1992). The Law therefore provides that it must be placed on a lorry and the vehicle movement can take place only during daylight hours and without any particular escort.
Exceptional transports and vehicles are subjected to specific permissions for circulation, issued by the owner or dealer for highways, national and military roads and by Regions for the remaining road networks.
This type of transport requires a long and careful planning with regard to the study of paths: in fact it must be verified that the roads have appropriate radii of curvature to allow easy movements of the lorry, that the slope and width of the road are always suitable and that along the way not to gather damage to bridges, power lines and buildings (penalty the payment of a compensation).
The securing of the cargo has to be performed carefully to avoid damage to vehicles and injuries to passengers and other road users and traffic jams caused by the loss of the cargo itself.

Figure 7.3 The footbridge placed on the lorry and directed towards the building site (Crocetti, 2010).

7.1.6 Launching with mobile crane and fixing of the footbridge to the supports

Once arrived on site, the footbridge will be moved with a mobile crane from the lorry to the correct position on the abutments and then it will be fixed to steel supports with metal eyelets. Navigation on the Viskan will be interdicted during the launching.

Figure 7.4 The footbridge during the lifting from the lorry towards the abutments (Crocetti, 2010).

WORKING PHASE	SHEET PHASE F6
	LAUNCHING AND FIXING OF THE FOOTBRIDGE
Description of works	This phase includes: the positioning of the footbridge in the right position with the mobile crane and then its fixing to the abutments by metal eyelets.
Risks	Knocks and bumps, cuts and abrasions, materials falling from above, injuries from manual handling of loads, interference machineries/workers.
Security measures and coordination	The placement of the footbridge will be done through the use of a mobile crane.The area of operations must be properly demarcated and marked with warning signs and no one has to pass in it. Before use brakes, rotating beacon and acoustic devices should be checked. To avoid overturning the gradients, the presence of deep holes and lift must be verified,and the vehicle should be situated away from the edge of any excavation and special bases shall be used to redistribute the load. The passage of suspended loads over workers during lifting should be avoided: if this precaution cannot be observed it must report the ongoing operation to allow the evacuation of people in the affected area. Obligation to provide maintenance of traffic routes and to avoid the storage of materials in the vicinity of excavations and in areas that could hinder the normal circulation. It is not allowed to transport people on mechanical vehicles except for the driver. It is required to maintain, for each worker, personal protective equipment supplied in perfect conditions so that they can provide effective protection from specific risks present in the various construction phases of work performed.
Machineries and equipment	Lorry, mobile crane, electric drill, hand tools.
Risk assessment	Operation at high risk.

Figure 7.5 Just before the positioning of the bridge the edges are protected with a rubber mat (Crocetti, 2010).

7.1.7 Waterproofing and asphalting

The footbridge, after being mounted, must be waterproofed with welded bitumen mats and covered with a layer of asphalt in order to achieve a walking surface resistant and protective for the bearing structure (see constructive details in Paragraph 5.5).

WORKING PHASE	SHEET PHASE F7
	WATERPROOFING AND ASPHALTING
Description of works	The phase includes: achievement of a waterproof layer by laying and welding bitumen mats and implementation of the walking surface in asphalt.
Risks	Vibrations, heat and fire, acoustic injuries, injuries from manual handling of loads, tar, smokes and fumes, interference machineries/workers.
Security measures and coordination	While the waterproofing will be done by placing mats of bitumen and welding them to the upper part of the deck of the bridge with the gas torch, the realization of the walking surface takes place putting down a traditional or high roughness antiskid asphalt wearing course (final thickness of the compressed layer about 3 cm).
Machineries and equipment	Gas torch, vibrating plate, hand tools.
Risk assessment	The level of risk associated is low.

Figure 7.6 The welded bitumen mats placed on the upper part of the deck just before the realization of the walking surface in asphalt (Crocetti, 2010).

Figure 7.6 The asphalting phase carried on with hand tools and the vibrating plate on the background (Crocetti, 2010).

7.1.8 Dismantling and closing down of the building site

Dismantling and closing down of the building site will take place only after the completion of all other working phases.

In particular, the sub-phases that will occur will be: cleaning of the area and removal of the fence, the closure of the electrical and hydraulic connections, removal of the barracks and removal of the traffic signs.

WORKING PHASE	**SHEET PHASE F8**
	DISMANTLING AND CLOSING DOWN OF THE BUILDING SITE
Description of works	The phase includes: cleaning of the area and dismantling of the fences, removal of barracks, removal of the traffic signs and closure of connections to water and electrical systems.
Risks	Knocks and bumps, cuts and abrasions, electrical shock, injuries from manual handling of loads, cargo handling hazards.
Security measures and coordination	Personnel on the ground supporting the drivers must remain outside the scope of the vehicles. It is not allowed to transport people on mechanical vehicles except for the driver. It is required to maintain, for each worker, personal protective equipment supplied in perfect conditions so that they can provide effective protection from specific risks present in the various construction phases of work performed. Particular care must be taken in the presence of unauthorized personnel in the working area surrounding the site and during transportation of the material outside the site.
Machineries and equipment	Truck, mobile crane, skid steer loader, hand tools.
Risk assessment	The level of risk associated is low.

7.2 Maintenance Plan and Building Booklet

The maintenance of a construction is intended to ensure its use, to maintain its economic value and to preserve its performance during its whole service life, promoting the prescriptive and technical adjustment.
The Maintenance Plan is the means by which the Property (the municipality of Borås) will relate to the building: it will describe the procedures in order to collect and record the information and to take actions to set up and organize in an efficient, both technically and economic, the maintenance service.
This document is the means by which the building contractor relates to the building under the management of a contract of programmed maintenance providing the technical operators the necessary instructions for a proper maintenance, using an appropriate technical language, and at the same time is the instrument that indicates a system of checks and actions to be executed at predetermined time intervals in order to ensure an efficient management of the building and its parts over the years.

The Maintenance Plan pursues the following objectives:

- technical and functional: establishing a system for the collection of "basic information" and for the bringing up to date of the "feedback information" as a result of the actions, which enables, through the implementation and constant updating of the "information system", to learn and maintain properly the construction and its parts; allowing the identification of the most appropriate maintenance strategies in relation to the characteristics of the construction; educating technical operators on the inspection and maintenance operations to be performed, ensuring a correct and efficient implementation of the actions.

- economic: optimizing the use of the building and extending the service life with the implementation of targeted maintenance interventions; achieving savings by reducing failures and the time of non-use of the construction; enabling a more efficient and more economical planning and organization of the maintenance service.

The Maintenance Plan for the Fish Belly Bridge can be articulated with checks that take place annually, every five, ten and twenty-five years.

During the annual checks:

- Abutments shall be checked once in spring time and once in autumn time and the vegetation shall be removed.
- Railings shall be checked and possibly repaired, if they are damaged.
- Mechanical fasteners shall be checked and bolts shall be tightened. Tightening should occur between September and October.
- Wearing parts at the sides and at the ends of the bridge shall be checked. Holes or small openings where water and/or debris can enter upon, are not accepted.

During the quinquennal checks:

- Paving and waterproof layer shall be checked. Possible damage shall immediately be retrofitted.
- Moisture content (MC) shall be measured on 15 different points in the underside of the deck and its mean value of MC should be lower than 18%. If MC in one or more points is considerably larger than MC mean there is a risk for leakage; if the leakage has already occurred, the waterproof membrane and the asphalt paving should be removed around the damaged point (within a distance of 2 m from the damaged zone). Then the deck should be dried and the waterproof membrane and asphalt paving restored.

During the decennary checks:

- Surface treatment (coating) of the elements of the bridge shall be checked by professional staff, who will decide if retrofitting or simple repainting will be applied.
- Damaged coating on steel parts shall be steel brushed and improved by means of zinc-based coating.

Finally after 25 years:

- the asphalt paving and waterproof membranes, bridge joints and wearing plates shall be removed and replaced with new ones.

For a better understanding and a rational management of the maintenance interventions it has been decided to divide the construction in sub-elements, namely:

bearing structure, parapets and abutments and for each the specific features of the intervention and the professional figures involved have been specified.

1 _ BEARING STRUCTURE				
1.1 _ DECK				
Worker	**Description of the action**	**Frequency**	**Risks**	**Security measures**
Specialized technician	Measurement of moisture content in 15 different points in lower part of the deck: the average value must be lower than 18%. If the value in some point exceeds the mean value there is the risk of leakage; if it is already done, asphalt and bitumen mat must be removed (within 2 m of the damaged area), the deck must be dried and new mat and asphalt should be placed.	Quinquennal	Falling from height	See sheet BB1
Painter	Checking the integrity of the protective layer of paint and, according to the conditions, proceeding to a local or total paint.	Decennary	No specific risk	
1.2 _ METAL FLASHINGS				
Worker	**Description of the action**	**Frequency**	**Risks**	**Security measures**
Tinsmith	Checking the conditions of the protective metal sheets. Holes or small openings where water and debris can enter are not accepted.	Annual	No specific risk	
Tinsmith	Removing all flashings and replacement with new items.	Every 25 years	Falling from height	See sheet BB1
1.3 _ COVERINGS				
Worker	**Description of the action**	**Frequency**	**Risks**	**Security measures**
Asphalter	Checking the conditions of the coatings (in particular the layer of asphalt). Holes or small openings where water and dirt can enter are not accepted.	Annual	No specific risk	
Asphalter, waterproofe r	Checking the asphalt paving and the waterproof layers. Any damage must be repaired immediately.	Quinquennal	See sheets IA1 and IA2 (Appendix D)	See sheets IA1 and IA2 (Appendix D)
Asphalter, waterproofe r	Removal of the asphalt layer and the waterproof layer and replacement with new ones.	Every 25 years	See sheets IA1 and IA2 (Appendix D)	See sheets IA1 and IA2 (Appendix D)
1.4 _ METAL FASTENERS				
Worker	**Description of the action**	**Frequency**	**Risks**	**Security measures**

Specialized technician	Check of all metal fasteners (screws, bolts, nails) and tightening of loose bolts. The tightening should take place between September and October. Removal of any damaged fastener.	Annual	Falling from height	See sheet BB1
Specialized technician	Removal of all metal fastener and replacement with new ones.	Every 25 years	Falling from height	See sheet BB1

2 _ PARAPETS				
2.1 _ WOODEN ELEMENTS				
Worker	**Description of the action**	**Frequency**	**Risks**	**Security measures**
Carpenter	Controllo dei parapetti in tutte le loro parti e riparazione in caso di danneggiamento.	Annual	Falling from height	See sheet BB2
Painter	Checking the integrity of the protective layer of paint and, according to the conditions, proceeding to a local or total paint.	Decennary	No specific risk	
2.2 _ METAL FASTENERS				
Worker	**Description of the action**	**Frequency**	**Risks**	**Security measures**
Specialized technician	Check of all metal fasteners (screws, bolts, nails) and tightening of loose bolts. The tightening should take place between September and October. Removal of any damaged fastener.	Annual	Falling from height	See sheet BB1
Worker	**Description of the action**	**Frequency**	**Risks**	**Security measures**
Specialized technician	Removal of all metal fastener and replacement with new ones.	Every 25 years	Falling from height	See sheet BB1

3 _ ABUTMENTS				
3.1 _ STEEL SUPPORTS				
Worker	**Description of the action**	**Frequency**	**Risks**	**Security measures**
Specialized technician	Control of conditions of the reinforced concrete abutments and of the zinc-coated steel supports of the footbridge, to be performed once in the spring and once in autumn. Removal of any vegetation.	Every 6 months	Falling from height	See sheet BB2
Worker	**Description of the action**	**Frequency**	**Risks**	**Security measures**

Smith	Removal by brushing of the old zinking and applying of a new layer of zinc-based coating.	Decennary	Falling from height	See sheet BB2

Then as already mentioned, given the close link between the Maintenance Plan and the Building Booklet, which contains the information related to the prevention and protection from risks which workers are exposed to during the execution of maintenance workmanships, it has been decided to integrate the two documents in order to make easier to understand the logical sequence of operations, risks and safety measures.
The sheets compiled for the Maintenance Plan, showing security measures to be implemented to prevent the specific risks identified, are therefore presented below:

RISKS DUE TO A FALL FROM HEIGHT DURING OPERATIONS ON THE OUTER SIDES OF THE BRIDGE
Sheet **BB1**
For all the workmanships and maintenance operations which require that operators would be operating on the outer side of the bridge (namely: measurement of the moisture content at the bottom of the deck, check, removal and replacement of flashings, metal fasteners and wooden studs) it is necessary to use a truck-mounted platform which has a minimum working outreach of 22 m. In this way it will be possible cover the entire length of the bridge by placing the vehicle on only one river bank. In the event that the basket has four sides closed the only safety measures that operators will adopt will be wearing the appropriate PPE (Personal Protection Equipment). In the case that the drum has one or more sides open of the personnel must wear safety belts made with the body harness device and fall protection device designed to limit the possible fall of no more than 1,5 m. Obviously the building contractor will be required to provide guidance to workers regarding the use and maintenance of the PPE.

RISKS DUE TO A FALL FROM HEIGHT DURING OPERATIONS ON THE INNER SIDES OF THE BRIDGE
Sheet **BB2**
For all the workmanships and maintenance operations which require that operators would be operating on the inner side of the bridge (replacement of parts of the parapet) with the risk of falling from a height into the water it is necessary that, in addition to PPE, workers use safety belts made with body harness device and fall protection device designed to limit the possible fall of no more than 1,5 m. The coupling of the belt will take place to metal rings fixed to the deck of the bridge and spaced at a distance of 2 m one from another. Obviously the building contractor will be required to provide guidance to workers regarding the use and maintenance of the PPE. In the event that the workers operate on the abutments (removal of vegetation, check of steel supports) for the coupling of the safety belts metal rings will be placed on the upper chords of the lenticular beams.

8 Final remarks

8.1 Conclusions

The theme of this Thesis has been a complete and accurate study of the design process that should be put into practice during the design and subsequent construction of any civil engineering structure.

The needs analysis and the resulting definition of the characteristics that the work must have, the understanding by the designer (or design team) of constraints and opportunities that the environmental context provides, and the realization of physical models for understanding the static behaviour of the structure are key points that allow the realization of projects fully satisfactory and adequate to the demands of the clients.

having addressed in this Thesis the issue concerning the design of a complex and at the same time simple structure as that of a footbridge provided the opportunity to apply to a practical case what usually is never taught and that is learned instead during the everyday work.

It has been moreover extremely satisfying to see how the proposal of an idea developed during the conceptual design process has ultimately transformed into a structure viable in reality and which, in particular, has significant static potential (its static behaviour is similar to that of an arch with a tie rod) combined with aesthetic qualities and an extreme structural simplicity that make it as product that fits the requirements made by the clients at the beginning of Competition.

The structure meets all verifications imposed by Eurocodes both for the Ultimate Limit State and for Serviceability Limit State and guarantees, if properly constructed and maintained, a long service life considering the extreme care taken in the design of constructive details.

8.2 Limitations

Since the contents of this Thesis are based on literary studies and direct analysis of a design process actually happened, it is obvious that they cannot be regarded as a fixed model to be followed but rather they constitute a possible example of modus operandi to reference in similar situations.

However, recommendations such as new documentation work, a further needs analysis (perhaps partly based on interviews with residents) and implementation of different technological or structural solutions would be a good starting point to new and completely different structures.

8.3 Future developments

Given the breadth of the spectrum that the design process of a complex structure such as a bridge deals, this Master's project covers only a part and for this further research should be made in order to give a complete answer to all the problems arisen during the design process itself. Below some hints concerning possible developments as subjects for future Master's Thesis are presented:

- this Master's Thesis has focused only on a wooden structure but it would be interesting to try to work out a solution which also includes other materials (for example, a composite timber-concrete or steel structure).

- Calculations and verifications according to EC5 have been based on the realization of a very simple model in SAP 2000, which works through the Finite Element Method. It might be of some interest to achieve a thorough analysis of the actual behaviour of the structure through more refined models and through the usage of other softwares (Abaqus for example).

- The design process described in this Thesis stops after having done the verification of structural elements and after the precise definition of constructive details: a possible and obvious prosecution of the work could get more into methods and phases regarding the erection of the bridge and the organization of the building site.

- Finally the Author suggests that a work similar to the one carried one in this Master's project can be done on a bridge model made built during the Competition, given the validity of some of the proposals.

9 References

Anon., (1999): *Decreto del Presidente della Repubblica 21 dicembre 1999, n. 554: regolamento d'attuazione della legge quadro in materia di lavori pubblici.*

Anon., (1992): *Decreto Legislativo 30 aprile 1992, n. 285: Nuovo Codice della Strada.*

Anon., (2008): *Decreto Legislativo 9 aprile 2008, n. 81: Testo Unico sulla salute e sicurezza sul lavoro.*

Anon., (2002): *Eurocode 0: Basis of structural design/Amendment A1 – Annex A2: Application to bridges* (EN 1990:2002/A1)

Anon., (1996): *Eurocode 1: Basis of design and actions on structures* Part 2-1: *Actions on structures* (EN 1991-2-1:1996).

Anon., (1997): *Eurocode 1: Basis of design and actions on structures* Part 2-4: *Wind actions* (EN 1991-2-4:1997).

Anon., (1998): *Eurocode 1: Basis of design and actions on structures Part 3*: *Traffic loads on bridges* (EN 1991-3:1998).

Anon. a, (2004): *Eurocode 5: Design of timber structures Part1-1: General-Common rules and rules for buildings* (EN 1995-1-1:2004).

Anon. b, (2004): *Eurocode 5: Design of timber structures Part 2: Bridges* (EN 1995-2:2004)

Barizza, S., (1986): Il ponte dell'Accademia: una storia veneziana. *Venetica*, July-December 1986, pp. 128-147.

Blass, H., (1995): *Timber engineering, Step 1*. Centrum Hout, Almere, The Netherlands.

Brüninghoff, H., (1993): The Essing Timber Bridge, Germany. *Structural Engineering International*, May 1993, pp. 70-72.

Conzett, J., (1997): The Traversina footbridge, Switzerland. *Structural Engineering International*, May 1997, pp. 92-94.

Crocetti, R., (2010): *Drawings and pictures of wooden footbridges*.

Dietrich, R., (1998): *Faszination Brücken: Baukunst, Technik, Geschichte*. Callwey, München, Germany.

Federation International du Beton (2005): *Guidelines for the design of footbridges*. FIB, Lausanne, Switzerland.

Funis, F., (2000): Il ponte ligneo sul Cismon e le altre tre invenzioni di Palladio. *Bollettino Ingegneri*. January 2000, pp. 7-18.

Goral, J., (2007): *Risk Management in the Conceptual Design Phase of Building Project.* Master's Thesis. Department of Civil and Environmental Engineering, Chalmers University of Technology, Publication no. 07:124, Göteborg, Sweden, 2007.

Herzog, T., (2004): *Timber construction manual.* Birkhäuser, Zurich, Switzerland.

Killer, J., (1985): *Die Werke der Baumeister Grubenmann*. Birkhäuser, Basel, Switzerland.

Kroll, E., Condoor, S., Jansson, D., (2001): *Innovative Conceptual Design*. Cambridge University Press, Cambridge, UK.

Maggi A., Navone N., (2002): *John Soane e i ponti in legno svizzeri : architettura e cultura tecnica da Palladio ai Grubenmann*. Centro Internazionale di Studi di Architettura Andrea Palladio, Vicenza, Italy.

Maggi A., Navone N., (2003): *John Soane and the wooden bridges of Switzerland : architecture and the culture of technology from Palladio to the Grubenmanns*. Centro Internazionale di Studi di Architettura Andrea Palladio, Vicenza, Italy.

Miozzi, E., (1933): Il ponte di legno sul Canal Grande a Venezia. *Annali dei lavori pubblici*, n. 5, 1933, pp. 431-442.

Nordic Timber Council (1997): *Competitiveness of timber bridges*. NTC, Stockholm, Sweden.

Paolillo, G., (1999): Il ponte di Traiano sul Danubio presso Drobeta nella Dacia Inferior. *Bollettino Ingegneri*, December 1999, pp. 8-10.

Piazza, M., (2007): *Strutture in legno: materiale, calcolo e progetto secondo le nuove normative europee*. Hoepli, Milano, Italy.

Polastri, A., (2006): *Pedestrian bridges with glulam beams and LVL deck.* Master's Thesis. Department of Civil and Environmental Engineering, Chalmers University of Technology, Publication no. 06:12, Göteborg, Sweden, 2006.

Populin, E., (1998): *Il ponte dell'Accademia a Venezia, 1843-1986*. Il Cardo, Venezia, Italy.

Puppi, L., (1999): *Andrea Palladio: opera completa*. Electa, Milano, Italy.

Ritter, M., (2005): *Timber bridges: design, construction, inspection and maintenance.* University Press of the Pacific, Honolulu, USA.

Ryall, M., Parke G., Harding J., (2000): *The manual of bridge engineering.* Thomas Telford Publishing, London, UK.

Tampone, G., (2000): Sulle caratteristiche strutturali ed esecutive dei ponti in legno di Palladio. *Bollettino Ingegneri*, January 2000, pp. 2-6.

Thelandersson S., Larsen H., (2003): *Timber Engineering*. John Wiley and Sons, Chicester, UK.

Appendix A: diagrams of internal actions found with SAP 2000®

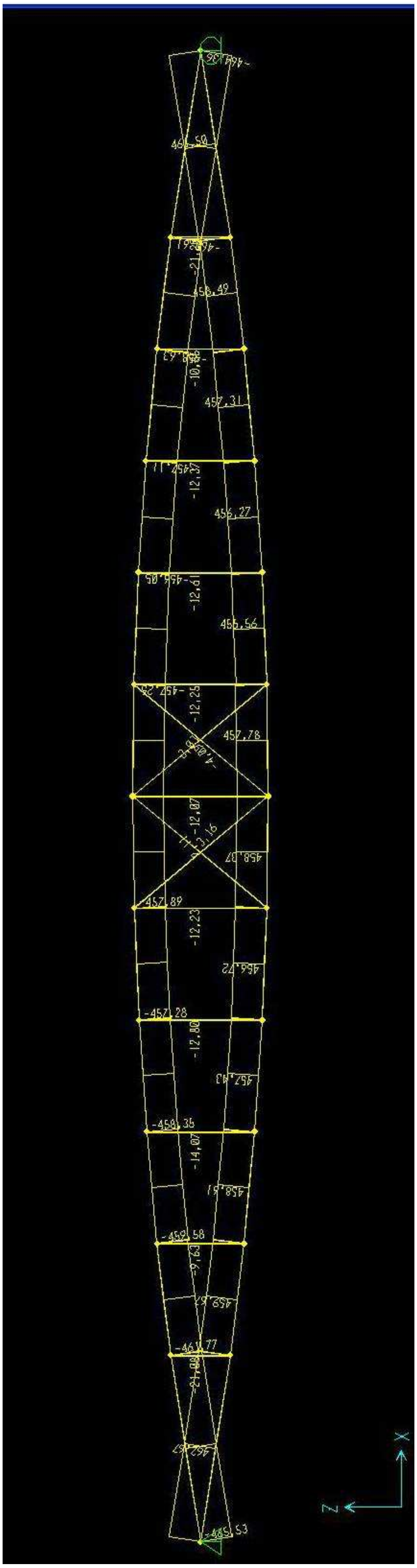
In this Appendix are put all the diagrams of the internal actions found using the software SAP 2000®.

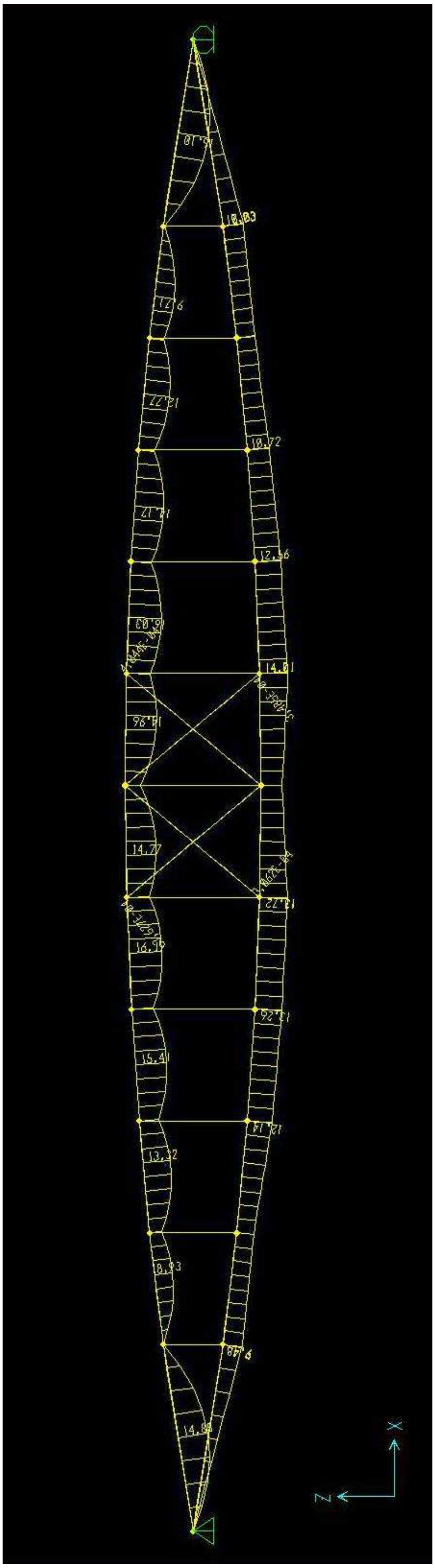
The Load cases are three:

1. Uniformly distributed load on the whole length of the bridge
2. Uniformly distributed load on on half length of the bridge
3. Uniformly distributed load on a quarter of the length of the bridge

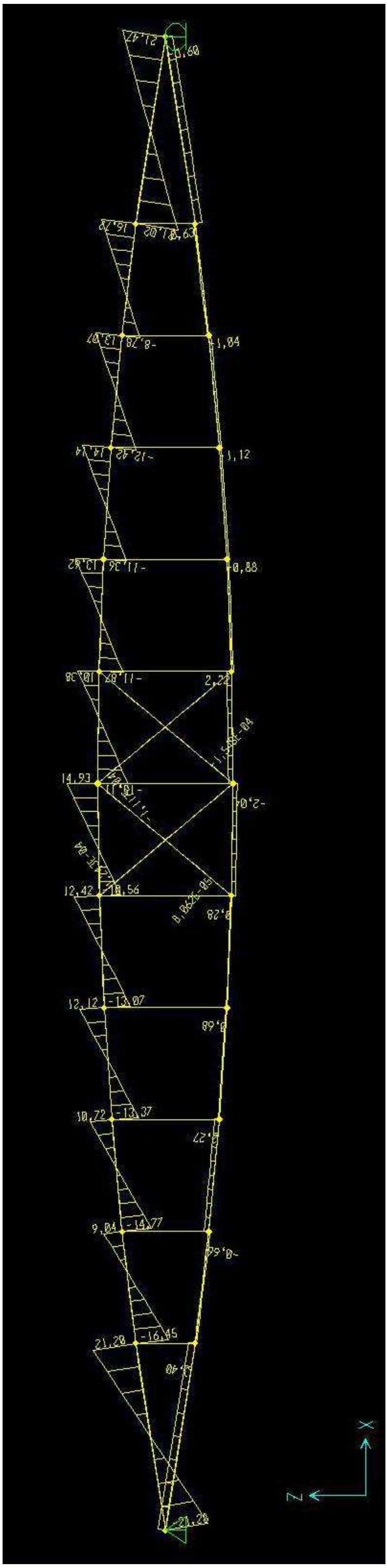
For every Load case diagrams of axial forces, bending moments, shear stresses and deformed shape will be shown.

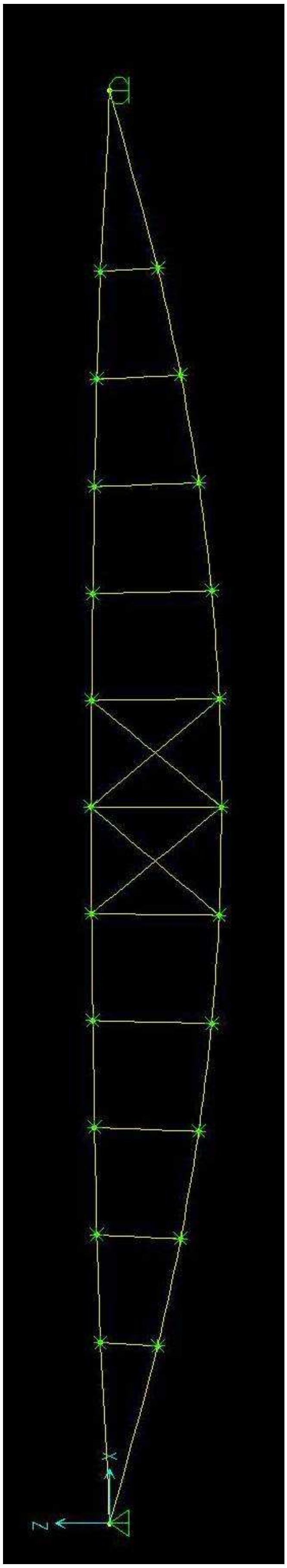
Load Case 1: Axial forces and bending moments



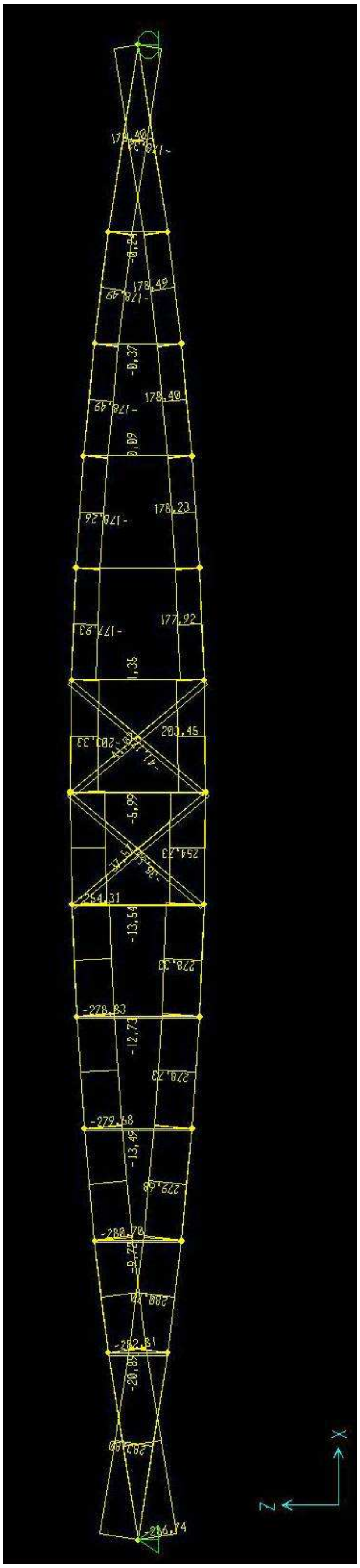


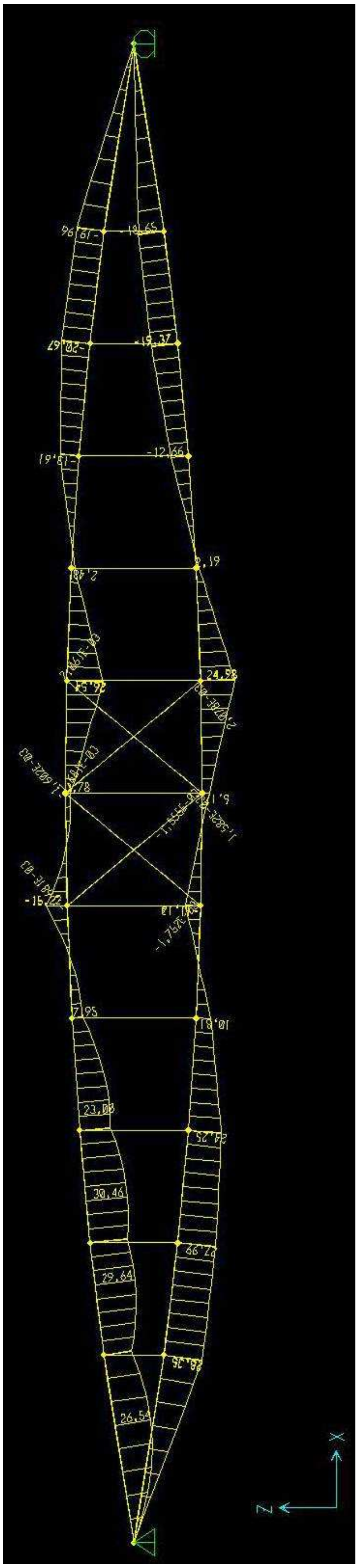
Load Case 1: shear stresses and deformed shape



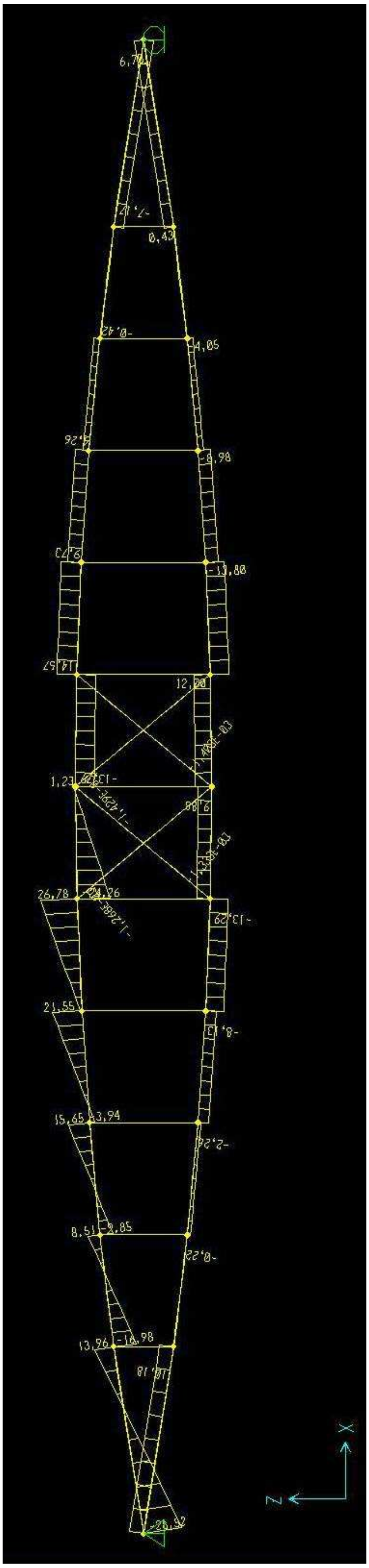


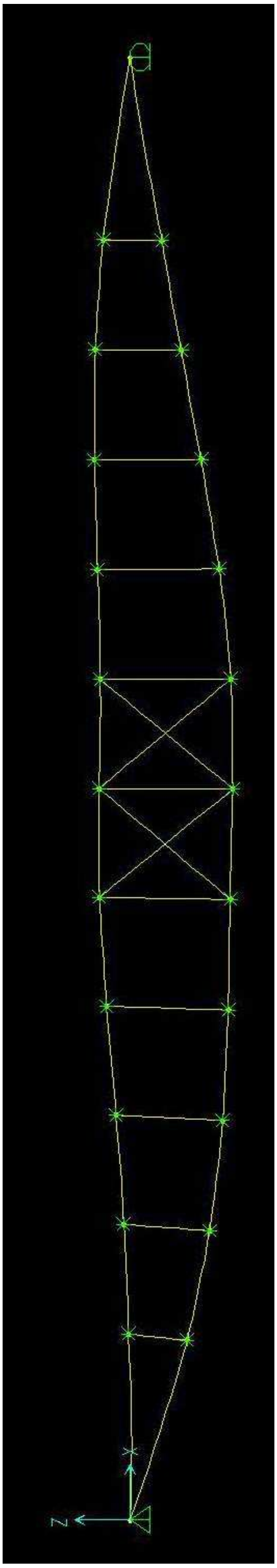
Load Case 2: Axial forces and bending moments



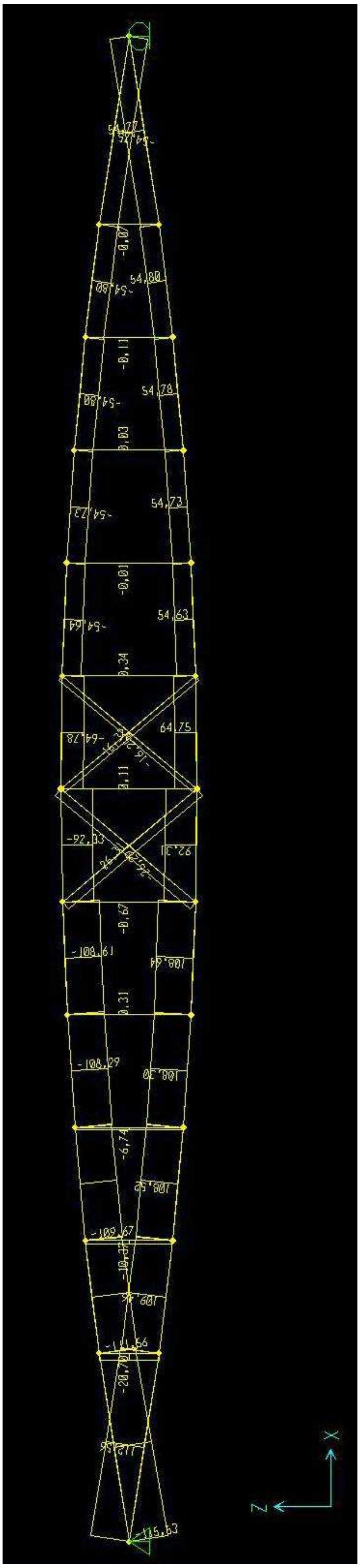


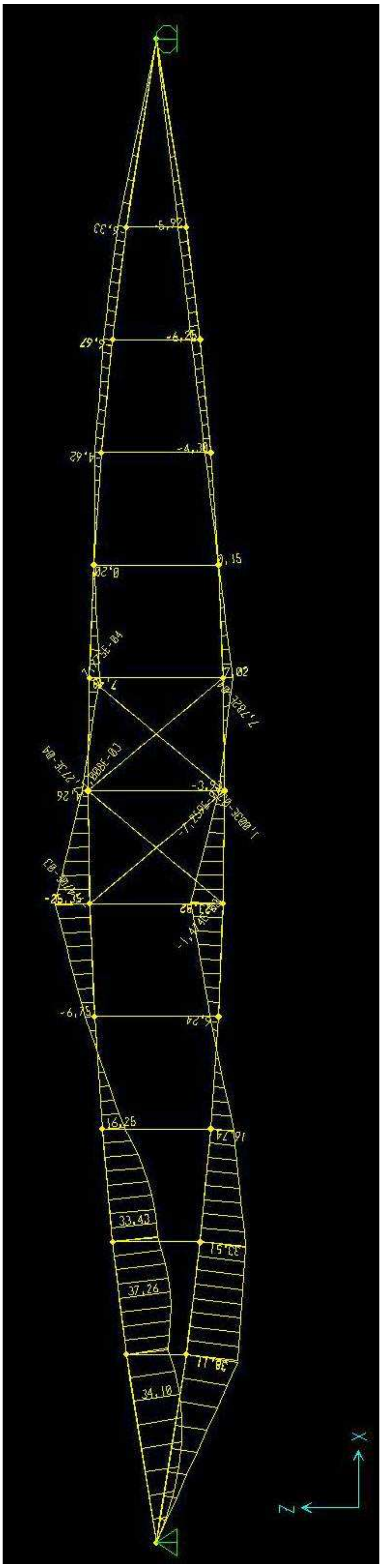
Load Case 2: shear stresses and deformed shape



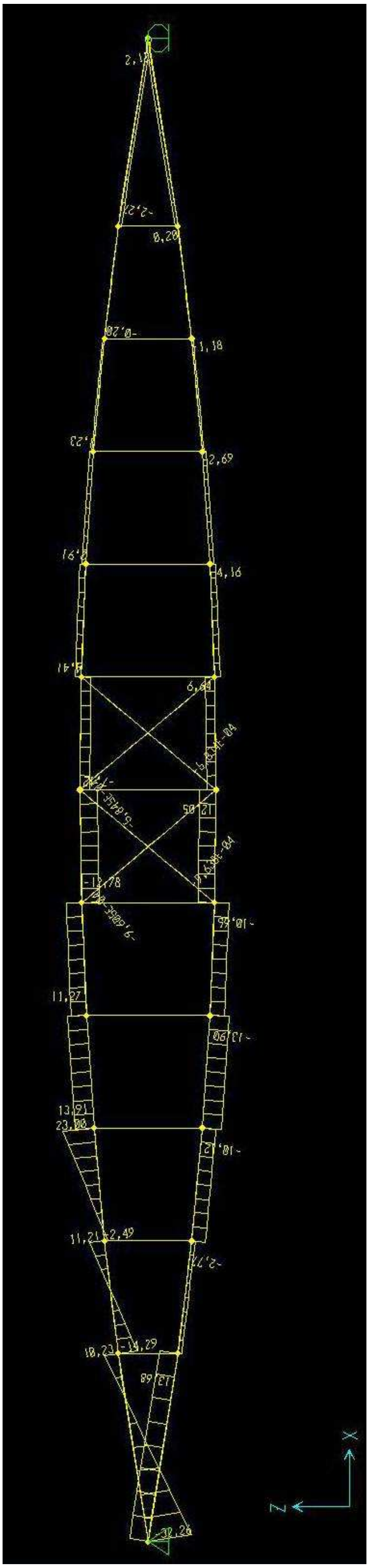


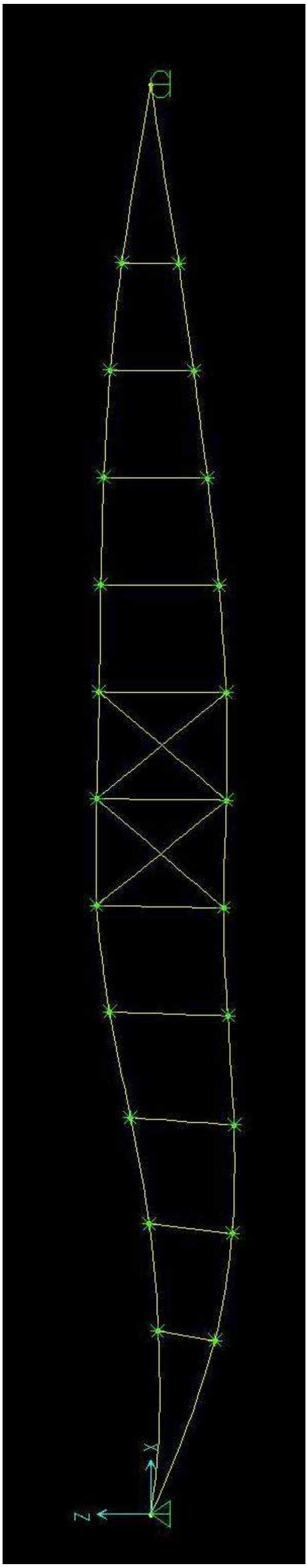
Load Case 3: Axial forces and bending moments





Load Case 3: shear stresses and deformed shape





Appendix B: Calculation of the connection between the vertical stud and the upper chord

The metal fastener used are bolts 460 mm long and with a diameter of 16 mm; the characteristic strengths are reported below:

d	16	mm
length	460	mm
$f_{u,k}$	400	MPa
$f_{y,k}$	320	MPa

According to EC5 Part 1-1 point 8.5.1.1 the minimum values of spacing and edge distance for bolts are:

α	90	
a_1	64	mm
a_2	64	mm
$a_{4,t}$	64	mm
$a_{4,c}$	48	mm

for the beam (compression perpendicular to the grain) and

α	0	
a_1	80	mm
a_2	64	mm
$a_{3,c}$	64	mm
$a_{4,t}$	48	mm
$a_{4,c}$	48	mm

for the stud (compression parallel to the grain).

The characteristic embedment strengths for the two elements are:

$$f_{h,90,k,beam} = \frac{f_{h,0,k}}{k_{90}\sin^2\alpha + \cos^2\alpha} = 16{,}46 MPa$$

$$f_{h,0,k,stud} = 0{,}082(1-0{,}01d)\rho_k = 26{,}17 \text{ MPa}$$

The characteristic value for the yield moment for bolts is:

$$M_{y,Rk} = 0{,}3 f_{u,k} d^{2,6} = 162141 Nmm$$

Then it is possible to calculate the characteristic load-carrying capacity for bolts that is:

$$F_{v;RK} = \begin{cases} f_{h,1,k} t_1 d = 30289{,}87N \\ 0{,}5 f_{h,2,k} t_2 d = 45019{,}97N \\ 1{,}05 \dfrac{f_{h,1,k} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \dfrac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} d t_1^2}} - \beta\right] + \dfrac{F_{ax,Rk}}{4} = 12929{,}97N \\ 1{,}15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk} f_{h,1,k} d} + \dfrac{F_{ax,Rk}}{4} = 11776{,}68N \end{cases}$$

So $F_{v,RK} = 11{,}76KN$.

The effective number of bolts for the two different elements are:

$n_{ef} = n = 1$ for the beam

$n_{ef} = 2{,}12$ for the stud

Finally it is possible to calculate the characteristic strength of the connection. Concerning the beam the R_k is:

$F_{v,Rk}$	11776,68	N
$R_{k,row} = 2 * F_{v,Rk} * n_{ef}$	23553,36	N
$R_{k,tot} = 3 * R_{k,fila}$	70660,09	N

While for the stud is:

$F_{v,Rk}$	11776,68	N
$R_{k,row} = 2 * F_{v,Rk} * n_{ef}$	49855,40	N
$R_{k,tot}$	49855,40	N

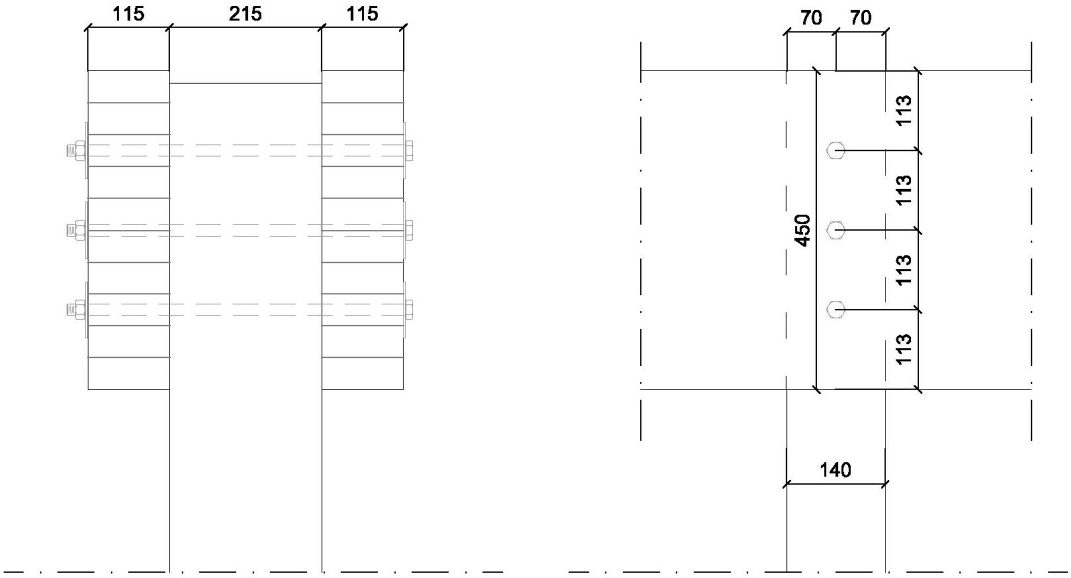
And between the two values the lower must be chosen. The design strength of the connection is:

$R_{d,connection} = 35{,}90$ KN (using $\gamma_M = 1{,}25$ and $k_{mod} = 0{,}9$)

While the compressive force acting on it is

$F_d = 21$ KN

So the connection is verified.


115
215
115
70
70
113
113
113
113
450
140

Appendix C: calculation of the resultant wind force

According to EC1 Part 2-4 Point 6.1 the resultant wind force F_W is obtainable from the following equation:

$$F_W = q_{ref} \cdot C_e(z_e) \cdot C_d \cdot C_f \cdot A_{ref}$$

Getting from Appendix A.16 the reference wind velocity for the area of Borås (v_{ref} = 26 m/s) and assuming the air density ρ equal to 1,25 kg/m^3, it is then possible to calculate q_{ref}, that is the reference mean pressure of the wind:

$$q_{ref} = \frac{\rho}{2} \cdot v_{ref}^2 = 16{,}25 \text{ N/m}^2$$

Being the bridge placed in the city centre the terrain category is the number IV and so it is possible to get the exposure coefficient C_e:

$C_e = 1{,}5$

Concerning C_d, the dynamic coefficient, it is easily findable at the Point 9.3 and its value, considering also the boundary conditions, is equal to:

$C_d = 0{,}95$

To get C_{fx} , the force coefficient in x-direction, it is necessary to calculate first the following data:

$C_{fx,0}$	2,3	
l	22	m
b	2,4	m

hence λ=9,17

A_c	29,98	m^2
A	18,64	m^2
φ	0,62	

hence ψ_λ=0,9

And finally:

$$C_{fx} = C_{fx,0} \cdot \psi_\lambda = 2{,}07$$

The reference area A_{ref}, using h_{ref} of 3 m, is equal to:

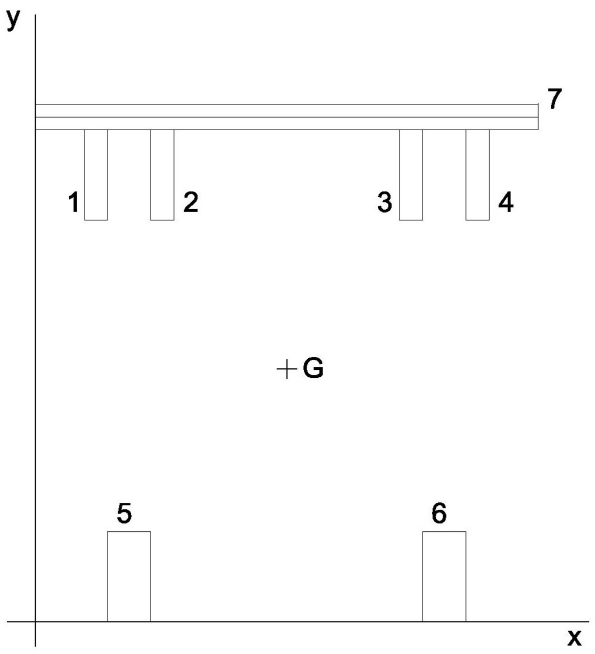
$$A_{ref} = 66 \text{ m}^2$$

In the end the resultant wind force value is:

$F_W = 31{,}64$ KN.

Appendix D: calculation of the fundamental frequency of the bridge

In order to be able to calculate the fundamental frequency of the bridge first of all the inertia of the cross section must be found.
As shown in the figure the cross section analyzed is the middle one.



The areas of the different elements are:

A_1	51750	mm^2
A_2	51750	mm^2
A_3	51750	mm^2
A_4	51750	mm^2
A_5	96750	mm^2
A_6	96750	mm^2
A_7	315000	mm^2
A_{TOT}	715500	mm^2

And so the static moments S_x and S_y are the following:

S_{x1}	115402500	mm^3
S_{x2}	115402500	mm^3
S_{x3}	115402500	mm^3
S_{x4}	115402500	mm^3
S_{x5}	22252500	mm^3
S_{x6}	22252500	mm^3

S_{x7}	790650000	mm^3
S_{xTOT}	1296765000	mm^3

S_{y1}	15525000	mm^3
S_{y2}	32602500	mm^3
S_{y3}	96772500	mm^3
S_{y4}	113850000	mm^3
S_{y5}	45472500	mm^3
S_{y6}	196402500	mm^3
S_{y7}	393750000	mm^3
S_{yTOT}	894375000	mm^3

It is possible then to find out the position of the G point (setting the values of S_x and S_y equal to zero):

x_G	1250	mm
y_G	1812	mm

The moments of inertia about the x-axis are:

J_{x1}	873281250	mm^4
J_{x2}	873281250	mm^4
J_{x3}	873281250	mm^4
J_{x4}	873281250	mm^4
J_{x5}	1632656250	mm^4
J_{x6}	1632656250	mm^4
J_{x7}	416745000	mm^4
$J_{x,TOT}$	6,84617E+11	mm^4

And about the G point-axis then become:

$J_{x1,G}$	9700227000	mm^4
$J_{x2,G}$	9700227000	mm^4
$J_{x3,G}$	9700227000	mm^4
$J_{x4,G}$	9700227000	mm^4
$J_{x5,G}$	2,45304E+11	mm^4
$J_{x6,G}$	2,45304E+11	mm^4
$J_{x7,G}$	1,55208E+11	mm^4

The calculation of EJ for the whole section it is carried on finding the EJ value for every element and then summing them all:

$E_{0,mean,gl}*J_{x1,G}$	1,12523E+14	Nmm^2
$E_{0,mean,gl}*J_{x2,G}$	1,12523E+14	Nmm^2
$E_{0,mean,gl}*J_{x3,G}$	1,12523E+14	Nmm^2
$E_{0,mean,gl}*J_{x4,G}$	1,12523E+14	Nmm^2
$E_{0,mean,gl}*J_{x5,G}$	2,84553E+15	Nmm^2
$E_{0,mean,gl}*J_{x6,G}$	2,84553E+15	Nmm^2
$E_{0,mean,KQ}*J_{x7,G}$	1,62968E+15	Nmm^2
EJ_{TOT}	112522633,20	Nm^2

But in order to calculate the frequency EJ_{TOT} should be in Nm^2/m, so it has to be divided by the width of the bridge (2,5m) resulting then:

$$EJ_{TOT} = 45009053 \frac{Nm^2}{m}$$

Then *m*, mass per unit area of the bridge is calculates as follows:

$$m = \frac{\left(\rho_{k,glulam} A_{glulam} + \rho_{k,Kerto} A_{Kerto}\right)}{2,5} = 121,35 \frac{kg}{m^2}$$

And finally the fundamental frequency of the bridge has been obtained by the following expression:

$$f_{vert} = \frac{\pi}{2l} \sqrt{\frac{EJ}{m}} = 1,975 Hz$$

Appendix E: sheets concerning the main working sub-phases

EXCAVATION DONE BY MACHINE			
Sheet **OS1**			
Activities and machineries used	**Associated risks**	**Security measures depending on the contractor**	**Security measures depending on the worker**
Truck, excavator with hammer/bucket	Collision	Provide transit paths for the excavation and transport vehicles. Prohibited the approach to the machineries to all those who are not directly involved in such works. Prohibited the presence of personnel during reversing.	Keep a safety distance from vehicles in motion. Pay attention to acoustic and flashing light signals and safety signs.
	Overturning	The paths must have an adequate transversal slope. The edge of the trenches must be properly defined.	The vehicle must be placed on a solid and flat base. If maneuvers are particularly complex the driver has to be assisted by a person on the ground. Non involved operators must keep a safety distance.
Excavator with hammer/bucket	Noise	Basing on evaluation of the level of personal exposure providing appropriate personal protective equipment (ear defenders), including information for use. Performing regular maintenance.	Use personal protective equipment (headphones or earplugs).
	Throwing of stones or earth	Prohibited the presence of people near the vehicles in motion.	Keep a safety distance.
	Falling people from the edges of the trench	Setting up parapets, barriers or signs on the edge of the excavation.	To get up and down from the bottom of the trench use stairs or walkways.
	Falling material in the trench	Prohibited the storage of materials of whatsoever nature in the vicinity of edges of the trench. Providing appropriate personal protective equipment (helmets) with related information for use.	Do not collect soil or other materials near the edges of the trench. Use appropriate personal protective equipment.
	Dusts	Providing appropriate personal protective equipment (dust masks) with related information for use.	Sprinkle the ground with frequency. Use appropriate personal protective equipment.

EXECUTION OF WOODEN FORMWORKS			
Sheet **SC1**			
The process for its nature, can occur within an excavation, trench or near slopes: in all cases be required to check the conditions of stability of the ground so that there will be no landslides. For the preparation of the tables, it provides for the continued use of circular saw and in this case must be provided by Company and used by the operators of personal protective equipment to prevent cuts and abrasions as well as inhalation of dust.			
Activities and machineries used	**Associated risks**	**Security measures depending on the contractor**	**Security measures depending on the worker**
Hand tools	Contact with equipment	Providing appropriate personal protective equipment (gloves and safety footwear) with related information for use.	Use personal protective equipment. Check frequently the conditions of the tools with particular regard to the strength of connection between wooden handles and metal elements.
Circular saw	Flying particles	Providing appropriate personal protective equipment (glasses) with related information for use.	Correct and conscious use of the provided tools. The univolved personnel have to keep a safety distance.
	Contact with moving parts	Authorizing to use only competent personnel, previously provided with personal protective equipment.	In any case the protection shall be removed from the instrument. Keep the work area tidy and free from waste materials.

INSTALLATION OF REINFORCED BARS			
Sheet **SC3**			
Activities and machineries used	**Associated risks**	**Security measures depending on the contractor**	**Security measures depending on the worker**
Hand tools	Contact with equipment	Providing appropriate personal protective equipment (gloves and safety footwear) with related information for use.	Use personal protective equipment. Check frequently the conditions of the tools with particular regard to the strength of connection between wooden handles and metal elements.
Mobile crane	Material falling from above	Providing appropriate sling ropes and adequate containers (metal baskets) for small materials, detailed information on use and appropriate protective equipment. Demarcation of the affected processing area and testing of hooks (with safety devices) and cables with the stated maximum capacity.	The slings should be performed correctly. During the lifting of materials following the safety rules exposed. Use personal protective equipment. Operators should work in a coordinated way and univolved personnel should keep a safety distance.
Electric welder	Electrical	The instrument must be used by skilled personnel. The supply shall be provided by regulatory electric panel. The electrical cables must be complying with EC rules and suitable for mobile installation.	Check the condition of the cables and the integrity of the electrode holder and report any damage immediately. Place the instrument outside of the reinforcement and place the cables to avoid damage due to shock or mechanical wear.

CONCRETE CASTING WITH MIXER TRUCK			
Sheet **SC4**			
Activities and machineries used	**Associated risks**	**Security measures depending on the contractor**	**Security measures depending on the worker**
Mixer truck	Jets, splashes	Providing appropriate personal protective equipment with related information for use.	Workers must wear appropriate protective work clothing and use the required PPE.
	Overturning	The paths should not have excessive transversal slopes.	The vehicle must be placed on a solid and flat base. Non involved operators must keep a safety distance.

POSITIONING OF THE FOOTBRIDGE ON THE LORRY			
Sheet **CP1**			
Activities and machineries used	**Associated risks**	**Security measures depending on the contractor**	**Security measures depending on the worker**
Hand tools	Contact with equipment	Providing appropriate personal protective equipment (gloves and safety footwear) with related information for use.	Use personal protective equipment. Check frequently the conditions of the tools with particular regard to the strength of connection between wooden handles and metal elements.
Mobile crane	Overturning	Leaving the vehicle to qualified and reliable personnel. Check the proper running of safety systems. The working area must be as flat as possible.	Make sure that all the stabilizers are fully extended. The vehicle must be placed only on flat ground; wooden planks should be placed under the stabilizers to increase the supporting area. Do not use the vehicle in case of strong wind. Avoid reaching the limit conditions and follow the instructions provided by the manufacturer.
	Material falling from above	Providing appropriate personal protective equipment (protective helmet) with related information for use. The area under the arm and the rope must be inhibited in transit with the use of appropriate barriers.	The slings should be correctly performed. Authorised operators can pass only in the working area outside the maneuvering area and must wear the PPE required. When the load is overhung: do not climb on the load, use protective helmets with chin straps and keep a safety distance. Accompany the load with ropes during lifting.

LAUNCHING OF THE FOOTBRIDGE			
Sheet **VP1**			
Activities and machineries used	**Associated risks**	**Security measures depending on the contractor**	**Security measures depending on the worker**
Hand tools	Contact with equipment	Providing appropriate personal protective equipment (gloves and safety footwear) with related information for use.	Use personal protective equipment. Check frequently the conditions of the tools with particular regard to the strength of connection between wooden handles and metal elements.
Mobile crane	Overturning	Leaving the vehicle to qualified and reliable personnel. Check the proper running of safety systems. The working area must be as flat as possible.	Make sure that all the stabilizers are fully extended. The vehicle must be placed only on flat ground; wooden planks should be placed under the stabilizers to increase the supporting area. Do not use the vehicle in case of strong wind. Avoid reaching the limit conditions and follow the instructions provided by the manufacturer.
	Material falling from above	Providing appropriate personal protective equipment (protective helmet) with related information for use. The area under the arm and the rope must be inhibited in transit with the use of appropriate barriers.	The slings should be correctly performed. Authorised operators can pass only in the working area outside the maneuvering area and must wear the PPE required. When the load is overhung: do not climb on the load, use protective helmets with chin straps and keep a safety distance. Accompany the load with ropes during lifting.
	Contact with power lines	Operator should be assisted by personnel on the ground during the operations of the crane. Provide personnel of radio devices to facilitate real-time communication.	Keep the arm of the vehicle at the point of maximum extension and approach to high voltage power lines at a safety distance of not less than 20 meters. Pay attention to oscillations of the rope. In case of strong winds immediately stop lifting operations.

LAYING OF THE BITUMEN MATS			
Sheets **IA1**			
Operators must pay attention to the use of fire to weld the bitumen mats. It will also important to take care of the posture during the work and the manual handling of loads.			
Activities and machineries used	**Associated risks**	**Security measures depending on the contractor**	**Security measures depending on the worker**
Gas torch	Heat/flames	Provision of emergency portable fire extinguishers, safety signs (no smoking, etc.) and emergency procedures in case of fire.	Keeping the flames from a safety distance from flammable and easily combustible materials , and in particular from the gas cylinders. Operators must maintain order in the workplace and remove the waste materials at the end of each phase.
	Manual handling of loads	The manual handling will be preceded by and adequate training and information, subjected to verification of the health of workers.	Take precautions such as handling by mechanical means and load distribution. Making the load easily grasped without risk of injury.

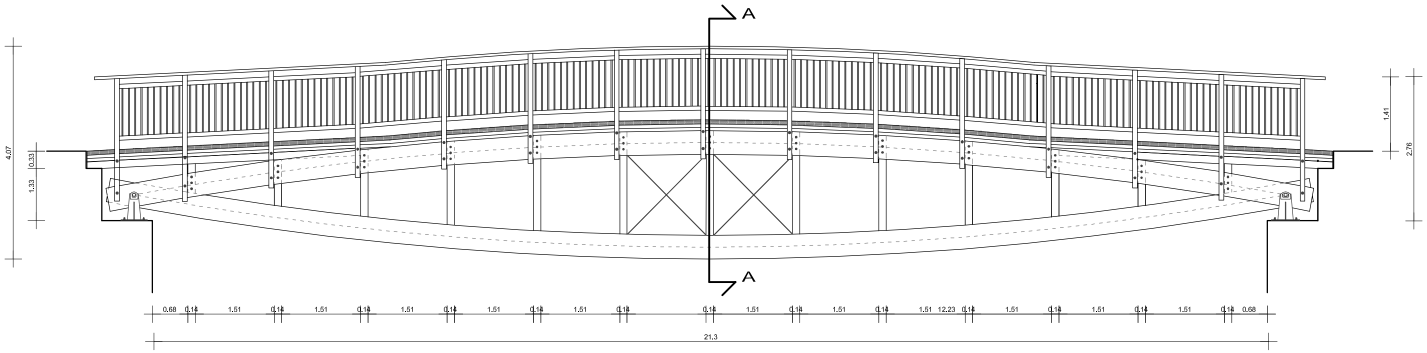
ASPHALTING			
Sheet **IA2**			
Activities and machineries used	**Associated risks**	**Security measures depending on the contractor**	**Security measures depending on the worker**
Hand tools	Contact with equipment	Providing appropriate personal protective equipment (gloves and safety footwear) with related information for use.	Use personal protective equipment. Check frequently the conditions of the tools with particular regard to the strength of connection between wooden handles and metal elements.
Vibrating plate	Heat, vapours	Provide appropriate personal protective equipment (gloves, safety shoes, masks, glasses, clothing) including related nformation on use.	Use PPE (gloves, safety shoes, masks, glasses, clothing).
	Noise	Preliminary investigation on the noise of the machineries, adapting to technical progress by adopting new less noisy machines and renovation of worn mechanical parts.	Use PPE (caps, ear plugs), especially for workers employed in using the plate.
	Contact with moving parts	Authorizing to use only competent personnel, previously provided with personal protective equipment.	Use work equipment according to information received by the employer. In any case the protection shall be removed from the instrument. Do not perform operations on its own initiative that may compromise the safety of themselves or of other workers. Report any defects in work equipment.

Appendix F: technical drawings

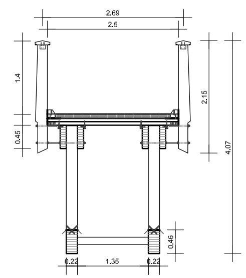
This Appendix contains:

- Plate 1: plan, cross section and side view of the bridge
- Plate 2: structural details
- Plate 3: building site layout _ launching of the bridge
- Plate 4: building site layout _ maintenance phase

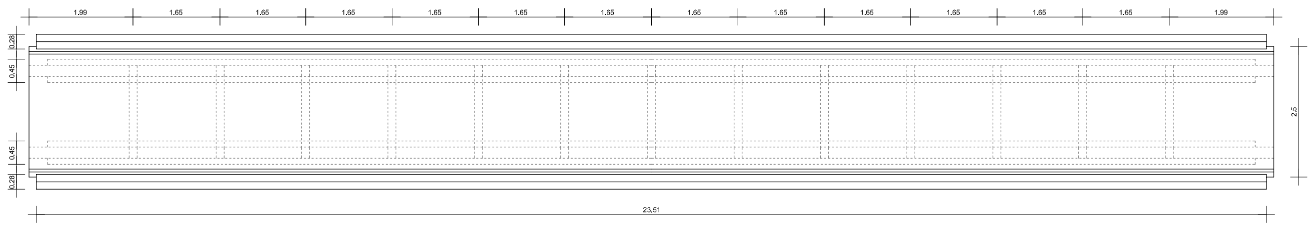
SIDE VIEW



CROSS SECTION A-A



PLAN



UNIVERSITÀ DEGLI STUDI DI TRENTO

CHALMERS

FACULTY OF ENGINEERING
MASTER SCIENCE PROGRAMME IN ARCHITECTURAL ENGINEERING

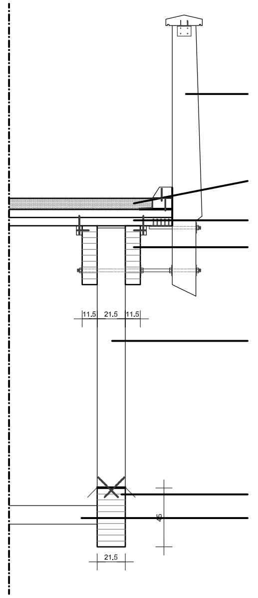
FISH BELLY BRIDGE

PLATE 1: PLAN, CROSS SECTION AND SIDE VIEW
OF THE BRIDGE

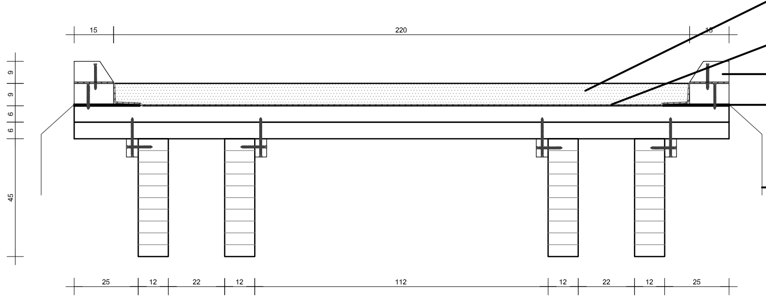
SCALE: 1:50

MATTIA TOSI _114954

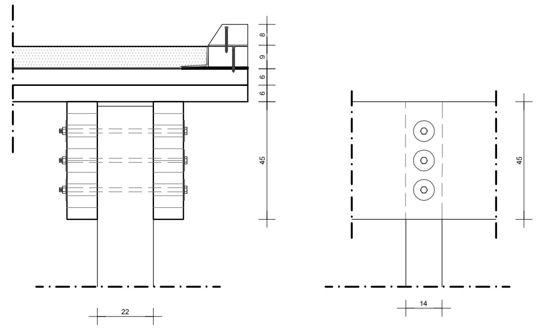
DETAIL: CONNECTION UPPER CHORD/UPRIGHT - Scale 1:20



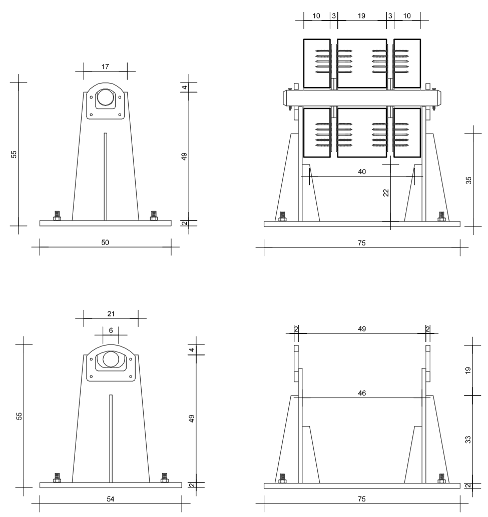
DETAIL: DECK CROSS SECTION - Scale 1:10



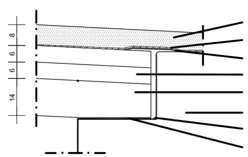
DETAIL: CONNECTION VERTICAL STUD/UPPER CHORD - Scale 1:10



DETAIL: STEEL SUPPORTS - Scale 1:10



DETAIL: EDGE OF THE DECK - Scale 1:10



UNIVERSITÀ DEGLI STUDI DI TRENTO

CHALMERS

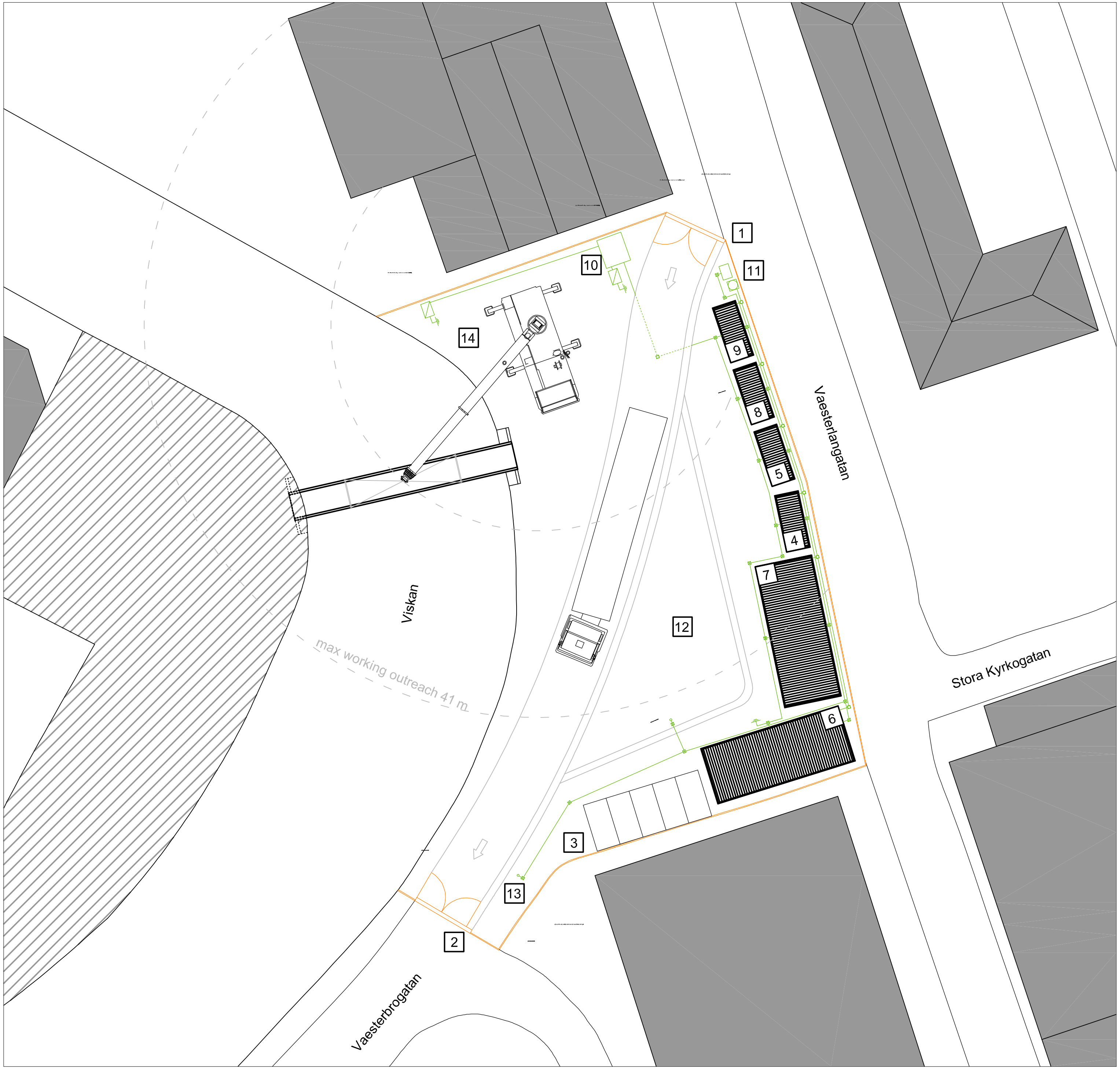
FACULTY OF ENGINEERING
MASTER SCIENCE PROGRAMME IN ARCHITECTURAL ENGINEERING

FISH BELLY BRIDGE

PLATE 2: STRUCTURAL DETAILS

SCALE: 1:10 AND 1:20

MATTIA TOSI _114954



1 Entry to the building site (vehicles/pedestrians)
2 Exit from the building site (vehicles/pedestrians)
3 Employees car park
4 Construction Management
5 Contractor Technical Direction
6 Changing rooms/showers
7 Canteen
8 Infirmary
9 WC
10 Main electrical wiring box
11 Water meter
12 Goods check and acceptance area
13 Vehicles cleaning area
14 On-site working area

UNIVERSITÀ DEGLI STUDI DI TRENTO

CHALMERS

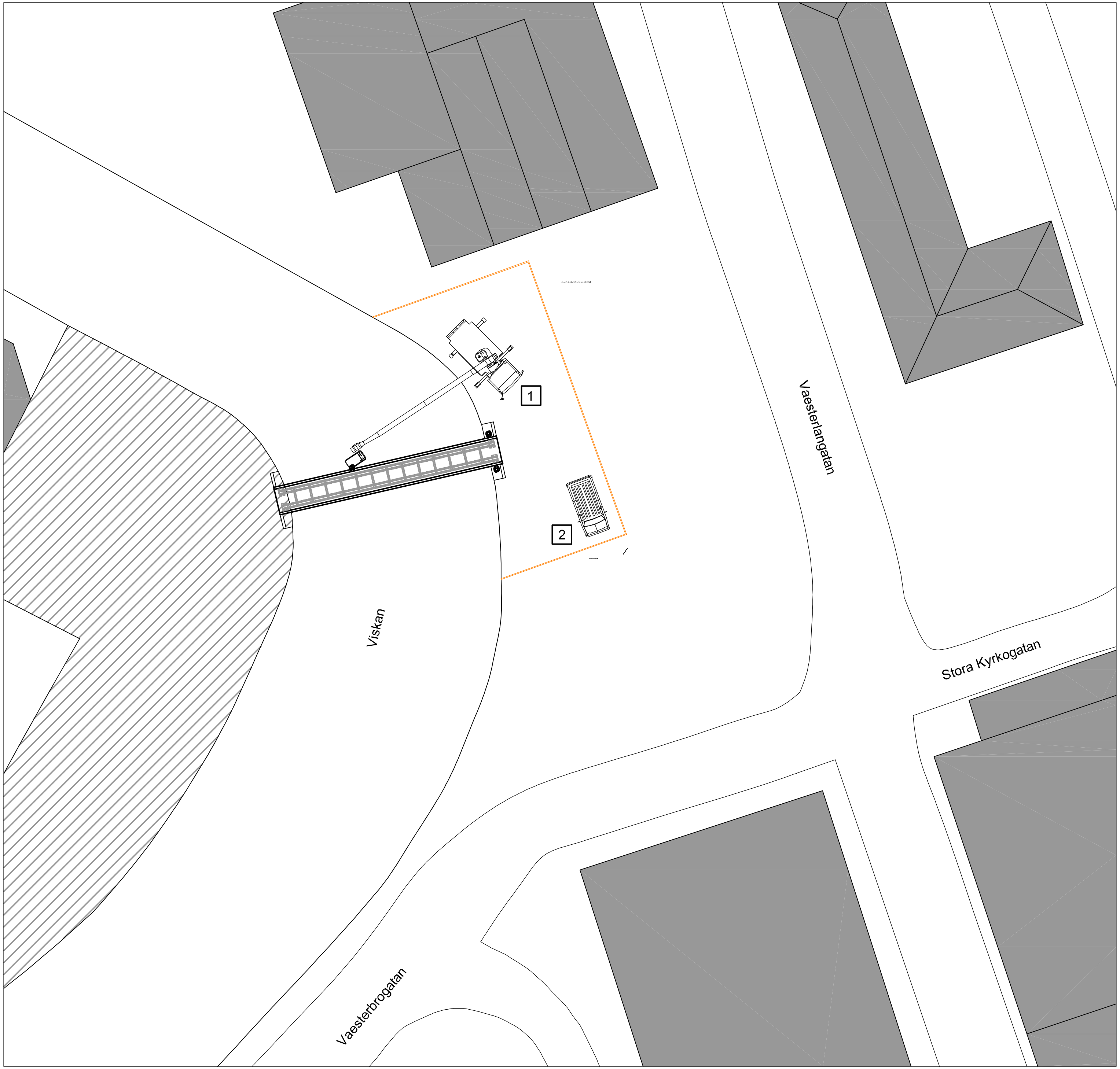
FACULTY OF ENGINEERING
MASTER SCIENCE PROGRAMME IN ARCHITECTURAL ENGINEERING

FISH BELLY BRIDGE

PLATE 3: BUILDING SITE LAYOUT _ LAUNCHING OF THE BRIDGE

SCALE: 1:500

MATTIA TOSI _114954



UNIVERSITÀ DEGLI STUDI DI TRENTO

CHALMERS

FACULTY OF ENGINEERING
MASTER SCIENCE PROGRAMME IN ARCHITECTURAL ENGINEERING

FISH BELLY BRIDGE

PLATE 4: BULDING SITE LAYOUT _ MAINTENANCE PHASE

SCALE: 1:500

MATTIA TOSI _114954