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DEPARTMENT OF CIVIL ENGINEERING

DEPARTEMENT DE GENIE CIVIL

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DEPARTMENT OF CIVIL, ARCHITECTURAL AND ENVIRONMENTAL ENGINEERING

DESIGN STRATEGIES AND ERECTION PERFORMANCES OF TEMPORARY BRIDGE. CASE STUDY: TEMPORARY BRIDGE ON NACHTIGAL HYDROELECTRIC POWER DAM PROJECT IN NACHTIGAL-CAMEROON.

A thesis submitted in partial fulfilment of the requirements for the degree

Of Master of Engineering (MEng) in Civil Engineering

Curriculum: **Structural Engineering**

Presented by:

KAPGANG DJOFANG Igor Gemael

Student number: **16TP21152**

Supervised by:

Prof. Carmelo MAJORANA

Co-supervised by:

Dr. Emanuele MAJORANA

Dr. Guillaume Hervé POH'SIE

Academic year: 2020/2021

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DEDICATION

I dedicate this endeavor work to my father

Emmanuel Kapgang.

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LIST OF ABBREVIATIONS AND SYMBOLS

EC	Eurocode
EN	European Norm
HSS	High Strength Steel
SLS	Serviceability Limit State
ULS	Ultimate Limit State
LSB	Logistic Support Bridge
ADT	Average Daily Traffic
CARC	Chemical Agent Resistant Coating
IRB	Improved Ribbon Bridge
CHS	Circular Hollow Section
NATO	North Atlantic Treaty Organization
WHO	World Health Organization
USEPA	United States Environmental Protection Agency

SYMBOLS

$G_{k,j}$	Permanent loads
$Q_{k,i}$	Variable loads
A_d	Accidental load
$\gamma_{G,j}$	Partial factors of permanent loads
$\gamma_{Q,i}$	Partial factors of variable loads
ψ	Multipliers for the characteristic values of variable loads
α_{Qik}	Values of adjustment factors
TS_k	Tandem Load
UDL_k	Uniform distributed load
$q*_{fk}$	Traffic load on footways
T_k	Temperature load
A_c	Slab section area
ε	Thermal coefficient
E_W	Wood elastic modulus
T_{beam}	Temperature load per beam
y_G	Coordinate y of gravity center
A_{steel}	Area of main beam
A_{slab}	Effective area of slab
A_{id}	Area of ideal mixed section
J_{id}	Moment of inertia of ideal mixed section
$W_{up,steel}$	Resistance modulus relative to steel upper section
$W_{low,steel}$	Resistance modulus relative to steel lower section
$\sigma_{up,steel}$	Stress at steel upper section

$\sigma_{low,steel}$	Stress at steel lower section
V_{Ed}	Design shear value
$V_{pl,Rd}$	Design plastic shear resistance
A_v	Resisting shear area
$V_{b,Rd}$	Shear buckling resistance
P_{Rd}	Design shear resistance of a stud
d	Bolt diameter
N_{Ed}	Design axial value
$N_{b,Rd}$	Axial buckling resistance
χ	Reduction factor
$\bar{\lambda}$	Slenderness
σ_s	stress in steel reinforcement at SLS
y	Neutral axis position
J	Moment of inertia of the section

ABSTRACT

The main objective of this work was to minimize the weight of a temporary bridge structure without compromising its strength and load-bearing capacity. Modular bridge structures are commonly used in areas where space and weight and time of erection are major limitations. The case study used for the various analyses was a temporary bridge of 16.5 meters long made of steel panels of structural class S235 located at the Nachtigal hydroelectric power dam construction project in the Center Region of Cameroon. To achieve our objective, a literature review was conducted first to understand the concept of temporary bridges, their design requirements, and the implementation mechanisms of these types of bridges. Secondly, the modeling of the structure was performed and the static analysis was done using Midas/Civil 2019 and SAP 2000 version 22 software. The verifications were done using Microsoft Excel 2019 software and the execution drawings by AutoCAD 2020. The aforementioned verifications were carried out in accordance with the Eurocodes standards requirements by determining the state of stress inside the different panel elements that constitute the bridge, and by verifying the global and local stability of this bridge subjected to a defined loading model. A parametric analysis or to be more precise a sensitivity analysis was carried out by taking the thickness of the structural elements as a variable of structural improvement, using this turn, high strength steels (HSS) as structural steel of grades S450 and S690QL. The results obtained from the analyses revealed that from the optimal thicknesses obtained for each class of steel, it was possible to reduce the total weight of the structure by almost 40% while maintaining its strength. The structural improvement is not dissociated from the economic aspect and the problem of stability for HSS, so S450 was therefore agreed for an improvement that is both structurally and economically advantageous.

Keywords: Temporary bridge, design improvement, structural analysis, high strength steel, sensitivity analysis, modular bridge.

RESUME

L'objectif principal de ce travail était de minimiser le poids de la structure d'un pont temporaire sans toutefois compromettre sa résistance et sa capacite portante. Les structures de ponts modulaires sont couramment utilisées dans les zones ou l'espace et le poids sont des limitations majeures. L'étude de cas utilisée pour les différentes analyses était un pont a panneaux modulaires en acier de classe S235 de 16,57 mètres de long basé sur le projet de construction du barrage hydroélectrique de Nachtigal, dans la région du Centre au Cameroun. Pour atteindre notre objectif, une revue de la littérature a été réalisée dans un premier temps pour comprendre le concept de pont temporaire, les exigences de conception de ceux-ci, et les mécanismes de mise en œuvre de ces types de pont. Deuxièmement, la modélisation de la structure a été réalisée et l'analyse statique a été faite à l'aide des logiciels informatiques Midas/Civil 2019 et SAP 2000 version 22. Les vérifications ont été faites à l'aide du logiciel Microsoft Excel 2019 et les plans d'exécutions par AutoCAD 2020. Les vérifications susmentionnées ont été effectuées conformément aux normes des exigences des Eurocodes en déterminant l'état de contrainte à l'intérieur des différents éléments de panneaux qui constituent le pont, et en vérifiant la stabilité globale et locale de ce pont soumis à un modèle de chargement définit. Une analyse paramétrique ou pour être plus précis une analyse de sensibilité a été effectuée en prenant l'épaisseur des éléments de structure comme variable d'amélioration structurelle en utilisant cette fois ci des aciers à hautes résistance que sont, le S450 et le S690QL. Les résultats obtenus des analyses ont révélé qu'à partir des épaisseurs optimales obtenues pour chaque classe d'acier, l'on pouvait réduire le poids total de la structure de près de 40% tout en gardant sa résistance. L'amélioration d'une structure n'étant pas dissocier de l'aspect économique et du problème de stabilité pour des aciers à haute résistance, l'acier structurel de classe S450 a donc été considéré comme l'acier de structure le plus favorable pour une amélioration avantage, aussi bien sur le plan structurel qu'économique.

Mots clés : pont temporaire, amélioration de conception, analyse structurelle, acier de haute résistance, analyse de sensibilité, pont modulaire.

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GENERAL INTRODUCTION

Nowadays, temporary bridges are often used for quick installations and because of their lower cost compared to other conventional types of bridges. The Modular Bridge Technology has widely been used to increase the efficiency of design and production of bridge members. Various construction companies are involved in the design, analysis and production of these bridges due to their simplified structure and maneuverability. Modular bridge structures are specifically made for quick assembly, disassembly and easy transportation.

Design, analysis and structural improvement of modular bridges have been the focus of several studies. Design amelioration such as topology, shape and size are used to find the best design solution for modular bridges (Yang & Angeles, 2005). Shape and size amelioration help to find the best geometrical properties such as thickness, length, width, height, radius and angles which can reduce weight and improve performance (Taylor & Séquin, 2013). Stress is an important design criteria in engineering design (Uc, 2011). Stress-based design has been used in various engineering problems in the past years (Uc, 2011). High strength steel can bring along significant savings, but buckling, fatigue or deflection criteria often set the limit of the steel grade being used (Collin & Johansson, 2005) . With this in mind, structural engineers and other professionals are continuously researching and developing methodologies in order to have best savings when using HSS.

The main objective of this work was to ameliorate using high-strength steel the weight of the bridge structure so as to ease transportation to the different usage zones while keeping in mind the economic aspect of the construction. This is possible after the determination of the stress state of the bridge elements by the software Midas/Civil and SAP 2000 V22.

In order to achieve this objective, the study is divided into three chapters. The first part (Chapter 1) consists of a literature review on temporary bridges, design requirements and implementation mechanisms. The second part (Chapter 2) presents the methodology for this study, it elaborates on the collection of data, analysis and design procedures used. The third part (Chapter 3) is the application of the detailed methodology outlined in chapter 2, that is, using Midas/Civil 2019, SAP2000 to obtain solicitations, static analysis of the structure and sensitivity analysis results of the bridge.

CHAPTER 1. LITERATURE REVIEW

Introduction

Temporary steel bridges provide a rapid, flexible, durable and cost-effective solution for a variety of sites requiring a temporary crossing. These structures can be used to provide detours to direct traffic around road construction sites during bridge replacement, repair, or in emergency situations. Temporary steel bridges also allow safe and efficient access for workers, heavy off-road vehicles, machinery, and equipment. To understand how it works, it is necessary to have a view of everything which is around the concept of temporary bridge. It will begin by a general overview on temporary bridges, followed by different system of temporary bridge. Next, will be the presentation of design procedures. Some mechanism of construction phase will be given also.

1.1. General overview on temporary bridges

1.1.1. History of bridge design

Bridge is a structure built to cross physical obstacles such as a plane of water, a valley or a road, for the purpose of providing a passage over the obstacle (G. Tecchio, n.d.). He is many different designs that all serve unique purposes and apply to different situations. The design of the bridges varies according to the function of the bridge, the nature of the ground where the bridge is built or anchored, the material used and the funds available to build it. The actual beginning of the use of the bridges is not well known, so it is obvious to say exactly when and how the use of the bridges began. But our ancestors were hunters and collectors who had to travel long roads through rough terrain to find food, burning material or shelter. Because of this, it was necessary to overcome rivers, canyons or other obstacles. Naturally shaped bridges like a fallen tree were first used by man prehistoric before he could carry out a technical construction. The first constructions of wooden bridges were built with a maximum length of span the length of trees used, so another technique was necessary (Joiner, 2006).

1.1.1.1. The Roman period

The Romans developed later, 117-180 AD a main invention that separated them from other cultures of their time, they discovered natural cement. Pozzolan is loose volcanic sand found at Pozzoli near Naples. When mixed with lime, hydraulic cement is formed. The builders of the Roman Empire built mainly stone arch bridges with long spans, up to 30-50m, but many of them were destroyed for insufficient foundations (Brown, 1996). The next flagship period was the Middle Ages.

1.1.1.2. The Middle Ages

In the Middle Ages the construction of bridges and cement technology was lost after the fall of the Roman Empire. Not much activity for about 1000 years. Bridges were used to control traffic and as defenses they were made of heavy stone with narrow openings. Knowledge of the bridge must have been relearned around 1100-1500 AD (G. Tecchio, n.d.). . Another new way of doing things developed during the Renaissance and with the Inca civilization.

1.1.1.3. The Inca civilization

Around 1400 AD The rope bridge was developed by the Incas. This is the beginnings of the type of suspension bridge (Brown, 1996). The Renaissance period many Roman knowledge was rediscovered including the construction of bridges a Truss was first invented but not widely used until later in the 18th to 19th century.

1.1.1.4. xviiith - xixth Century

During the industrial revolution, with the new material iron and steel, the relationship between the dead weight and load capacity has enabled new types of bridges such as truss bridges. The already known knowledge of wooden truss bridges was developed to build the first steel truss bridges. These bridges crossed major rivers to build the infrastructure needed for the growing rail network. During this time, the first constructions of temporary steel bridges were used (G. Tecchio, n.d.). These bridges were mostly pontoon bridges developed and used independently by the military forces of different countries. With the outbreak of World War, and the rapid development of vehicles, temporary bridges became even more important (Brown, 1996).

1.1.1.5. Twentieth century

In this period, iron was completely replaced by steel and steel Truss bridge was widely used. Prestressed concrete was developed by Fressinet in 1940s, New techniques developed for the execution of deep foundations (drilled-large diameter piles…) allowed constructions in zones first considered inaccessible, suspension bridge was popular for long-span bridges(Tacoma Bridge, 1940), Cable-Stayed Bridge was first developed (G. Tecchio, n.d.).

1.1.2. Definition of temporary bridge

First developed for military use, temporary bridges are pre-engineered, modular structures that can be used to cross anything from drainage ditches, streams, and trenches to railway lines, or utility pipes that are buried close to the surface.

For the most part, temporary bridges are used to provide site access for construction vehicles and operatives, and are often seen bridging small culverts, rivers or sections of uneven terrain at the side of major motorways, or large building sites. They are generally between twelve and sixty meters long, and can be flat-packed for easy transportation and storage.

1.1.3. Importance of temporary bridge system

In every extraordinary case if it is war, natural disaster or whatever mobility is a key element for a solution. At the present time first connection to areas which are after a bridge collapse not reachable could be made also with other technologies like by air or water. For example, after a flood were in most cases the infrastructure is damaged first help and rescue is done by boats and helicopters. But for the clean-up work and reconstruction where heavy equipment is necessary, a temporary bridge often creates the only possible way to get to the affected area. If the need for a temporary bridge is very high but the delivering of a bridge system took a long time, local available machines and materials like timber or steel beams could be used for an improvised solution. In case of a natural catastrophe or modern warfare the speed of operation is most important. Therefore, flexible systems which can be rapidly build and deconstruct without complex preparation of the site are necessary. A good example gives Field Marshall Lord Montgomery who said after World War II that he could never rushed in such a speed forward without temporary bridge constructions. He was not the only man who saw temporary bridges as one of the key elements to win the Second World War (Zierhofer, 2011).

1.2. System of temporary bridges

A system bridge is a further development of improvised bridges and is nowadays the standard for a temporary bridge construction. System bridges are constructions with a special scheme which could be built in many variations. So, it is often possible that one bridge system could be used for several operations by flexibility in length and load capacity. The erection and deconstruction are standardized and explained in user manuals for each bridge system. Most system bridges are made out of steel whereby also aluminum alloys, kevlar, carbon or reinforced plastic are in use for some bridges (Worenz, 2018). In general system bridges could be separated in floating bridges for wet obstacles and fixed bridges for dry or wet obstacles. Compared to an improvised bridge a system bridge has several advantages. The knowledge about load capacity, erection time, needed material or required machines are just some of them. Examples of these bridge types include panel truss systems, pre-engineered steel girder bridges, railroad flatcar systems, and composite steel-concrete systems.

1.2.1. Panelized truss systems

It is a bridge construction system in which the beams are made up of removable modular panels. Panel truss systems are composed of panels of prefabricated trusses that bolt/pin together, floor beams that span between trusses, and a steel deck system. The bridge type was developed to be flexible enough to be used in a variety of conditions and to be deployed quickly in the field by inexperienced teams with little or no heavy equipment. The concept was refined during World War II and used during the war. Many suppliers have developed proprietary systems based on this concept. All of these bridges have standard parts, which can be assembled in a variety of configurations depending the length of the span and the loads supported. Configuration variables include span length, truss number and depth (D. of the Army, 1986).

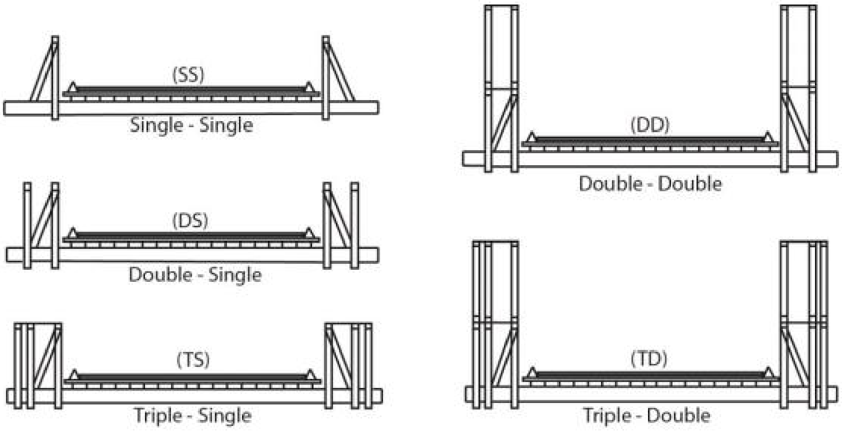


Figure 1.1. Example Configurations for Temporary Bridges

The floor beams are set on the truss, and the deck is set on the stringers, or floor beams if the floor is a structural deck. The decks are typically steel grate, skid-resistant steel plate, or steel plate with a skid-resistant overlay, the abutments for this type of bridge can be any variety of materials that will adequately support the truss. Some of the materials used have been steel piles, cast-in-place concrete, precast concrete blocks, timbers, cribbing and gabions.

A double-single (two trusses on each side, one panel deep) with a steel grate deck is shown in Figure 1.2. The abutment for the bridge is steel plate resting on timbers.

Figure 1.2. Panelized truss temporary bridge with two trusses on each side of roadway (fdotwww.blob.core.windows.net)

Typically, the trusses with the floor beams are assembled on one side of the feature to be crossed, and then rolled/launched to the other side. The launch often uses a “launching nose”, which is an angled set of panels at the leading end of the truss. An example of the installation of a panelized truss with a launching nose is shown in Figure 1.3. The launching nose is removed once the truss assembly has been set in place.

Figure 1.3. Installation of panelized truss with launching nose(www.maybeybridge.com)

In some cases, where the equipment is available, these bridges may be set with a crane. Once the truss assembly is in place, the decking is then installed, the back wall placed (which may be steel, concrete, or timber), and the approach graded. A photo sequence for the installation of a panelized truss bridge is presented in Figure 1.4. The figure shows (a) Stored panels, (b) Assembling truss and floor beams, (c) Assembling and attaching second truss, (d) Setting bridge with crane and dozer, (e) Installing steel deck panels, and (f) Placing wearing course on deck.

(a)

(b)

(c)

(d)

(e)

(f)

Figure 1.4. Sequence for the installation of a panelized truss bridge

1.2.2. Steel girder pre-fabricated deck

Another common type of temporary bridge is steel girder bridge with prefabricated deck panels. The deck panels may be steel, wood, or concrete. Contractors will periodically construct this type of bridge using salvaged girders. A temporary bridge erected after the existing bridge was damaged by flooding from Hurricane Irene in 2011 is shown in Figure 1.5. This bridge was built with salvaged girders, stress laminated wood deck panels, and precast concrete barrier, and using the existing approaches as the abutment.

Figure 1.5. Steel Girder temporary bridge (fdotwww.blob.core.windows.net)

1.2.3. Rail road flatcar system

Flat Cars have been used in rural areas for permanent bridging on low volume roads for many years. Using railroad flatcars concept as temporary bridging was developed by W.H. Wattenbug of the Lawrence Livermore National Laboratory in the 1990's. The modular system consists of a flatcar acting as a foundation and supports the half flatcars that serve as columns, which in turn support a flatcar that acts as a bent cap. The deck system consists of four flatcars, which are interlocked side-by-side. Figure 1.6 display the concept.

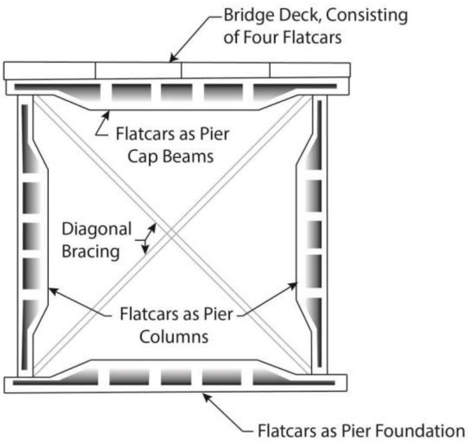


Figure 1.6. Railroad flatcar modular system schematic

1.2.4. Composite concrete and steel

The composite concrete and steel structure is a twin girder unit tied together with diaphragms that are precast into a concrete deck. This design takes full advantage of the composite action of the steel and concrete allowing for a light steel section. This type of construction was used extensively on the central artery project in Boston for bridges that were to be in use for approximately 10 years before removal.

The units shown in Figure 1.7 were initially installed as temporary spans but were later removed and reinstalled as permanent bridges in another location. The figure shows (a) Composite superstructure component prior to removal from temporary installation location, (b) Composite section after removal, (c) Stockpile of used composite sections awaiting shipping to new location, and (d) Permanent bridge built with used composite sections.

(a) (b) (d) (e)

Figure 1.7. Temporary span reuse (fdotwww.blob.core.windows.net)

1.3. Parameters for temporary bridge placement

The design, construction and erection of temporary bridges depend on certain parameters that are specific to each situation. Among these parameters there are also the length of the spans, waterway opening, and the expected loading.

1.3.1. Length

Prior to erecting a temporary bridge, the necessary span of the bridge must be determined. One factor driving the length of span is the hydraulic opening (in the cases of bridges spanning waterways) or required clear span for bridges spanning other features such as roads. Another factor determining length of span is the terrain and the approaches to the bridge. An example of this would be a relatively wide and deep ravine with a small creek at the bottom.

1.3.2. Waterway opening

If a temporary bridge is replacing a bridge with no history of flooding, overtopping, or other hydraulic issues, a temporary bridge of equal or greater waterway opening is often the preferred choice. If the agency is trying to optimize the size of the temporary bridge, a more detailed analysis accounting for acceptable risk would be necessary. Determining the required waterway opening for a temporary bridge not only requires an analysis of the watershed and the hydraulics of the waterway opening, but also requires an estimate of how long the temporary bridge will be in place, the importance of the road, and the agency's acceptable risk for overtopping. The hydraulic design requirements for a permanent bridge may require a minimum freeboard above the 50-year flood elevation (2 percent chance of reaching that in a given year) for a bridge that will be in service for 50 to 75 years. If the temporary bridge is only going to be in service for a limited time, for example 6 months to a year, an increased risk maybe acceptable, (e.g., 1 in 25 or 1 in 10 of exceeding the hydraulic capacity in a given year).

1.3.3. Expected loading

As with permanent bridges, temporary bridges must be designed to handle the traffic that is expected to use the bridge while it is in service. Depending on the traffic volumes, the percentage of trucks, and the availability of reasonable alternate routes, a temporary bridge may not need to be designed and built to carry full legal loads. If a temporary bridge is designed for less than full legal loads, a careful analysis should be done to justify it. If justified and built to handle only lighter traffic, the bridge must be posted for the allowable load and the owner should plan to take steps to enforce the load posting. If the bridge is designed for full legal loads, it may be posted at a lower level if recommended by the engineer or manufacturer. It may need to be posted at the legal load limit to prevent use by trucks traveling with an overweight permit.

1.4. General usage and maintenance

1.4.1. Installation Considerations

Because each type of bridge has its own unique characteristics, the installation considerations given here are general. See the manufacturer, designer, or engineer of the particular temporary bridge for detailed installation instructions.

Before any deployment, experienced bridge personnel (who have previously been trained to deploy the same bridge type) should supervise the deployment.

During the initial deployment of any temporary bridge, the manufacturer (if a manufactured type) or the Engineer (if a custom designed bridge) should provide on-site training and appropriate erection plans.

Whether the bridge is deployed, moved, or disassembled, any component that is damaged or missing should be immediately reported to the Engineer or manufacturer. Damaged

components, including the loss of the protective system, should be inspected by the Engineer or manufacturer and repaired or replaced before any further use of the bridge is permitted. The manufacturer or engineer should be consulted as needed. Any missing component must be replaced as per the manufacturer or engineer requirements.

1.4.1.1. Bridge Deployment and Site Security

All components of the bridge should be on site, inspected for damage, and inventoried before assembly and erection can begin.

Unless approved by the Engineer, there should be no deviation from the manufacturer recommended deployment procedures. In unusual cases where deviation is warranted, the manufacturer should be consulted.

Since these bridges represent a significant investment, each bridge site should be secured. The degree of security necessary will depend on site conditions, the risk of loss or damage, and the amount of acceptable risk the agency is willing to take.

Ideally, bridge components should not be delivered to the site until the foundations are completed and all materials and equipment necessary to erect the bridge are on site**.**

Typically, assembly and erection should be scheduled in such a manner that as much continuous work as possible is completed, and the time that the bridge components are on site in the un-erected position is limited.

1.4.2. Periodic inspections of pins and connections

Due to the nature of temporary bridges and depending the Average Daily Traffic (ADT) regular inspections should be performed above that required by the manufacturer. The bridge owner should determine the frequency of inspections. Cursory visual inspections should be performed.

1.5. Examples of temporary bridges

There exist a variety of temporary bridges, which according to dynamics can be classified into two large groups which are fixed bridges and mobile bridges.

1.5.1. Fixed bridges

That are bridges having no moving parts and stay in one place until they fail or are demolished. They are designed to stay where they are made to the point they are unusable or demolished.

1.5.1.1. The Bailey bridge

a. Generalities on Bailey bridge

It was developed by Sir Donald Bailey in 1941 who improved the Callender-Hamilton bridge system Developed in 1930. A key improvement over the Callender-Hamilton system is in the connection; the Callender-Hamilton requires bolted connections to standardized gusset plates to build up a truss, whereas Bailey Bridge panels can be connected by simple pins through pre-drilled holes (Russell & Thrall, 2013).

The Bailey bridge is a truss bridge system whereby the carriageway is between two main girders. The trusses in each girder are assembled by the standard panels which are connected from end to end. Transverse to them are the cross beams which are called transoms. The transoms are clamped to the bottom of the main girders to stiffening the bridge and provide a construction for the carriageway. For additional horizontal stiffening sway braces are necessary which are also mounted between the girders. On the transoms stringers are mounted to provide a substructure for the decking. Rakers are connected between the panels and the transoms to hold the trusses upright. For a horizontal connection of the trusses bracing frames and tie plates are necessary. One bridge set consists out of 33 different parts and 30 special tools for erection. A girder could consist out of one, two or three trusses which are mounted side by side. To increase span or load capacity it is possible to add a second or third story of trusses. Each story is connected with bolts to the lower elements. With this variety following possible configurations are possible.

Table 1.1. Abbreviation for single-lane panel bridge, Bailey type M2 (U. Army, 2010)

TYPE		USUAL	
TRUSS	**STORY**	**NOMENCLATURE**	**ABREVIATION**
Single	Single	Single-single	SS
Double	Single	Double-single	DS
Triple	Single	Triple-single	TS
Double	Double	Double-double	DD
Triple	Double	Triple-double	TD
Double	Triple	Double-triple	DT
Triple	Triple	Triple-triple	TT

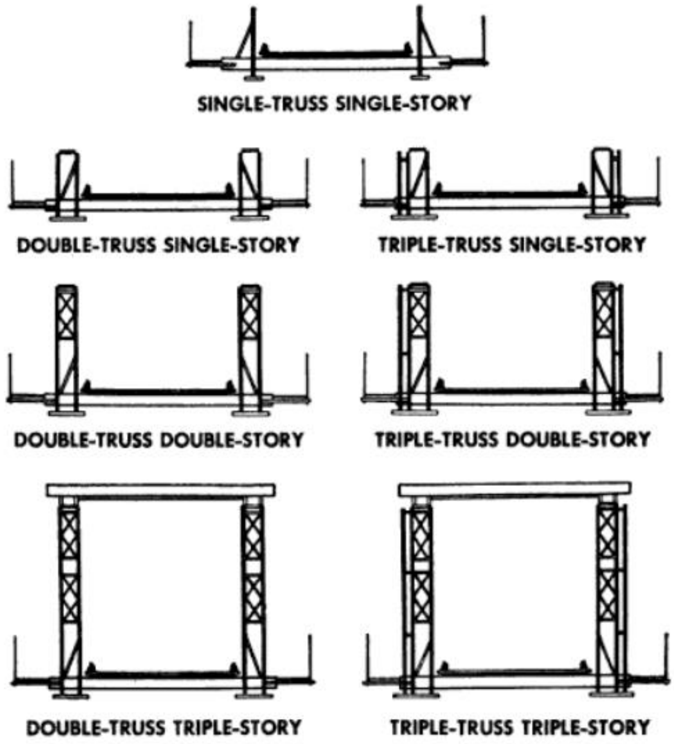


Figure 1.8. Abbreviation for single-lane panel bridge, Bailey type M2(U. Army, 2010)

A single truss with two or three stories is not possible because it would be not stable. For the necessary stability all three stories' configurations are braced on the top with transoms and sway braces.

The carriageway which is between the girders has a width of 12 feet and 6 inches. To get with a vehicle on the carriageway at each side a ramp is required. The slope for vehicles up to 50 tones must be under a rise/run ratio from 1 to 10. Over 50 tones a ratio of at least 1 to 20 has to be build. Additional at each side of the bridge a 2 feet 6 inches sidewalk for pedestrians could be mounted. At each end of a truss a bearing has to be mounted. That bearing sits on cylindrical bearings which rest on the base plate. To avoid settlements caused by soft soil a timber grillage under each base plate is obligatory (D. of the Army, 1986).

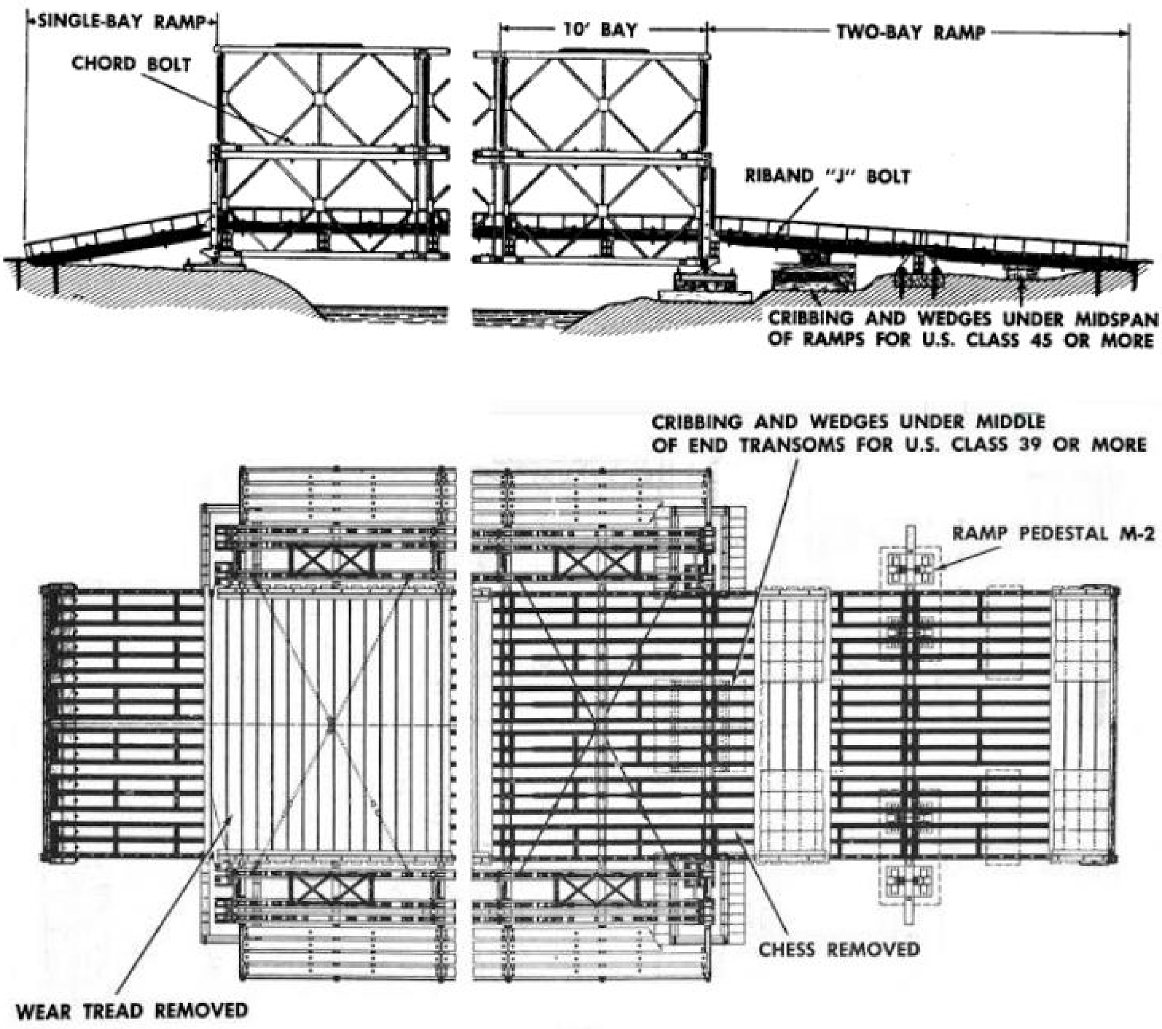


Figure 1.9. Section and floor plan from a Bailey Bridge (U. Army, 2010)

b. Main parts of the Bailey bridge

i. The standard panel

The panel is the main element of the bridge. It is a welded steel truss which is 10 feet long, 5 feet 1 inch high and 6 ½ inches wide. The horizontal steel beams are called chords. The transoms will be clamped with transom clamps on the top of to the bottom chord. Therefore, four steel elements with holes are welded between each chord. At the end of all chords there are male or female lugs to connect the panels with pins.

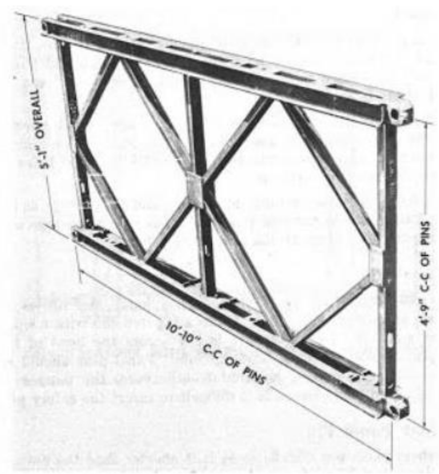


Figure 1.10. Bailey bridge standard panel

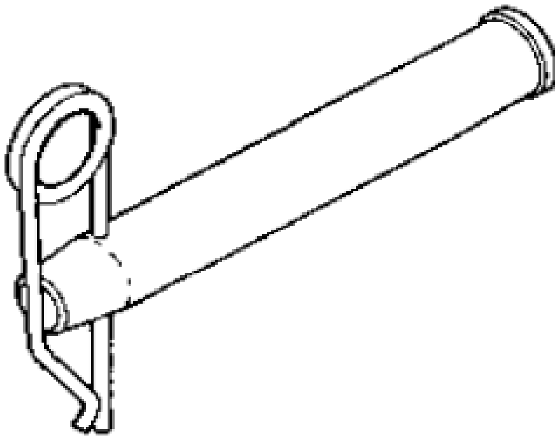


Figure 1.11. Bailey bridge connection pin

ii. Transom

The transom is places between the main girders and is the substructure for the carriageway system. It is a 10-inch steel I-beam with a length of 19 feet 11 inches. The weight of that part is 618 pounds and is usually carried by 8 mans with special lifting equipment. The underside provides six recesses where standard panels could be mounted. On the upper side 8 lugs are welded on whereby 6 lugs are needed for the connection to the stringers and two are necessary to connect the rakers. On the sides of the I-beam lugs exist to mount the sidewalk. In most cases a transom is placed at each end and in the middle of each standard panel. Additionally for a higher load capacity of the bridge a fourth could be added per bay. For a simplify erection of the transoms a transom roll could be temporary mounted on a standard panel.

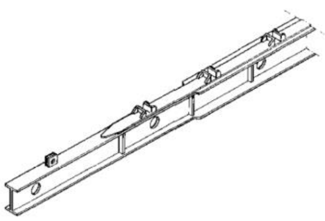


Figure 1.12. Bailey bridge transom beam

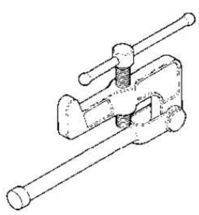


Figure 1.13. Bailey bridge transom roll

iii. The Raker and Bracing Frame

The bracing frame is a rectangular frame, 4 feet 3 inches (1.3 meters) by 1 foot 8 inches (50.8 centimeters) with a hollow conical dowel in each comer. It weighs 44 pounds (20.0 kilograms). The bracing frame is used to brace the inner two trusses on each side of the double- and triple-truss bridge. Bracing bolts attach the bracing frames horizontally to the top chords of the bridge, and vertically on one end of each panel in the second and third stories.

The raker is a 3-inch (7.6 centimeters) steel beam with a 2 3/8-inch (6.0 centimeters) flange. It is 3 feet 8 5/16 inches (1.11meters) long and weighs 22 pounds (10.0 kilograms). A raker connects the ends of the transom to the top of one end of each panel of the inner truss. This prevents the panels from overturning. An additional raker is used at each end of the bridge. Both ends of the raker have hollow dowels for the bracing bolts. The dowels fit through a hole in the panel and a hole in the transom.

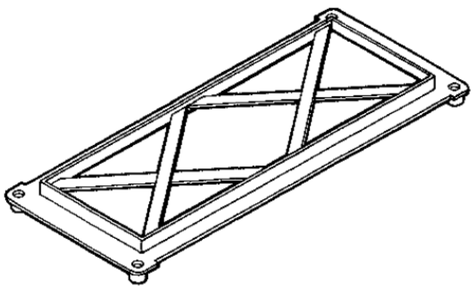


Figure 1.14. The Bailey bridge bracing frame (U. Army, 2010)

Figure 1.15. Bailey bridge raker(U. Army, 2010)

iv. The sway braces

The sway brace (Figure 1.15) is a 1 1/8-inch (2.9 centimeters) steel rod, hinged at the center, and adjusted by a turnbuckle. It weighs 68 pounds (30.8 kilograms). At each end is an eye, and a chain with a pin attached. This pin is inserted through the eye to the sway brace to the panel. The sway brace is given the proper tension by inserting the tail of an erection wrench in the turnbuckle and screwing it tight. The locknut is then screwed up against the turnbuckle. Two sway braces are required in the lower chord of each bay of the bridge, except the first bay of the launching nose, and in each bay of overhead bracing.

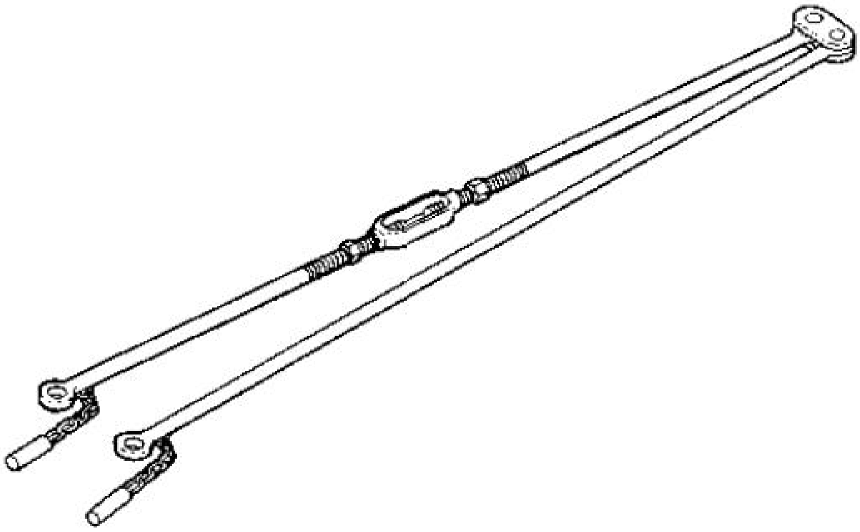


Figure 1.16. Bailey bridge sway brace (D. of the Army, 1986)

v. Stringer and Decking

Stringers (Figure 1.17-1.18) carry the bridge's roadway. Each stringer consists of three 4-inch (10.2 centimeters) steel beams, 10 feet (3.0 meters) long, joined by welded braces. There are two types of stringers: plain stringers weighing 260 pounds (118 kilograms) and button stringers weighing 267 pounds (122 kilograms). They are identical except that the latter has 12 buttons which hold the ends of the chess (roadway) in place. Each bay of the bridge has six stringers: four plain stringers in the middle, and a button stringer on each side. The stringers are positioned by the lugs on the top of the transoms. Decking, often referred to as deck or chess, form the road surface. A piece of chess is 2 inches (5.1 centimeters) by 8 3/4 inches (22.2 centimeters) by 13 feet 10 inches (4.2 meters). It is made of wood and weighs 65 pounds (29.5 kilograms). It is not ched at the ends to fit between the buttons of the bottom stringer. Each bay of the bridge contains 13 chesses, which lie across the stringers and are held in place by the buttons. Chess are held down by ribbands.

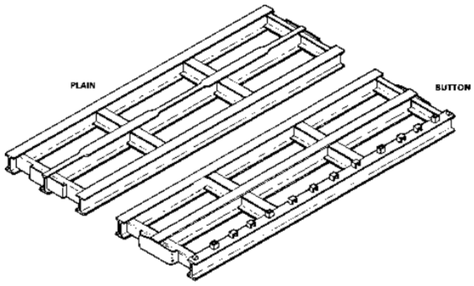


Figure 1.17. Bailey bridge stringer(U. Army, 2010)

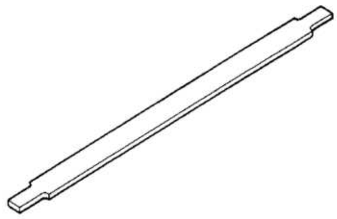


Figure 1.18. Bailey bridge decking(U. Army, 2010)

vi. End posts and Bearing

End posts (Figure 1.19) are used on both ends of each truss of the bridge to take the vertical shear. They are placed only on the story carrying the decking. They are 5-foot 8-inch (1.7 meters) columns made of two 4-inch (10.1 centimeters) channels and plates welded together. There are two types; male and female, having male and female lugs, respectively. These lugs are secured to the end panels of the bridge by panel pins placed through holes in the lugs.

The male and female end posts weigh 121 and 130 pounds (54.9 and 59.0 kilograms), respectively. End posts have a step to support a transom outside the panel at one end of the bridge. In jacking the bridge, the jack is placed under the step. The lower end of the end post has a bearing block with a semicircular groove which fits over the bearing. The bearing (Figure 1-20) spreads the load of the bridge to the base plate. A bearing is a welded steel assembly containing a round bar which, when the bridge is completed, supports the bearing blocks of the end posts. During assembly of the bridge, it supports the bearing block of the rocking roller. The bar is divided into three parts by two intermediate sections that act as stiffeners. The bearing is 4 5/16 inches (11.9 centimeters) high and weighs 68 pounds (30.8 kilograms). One bearing is used at each corner of a single truss bridge and two bearings per corner for a double- or triple-truss bridge.

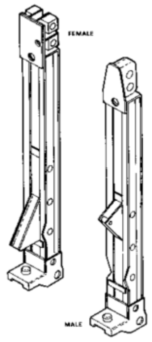


Figure 1.19. The Bailey bridge end posts (U. Army, 2010)

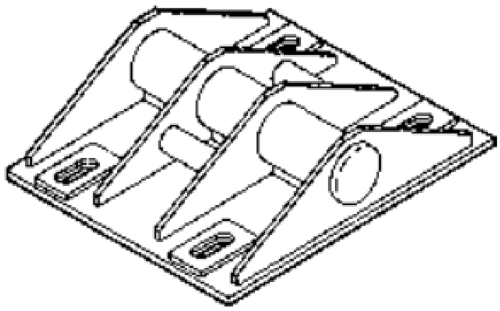


Figure 1.20. The Bailey bridge bearing(U. Army, 2010)

vii. Ramps and Ramp pedestal

Ramps (Figure 1.21-1.22) are similar to stringers but consist of three 5-inch (12.7 centimeters), instead of 4-inch (10.2 centimeters), steel beams. They are 10 feet (3.0 meters) long and are joined by welded braces. The lower surface of the ramp tapers upward near the ends. There are two types of ramps: plain ramps weighing 338 pounds (153 kilograms), and button ramps weighing 349 pounds (158 kilograms). They are identical except that. The latter have 12 buttons which hold the ends of the chess in place. The ends of the ramps fit into lugs on the transoms at the ends of the bridge. Ramp pedestals (Figure 1-22) are built-up welded steel assemblies weighing 93 pounds (42.2 kilograms). They prevent the transoms supporting multiple-length ramps from over turning and spread the transom load over the ground. They are held in place by spikes or pickets driven through holes in their base plates.

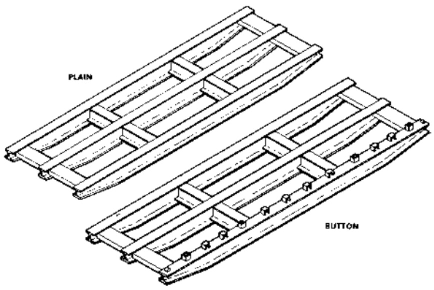


Figure 1.21. Ramps (U. Army, 2010)

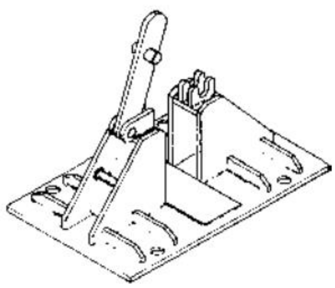


Figure 1.22. Ramps pedestal(U. Army, 2010)

c. Erection equipment

i. Roking roller, plain roller and transom roller

The rocking roller (Figure 1.23), weighing 206 pounds (93.4 kilograms), consists of three rollers housed in a balanced arm which fits over the bearing, and is free to rock on it. Two side rollers on the flange on each side of the rocking roller frame act as guides for the trusses. The side rollers can be removed from the flanges by removing split pins from spindles underneath the flange; they then remain loosely attached to the frame by a chain. The rollers distribute the bridge load along the bottom chord during launching. The maximum allowable load on one rocking roller is 30 tons (27.2 metric tons). The plain roller (Figure 1.24) is 2 feet 1 1/2 inches (64.8 centimeters) wide and weighs 116 pounds (52.6 kilograms). It consists of a welded housing containing a single roller split in two. The maximum allowable load on one roller is 10 tons (9.1 metric tons). Trusses of single-truss bridges can be carried on either half of the roller. Second and third trusses of triple-truss bridges are carried on both halves. The transom roller (Figure 1.25) is a roller having an outside diameter of about 1 7/8 inches (4.8 centimeters) (or 1 1/2-inches [3.8 centimeters] extra-heavy steel pipe) and a length of 6 5/8 inches (16.8 centimeters). The roller is fitted with bronze bushings at each end and revolves on a l-inch (2.5 centimeters) diameter steel pin mounted in a steel frame which is built up from standard steel bars and angles. The roller assembly is 8 inches (20.3 centimeters) long, 7 5/8 inches (19.4 centimeters) wide, and 5 3/4 inches (14.6 centimeters) high overall. It weighs about 12 pounds (5.4 kilograms). The roller is used to make the placement and removal of transoms easier during the assembly and disassembly of the bridge.

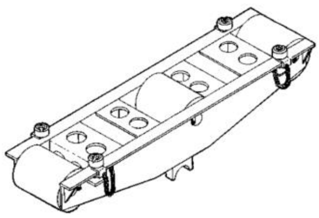


Figure 1.23. Rocking roller

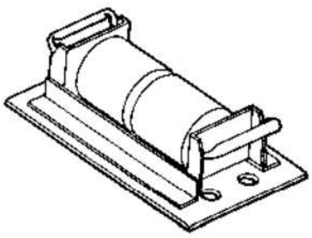


Figure 1.24. Plain rolle

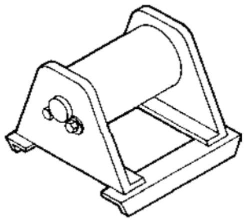


Figure 1.25. Transom roller

ii. The Jack, Panel lever and carrying bar and tongs.

The jack (Figure 2.28) is used to lift the bridge on and off the rocking rollers. It is a mechanical lifting jack (the type normally used in rigging, railroad, and construction work). It has a lifting range of 15 inches (38.1 centimeters) and a capacity on the top of 15 tons (13.6 metric tons). When the weight is carried on its toe, its capacity is only 7 1/2 tons (6.8 metric tons). Jacks from different manufacturers have different spacing (pitch) between the teeth, as listed in Table 2-2. Where jacks are lifting at the same point, all jacks used must have the same tooth pitch so they can be operated in unison. The jack weighs 128 pounds (58.1 kilograms). The panel lever (Figure 1.27), used in assembling the second and third trusses after the first truss is in place over the gap, is a wooden bar 7 feet 9 inches (2.4 meters) long weighing 48 pounds (21.8 kilograms). It has a fulcrum near the center and a lifting link at the end. The lifting link has a swiveling crosspiece which can be readily attached to the top of a panel by passing it through the upper chord and turning it. The upper end of the link slides in a slot the inner end of the slot is used when erecting the second truss, the outer end is used when erecting the third truss. The fulcrum is always placed on the top of the first truss. Two levers per panel are required, with two soldiers operating each lever. A wooden carrying bar is 3 feet 6 inches (1.1 meters) long and reinforced by a steel band at the middle. It is used to carry panels and transoms. It weighs 8 pounds (3.6 kilograms). Carrying tongs are steel and shaped like railroad tongs, as shown in Figure 1.28. These tongs are used to carry transoms by clamping them over the top flange. One soldier carries one of the two handles. Normally, four pair of tongs and eight soldiers are used to carry a transom.

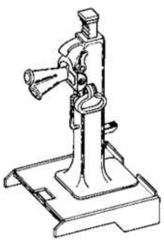


Figure 1.26. The jack(U. Army, 2010)

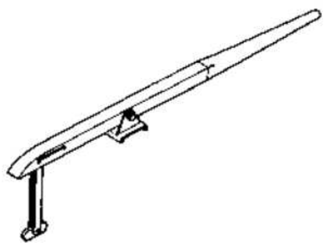


Figure 1.27. Panel lever(U. Army, 2010)

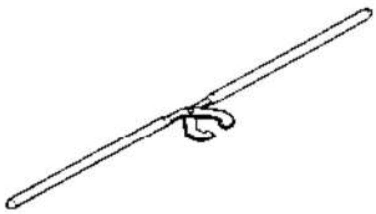


Figure 1.28. Carrying tongs(U. Army, 2010)

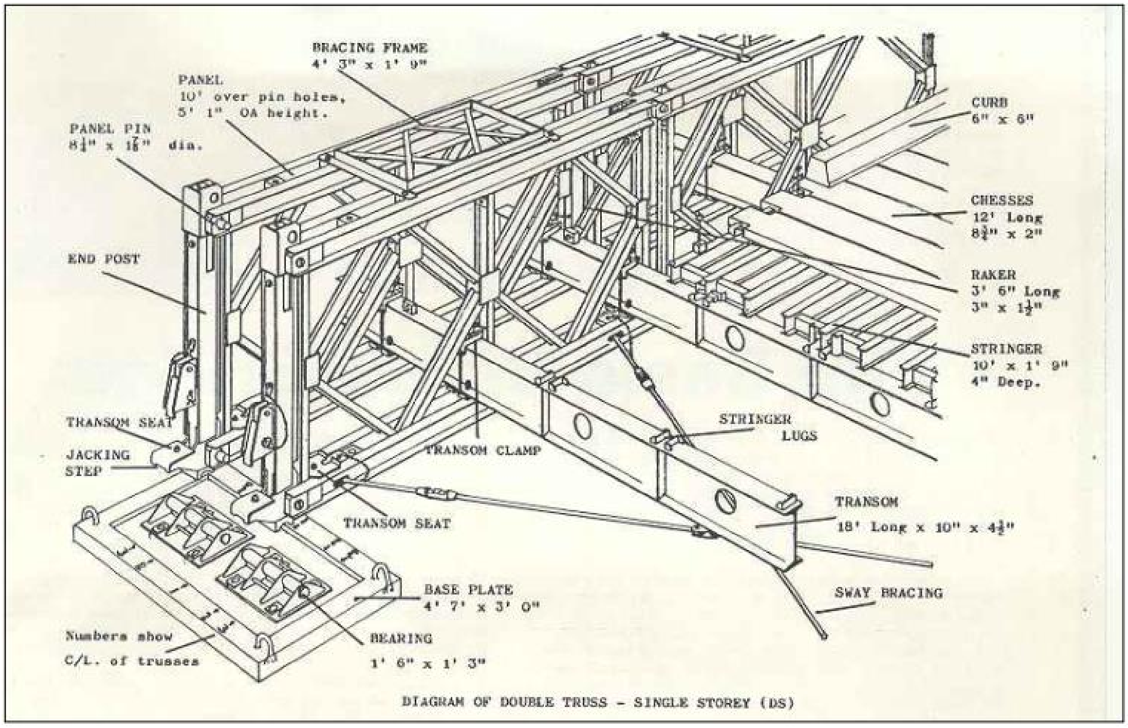


Figure 1.29. Diagram of double truss-singlr storey (Russell & Thrall, 2013)

d. Dead load of Bailey bridge

For the right calculation of the dead load and the needed amount of transportation vehicles a table for each system is given by the manufacturer. The calculation of their weight has to be made at the beginning of the planning phase for a comparison with the soil characteristics.

Table 1.2. Dead load of the Bailey bridge (D. of the Army, 1986)

Bailey bridge		
type		Weight (tones per bay)

SS		2.76
DS		3.41
TS		4.01
JJ		4.66
TD		5.88
DT		6.46
TT		8.29
	Launching-nose	
SS		1.00
DS		1.64
JJ		2.90
	Decking	
Chess and stell ribands		0.66
Stringers only		0.79
Wear treads		0.35
	Accessory	
Sidewalk		0.17

e. Load capacity

Following table shows the maximum load capacity for a single span bridge. The load is given in a military load classification which means different loads for tracked and wheeled vehicles.

Table 1.3. Maximum load capacity for a single span of Bailey bridge (D. of the Army, 1986)

`							
	SS	DS	TS	DD	TD	DT	TT
30	24						
40	20						
50	20	65/65					
60	20	60/60					
70	16	50/55					
80	16	45/50	80/80				
90	12	40/45	70/70				
100	12	35/40	60/60	90/90			
110		30/35	50/55	80/80	100/90		
120		20	40/45	65/70	90/90		
130		16	30/35	50/55	80/80	90/90	
140		12	24	40/45	60/65	75/80	
150			20	30/35	50/55	65/75	
160			1	24	40/45	60/65	80/90
170				16	30/35	50/60	75/85
180				12	20	40/45	65/75
190					16	30/30	50/55
200							35/40

1.5.1.2. The Mabey bridge

The Mabey Compact 200 was introduced in 1986 with taller panels to provide an increased module than the previous Compact 100, which enabled the bridge to span gaps up to 60.96 meters (200 ft) in single story truss construction. Another benefit of the taller panels was that a larger transom could be used enabling the provision for two-lane traffic. Compact 200 Super was introduced in 1992 with panels that had stronger chords, which further increased the bending capacity. The Logistic Support Bridge (LSB) was developed from the Compact 200 Super to meet the requirement of various military users. LSB features include military grillages and ramps, and the system is supported with technical documentation including a safety study, a reliability and maintainability study, training package and user manuals.

The capabilities of one Logistic Bridge Stock Module Set it as follows:

- Two separate 13 bay (40m) span bridges for Military Load Class 80T/110W.

OR

- One 17 bay (50 m) span bridge for MLC 80T/110W.

OR

- One 20 bay (60 m) span bridge for MLC 80T/110W.

One bridge of 80m overall length comprised of two 13 bay (40 m) spans joined with span junction equipment and supported by a 10 m pier for MLC 80T/110W. Cost for One Logistic Bridge Module Set is $1.25M, which includes transportation, training and manuals. Lead times for various components (ramps) ranges from 3 to 12 weeks. Currently there are approximately 1200 meters of bridge modules available for immediate.

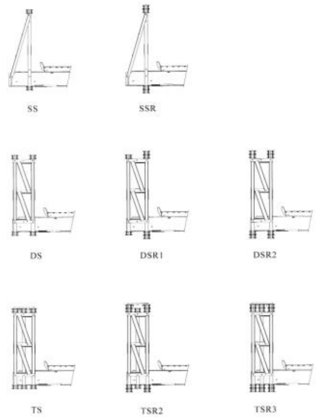


Figure 1.30. Different truss construction type of Mabey bridge (Worenz, 2018)

a. The description of truss construction

Table 1.4. Description of truss construction(*Mabey-Johnson-Bridge-Smartbook-Version-2-Pdf @ Fr.Scribd.Com*, n.d.)

CODE	NOMENCLATURE	DESCRIPTION OF TRUSS CONSTRUCTION
SS	SINGLE SINGLE	Each truss has a single panel line in a single-story format.
SSR	SINGLE SINGLE REINFORCED	Each truss has a single panel line in a single-story format, with a chord reinforcement attached to both the top and the bottom of the panel.
DS	DOUBLE SINGLE	Each truss has two panel lines in a single-story format.
DSR1	DOUBLE SINGLE REINFORCED ONE	Each truss has two panel lines in a single-story format, with a chord reinforcement attached to both the top and the bottom of the inner panel of each truss only.
DSR2	DOUBLE SINGLE REINFORCED TWO	Each truss has two panel lines in a single-story format, with a chord reinforcement attached to both the top and the bottom of both panels of each truss.

TS	TRIPLE SINGLE	Each truss has three panel lines in a single-story format.
TSR2	TRIPLE SINGLE REINFORCED TWO	Each truss has three panel lines in a single-story format, with a chord reinforcement attached to both the top and the bottom of the inner and outerpanels of each truss only.
TSR3	TRIPLE SINGLE REINFORCED THREE	Each truss has three panel lines in a single-story format, with a chord reinforcement attached to both the top and the bottom of all three panels of each truss.

The letter "H" is used after either the panel configuration or the chord reinforcement configuration to signify that Mabey Compact 200 Super Panels (MC411 & MC412) or Super Chord Reinforcements (MC304) are to be used to form the bridge trusses, instead of the original Compact 200 Standard Panels (MC200 & MC201) or Standard Chord Reinforcement (MC302).

for example:

"SSR" signifies those standard panels and standard chord are to be used in the bridge truss "SSRH" signifies those standard panels and super chord are to be used in the bridge truss "SSHR" signifies those super panels and standard chord are to be used in the bridge truss "SSHRH" signifies those super panels and super chord are to be used in the bridge truss.

A mix of standard and super panel stock is unusual, however, and it should be noted that the Logistic Bridging supplied to NATO is the current Compact 200 Super Panel Bridge consisting entirely of Super Panels and Super Chords. Note that the final bay at each end of each simply supported span is always of unreinforced truss construction, even if the bridge is otherwise of reinforced truss construction.

b. Bridge dimensions

All dimensions are in millimeters and are nominal, subject to manufacturing tolerances. All of the dimensions given to the top of the bridge trusses are actually those to the top of the chords. They are, therefore, correct for single panel truss constructions and for double panel truss constructions, where the Bracing Frames are fitted to the underside of the top chords of the Panels. For all triple panel truss constructions, however, 35mm must be added to these dimensions as the Bracing Frames are fitted on top of the chords.

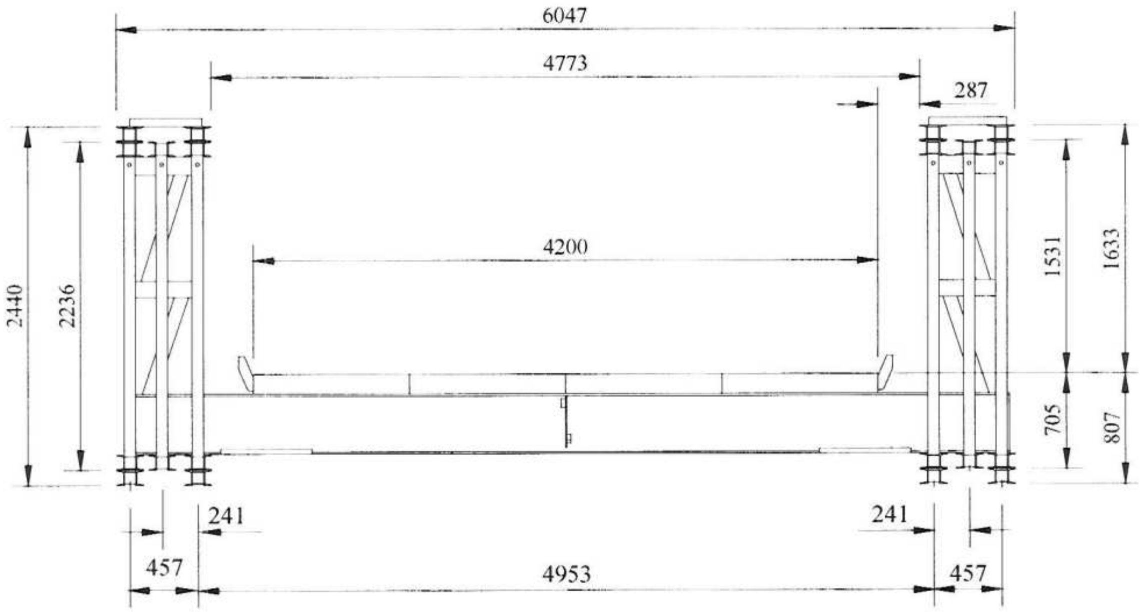


Figure 1.31. Transversal section of the Mabey bridge(*Mabey-Johnson-Bridge-Smartbook-Version-2-Pdf @ Fr.Scribd.Com*, n.d.)

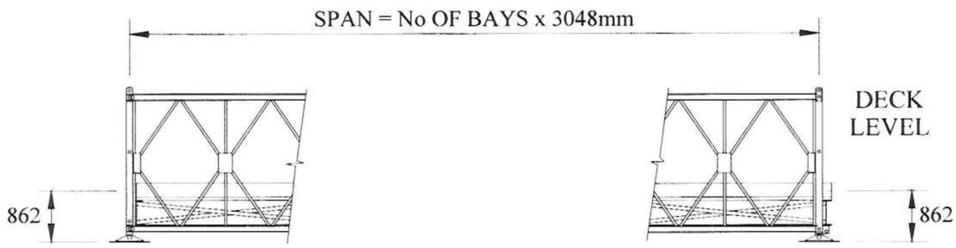


Figure 1.32. Longitudinal view of a single span of Mabey bridge(*Mabey-Johnson-Bridge-Smartbook-Version-2-Pdf @ Fr.Scribd.Com*, n.d.)

1.5.1.3. Acrow bridge

Acrow improved on the design of the Bailey Bridge, producing two unique systems. The first patent, which involved a modification of the original Bailey panels, came in 1973. Some of these improvements include trusses that use a higher-grade steel and thus are lighter and stronger than the Bailey panels. Additionally, the steel roadway deck panels more efficiently distribute the loads across the width of the bridge. Finally, the Acrow 700XS-series panels are 50% taller than the Bailey panels, standing 2.29 m (7 ft 6 in.) tall. The system was designed to carry the heaviest military tanks and earthmovers on the market. It can accommodate from one to three lanes and can span between 6 and 76 m (20 and 250 ft). Typically, the bridge is constructed on one side of the gap and cantilevered over the gap using a launching nose; alternatively, it can be erected with a crane.

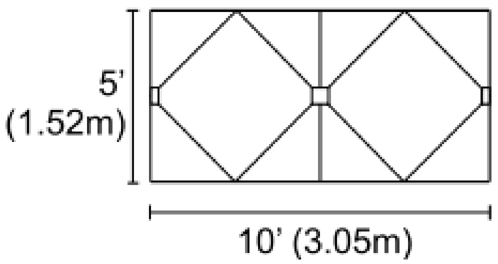


Figure 1.33. Rectangular lattice panel of Bailey bridge(Gerbo et al., 2016)

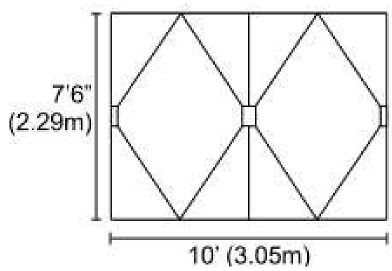


Figure 1.34. Rectangular lattice panel of Acrow bridge(Gerbo et al., 2016)

1.5.1.4. Bailey railway bridge

With the Bailey system it was also possible to build railway bridges up to a span of 60ft. By the fact that a railway bridge has to carry a much higher load than a bridge for land vehicles the girders are much bigger. Girders often have to be quadruple/double to enable a railway crossing the bridge with a speed of 20mph.

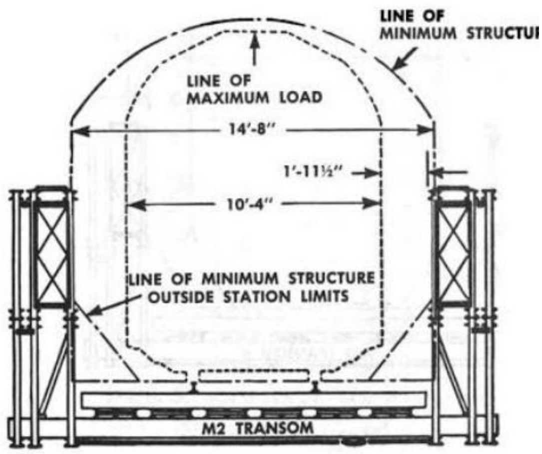


Figure 1.35. Bailey railway bridge(D. of the Army, 1986)

1.5.2. Floating bridge

Floating bridges for military and civil purpose have been known since centuries. There are several different systems by using different materials and techniques. Most time buoyancy is given by floating hollow structures like pontoons or boats.

1.5.2.1. The Bailey Pontoon Bridge

After first use as fixed bridge the system was a full success but it also has to fulfil the tasks of a floating bridge. Pontoons from older systems have been adjusted to the bailey system. The challenge to adjust the system for a floating bridge was the degree of rigidity which was tolerable for the girders. A minimum of rigidity was necessary to ensure that a load could be spread up on several pontoons but a too high rigidity would cause extreme bending moments. After some field tests and analyses, a special adapter was established who connected the bridge girders with the pontoons. With the new part it was possible to transfer the shear forces and reduce the bending moments between the pontoons. The bailey pontoon bridge consists out of a ramp and a landing bay which is spanned from the bank to a special four-pier landing pontoon on each side. Between those special landing pontoons, it was possible to cross a river of an unlimited width with the necessary number of pontoons, bailey panels and adapters. One floating unit has a buoyancy of 14.5 tones and the number of needed pontoons depends on the bridge class.

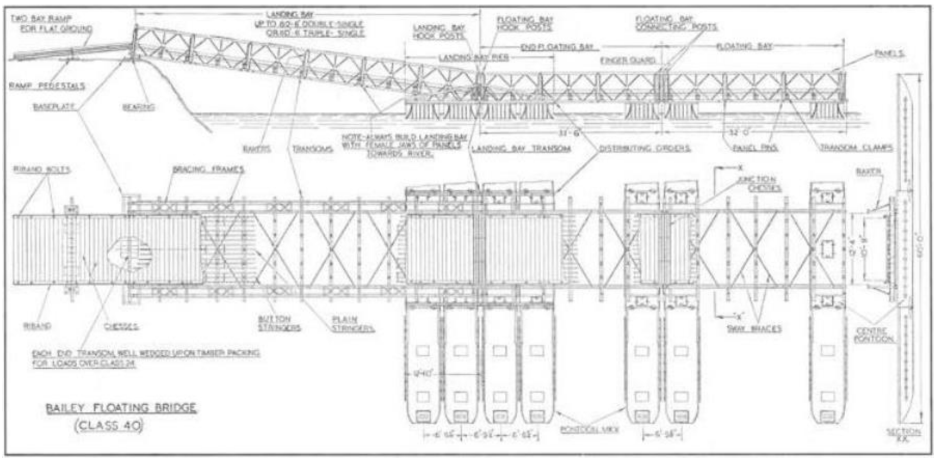


Figure 1.36. Bailey floating bridge (Joiner, 2006).

1.5.2.2. Improved Ribbon Bridge

The Improved Ribbon Bridge (IRB) is the newest generation of foldable floating bridges which allows a two-way traffic for road-legal vehicles and one-way traffic for vehicles with a bigger width. The maximum load capacity of the IRB is 80 tons for tracked vehicles and 96 tons for wheeled vehicles. The IRB can easily handle bank heights from around 2 meters. The operation site could be also in rough water as long the water depth is more than 2.2 meters. The elements are categorized in the main elements for the inner bridge and ramp elements for the connection to the banks, whereby each bay element consists out of two inner and two outer floating bodies. Those bay elements are foldable and have a shape of a W. An element automatically unfolds itself when it is in water (U. Army, 2010).

Figure 1.37. Improved ribbon bridge during use (*Fr.Scribd.Com)*

1.6. General description of load bearing systems

The nature was a role model for most load bearing systems by forming bridges like natural formed rock arches, fallen trees or a liana. Most existing load bearing systems could be separated into three groups, an arch construction, a rope construction or a beam construction. The difference of the classification is how a construction is transferring the external loads. The ideal arch and the ideal rope will be just stressed by a normal force. Whereby in a beam usually just bending stress occur.

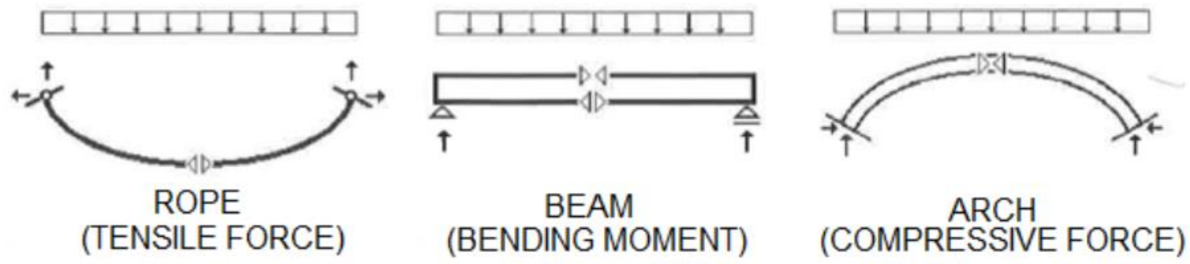


Figure 1.38. Main load bearing system**.**(*BRIDGE ENGINEERING*, n.d.)

1.6.1. Arch system

The optimum arch has to resist just against a compressive force. That is the case when all external loads form a result force which is at the same line as the center-of gravity line of the arch cross section. If the result force differ from the center-of gravity line additional bending moments will occur. Characteristic for an arch system is that the external vertical loads form vertical and horizontal loads at the supports. Those horizontal forces have to be channel in a save way to the foundation soil. Another method would be the use of a tieback which takes the horizontal stress of the construction. The first material which was used for arches was stone by the high compressive characteristic

1.6.2. Beam system

The load bearing effect of a beam system is achieved by the resistant of the bending moment. After an external load a compressive and a tension zone will be formed in the construction. The shearing resistance prevents that a single part of the construction doesn't shift away. At a usual single span girder which is supported at each end the maximum bending force is in the middle of the beam and the maximum shear stress is at each end. Most failures of this system are made by too much pressure at the compressive zone, too much tension at the tension zone or lateral movement of the compressive zone which is called torsional flexural buckling. The oldest material which was used for beam is wood by the characteristic that timber could handle compressive and tension forces.

1.6.3. Truss system

By the need of bridges with a higher span an improvement of the beam system ended in the 19th century with first truss constructions. It was the change from intuitive timber constructions to engineered constructions. Also, by the first use of iron in construction this form as load bearing system was getting more interesting. A typical truss system consists out of several triangular units whereby each truss beam gets forced by compressive or tension stress. All moments are excluded by the fact that all joints in a truss system are made as a kinematic pair. Another fact to avoid moments is that external loads are considered to act only at the nodes and not at the beams. By the fact that a truss system has a very low weight compared to its load bearing capacity such a system is often used for a temporary bridge construction. By that reasons the truss system, with its low dead load compared to the load capacity, is an ideal load bearing system for temporary bridge constructions. That will be also supported by the fact that a truss system is very flexible in its length.

1.7. Bridge design methods

1.7.1. Allowable Stress Design

Allowable Stress Design (ASD) is also referred to as the service load design or working stress design (WSD). The basic conception (or design philosophy) of this method is that the maximum stress in a structural member is always smaller than a certain allowable stress in bridge working or service conditions. The allowable stress of a material determined according to its nominal strength over the safety factor. Therefore, the design equation of the ASD method can be expressed as:

$$\sum \sigma_i \leq \sigma_{all} = \frac{\sigma_n}{FS} \tag{1.1}$$

where σ_i is a working stress due to the design load, which is determined by an elastic structural analysis under the design loading conditions. σ_{all} is the allowable stress of the constructional material. The σ_n is the nominal stress of the material, and FS denotes the safety factor specified in the design specification. Selection of allowable stress depends on several factors, such as the design code, construction materials, stress conditions.

The ASD method is very simple in use, but it cannot give a true safety factor against failure. All uncertainties in loads and material resistance are considered by using the safety factor in ASD. Although there are some drawbacks to ASD, bridges designed based on ASD have served very well with safety inherent in the system. Currently, ASD design method is still used in the bridge design specifications in Japan.

1.7.2. Load Factor Design

To overcome the drawbacks of the ASD design method, the ultimate load design method was developed in reinforced concrete design, which was modified as the Load Factor Method Design (LFD). In this method, different load multipliers were introduced, and the LFD design equation generally can be expressed as:

$$\sum \gamma_i Q_i \leq \emptyset R_n \tag{1.2}$$

where γ_i is a load factor and ϕ is the strength reduction factor, Qi and Rn are, respectively, load effect and nominal resistance.

1.7.3. Load and Resistance Factor Design

Currently, limit state design (LSD) is the most popular design concept for bridge design and widely used for many countries in the world. In the United States, it is known as load and resistance factor design (LRFD). Load and resistance factor design is a design methodology in which applicable failure and serviceability conditions can be evaluated considering the uncertainties associated with loads by using load factors and material resistances by considering resistance factors. The LRFD was approved by AASHTO in 1994 in the LRFD Highway Bridge Design Specifications.

$$\sum \eta_i \gamma_i Q_i \leq \emptyset R_n = R_r \tag{1.3}$$

Equation (1.3) is the basis of LRFD methodology In this equation (AASHTO, 2007). η_i is the load modifier, γ_i is the load factor, ϕ is the resistance factor, Q_i and R_n are load effect and nominal resistance, respectively.

Several limit states, including strength limit state, service limit state, the fatigue and fracture limit state, and the extreme event limit state, are included in this design method. The strength and stability are considered in the strength limit state design. In service limit state design, the stress, deformation, and drack width in service condition should be carefully checked. Stress ranges, stress cycles, and toughness requirement are considered in the fatigue and fracture limit state, and the survival of a bridge during a major earthquake or flood is considered in extreme event limit state. Though the current design specification in Japan is based on the ASD design, the LRFD method is also used for designing the Tokyo Gate Bridge in Japan.

1.8. Loads and Load Distribution

Bridge structures are designed to carry traffic during their service lives. Bridge loads are actions in the form of forces, deformations, or accelerations applied to a structure or its components. The load acting on the bridge structures are generally divided into two categories such as those acting on the superstructure, and those acting on the substructure. The major load components of highway bridges are dead load, live load (static and dynamic), environmental loads (temperature, wind, and earthquake), and other loads (collision, emergency braking).

1.8.1. Dead load

Gravity loads of constant magnitudes and fixed positions act permanently on the structure. Such loads consist of the weights of the structural system itself and of all other material and equipment permanently attached to the structural system. In the bridge design, the dead load denotes the constant load in a bridge due to the weight of the members, the supported structure, and permanent attachments or accessories.

Table 1.5. Specific weight of different materials**.**(*BRIDGE ENGINEERING*, n.d.)

Material	Unit Weight	Material	Unit Weight
Steel	77	Concrete	23
Cast iron	71	Cement mortar	21
Aluminum	27.5	Wood	8.0
Reinforced concrete	24.5	Bituminous material (water proofing)	11
Prestressed concrete	24.5	Asphalt pavement	22.5

1.8.2. Live load

Live load in bridge design generally refers to loads due to moving vehicles that are dynamic, or the loads that change their positions with respect to time. This is unlike building structure, where live loads are the occupancy loads, which are considered as static load (Taly, 1997). The live load has been increasing with the progress of time. For modern bridges, their service lives are generally decades or even more than a hundred years. Therefore, the appropriate calculations or predictions for future service loads are necessary. It exists specifications depending on the country like US, Japanese and European specification.

1.8.2.1. Live Load in US Specification

In the United States, the design vehicular live load is divided into three categories, namely design truck load, design tandem load, and design lane load.

The design truck specified in AASHTO Load and Resistance Factor Design (2007) specify that, the weight of the front axle is 35,000 N with double rear axles weighing 145.000 N, respectively. The spacing between the two 145,000 N axles needs to be varied from 4,300 to 9,000 mm to produce the extreme force effects. The tire contact area of a wheel consisting of one or two tires is assumed to be a single rectangle with 510 mm in width and 250 mm in length. The design tandem consists of a pair of 110,000 N axles spaced 1,200 mm apart, with a transverse spacing of 1,800 mm for the wheels. With regard to the design lane load, a uniformly distributed load of 9.3 N/mm shall be applied in the longitudinal direction with a width of 3,000 mm in the transverse direction. In the design, such loading conditions shall be considered to produce the extreme force effects like the effects of the design tandem together with the design lane load, the effects of one design truck with the variable axle spacing plus the design lane load, and for negative bending moment regions, 90% of the effects of two design trucks with a minimum spacing of 15,000 mm between the lead axles of one truck and the rear axle of the other truck, plus the 90% of the effects of the design lane load. In this case, the spacing between the two 145,000 N axles is taken as 4,300 mm.

1.8.2.2. Live Load in Japanese specification

Both truck load and lane load used in Japan bridge design specification are shown in Figure 1.40 and 1.41, respectively. The truck load, also denoted as T-load, is mainly used for designing the deck systems. The lane load, or L-load, is mainly used for designing the main girders. For the lane load, the distributed load of p1 and p2 are taken based on the loading conditions of the oversized vehicles. A-Type live load is used for municipal roads with a small number of oversize vehicles, while the B-Type live load is used for national highway, a prefectural road (highway), and a main (principal, trunk) road with large number of oversize vehicles. In addition, the loading intensities of p1 and p2 also vary according to the internal forces including shear force or bending moment, as indicated in Table 1.5.

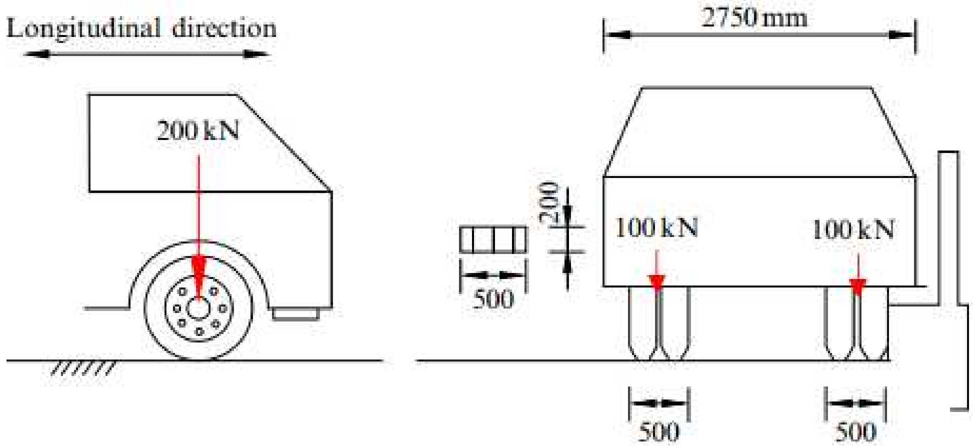


Figure 1.39. T-load in Japanese bridge design specification (*BRIDGE ENGINEERING*, n.d.)

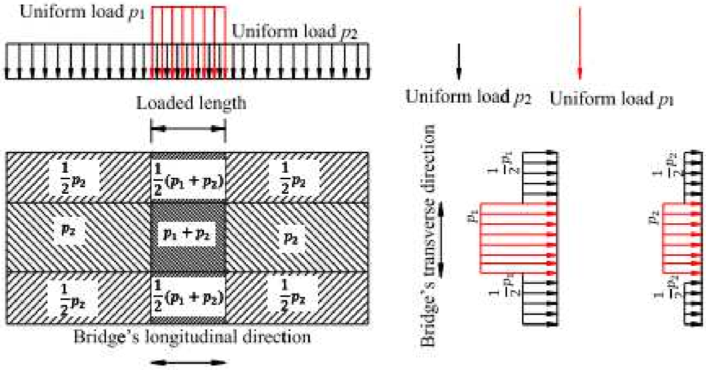


Figure 1.40. L-load in Japanese bridge design specification(*BRIDGE ENGINEERING*, n.d.)

Table 1.6. Distributed load intensities of the lane Load (*BRIDGE ENGINEERING*, n.d.)

		Live Load on Main Lanes (Width: 5.5 m)					
		Uniform Load p_1 (kN/m^2)		Uniform Load p_2 (kN/m^2)			
Load	Load Length D (m)	For Bending Moment	For Shear Force	L≤80	80<L≤130	130≤L	Live Load on Secondary Lanes
A-Live Load	6	10	12	3.5	4.3–0.01 L	3.0	50% of live load on main lanes
B-Live Load	10						

1.8.2.3. Live Load in European specification

The Eurocode EN 1991-2(2003) specifies four loading models to be considered as the road traffic effects for the ultimate limit states with particular serviceability verifications. The Load Model1 (LM1) represents normal traffic which consists of Tandem System (TS) with double-axle concentrate loads 1.2 m apart (center to center) and UDL to include the effect of lorries and cars. The LM1 should be used for both general and local verifications. The TS and UDL have the intensity of $\alpha_Q Q_k$ (for each axle) and $\alpha_q q_k$, respectively. The adjustment factors α_Q and α_q are specified in the National Annex. The Q_k is distributed on two wheels 2 m apart (center to center) over the contact surface of 0.4 m each. On each lane, only one TS (as shown in Fig. 1.41) should be used with at most three loading lanes. The intensity for each loading lane is specified in Table 1.7. The q_k should apply on all the unfavorite part of the influence surface, whose intensity is

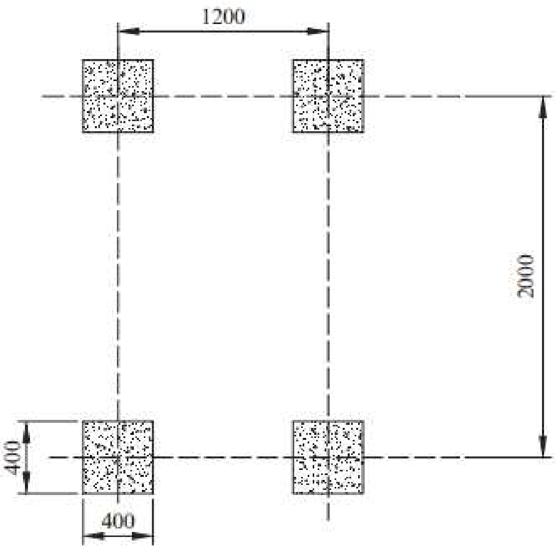


Figure 1.41. Details of one single Tandem in Eurocode(*EN 1991-2*, 2003)

Table 1.7. Intensity of Axle load for TS (*EN 1991-2*, 2003)

Lane Number	Tandem System (TS) Axle Load Q_k (kN)
1	300
2	200
3	100
≥4	0

Table 4.4 Intensity of Uniform Distributed Load (UDL)

Lane Number (*i*)	Uniform Distributed Load (UDL) q_k (kN/m^2)
1	9
≥2 (and other remaining areas)	2.5

1.9. Construction phase

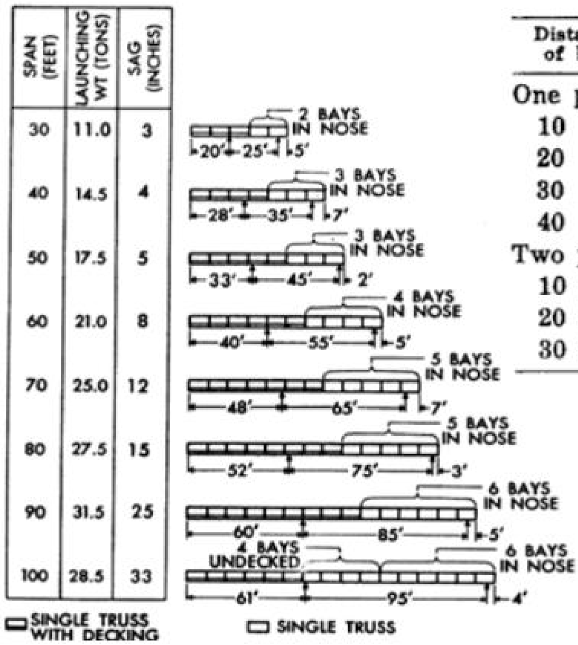
The usual construction process of a temporary bridge construction is the launching method. Therefore, a launching nose, rocking rollers and plain rollers are needed. Another construction scheme is an erection by lifting the bridge in position by one or two cranes. Beside cranes also helicopters could be used for construction which is more the exemption.

1.9.1. Launching method

The launching method is besides lifting the whole bridge by crane or other schemes the usual way how to build a temporary bridge construction. For most bridge types that process could be done with manpower. For bigger types like a triple-triple bridge, machines like a bulldozer or an excavator are necessary. While the bridge is a hug cantilever during that process it has to be always sure that there is enough weight behind the rollers to avoid tipping into the gap. This weight will be reached by mounting several panels at the end of the bridge or if not, enough place is available by the use of counterweights. Additionally, a launching nose is temporarily mounted on the front. This launching nose has no stringers or decking and is lighter compared to the finished bridge construction.

The temporary construction is mounted with an angle which is required to compensate the bigger deflection during the cantilever phase. This angle and the resultant vertical lift of the nose are variable by the use of special links between the standard panels. For a bigger lift the special connector is mounted more at the end of the launching nose. To know where such a link has to be placed is shown in a table which also gives the resultant vertical lifting of the nose. If the far-bank is lower than the near-bank the links for an angle of the launching nose are not needed. The amount of required panels for the launching nose is shown in tables and depends on the span and type of the final bridge structure (D. of the Army, 1986) While the bridge is pushed over rocking and plain rollers the construction is higher during the erection process. To get the bridge in its final position it is necessary to jack the ridge down. When the bridge is placed on both banks it could be started with the deconstruction.

The deconstruction of the launching nose and the additional elements which were needed for the counterweight. When this step is done the end posts and end transoms have to be installed before the jacking down process could be started. The process is done by several steps which are always supported by construction timber in case a jack will fail. It doesn't matter which side of the bridge will be jacked down first. Important is that both girders at one side will be jacked down continuously at same speed to guarantee the same load bearing on all jacks. When the end posts sit on the bearings the bridge could be finalized for the use.



Distance of links from tip of launching nose (feet)	Resulting vertical lift of tip of nose (inches)
One pair of links:	
10	13½
20	27
30	40½
40	54
Two pairs:	
10 and 40	67½
20 and 40	81
30 and 40	94½

Figure 1.42. Different types of launching noses(U. Army, 2010)

1.9.2. Lifting in

It exists many types of lifting in that are the lifting in by crane, lifting in by helicopter and Crane assisted launching.

1.9.2.1. Lifting in by crane

An easier and often faster method to construct a temporary bridge is the installation by crane. This method requires the availability of a crane with a sufficient load bearing capacity. Therefore, a previous erected or partially preassembled bridge will be lifted in position by one or two cranes. For that process usually mobile cranes or excavators for smaller bridges are used. (Arthur Ward Buckingham, 1911) For the lifting process it is important that the crane has the right equipment to spread the rope to same wide like the bridge that each truss will be stressed by just vertical loads. By using that method, the preparation of the bearing units have to be done before the lifting process starts. Most bridges have special devices close to the nodes where the structure could be connected to the crane. In case that a bridge construction is too heavy for an availably crane the bridge could be lifted in without the decking and final construction will be done followed by hand or machine. Compared to a usual erecting process by launching this method is commonly used for single span bridges and for sites where the erection of a longer bridge as counter weight is not possible. Also, a lifting process needs less bridge elements which are typically needed for the launching process.

1.9.2.2. Lifting by helicopter.

A special form of lifting in which is usually just done by military is the use of a helicopter. This form of lifting a bridge in position is very rarely and is just used when other construction methods are not possible by different facts. The weight of the bridge is limited by the maximum load capacity of the helicopter. Also, it is not possible to transport a complete finished bridge. For the transportation the decking has to be removed from the bridge to enable a sufficient air flow for the aircraft.

Figure 1.43. Bridge lifted in by a mobile crane(*Index @ Www.Estrepublicain.Fr*, n.d.)

Figure 1.44. Bridge lifted in by helicopteter (*Index @ Www.Lindependant.Fr*, n.d.)

1.9.2.3. Crane assisted launching

The crane assisted launching method is a mixture between a lift in by crane and the typical launching method. For that construction method a crane with a sufficient load capacity, rocking and plain rollers are needed. The bridge will be erected as usual on the rollers with the difference that no launching nose will be mounted on the bridge. If enough elements are mounted on the bridge the launching method could start. Therefore, the crane which is usual on the opposite bank will be connected to the end of the bridge. The function of the crane is to prevent a tip over and compensate the deflection during construction phase. Additionally, the crane is helping a little at the pushing process over the obstacle whereby it is not a replacement of man or machines which are the main pushing force. This construction method could be used when a bridge doesn't fulfill the requirements of a lift in by crane for example by a too heavy weight of the bridge, a too wide obstacle or not enough place on site for two mobile cranes. The advantages compared to a launching process are that fewer elements are required for the erection process. The launching nose and the elements for a counterweight could be saved by a crane assisted launching.

Figure 1.45. Crane assisted launching process (*Index @ Www.Lindependant.Fr*, n.d.)

1.9.3. Inspection during use

Before a finished bridge is going into service most temporary bridges have to be checked from authorized personnel. Additional inspections are necessary in temporal intervals which are given in the user manuals. Thereby the superstructure and especially the connections have to be checked if they are loose by traffic. Also, the bridge construction has to be checked after cases of Accidents where the superstructure is involved, determination of any damage, which could affect the structural stability, bigger storms, flooding, bigger traffic accidents on bridge, earthquakes, visible settlements, unapproved use by heavy load transportations.

1.9.4. Deconstruction

The deconstruction of temporary bridge constructions is usual like the erection process in opposite way. The deconstruction process is in general much faster because the exactly adjustment of rollers and jacks lapse. During the deconstruction all parts of the bridge system should be inspected on possible damage. Also screw connections should be lubricated to prevent corrosion and enable an easy construction during next use.

conclusion

In this chapter, the concept of temporary bridge and design procedure has been introduced, presenting a review of the previous research carried out. It is therefore necessary to carry out in-depth investigations on structural improvement of steel temporary bridge. The next chapter presents the method and theories involved in the design and inprovement of a steel truss temporary bridge subject to a conventional load model.

CHAPTER 2. METHODOLOGY

Introduction

Following the literature review done in the previous chapter which enabled us to have a broad knowledge on the concept of Temporary bridges notably on its genesis, the design strategies and the erection performances, this chapter will focus on the description of the methodology of work. The methodology is the part of the study that establishes the research procedure after the definition of the problem, so as to achieve the set objectives. It is partitioned in different sections, the first being a general recognition of the site done by documentary research. This is followed by data collection that will enable the modelling and analysis of a Prefabricated Modular Steel Bridge. Thereafter, this chapter will focus on the description of the design procedures and the governing equations used by analytical and numerical procedures which are intended to be used for the study and structural improvement of Steel Truss Bridge. For this study, the definition of loads and the design of the structural members will be done using the European norms. Nowadays, modern software makes analysis ever increasing number of structural problems. However, the results of these analyses are strongly dependent on the assumptions made and the understanding of the working principles of the software used, so care is always recommended when adopting numerical solutions. An overview of softwares used in this thesis and modelling will be made.

2.1. Site recognition

Recognition of the site was done through research from available documents in order to know on one hand the general physical characteristics (geographical location, climate, hydrology and geology), and other hand the socio-economic parameters in the other side.

2.1.1. Site visit

Some pictures of the project site were taken on the site. The activity on the site and the purpose of the temporary bridge was obtained by questioning the population present on the site project.

2.1.2. Questionnaire

A questionnaire was addressed to the direct project impact zone that is the population present on the construction site of hydroelectric dam and population from Nachtigal concerning the purpose of a temporary bridge, the climate in the region, the economic activities.

2.2. Data collection

The main type of data required for the purpose of this research is the architectural and structural data of the bridge.

2.2.1. Architectural data

These data collected gives information on the type and geometry of the bridge, the surface area and the specific use of the structure.

2.2.2. Structural data

These data collected gives information on the section's properties of structural elements, the characteristic of materials used as well as the load applied on the structure.

2.3. ANALYSIS AND LOADING METHODS

A common situation in structural mechanics is that a structure is only affected by static forces. If the structure is affected by a dynamic force i.e., a force that varies with time, it may have a different response compared with the response of a static force. This section is dedicated to procedures used to outcome steel bridge reactions under static and dynamic loads.

2.3.1. Static and dynamic loading method

Different types of actions act on the bridge. Regarding our study, we will focus on permanent, variable loads. Permanent loads are represented by $G_{k,j}$; $Q_{k,i}$ express variable loads.

2.3.1.1. Load combinations

A load combination defines a set of designed values used for the verification of the structural reliability for a limit state under the simultaneous influence of different actions. The designed value is the value obtained by multiplying the representative value by a partial factor.

Eurocodes

The main codes applied in this thesis are:

- EN 1990 Eurocode 0: Basis of structural design
- EN 1991 Eurocode 1: Actions on structures
- EN 1993 Eurocode 3: Design of steel structures

a. Static load cases

As recommended in EN 1990, the following rules have been considered for the combination of loads with respect to static loads applied in a structure. In the case of a bridge, it is consider:

i. Fundamental combination

This combination is used for Ultimate Limit State (ULS) associated to determining of structure resistance and is given by equation 2.1:

$$\sum_{j} \gamma_{G,j} * G_{k,j} + \gamma_{Q,1} * Q_{k,1} + \sum_{i>1} \gamma_{Q,i} * \psi_{0,i} * Q_{k,i} \tag{2.1}$$

Where the coefficients $\gamma_{G,j}$ *and* $\gamma_{Q,i}$ are partials factors which minimize the loads which tend to reduce the solicitations and maximise the ones that increase them. The recommended values preconized by the Eurocode 0 for the partial safety factors are given in table 2.1.

Table 2.1. Partial safety factors for ULS combination *(EN 1990)*

Partial factor	Favorable	Unfavorable
$\gamma_{G,j}$	1.35	1.00
$\gamma_{Q,1}$	1.50	0.00
$\gamma_{Q,i}$	1.50	0.00

ii. Characteristic combination (rare)

Usually used for non-reversible Serviceability Limit States (SLS), this combination (2.2) has to be used in the verifications with the allowable stress method:

$$\sum_{j} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \psi_{0,i} * Q_{k,i} \tag{2.2}$$

iii. Frequent combination

Frequent combination (2.3) is recommended for reversible SLS:

$$\sum_{j} G_{k,j} + P + \psi_{1,1} * Q_{k,1} + \sum_{i>1} \psi_{2,i} * Q_{k,i} \tag{2.3}$$

iv. Quasi-permanent combination

Generally used for long-term effects, it is given by equation (2.4).

$$\sum_{j} G_{k,j} + P + \sum_{i\geq 1} \psi_{2,i} * Q_{k,i} \tag{2.4}$$

The values recommended for the reduction factors are given in Table 2.2.

Table 2.2. Multipliers for the characteristic values of variable loads *(EN 1990)*

	Temperature load	**Wind load**	**Traffic load**	
			TS	**UDL**
ψ0	0.6	0.6	0.75	0.4
ψ1	0.5	0.2	0.75	0.4
ψ2	0.5	0	0	0

b. Dynamic load cases

During the nominal life of a structure, actions such as explosions, fire can occur accidentally. In prevision of such an event, EN 1990 formulated a combination used for the ULS related to the design of accidental actions. Equation (2.5) presents the accidental load combination:

$$G + P + A_d + (\psi_1 \; or \; \psi_2)Q_{k,1} + \sum \psi_2 \, Q_{k,i} \tag{2.5}$$

Where:

G: is the self-weight of the structure

A_d: is the design value of the accidental load

$Q_{k,1}$: is the characteristic value of the leading variable load

$Q_{k,i}$: are the characteristic values of the accompanying variable loads

The values of ψ_1 and ψ_2 depend on the relevant accidental design situation.

2.3.1.2. Load actions

This section focuses on the calculation of loads acting on the bridge. These loads include permanent and variable loads. Blast load and its parameters will also be presented.

i. Permanent loads

Dead loads represent actions which remain relatively constant over the structure life cycle. They comprise the weight of bridge's structural elements (g_1) such as steel tube, slab, and the others. Weight of non-structural elements (g_2) is also included in dead loads.

ii. Live loads

The live load on the bridge is moving load throughout bridge length. The moving loads are vehicles, pedestrians, etc.

iii. Traffic loads

The number and the width of notional lanes result from Table. 2.3 (*EN 1991-3*, 1997).

Table 2.3. Number and the width of notional lanes (*EN 1991-3*, 1997)

Carriageway width w	Number of notional lanes	Width of a notional lane w_l	Width of the remaining area
$w < 5{,}4$ m	$n_1 = 1$	3 m	w - 3 m
$5{,}4 \text{ m} \leq w < 6 \text{ m}$	$n_1 = 2$	$\frac{w}{2}$	0
$6 \text{ m} \leq w$	$n_1 = Int\left(\frac{w}{3}\right)$	3 m	$w - 3 \times n_1$
NOTE For example, for a carriageway width equal to 11m, $n_1 = Int\left(\frac{w}{3}\right) = 3$, and the width of the remaining area is 11 - 3×3 = 2m.			

There are two types of traffic loads that are vertical load (Load Model) and Horizontal load.

- **Vertical forces**

For the complete analysis of the vertical forces, the traffic Load Model 1 (LM1) has been considered. This Load Model is constituted of a tandem load (TS) and a uniformly distributed load (UDL) (see *EN 1991-2*, 2003). Table 2.4 recaps the values to consider for loads due to traffic.

Table 2.4. Characteristic values of load model 1(*EN 1991-2*, 2003)

Location	Tandem system *TS*	*UDL* system
	Axle loads Q_{ik} (kN)	q_{ik} (or q_{rk}) (kN/m^2)
Lane Number 1	300	9
Lane Number 2	200	2,5
Lane Number 3	100	2,5
Other lanes	0	2,5
Remaining area (q_{rk})	0	2,5

In the following we adopt the load group gr1a from Table 2.5. It is displayed in equation (2.6)

$$gr1a = TS_k + UDL_k + q*_{fk} \tag{2.6}$$

Where $q*_{fk}$ represents the uniformly distributed load on footways and cycle tracks.

Table 2.5. Assessment of groups of traffic loads (EN 1991-2, 2003)

		CARRIAGEWAY						FOOTWAYS AND CYCLE TRACKS
Load type		Vertical forces				Horizontal forces		Vertical forces only
Reference		4.3.2	4.3.3	4.3.4	4.3.5	4.4.1	4.4.2	5.3.2-(1)
Load system		LM1 (TS and UDL systems)	LM2 (Single axle)	LM3 (Special vehicles)	LM4 (Crowd loading)	Braking and acceleration forces [a]	Centrifugal and transverse forces [a]	Uniformly Distributed load
Groups of Loads	gr1a	Characteristic values						Combination value [b]
	gr1b		Characteristic value					
	gr2	Frequent values				Characteristic value	Characteristic value	
	gr3 [d]							Characteristic value [c]
	gr4				Characteristic value			Characteristic value
	gr5	See annex A		Characteristic value				
	Dominant component action (designated as component associated with the group)							

[a] May be defined in the National Annex (for the cases mentioned).
[b] May be defined in the National Annex. The recommended value is 3 kN/m^2.
[c] See 5.3.2.1-(2). One footway only should be considered to be loaded if the effect is more unfavourable than the effect of two loaded footways.
[d] This group is irrelevant if gr4 is considered.

- **Horizontal forces**

For the analysis of the horizontal forces the braking and acceleration force have been considered.

A characteristic braking force, Q_{lk}, is a longitudinal force acting at the surfacing level of the carriageway Q_{lk}, limited to 900 kN for the total width of the bridge, is calculated as a fraction of the total maximum vertical loads corresponding to Load Model 1 and applied on Lane Number 1.

This force is given by the equation (2.7).

$$Q_{lk} = 0.6\,\alpha_{Q1}(2Q_{1k}) + 0.10\alpha_{q1}q_{1k}w_1L \tag{2.7}$$

Where:

$180\alpha_{Q1}$kN$\leq Q_{lk} \leq 900$ kN

$\alpha_{Q1} = \alpha_{q1} = 1$ (for one lane)

$Q_{1k} = 180 + 2.7$L for $0 \leq$ L ≤ 1.2 m

$Q_{1k} = 360 + 2.7$L for L ≥ 1.2 m

L = length of the deck or of the part of it under consideration

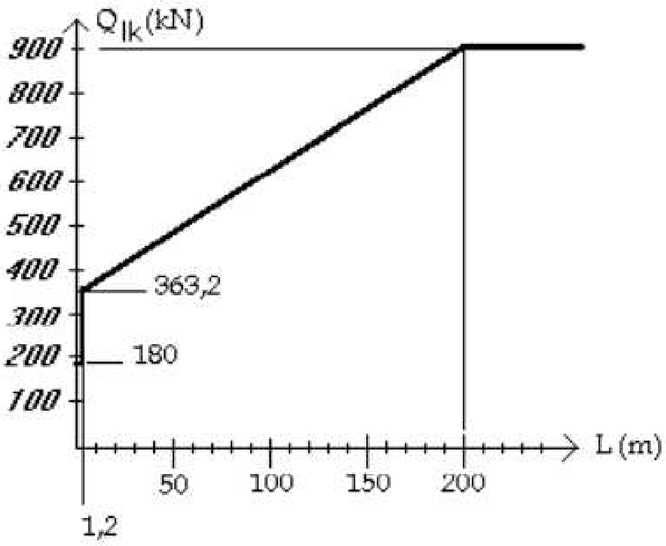


Figure 2.1. Variation of Q_{lk} depending on the braking distance ***(EN 1993-3, 2007)***

iv. Wind load

The general expression of wind force F_w acting on a structure or a structural component can be determined directly by using equation (2.8)

$$F_w = c_s.c_d.c_f.q_p(z_e).d_{tot} \tag{2.8}$$

Where :

$q_p(z_e)$ is the peak velocity pressure at reference height Ze

d_{tot} is the total depth of the structural element z_e

$c_s.c_d$ is the structural factor

c_f is the force coefficient

The peak velocity pressure at height z is expressed by equation (2.9)

$$q_p(z) = \frac{1}{2}.\left(1 + 7.I_v(z)\right).\rho.v_m^2(z) = c_e(z).q_b \tag{2.9}$$

Where:

ρ is the air density

$v_m(z)$ is the mean wind velocity at height z and is given by equation (2.10)

$$v_m(z) = c_r(z).c_o(z).v_b \tag{2.10}$$

$c_r(z)$ is the roughness coefficient

$c_o(z)$ is the orography coefficient

v_b is the basic wind velocity

$I_v(z)$ is the turbulence intensity and can be obtained from equation (2.11)

$$(I_v(z) = \frac{k_l}{c_o(z).ln(z/z_o)} \tag{2.11}$$

k_l is the turbulence factor

z_0 is the roughness length

$$v_b = c_{dir} * c_{season} * v_{b,0} \tag{2.12}$$

c_{dir} is the directionl factor

c_{season} is the season factor

Roughness length and coefficients depends on terrain category and parameters. The procedure to find values of different coefficients is detailed in (*EN 1991-1-4*)

The wind used for design will be wind acting on:

- Bridge during its service life, without traffic;
- Bridge during its service life, with traffic;
- Bridge under construction (finished and most critical case).

v. Temperature load

Temperature load has two components, the uniform one and the vertical non-linear one.

- **Uniform temperature component**

It depends on the minimum and maximum temperature which a bridge will achieve. Having the minimum shade air temperature (T_{min}) and maximum shade air temperature (T_{max}) for the site, we can derive minimum and maximum uniform bridge temperature components $T_{e.min}$ and $T_{e.max}$ by using the graph on Figure. 2.2.

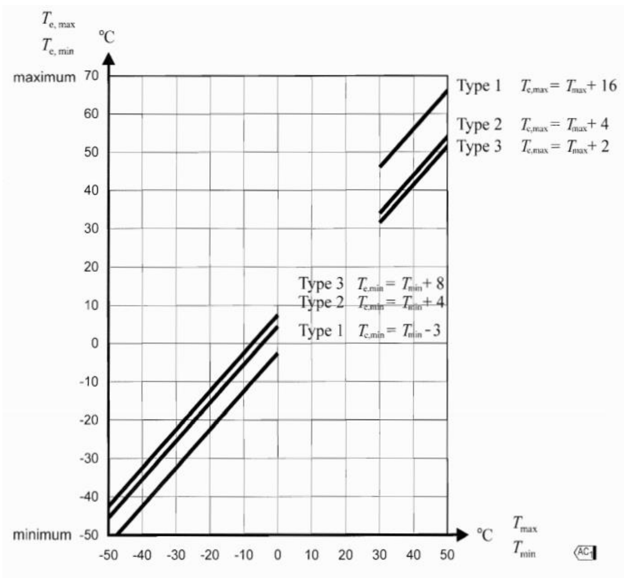


Figure 2.2. Correlation between minimum/maximum shade air temperature and minimum/maximum uniform bridge temperature component *(EN 1991-1-5,* 2011*)*

The initial bridge temperature to is important for calculating contraction down to the minimum uniform bridge temperature component and expansion up to the maximum uniform bridge temperature component.

Thus the characteristic value of the maximum contraction range of the uniform bridge temperature component, ΔTN,con should be taken as :

$$\Delta TN, con = T_0 - T_{e,min} \quad (2.13)$$

And the characteristic value of the maximum expansion range of the uniform bridge temperature component, ΔTN,exp should be taken as :

$$\Delta TN, exp = T_{e,max} - T_0 \quad (2.14)$$

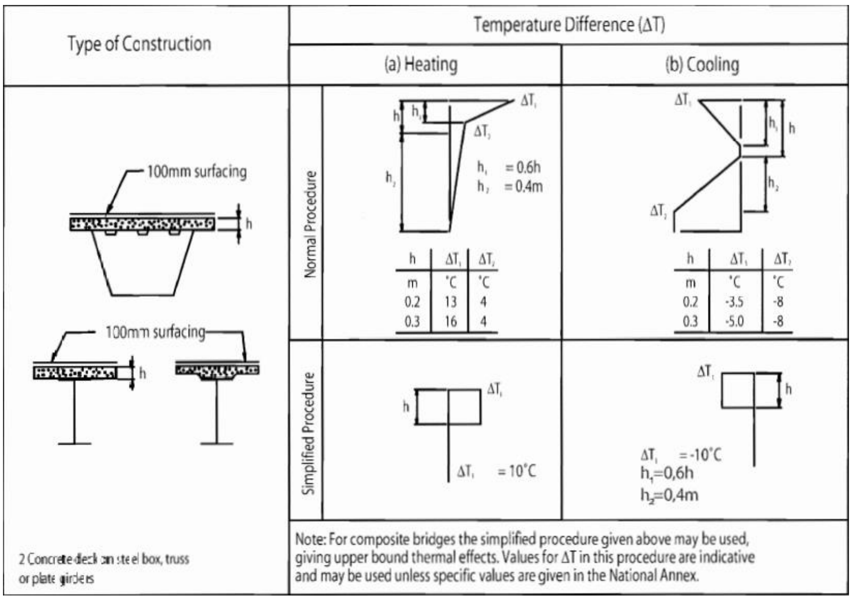
The overall range of the uniform bridge temperature component is

$$\Delta TN = T_{e,max} - T_{e,min} \quad (2.15)$$

- **Vertical non-linear component**

The vertical temperature gradient applied on bridge deck was defined according to Table 2.6 in (*EN 1991-1-5*, 2011).

Table 2.6. Temperature differences for bridge decks type 2: composite decks (*EN 1991-1-5*, 2011)



The load (T_k) due to differential thermal variation (between slab and metal beams) is expressed by equation (2.16):

$$T_k = A_W.\varepsilon.E_W \quad (2.16)$$

Where:

A_W	is the slab area
ε	is a thermal coefficient
E_W	is the wood elastic modulus with

Because of the study of an effective section of the bridge, it is necessary to apply the corresponding temperature load. The force T_{beam} that will be considered is:

$$T_{beam} = T_k/n \quad \text{Where n is the number of girders.} \quad (2.17)$$

2.3.2. Structural analysis and design methods

In this section, the software used for the modelling will be presented, and verifications that has to be made for the analysis.

2.3.2.1. Software used for modelling and analysis of the structure

Numerical modelling in civil engineering is used as a tool that facilitates the engineers to evaluate the behavior of structures. The numerical methods are convenient, and less time-consuming for the analysis of redistribution of stresses and designing of structures. Numerical methods give the exact mathematical solution for the problem based on the engineering judgment and input parameters like physical and strength parameters of steel structure.

To study the design improvement of the steel temporary bridge, Midas/Civil, SAP2000 version 22 and Microsoft Excel 2019 will be used. The modelling of the structure will be done on SAP2000 version 22 and Midas/civil 2019 and scientific representation on Excel. Modelling will consist of creating the appropriate material, section properties, loads and combinations. The steel elements shall be drawn according to plan and the supports conditions assigned to be fixed and pinned where concerned. The structure shall be loaded with respect to specific load patterns discussed in section 2.3.1 The load combinations will be defined prior to the analyses to satisfy the ULS and SLS conditions discussed in section 2.3.2.

a. MIDAS/civil 2019 description

To do static analysis of the bridge under permanent and live loads, MIDAS/civil was used.

The different modules that will be used are presented as follows:

- Properties: this section is meant for definition of material and section properties of different elements (Top and Bottom Cord, Crossing Tube, Diagonal Element)
- Boundary: here, the definition of boundary restrains, rigid and elastic links of our bridge.
- Loads: loads cases are defined here (self-weight, moving loads, element loads, nodal loads).
- Results: different results (displacements, stresses, moments, axial, shear forces…) can be displayed.

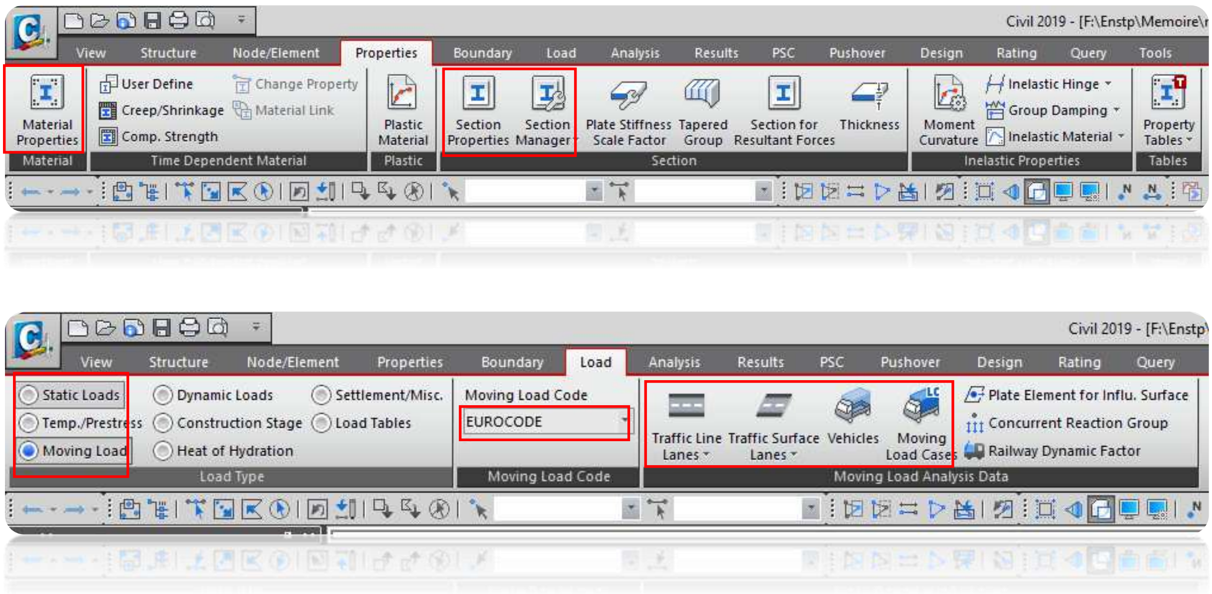


Figure 2.3. Midas/Civil 2019 menu

b. SAP 2000 version 22 description

To do static analysis of the bridge under permanent and live loads, SAP 2000 version 22 was used.

The different modules that will be used are presented as follows:

- Define: this section is meant for definition of material and section properties of different elements (Top and Bottom Cord, Crossing Tube, Diagonal Element)
- Assign: here the definition of boundary restrains, rigid and elastic links of our bridge.
- Loads: loads cases are defined here (self-weight, moving loads, element loads, nodal loads).
- Results: different results (displacements, stresses, moments, axial, shear forces…) can be displayed.

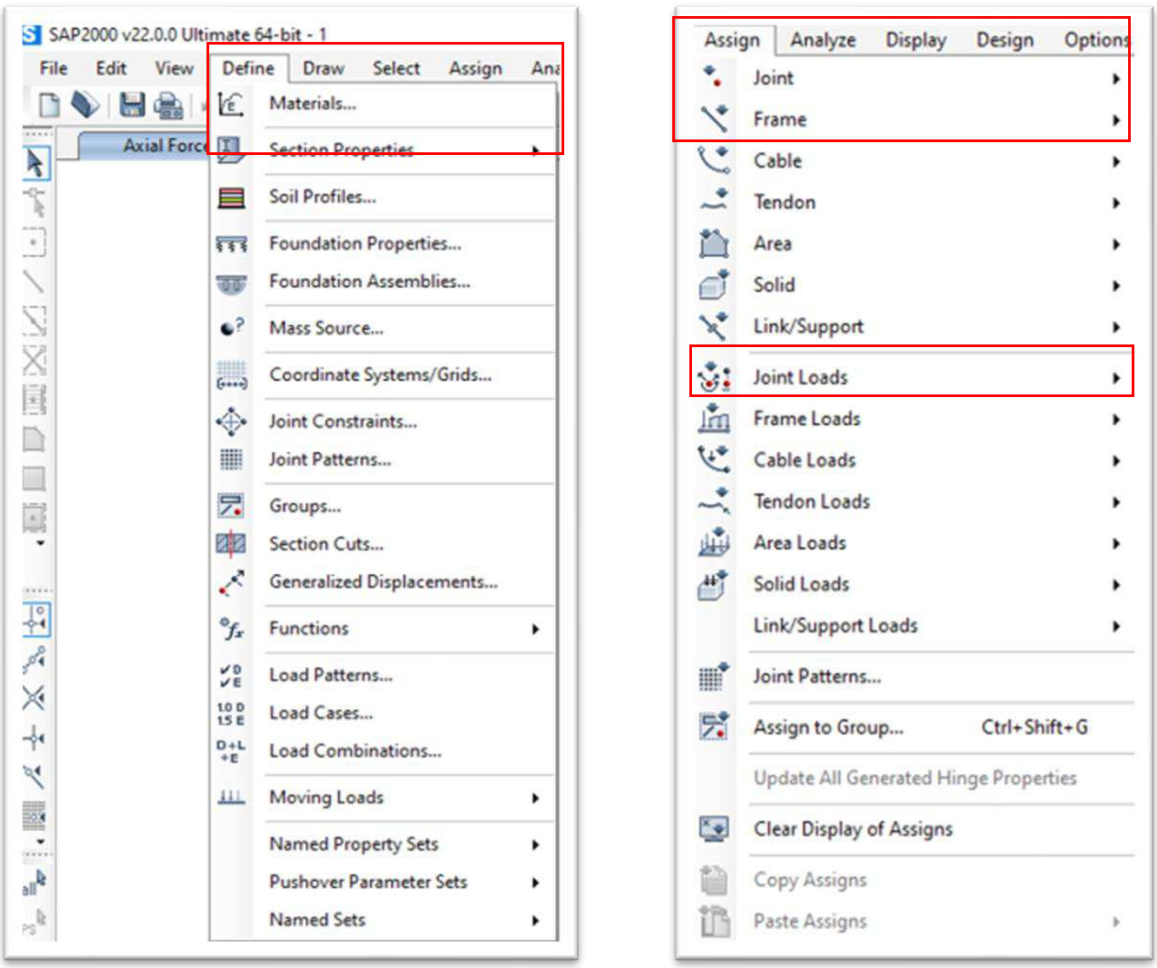


Figure 2.4. SAP 2000 menu

2.3.2.2. Design of the steel structure

The structure will be designed according to the corresponding limit states in such a way to sustain all actions acting upon it during its intended life. This implies it will be designed having adequate structural stability (ultimate limit states) and remain fit for the use it is required (serviceability). Before any element is designed, it needs to be classified according to its capacity to develop plastic hinges and rotation deformations. The classification proposed by EC3 1-1 will be used.

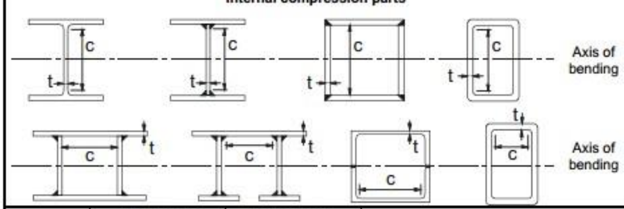
a. Classification of the sections

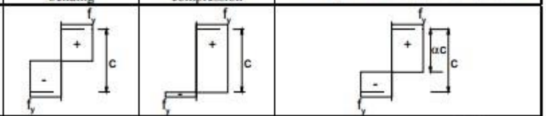
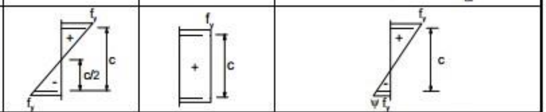
The sections of the members to be design are going to be classified as class 1, 2, 3, or 4. The norm defines these classes as follow:

- Class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance.
- Class 2 cross-sections are those which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling.
- Class 3 cross-sections are those in which the stress in the extreme compression fiber of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
- Class 4 cross-sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section.
- The sections are going to be classified according to Table 2.6, 2.7 and 2.7.

Table 2.7. Maximum width-to-thickness ratios for internal compression parts.(En, 2005a)

Internal compression parts



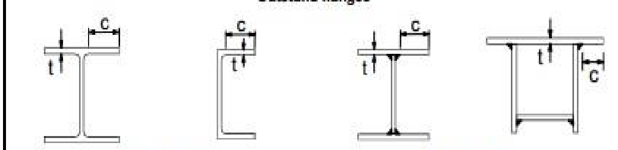
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{36\varepsilon}{\alpha}$
2	$c/t \le 83\varepsilon$	$c/t \le 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \le 124\varepsilon$	$c/t \le 42\varepsilon$	when $\psi > -1$: $c/t \le \dfrac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \le -1^{*)}$: $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

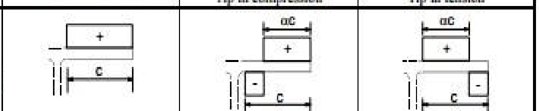
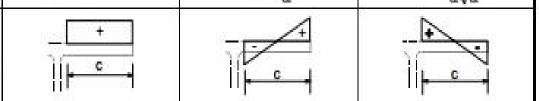
$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

*) $\psi \le -1$ applies where either the compression stress $\sigma \le f_y$ or the tensile strain $\varepsilon_y > f_y/E$

Table 2.8. Maximum width-to-thickness ratios for compression parts (outstand flanges). (En, 2005a)

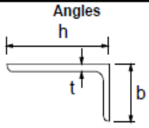
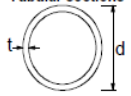
Outstand flanges



Class	Part subject to compression	Part subject to bending and compression: Tip in compression	Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \le 9\varepsilon$	$c/t \le \dfrac{9\varepsilon}{\alpha}$	$c/t \le \dfrac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \le 10\varepsilon$	$c/t \le \dfrac{10\varepsilon}{\alpha}$	$c/t \le \dfrac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \le 14\varepsilon$	$c/t \le 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

Table 2.9. Maximum width to thickness ration for members in bending and/or compression. (En, 2005a)

	Angles	
Refer also to "Outstand flanges" (see sheet 2 of 3)		Does not apply to angles in continuous contact with other components
Class	Section in compression	
Stress distribution across section (compression positive)		
3	$h/t \le 15\varepsilon : \frac{b+h}{2t} \le 11{,}5\varepsilon$	
	Tubular sections	
		
Class	Section in bending and/or compression	
1	$d/t \le 50\varepsilon^2$	
2	$d/t \le 70\varepsilon^2$	
3	$d/t \le 90\varepsilon^2$ NOTE For $d/t > 90\varepsilon^2$ see EN 1993-1-6.	

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71
	ε^2	1,00	0,85	0,66	0,56	0,51

b. Material characteristics

S235 steel is a non-alloy carbon structural steel grade, according to EN 10025-2 standard, S235 material mainly has 3 grades of quality: S235JR (1.0038), S235J0 (1.0114), S235J2 (1.0117).

S235 has good mechanical characteristics, such as plasticity, toughness and weldability, some strength and good cold bending properties.

S235 Signification:

- 'S' is the abbreviation for structural steel;
- '235'refers to minimum yield strength (MPa) for steel thickness ≤ 16mm;
- 'JR' indicates the quality level related to the energy value of the charpy impact test ≥ 27 J, at ambient temperature 20°C;
- 'J0' means Charpy impact test ≥ 27J at 0°C.
- 'J' Charpy impact test ≥ 27J at -20°C.
- 'C' means the material is suitable for cold flanging, cold bending, cold forming or cold drawing, e.g., S235JRC (1.0122)

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- 1.0038 is the steel number of this steel.

S235 material can be made into many steel products, such as H-beam, I-beam, steel channel, steel plate, steel angle, steel tube, wire rod and nails, etc. and these products are widely used in general requirements for welding structures and parts such as bridges, transmission towers, boilers, steel frame factories, shopping malls and other buildings.

For high strength steel like the S690QL in addition to previous specifications, there are some complementary aspects to maintain the efficacity of the steel. For example :

-Typical surface treatment: Shot blasting, painting and marking.

- Heat treatment: annealed;

- Z25, Z35 trials, simulated treatment (TTAS/PWHT);

- US controls: (S1E1 to S3E4 and ASTM SA578 level B & C).

c. Ultimate Limit States design checks for steel members (ULS)

Ultimate limit states are those that relate to the failure of a structural member or a whole structure. Design verifications that relate to the safety of the people in and around the structure are ultimate limit state verifications. The design verifications will be done with respect to EC3 1-1, 2005.

i. Design of tension or compression members

The design for a tension or compression member must be such that the design actions are lower than the resisting forces and moments.

Tension member

For a tension member, check that,

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1 \qquad (2.18)$$

Where, N_{Ed} is the design tension load and $N_{t,Rd}$ is the resisting tensile force of the element and is the minimum between $N_{pl,Rd}$ and $N_{u,Rd}$ given in equations (2.19) and (2.20).

$$N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} \tag{2.19}$$

$$N_{u,Rd} = \frac{0.9A_{net}f_u}{\gamma_{M2}} \tag{2.20}$$

ii. Compression member

For a compression member, it will be important to check that,

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1 \tag{2.21}$$

Where $N_{c,Rd}$ should be determined by equation (2.22).

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} \quad \text{For class 1, 2 and 3 cross sections} \tag{2.22}$$

The steel element shall also be checked for buckling. To do so, the design compressive force should be lower than the buckling resistance force.

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1 \tag{2.23}$$

Where, $N_{b,Rd}$ is the design buckling resistance of the compression member and is given by equation (2.24) and (2.25).

$$N_{b,Rd} = \frac{\chi Af_y}{\gamma_{M1}} \quad \text{For class 1, 2 and 3 cross sections} \tag{2.24}$$

$$N_{b,Rd} = \frac{\chi A_{eff}f_y}{\gamma_{M1}} \quad \text{For class 4 cross sections} \tag{2.25}$$

Where, χ is the buckling factor which reduces the resisting axial force of the whole element. To determine χ, the appropriate buckling curve is first selected (see Figure 2.6) which is given according to Table 2.10 with respect to the section's characteristics and steel type.

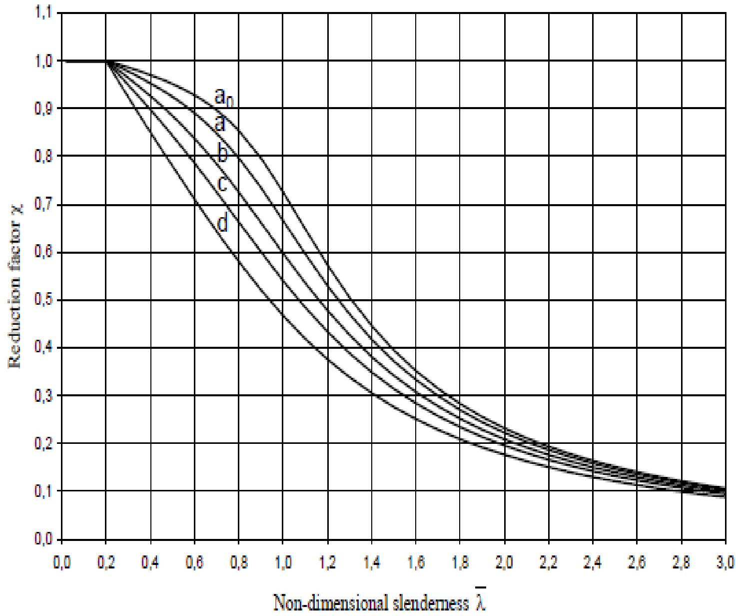
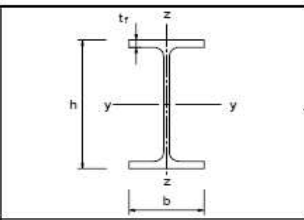
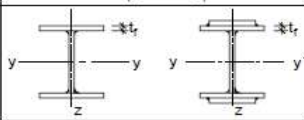
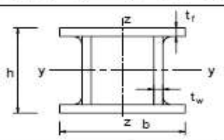
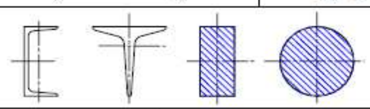
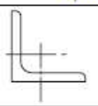


Figure 2.5. Buckling curves (source: EC3 1-1 , 2005).

Table 2.10. Selection of buckling curve for a cross section (EN 3-1, 2005)

Cross section		Limits		Buckling about axis	Buckling curve S 235 S 275 S 355 S 420	Buckling curve S 460
Rolled sections		$h/b > 1,2$	$t_f \le 40$ mm	y – y	a	a_0
				z – z	b	a_0
			$40 \text{ mm} < t_f \le 100$	y – y	b	a
				z – z	c	a
		$h/b \le 1,2$	$t_f \le 100$ mm	y – y	b	a
				z – z	c	a
			$t_f > 100$ mm	y – y	d	c
				z – z	d	c
Welded I-sections		$t_f \le 40$ mm		y – y	b	b
				z – z	c	c
		$t_f > 40$ mm		y – y	c	c
				z – z	d	d
Hollow sections		hot finished		any	a	a_0
		cold formed		any	c	c
Welded box sections		generally (except as below)		any	b	b
		thick welds: $a > 0,5t_f$ $b/t_f < 30$ $h/t_w < 30$		any	c	c
U-, T- and solid sections				any	c	c
L-sections				any	b	b

The non-dimensional slenderness, $\bar{\lambda}$ is computed using equation (2.26) and equation (2.27).

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \frac{l_{cr}}{i}\left(\frac{1}{\lambda_1}\right) \qquad \text{For class 1, 2 and 3 cross sections} \qquad (2.26)$$

$$\bar{\lambda} = \sqrt{\frac{A_{eff}f_{yk}}{N_{cr}}} = \frac{l_{cr}}{i}\frac{\sqrt{\frac{A_{eff}}{A}}}{\lambda_1} \qquad \text{For class 4 cross sections} \qquad (2.27)$$

Where:

l_{cr} is the buckling length in the buckling plane considered.

i is the radius of gyration about the relevant axis

$\lambda_1 = 93.9\varepsilon$.

N_{cr} is the elastic critical force for the relevant buckling mode based on the gross cross-sectional properties and is given by Euler's equation (2.28).

$$N_{cr} = \frac{\pi^2 EJ}{{l_0}^2} \qquad (2.28)$$

iii. Design of beam elements

The design procedure for the beam-column elements will be done as follow:

Bending and axial check, bending and axial interaction

The design value of bending moment at each cross-section shall satisfy:

$$\frac{M_{Ed}}{M_{C,Rd}} \leq 1 \qquad (2.29)$$

Where M_{Ed} is the design moment and $M_{C,Rd}$ is the resisting moment.

For class 1 or 2 sections

$$M_{C,Rd} = M_{pl,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} \tag{2.30}$$

For class 3 sections

$$M_{C,Rd} = M_{el,Rd} = \frac{W_{el,min} f_y}{\gamma_{M0}} \tag{2.31}$$

For class 4 sections

$$M_{C,Rd} = M_{el,Rd} = \frac{W_{eff,min} f_y}{\gamma_{M0}} \tag{2.32}$$

Where:

f_y is the yielding strength

W_{pl} is the plastic section modulus

$W_{el,min}$ is the elastic section modulus

$W_{eff,min}$ is the effective section modulus

As earlier defined, beams are subjected to axial and flexural load. In a more conservative approach, the design proposal states that the condition given by the Navier's equation (2.33) should hold.

$$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{Ed}}{M_{c,Rd}} \leq 1 \tag{2.33}$$

In which N_{Ed} is the design axial force and M_{Ed} the design moment acting on the element at the cross-section under consideration, $N_{c,Rd}$ is the cross-section axial resistance, and $M_{c,Rd}$ is the cross-section moment resistance.

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1 \tag{2.34}$$

Where $N_{c,Rd}$ is given by equation (2.35)

$$N_{c,Rd} \leq \frac{Af_y}{\gamma_{M0}} \tag{2.35}$$

For a given section, there is no reduction to the major axis plastic moment resistance provided the conditions hold:

$$N_{Ed} \leq 0.25N_{pl,Rd} \tag{2.36}$$

$$N_{Ed} \leq \frac{0.5\, h_w t_w f_y}{\gamma_{M0}} \tag{2.37}$$

iv. Shear check, shear and bending moment interaction

The design value of the shear force, V_{Ed} at each cross-section shall satisfy equation (2.38).

$$\frac{V_{Ed}}{V_{c,Rd}} \leq 1 \tag{2.38}$$

For plastic design (which shall be the one considered), $V_{c,Rd}$ is the design plastic shear resistance, $V_{pl,Rd}$ given by equation (2.39)

$$V_{pl,Rd} = \frac{A_v\left(\frac{f_y}{\sqrt{3}}\right)}{\gamma_{M0}} \tag{2.39}$$

Where:

f_y is the yielding strength

A_v is the shear area

If the shear force is less than half the plastic shear resistance ($V_{Ed} < 0.5V_{pl,Rd}$), its effect on the moment resistance may be neglected. If V_{Ed} exceeds 50% of $V_{pl,Rd}$, the reduced plastic shear resistance is calculated using a reduced yield strength given by equation (2.40).

$$f'_y = (1-\rho)f_y \tag{2.40}$$

Where, $\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2$.

For element acting as column elements shall also be checked for buckling since they are subjected to compressive force. The stability of the column is given by the equation (2.41).

$$\frac{N_{Ed}\gamma_{M1}}{\chi_{min}Af_y} + \frac{M_{eq,Ed}\gamma_{M1}}{W_{pl}f_y\left(1-\frac{N_{Ed}}{N_{cr}}\right)} \leq 1 \tag{2.41}$$

Where:

$M_{eq,Ed}$ =0.75M_{max}

χ_{min} is the buckling factor

N_{cr} is the elastic critical force for the relevant buckling mode

v. Design of the connections

In this structure, bolted connections were used the most.

- **Shear resistance for one bolt**

The design resistance for a bolt in shear is given by equation (2.42)

$$F_{v,Rd} = \frac{\alpha_v\, f_{ub}\, A_b}{\gamma_{Mb}} \tag{2.42}$$

Where:

$$\alpha_v = \begin{cases} 0{,}6 \text{ for classes } 4.6, 5.6 \text{ and } 8.8 \\ 0{,}5 \text{ for classes } 4.8, 5.8 \text{ and } 10.9 \end{cases}$$

f_{ub} is the ultimate tensile stresses of the bolt

A_b is the gross area of the bolt given by $\frac{\pi d^2}{4}$

γ_{Mb} is the partial safety factor for bolts = 1,25

- **Traction resistance for one bolt**

The design resistance for a bolt in tension is given by equation (2.43)

$$F_{t,Rd} = \frac{0{,}9\ f_{ub}\ A_s}{\gamma_{Mb}} \tag{2.43}$$

With A_s being the resisting area of the bolt

If shear and traction are present on the bolt, the following verification must be made:

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4F_{t,Rd}} \leq 1 \tag{2.44}$$

vi. Bearing resistance for the plate

For the plate involved in the connection, the bearing resistance is given by equation (2.45)

$$F_{b,Rd} = \frac{K_1\ \alpha_b\ f_u\ d_0\ t}{\gamma_{M2}} \tag{2.45}$$

Where:

$$\alpha_{b,int} = \min\left\{\left(\frac{p_1}{3d_0} - 0{,}25\right); \frac{f_{ub}}{f_u}; 1\right\}$$

$$\alpha_{b,ext} = \min\left\{\left(\frac{e_1}{3d_0}\right); \frac{f_{ub}}{f_u}; 1\right\}$$

$$K_1 = \min\left\{\left(\frac{2{,}8e_2}{d_0} - 1{,}7\right); 2{,}5\right\}$$

f_u is the ultimate tensile strength for the plate

d_0 is the hole diameter

t is the thickness of the plate

e1, e2, p1, p2 are the constructive dispositions on the plate and are represented in Figure 2.6.

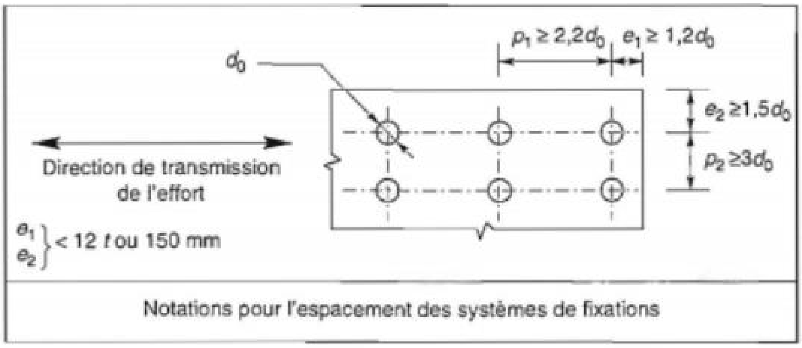


Figure 2.6. Spacing of the holes on the plate (Morel, 2005).

vii. Fatigue assessment

The fatigue verification procedure is performed for those details that are determinant for the fatigue performance of the bridge.

The bridge is used as a temporary bridge (less than 3 years of use compared to 100 years for others normal bridges) as part of the hydroelectric development of Nachtigal project to allow the passage of MC 305 which are compact hydraulic mini-drills of Dimensions 4.3 m long, 1.54 m wide, 2.19 m high and 3000 Kg weight (see the annex). The assumption of the number of heavy vehicles listed in the area can allow us, according to the Eurocodes, to classify it in a traffic class lower than traffic class number 4 for 'Local roads with low flow rates of lorries' in Table 2.11 that is with a number N_{obs} of heavy vehicles crossing this bridge lower than 0.05 x 10^6 cycles per year, and it should also be noted that the bridge has only one lane with w = 3.14 m of carriage way.

Table 2.11. Indicative number of heavy vehicles expected per year and per slow lone (Env & Pa, 1997)

Traffic categories		N_{obs} per year and per slow lane
1	Roads and motorways with 2 or more lanes per direction with high flow rates of lorries	$2,0 \times 10^6$
2	Roads and motorways with medium flow rates of lorries	$0,5 \times 10^6$
3	Main roads with low flow rates of lorries	$0,125 \times 10^6$
4	Local roads with low flow rates of lorries	$0,05 \times 10^6$

For these reasons, it is observed that for this project the number of cycles (N_{ods}) of heavy vehicles is low, lower than the smallest cycle to consider for fatigue dimensioning listed in the Eurocode.

Because of improving a steel bridge using the high strength steel, according to the Wöhler curve (or stress life Diagram) Figure 2.7, increasing the resistance $\boldsymbol{\sigma s}$ of a steel also increases its ultimate resistance $\boldsymbol{Sut}$, its fatigue strength $\boldsymbol{s_f}$ and the endurance limit $\boldsymbol{Se}$ but considerably reduce the number of stress cycles N before the failure. So, for a high strength steel, a high factor of safety $\boldsymbol{n}$ against failure due to fatigue for a low number of stress cycles, but not reaching a high number of stress cycles.

With: $$\left(\boldsymbol{n} = \frac{\boldsymbol{s_f}}{\boldsymbol{\sigma_{applied}}}\right) \qquad (2.46)$$

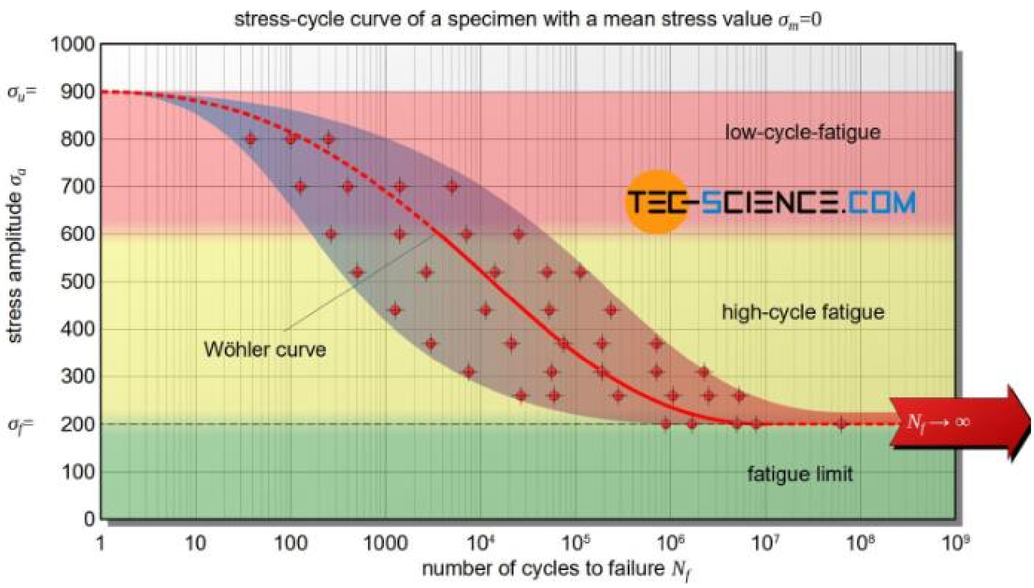


Figure 2.7. Stress-cycle curve (www. tec-science.com)

Since temporary bridges are not sized for a large number of load cycles, the use of high strength steels would allow greater resistance Sf to fatigue failure for the low stress cycle N to which it will be subjected during its lifetime. So as conclusion for this project, it will not be useful to check the fatigue.

d. Serviceability Limit States check for steel members (SLS)

Serviceability limit states concern the functioning of the structure under normal use, the comfort of the people using the structure and the appearance of the structure. Serviceability limit states may be irreversible or reversible.

Irreversible limit states occur where some of the consequences remain after the actions that exceed the limit have been removed, e.g., there is permanent deformation of a beam or cracking of a partition wall. Reversible limit states occur when none of the consequences remains after the actions that exceed the limit have been removed, i.e., the member stresses are within its elastic region.

Criteria that are considered during serviceability limit state design checks are:

- Deflections that affect the appearance of the structure, the comfort of its users and its functionality,
- Vibrations that may cause discomfort to users of the structure and restrict the functionality of the structure,
- Damage that may affect the appearance or durability of the structure and,
- Stress limitation in the section

The Eurocodes do not specify any limits for serviceability criteria, but limits may be given in the National Annexes. The limits should be defined for each project, based on the use of the member and the Client's requirements. For our structure and requirements, it will be consider the following checks:

- For the stress limitation, $\sigma \leq 0.8f_y$,
- Deflection, $d < l/250$ where, l is the span length and $d < h/300$ for columns

2.4. Parametric analysis

A parametric study is a study that deals with the influence of parameters on the solution of a particular problem. Many different parameters may influence the improvement of a temporary bridge construction as section of element, the global geometry and quality of material. Thus, this section starts by providing the determination of the appropriate section for the steel consider, and ends with the results comparison in term of weight and total cost of the entire structure.

2.4.1. Selection of the steel class and determination of the optimal section

The strength class of the steel use is an important parameter to take into account for improving a steel structure. It will be necessary to this part to reduce as much as possible the total weigh of our bridge using high strength steel commonly find on the market, the S 450 and the S690 QL.

Algorithm of parametric procedure is given by Figure 2.8

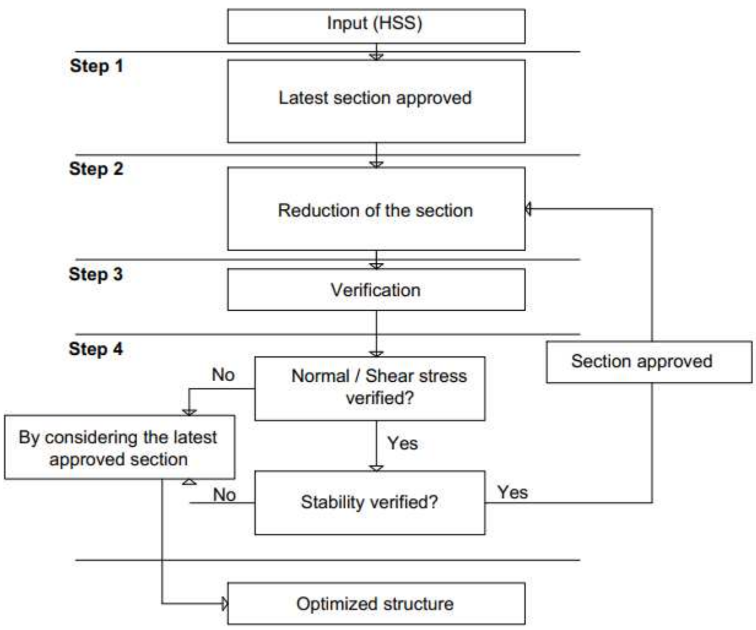


Figure 2.8. Algorithmic procedure for optimal thickness selection

2.4.2. Cost comparison

It means the process of developing an estimate of the cost the structure depending on the material use and the section of each element.

Conclusion

The aim of this chapter was to present steps necessary to design and improve a temporary bridge structure. It started off by establishing the procedure of collecting data from the field to material properties. In order to perform correct analyses, a clear methodology to obtain the loads acting on the structures and the various loads combination was done. The software used for this thesis was shown. The design at limit state follows a clear procedure and for the case study, the standard to satisfy these limit states were explained. In view of all the above, the rest of this work will consist in giving the detailed results of this methodology in the next chapter.

CHAPTER 3. PRESENTATION OF RESULTS AND INTERPRETATION

Introduction

This chapter presents the results obtained from the previously detailed methodology outlined in chapter 2. This application will integrate in particular the general presentation of Nachtigal village to which our case study belongs, followed by the presentation of the project. The case study to be modelled and its corresponding material properties with respect to the methodology discussed will be presented. The loads acting on the structure are computed to obtain the solicitations and deflections. The various data collected are for the static analysis followed by the results obtained by this analysis. Interpretation of results will gradually proceed with their presentation and finally, the last part of this chapter will be devoted to possible solutions.

3.1. General presentation of Nachtigal village.

The presentation of Nachtigal village will be the subject of this section. The village will be presented geographically, the climate, the hydrology and economic conditions in order to know the different conditions which generally have a major influence on the design of structures.

3.1.1. Geographical location

Nachtigal is a small village of Mbam-et-Kim department in the Center region of Cameroon. It is located at: 4° 21′ 54″ N and 11° 37′ 57″ E. The site is on the Sanaga River at Nachtigal approximately 65km NE of Yaounde, Cameroon as shown on Figure 3.1.

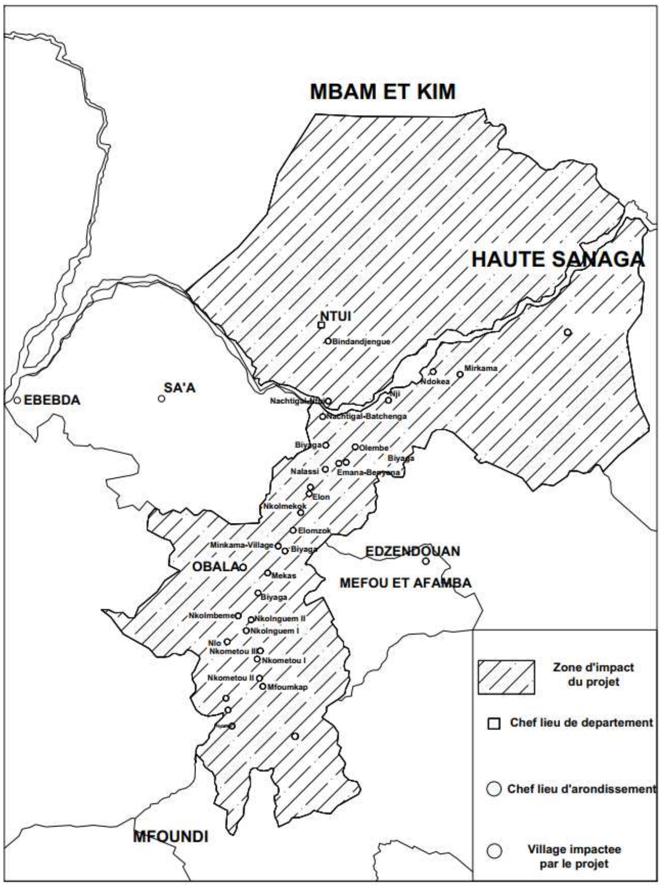


Figure 3.1. Project location

3.1.2. Climate and air quality

The climatic conditions around Nachtigal are mainly equatorial with two rainy seasons and two dry seasons of unequal duration.

Air quality in the project area has been assessed to be generally good with the main sources of air pollution identified as intermittent and irregular bush fires and road traffic, particularly in the dry season. Baseline studies undertaken in 2014 included one-time measurements (day and night) of ambient air quality at 4 locations in 3 villages around the project area, construction camps, villages and roads. Measurements were undertaken for sulphur dioxide (SO_2), nitrogen dioxide (NO_2), carbon monoxide (CO), ozone (O_3), volatile organic compounds (VOCs), dust suspensions lower than 10 microns (PM10) and dust suspensions lower than 2.5 microns (PM2.5) (Aida et al., 2017). Compared to national and international standards (WHO and USEPA), the results obtained showed that the reported concentrations of gaseous pollutants were below assessment limits, reflecting a lack of pollution from stationary sources (industry, urbanization, fuel operating units). Elevated PM10 (dust) were identified due to unpaved roads, local traffic (on National Road No. 1) and domestic activities such as the use of firewood, slash and burn agriculture and bush fires. Figures 3.2 and 3.3 presents respectively the rainfall and the temperature of Nachtigal village.

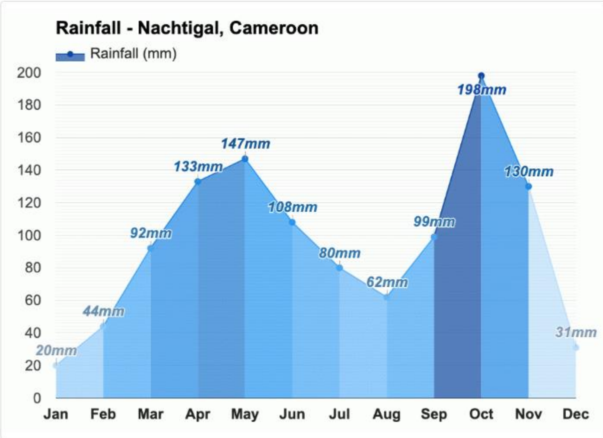


Figure 3.2. Rainfall-Nachtigal, Cameroun

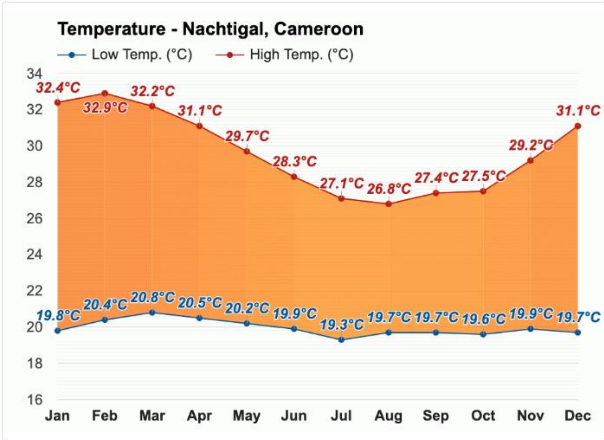


Figure 3.3. Temperature-Nachtigal, Cameroun

3.1.3. Surface water hydrology

The Nachtigal project is located on the Sanaga River which has a total catchment area of 129,000km^2 (a quarter of the total watershed in Cameroon). The river is 918km long with flow rates reaching 2,100m^3/s. The flow of the Sanaga River and its tributaries have historically been regulated by the dams and reservoirs at Mapé, Mbakaou, Bamendjin, and more recently the Lom Pangar dam. The commissioning of the Lom Pangar reservoir (upstream of Nachtigal) has significantly altered the hydrology and flow regions of the Sanaga River, such that at Nachtigal there are now two clearly defined seasons, namely a dry season when flows are regulated to 650 m^3/s by water retention at Lom Pangar and Mbakaou and a rainy season when water flows are above the design capacity of 980m^3/s. The Sanaga River and its tributaries are a main source of water for the surrounding villages.

3.1.4. Flora and vegetation

The project area is located in the semi-deciduous Guinea-Congolese rainforest which is a transition zone between forest and savannah. Vegetative cover is mainly defined by forest galleries along the river and shrubby savannah maintained as a result of bush fires and logging. Strips of forests have been cleared for agricultural purposes. There are no protected areas in the study area. Inventories conducted in the impacted zones (construction areas, flooded zone) revealed a total of 366 plant species, of which 15 are tree species with commercial value and nearly 155 species, not endemic to the study area, are used locally in traditional medicines and other traditional uses (nontimber forest products). Several species are endangered or critically endangered according to the International Union for Conservation of Nature (IUCN) i.e., *Ledermanniella sanagaensis* (aquatic), CR; *Ledermanniella thalloïdea* (aquatic), EN; *Marsdenia abyssinica* (riparian/semi-aquatic), CR; *Hymenodictyonpachyantha* (terrestrial), EN; *Ledermanniella sanagaensis* which is endemic to the Nachtigal falls (Project location) and *L. thalloïdea* which is endemic to the Sanaga watershed.

3.1.5. Socio-economic and cultural environment

The total population of the villages based on extrapolations from the National Census (2005 National census) data is 802 people. The project site is located on land which is on the boundaries between the Beti ethnic group in the Central Region and the Mvoute (Babouté) ethnic group of the Eastern Region of Cameroon. The migrant population in the project area who are mostly fishermen include people from Mali, northern Cameroon as well as people from the Tikar and Gbaya ethnic group in Southern Cameroon. There are also migrants involved in businesses and agriculture. Detailed socio-economic surveys were undertaken to establish baseline socio-economic conditions around the project area of influence. Several sacred sites, trees and forests have been identified too. The dominant form of land ownership is community ownership (customary land tenure) where the traditional local leaders serve as trustees and land administrators. Individual families receive rights to use the land and ownership of the land is generally hereditary being transferred from father to son. Women typically have access to parts of land after the death of the head of the household but the size of the land they can inherit is relatively smaller when compared to the size men can inherit.

3.1.6. Physical description of the site

Some pictures of the site are presented in Figure 3.4 and 3.5.

Figure 3.4. Side view of the Nachtigal Hydropower site

Figure 3.5. Aerial view of the Nachtigal Hydropower site

3.2. Presentation of the project

This section is dedicated to the presentation of the project through its geometrical data in the first phase and its material properties in the next phase.

3.2.1. Geometrical data

Geometric data are essentially the bridge geometry with its different cross sections or views.

3.2.1.1. Bridge geometry

The steel bridge under this study is a temporary truss bridge. It consists of one span which is 16.50 m long. The bridge deck is in timber (Dabema), centre-to-centre spacing between top chord element are 1.2m for lateral one and 0.8m for the middle one. Bridge transversal section is 3.15 m wide.

a. Longitudinal section

The longitudinal section of the study case bridge is illustrated by the Figure 3.6. with more details about connections in Annexes 1,2,3, and 4.

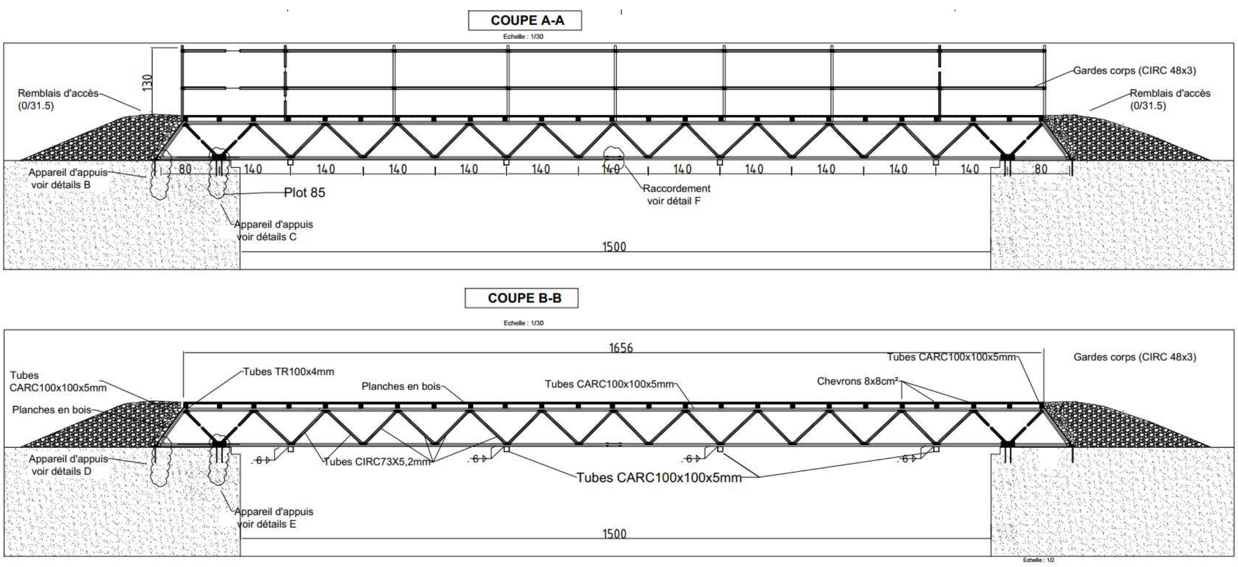


Figure 3.6. Longitudinal section of the steel truss bridge Nachtigal.

b. Transversal section

The total width of the bridge is 3.5m and it consists of a 1 lane roadway with a width 3.15m. Figure 3.7 illustrates the transversal section of the bridge with transversal bracing. More details about connections in Annex 4.

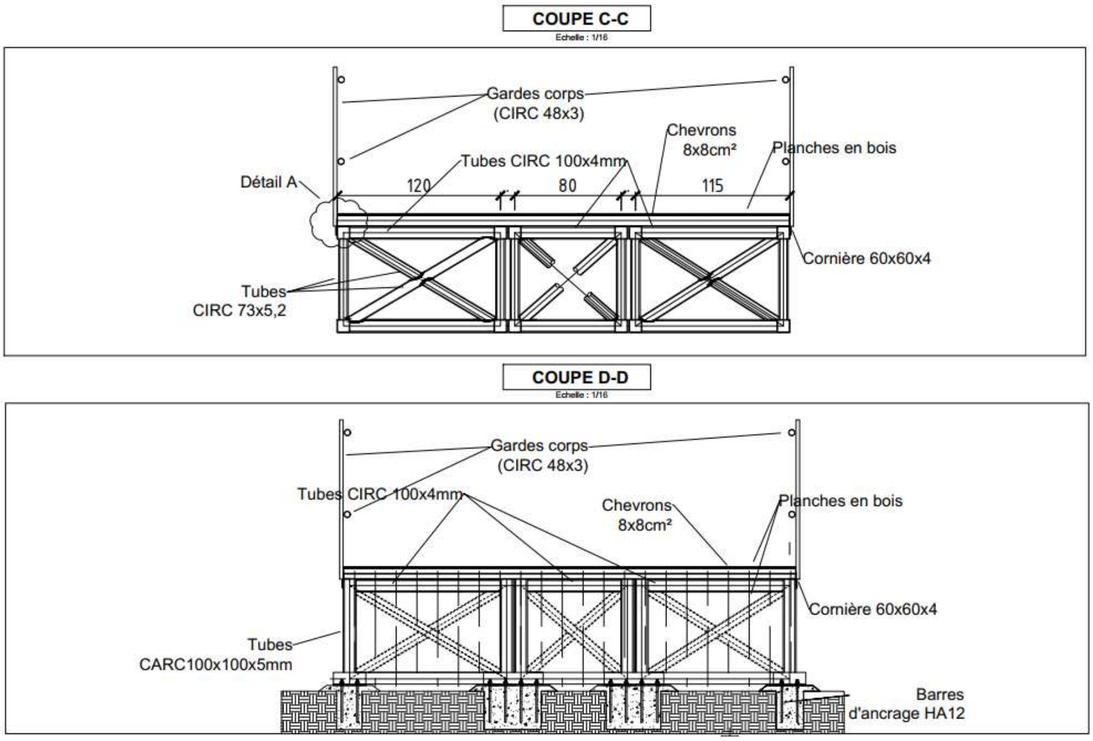


Figure 3.7. Tranversal section of the steel truss bridge Nachtigal.

c. Structural system plan view

Nachtigal temporary truss bridge plane view consists of a system of panels disposed as girders spaced of respectively 1.20m, 0.80m, and 1.50m above which there is a timber decking (Debama or Atui in local name). Illustration is given in Figure 3.8.

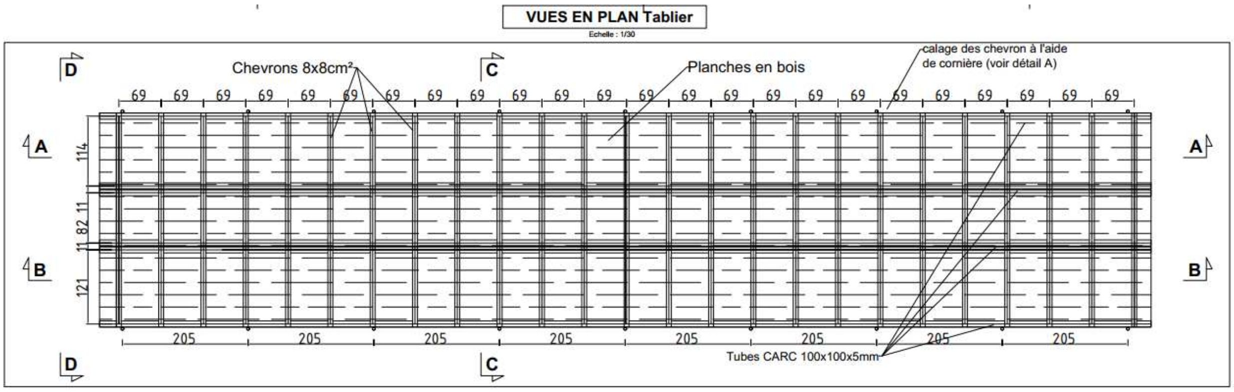


Figure 3.8. Plane view of the project

d. Lifting method plan

The method of setting up temporary bridges is very important because it induces other stresses in the structure which are important to take into account in the design. It is therefore necessary to produce plans for the lifting method. Figure 3.9 represent describe the lifting method of this project.

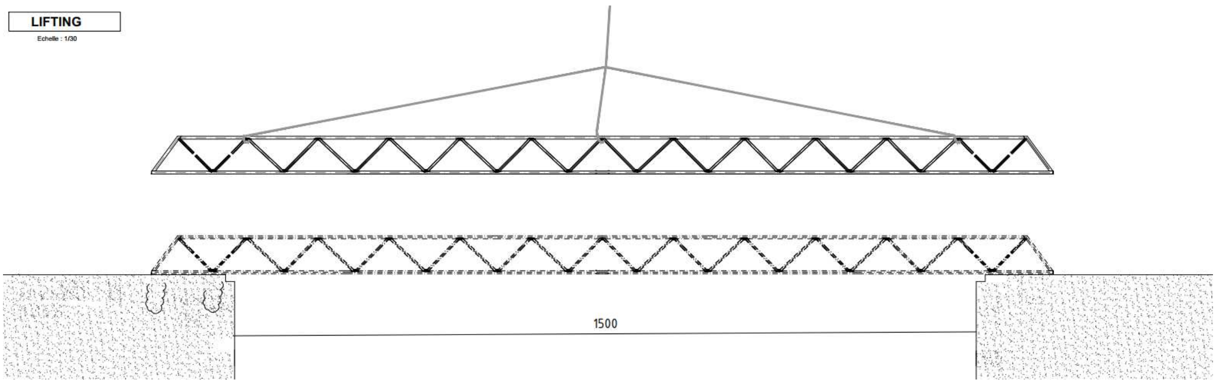


Figure 3.9. Lifting plan

3.2.2. Statistical data

Statistic data will deal with presentation of data linked to the characteristics of timber and steel used.

3.2.2.1. Characteristics of slab

The slab is in timber and principal characteristics used in this analysis are reported in Tables 3.1.

Table 3.1. Mechanical properties of timber (Bois & Grume, n.d.)

PROPRIÉTÉS PHYSIQUES — **PROPRIÉTÉS MÉCANIQUES ET ACOUSTIQUES**

Les propriétés indiquées concernent les bois arrivés à maturité. Ces propriétés peuvent varier de façon notable selon la provenance et les conditions de croissance des bois.

	Moyenne	Écart-type		Moyenne	Écart-type
Densité* :	0,70	0,06	Contrainte de rupture en compression* :	57 MPa	6 MPa
Dureté monnin* :	4,4	1,6	Contrainte de rupture en flexion statique* :	98 MPa	13 MPa
Coeff. de retrait volumique :	0,55 %	0,10 %	Module d'élasticité longitudinal* :	15190 MPa	2027 MPa
Retrait tangentiel total (RT) :	8,5 %	1,2 %			
Retrait radial total (RR) :	3,8 %	0,6 %	(* : à 12% d'humidité, avec 1 MPa = 1 N/mm²)		
Ratio RT/RR :	2,2				
Pt de saturation des fibres :	27 %		Facteur de qualité musicale :	106,9 mesuré à 2556 Hz	
Stabilité en service :	moyennement stable				

3.2.2.2. Structural steel

Materials features of structural elements reported in Table 3.2, Table 3.3 and 3.4

Table 3.2. Steel S235 mechanical properties

Structural Steel			
Designation	**S 235 (NF EN 10025-2)**		Units
Characteristic Ultimate Strength	f_{uk}	360	MPa
Characteristic Yield Strength	f_{yk}	235	MPa
Elastic Modulus	E_s	210000	MPa
ULS Safety Factor	g_s	1.05	
Design Yield Strength	f_{yd}	223.8095238	MPa
Density	ρ	7850	kg/m^3
Unit weight	γ	78.5	kN/m^3
Shear modulus	G	80769.23077	MPa
Poisson's ratio in elastic range	ν	0.3	
Coefficient of linear thermal expansion	α	0.000012	$°K^{-1}$

Table 3.3. Steel S450 mechanical properties

Structural Steel			
Designation	**S 450 (NF EN 10025-2)**		Units
Characteristic Ultimate Strength	f_{uk}	550	MPa
Characteristic Yield Strength	f_{yk}	440	MPa
Elastic Modulus	E_s	210,000	MPa
ULS Safety Factor	g_s	1.05	
Design Yield Strength	f_{yd}	419.04	MPa
Density	ρ	7850	kg/m3
Unit weight	γ	78.5	kN/m3
Shear modulus	G	80,769.23077	MPa
Poisson's ratio in elastic range	ν	0.3	
Coefficient of linear thermal expansion	α	0.000012	$°K^{-1}$

Table 3.4. Steel S690 QL mechanical properties

Structural Steel			
Designation	**S690Q/QL/QLI (NF EN 10025-6)**		Units
Characteristic Ultimate Strength	f_{uk}	770	MPa
Characteristic Yield Strength	f_{yk}	690	MPa
Elastic Modulus	E_s	210,000	MPa
ULS Safety Factor	g_s	1.05	
Design Yield Strength	f_{yd}	657.1428571	MPa
Density	ρ	7850	kg/m3
Unit weight	γ	78.5	kN/m3
Shear modulus	G	80769.23077	MPa
Poisson's ratio in elastic range	ν	0.3	
Coefficient of linear thermal expansion	α	0.000012	$°K^{-1}$

3.2.2.3. Bolt

Material features of bolted elements are reported in Table 3.5. The bolts have a nominal diameter of 20 mm (M 20) and of class 10.9.

Table 3.5. Bolt M20 mechanical property

Bolts M20			
Designation		10.9	
Nominal Ultimate Strength	f_{tb}	1,000	MPa
Nominal Yield Strength	f_{yb}	900	MPa
Elastic Modulus	E_s	210,000	MPa
ULS Safety Factor	γ_{M2}	1.25	
Design Yield Strength	$f_{yd,b}$	720	MPa

3.2.2.4. Plates

Material features of plate elements are reported in Table 3.6.

Table 3.6. Plate mechanical properties**.**

Designation			
Characteristic Yield Strength	f_{yk}	355	MPa
Characteristic Tensile Strength	f_{tk}	510	MPa
Thickness of plate	t	15	mm

3.3. Structural analysis of steel truss bridge

Eurocodes and principles enumerated in chapter 2 enable to calculate the different load values and do the static analysis verifications.

3.3.1. Loads determination

In this section, the permanent and variable load values acting on the bridge are determined.

3.3.1.1. Self-weight of structural elements (g1)

Table 3.7 shows a self-weight estimation concerning structural steel ($g_{1,\ s}$) and timber slab ($g_{1,\ t}$). In order to take into account, the weight of the connections and stiffening plates, the weight of the steel is increased by a 10% factor.

Table 3.7. Self-weight of structural elements**.**

Nature	Description	Value	Unit
$g_{1,s}$	Self-weight of steel structural component	78.5	kN/m^3
$g_{1,t}$	Self-weight of timber structural component	6.90	kN/m^3

3.3.1.2. Self-weight of non-structural elements (g2)

Self-weights of non-structural elements are summarised in Table 3.8

Table 3.8. Self-weight of non structural elements.

Nature	Description	Value	Unit
G2	Guardrail	1.5	kN/m

3.3.1.3. Live loads

For traffic distribution, load values of traffic are reported in Table 3.9

Table 3.9. load value

	Lane 1	Residual area	Footway
Width of the notional lane [m]	3.15	0	0
Uniformly distributed load UDL [kN/m^2]	9	2,5	3
Concentrated load TS [kN]	300	0	0

3.3.1.4. Wind load

With the information on wind load design given in section 2.3.1 well known, proceeding to an excel implementation permits to find the wind force acting on the structure and then distribute it to the wind braces. The known data is summarized in the Table 3.10. This project faces a terrain of category II (z_0=0.05 m and z_{min}=2 m) as stipulated in the same table. The fundamental reference velocity considered is $V_{b,0}$= 22m/s according to Figure 3.10.

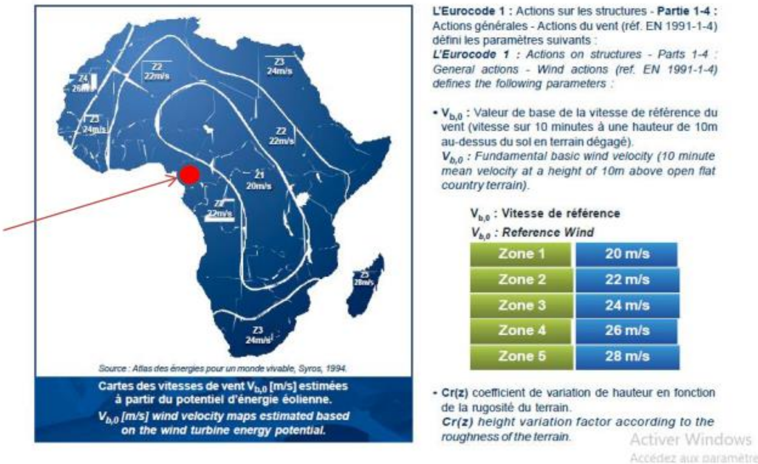


Figure 3.10. Basic wind velocity in Africa(Iv, 1991)

Table 3.10. Terrain categories and terrain parameters(Env, 2000)

Terrain category	z_0 m	z_{min} m
0 Sea or coastal area exposed to the open sea	0,003	1
I Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1,0	10
NOTE: The terrain categories are illustrated in A.1.		

The values of the coefficients at height z=3 m used in this process are reported in Table 3.11.

Table 3.11. Coefficients of wind

Distance between Girders	**d**	**1.2**	**m**
Total Width of Bridge	b	3.39	m
Total Length of Bridge	L	16.56	m

Bracing Spacing	s	1.4	m
Return Period (Years)	T	50	
Reference Height for external Wind Action (m)	z_e	5	m
Air Density	r	1.25	kg/m3
Fundamental Basic Wind Velocity	$v_{b,0}$	22	m/s
Roughness Length (m)	z_0	0.05	m
Reference Roughness Length	$z_{0, ref}$	0.05	m
Minimum Height (m)	z_{min}	2	m
Directional Factor	c_{dir}	1	
Season Factor	c_{season}	1	
Orography Factor	$c_o(z_e)$	1	
Turbulence Factor	k_l	1	
Probability of Exceedance	P	0.02	
Probability Factor	c_{prob}	1	
Terrain Factor	k_r	0.19	
Roughness Factor	$c_r(z_e)$	0.87	
Basic Wind Velocity (m/s)	v_b	22	m/s
Mean Wind Velocity (m/s)	v_m	19.24	m/s
Turbulence Intensity	$I_v(z_e)$	0.21	
Peak Velocity Pressure (kPa)	$q_p(z_e)$	0.58	kPa
Distribution Coefficient	u	1	

The wind force computation depending on the phase considered is reported in Table 3.12.

Table 3.12. Wind force computation

Parameter	**Action on bridge**	**Phase 0**	**Phase 1 Loaded**	**Phase 1 Unloaded**
Depth of Velocity Pressure (m)	d_{tot}	1.00	4.00	3.25
Width to Depth Ratio	b/d_{tot}	3.39	0.84	1.04
Reference Area (m2)	A_{ref}	16.56	66.24	53.82
Wind Load Factor for Bridges	C_f	3.60	1.80	1.70
Wind Pressure (kPa)	q_w	2.10	1.05	0.99
Wind Force on Girders (kN)	F_x	34.79	69.58	53.39
Average Wind Force on a Brace (kN)	$F_{x,avg}$	2.94	5.88	4.51
Wind Torsional Moment (kN.m)	M	5.21	20.35	16.26
Wind Distributed Load (kN/m)	p	0.13	0.51	0.40

3.3.1.5. Temperature load

Table 3.13 presents temperature load calculation.

Table 3.13. Temperature load computation

Thermal Actions			
Coefficient of Thermal Expansion	α_t	0.000021	/°C
Timber Cross Sectional Area	A_t	0.3	m^2
Total Length of Bridge	L	16.5	m
Differential Thermal Variation			
Differential Temperature Change	ΔT	10	°C
Strain	ε	0.00012	
Force due to Differential Thermal Variation	T_{diff}	1133.12	kN
Force due to Differential Thermal Variation per Girder	$T_{diff,i}$	188.85	kN
Uniform Thermal Variation			
Uniform Change in Temperature	ΔT	25	°C
Strain	ε	0.0003	
Bridge Elongation	ΔL	0.0048	m

3.3.1.6. Braking load

It has been considered that the braking distance is equal to the total length of the bridge, i.e., L= 16.5 ≥ 1.2 m then, the force is given by:

$$Q_{lk} = 0.6\ \alpha_{Q1}(2Q_{1k}) + 0.10\alpha_{q1}q_{1k}w_1L$$

$$Q_{lk} = 0.6 * 1 * (360 + 2.7 * \ 16.5) + 0.10 * \ 9 * \ 3.15 * \ 16.5$$

$$Q_{lk} = 289.50\ kN$$

3.3.2. Load combination

Load combination takes into account all the types of loads listed in the previous paragraphs. The generated load combination is given in the Annex 1.

3.3.3. Numerical modelling and verification

This section presents an overview of the numerical model of the project and the associated verifications for this work.

3.3.3.1. Numerical modelling

The numerical models were done on two software: Midas/Civil 2019 (Figure 3.11) and SAP 2000 version 22 (Figure 3.12). The existing model was done with steel grade S235.

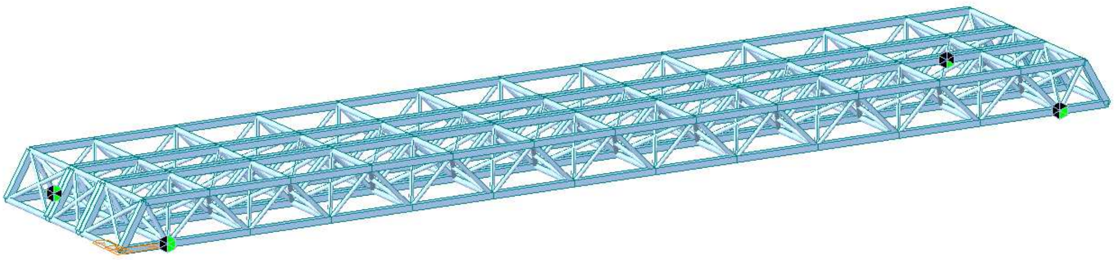


Figure 3.12. Numerical model of the steel truss bridge on Midas/Civil 2019

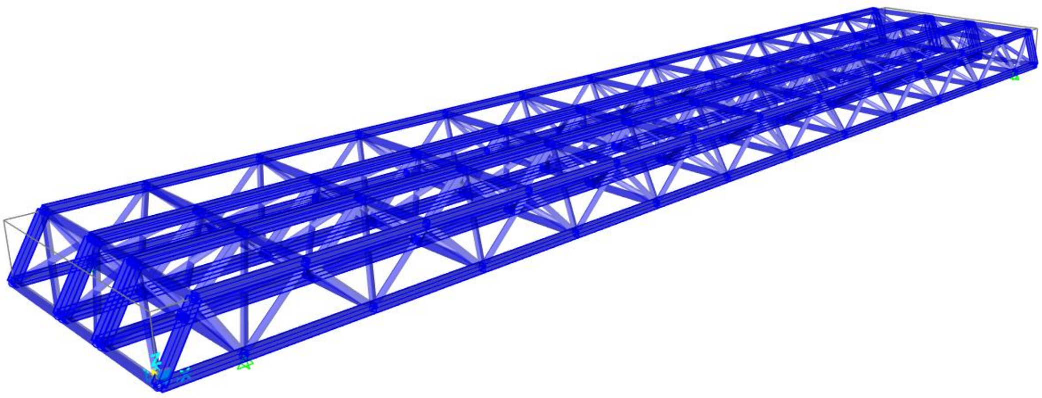


Figure 3.13. Numerical model of the steel truss bridge on SAP 2000 V22

For the verifications, the values of the maximum reactions for the most unfavorable combinations of the actions on the structure were taken into account. The values of solicitations used for this analysis are those obtained from Midas/Civil, but it is important to note that these values do not differ from those obtained from the SAP 2000 analysis by more than 10%.

3.3.3.2. Verifications

a. Verification at Ultimate Limit State

Stresses depend on a given section geometrical characteristics. Verification will be done for each section.

i. Geometrical characteristics

- Top and bottom chord elements

On the top and bottom chords, a rectangular CARC 100x100x5mm tube with its characteristics presented in Table 3.14 is used.

Table 3.14. Top and bottom chord element goemetry

Top and Bottom Cord: CARC 100X100X5 mm		
Width of Steel Tube (mm)	a	100
Thickness of Steel tube (mm)	s_1	5
Height of Steel (mm)	h_{steel}	100
Cross Sectional Area of Steel (mm²)	A_s	1,900
Height of Steel Centroid (mm)	y_G	50
Resistance Modulus of Steel (mm³)	W	57,316.67

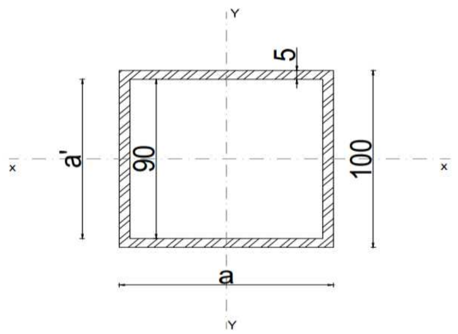


Figure 3.14. CARC 100x100x5 mm

- Top and bottom Crossing tubes

The element used for the top and bottom crossing tubes is a CIRC 100x4mm with characteristics given in Table 3.15.

Table 3.15. Crossing tube element goemetry

Crossing Tube: CIRC 100X4 mm		
Diameter of Steel Tube (mm)	D	100
Thickness of Steel Lower Flange (mm)	s_1	4
Cross Sectional Area of Steel (mm²)	A_s	1,205.76
Height of Steel Centroid (mm)	y_G	50
Resistance Modulus of Inf Steel (mm³)	W	27,828.94

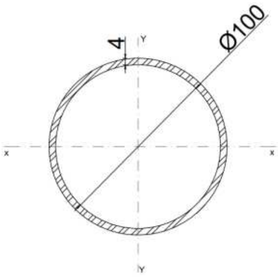


Figure 3.15. CIRC 100x4 mm

- **Diagonal elements**

For the diagonal elements, CIRC 73x5.2mm with characteristics given in Table 3.16 are used.

Table 3.16. Diagonal element goemetry

Diagonal Element: CIRC 73x5.2 mm		
Diameter of Steel Tube (mm)	D	73
Thickness of Steel Lower Flange (mm)	s_1	5.2
Cross Sectional Area of Steel (mm²)	A_s	1,107.03
Height of Steel Centroid (mm)	y_G	36.5
Resistance Modulus of Inf Steel (mm³)	W_{inf}	17,530.18

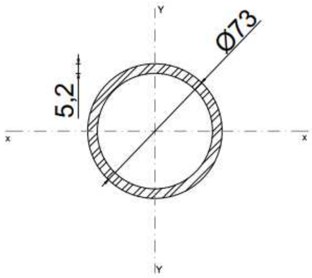


Figure 3.16. CIRC 73x5.2 mm

ii. Stress Verifications

According to the Navier formula given in Chapter 2, the verification of the section in terms of stress is proceeded in an excel sheet. Results of stresses in the elements have to be less than the design yield strength of S235 (f_{yd}= 223.80) and are given in Table 3.17. It is important to note that verification is made with the maximal action acting on the considered member using the ULS envelope combination. Figure 3.14 shows the solicitations on the whole structure.

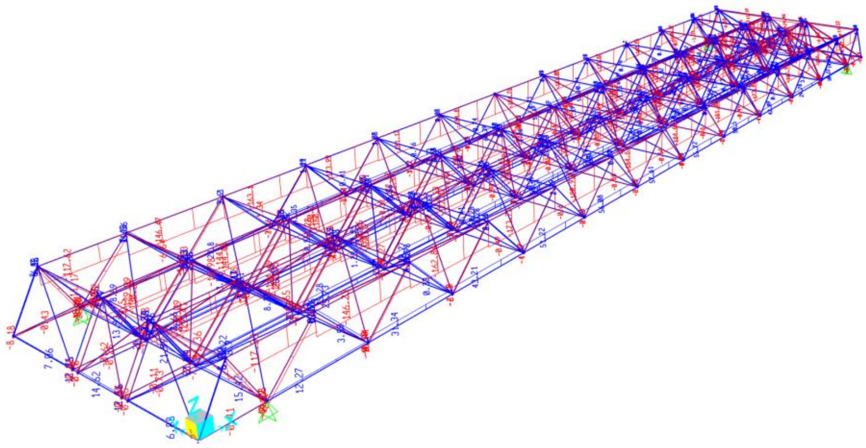


Figure 3.17. Solicitation on the structure in Software

Table 3.17. Stress verification on the elements

Top Chord of external frame					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	**$s_{Steel,Sup}$**	**$s_{Steel,Inf}$**
Envelope of the ULS combinations	1.17	2.62	-319.62	**-147.80**	**-188.63**
Verification	OK				
Top Chord of internal frame					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	**$s_{Steel,Sup}$**	**$s_{Steel,Inf}$**
Envelope of the ULS combinations	0.9	1.14	-280.54	**-131.95**	**-163.35**
Verification	OK				
Bottom Chord of external frame					
	Maximum Design Internal			**Stresses**	

	reactions				
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	$s_{Steel,Sup}$	$s_{Steel,Inf}$
Envelope of the ULS combinations	1.17	1.63	110.13	**78.37**	**37.55**
Verification	OK				

Bottom Chord of internal frame					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	$s_{Steel,Sup}$	$s_{Steel,Inf}$
Envelope of the ULS combinations	**1.002**	**2.12**	**120.30**	**80.79**	**45.83**
Verification	OK				

Crossing Elements Top					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	$s_{Steel,Sup}$	$s_{Steel,Inf}$
Envelope of the ULS combinations	0.25	20.1	1.67	**10.36**	**-7.59**
Verification	OK				

Transversal Element bottom					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	$s_{Steel,Sup}$	$s_{Steel,Inf}$
Envelope of the ULS combinations	**0.8**	**12.3**	**-0.31**	**28.48**	**-29.01**
Verification	OK				

Diagonal Elements in Tension					
Combination	**Maximum Design Internal reactions**			**Stresses**	
	M (kNm)	**T (kN)**	**N (kN)**	$s_{Steel,Sup}$	$s_{Steel,Inf}$
Envelope of the ULS combinations	0.00	0.00	26.17	**23.63**	**23.63**
Verification	OK				

Diagonal Elements in Compression					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	$s_{Steel,Sup}$	$s_{Steel,Inf}$
Envelope of the ULS combinations	0.00	0.00	-45.17	**-40.80**	**-40.80**
Verification	OK				

Brace Elements in Tension					
Combination	**Maximum Design Internal reactions**			**Stresses**	
	M (kNm)	**T (kN)**	**N (kN)**	$s_{Steel,Sup}$	$s_{Steel,Inf}$
Envelope of the ULS combinations	0.00	0.00	21.34	**19.27**	**19.27**
Verification	OK				

Brace Elements in compression					
Combination	**Maximum Design Internal reactions**			**Stresses**	
	M (kNm)	**T (kN)**	**N (kN)**	$s_{Steel,Sup}$	$s_{Steel,Inf}$
Envelope of the ULS combinations	0.00	0.00	-32.72	**-29.55**	**-29.55**
Verification	OK				

iii. Resistance to buckling

The verification of the elements in terms of resistance to buckling has been compiled in an excel sheet and is given in Table 3.18.

Table 3.18. Buckling resistance of element

Top chord Element CARC 100X100X5mm		
Compression Force (kN)	$N_{ed, max}$	319.6
Reduce slenderness	$\bar{\lambda}$	0.3
Critical axial force	N_{cr}	3,030,497.2
Imperfection factor	α	0.2
value to determine the reduction factor X	ϕ	0.6
Reduction factor	χ	0.9
Buckling resistance (kN)	$N_{b,Rd}$	370.9
The buckling Resistance $N_{b,Rd}$ > Ned, No buckling		

Crossing Tube: **CIRC 80X4 mm**		
Compression Force (kN)	$N_{ed, max}$	12.3
Reduce slenderness	$\bar{\lambda}$	0.9
Critical axial force	N_{cr}	281,634.4
Imperfection factor	α	0.2
value to determine the reduction factor X	ϕ	1.1
Reduction factor	χ	0.7
Buckling resistance (kN)	$N_{b,Rd}$	167.0
The buckling Resistance $N_{b,Rd}$ > Ned,No buckling		

Diagonal Tube: CIRC 50X5.2 mm		
Compression Force (kN)	$N_{ed, max}$	45.2
Reduce slenderness	$\bar{\lambda}$	0.6
Critical axial force	N_{cr}	687,375.1
Imperfection factor	α	0.2
value to determine the reduction factor X	ϕ	0.7
Reduction factor	χ	0.9
Buckling resistance (kN)	$N_{b,Rd}$	200.4
The buckling Resistance $N_{b,Rd}$ > Ned, No buckling		

iv. Bolt verification

The verification of the bolt in terms of shear resistance and traction resistance has been proceeded in an excel sheet. The shear and traction resistance of one bolt should be greater than the acting action on the considered bolt. The results with maximum external action is given in Table 3.19.

Table 3.19. Bolt resistance

Resistance of bolt: M20 class 10.9		
Normal force (kN)	N_{Ed}	319.6
Shear force (kN)	T_{Ed}	13.9
Re. coefficient	α_v	0.5
Ultimate tensile stresses (Mpa)	f_{ub}	1,000.0
Gross area of the bolt (mm^2)	A_b	314.2
partial safety factor	γ_{Mb}	1.3
Diameter of bolt (mm)	d	20.0
Number of bolts	n_b	4.0
Shear resistance for one bolt (kN)	$F_{v,Rd}$	125.7
Traction resistance for one bolt (kN)	$F_{t,Rd}$	226.2
shear acting on one bolt (kN)	$F_{v,Ed}$	79.9
Traction acting on one bolt (kN)	$F_{t,Ed}$	3.5

From the previous table, it is observed that,

$$\mathrm{F}_{v,Rd} \geq F_{v,Ed}$$

$$F_{t,Rd} \geq F_{t,Ed}$$

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4\,F_{t,Rd}} = 0.64 \leq 1$$

v. Bearing resistance of the plate

Using the solicitations at the support, the bearing resistance of the plate is calculated in excel spreadsheet for interior and exterior holes. The result is given in Table 3.20.

Table 3.20. Resistance of plate

Bearing Resistance of Bolted Plate			
Nominal Ultimate Strength	f_{tb}	1,000.0	MPa
Characteristic Tensile Strength	f_{tk}	510.0	MPa
Diameter of Bolt	d	20.0	mm
Diameter of Holes	d_0	22.0	mm
Thickness of Brace	t	15.0	mm
ULS Safety Factor	γ_{M2}	1.3	
Design Action Force	$F_{b,Ed}$	79.9	kN
Longitudinal distance from exterior hole	e_1	40.0	mm
Transversal distance from exterior hole	e_2	50.0	mm
Longitudinal distance form interior hole	p_1	80.0	mm
Transversal distance from interior hole	p_2	100.0	mm
Coefficient	α_{ext}	0.7	
Coefficient	α_{int}	0.9	
Coefficient	k_{ext}	2.5	
Coefficient	k_{int}	2.5	
Design Resistance from exterior hole	$F_{b,Rd\ ext}$	185.5	kN
Design Resistance from interior hole	$F_{b,Rd\ int}$	294.5	kN
Verification		OK	

From the previous table, $F_{b,Rd\ int} \geq F_{b,Rd\ ext} \geq F_{b,Ed}$. So, the verification is satisfied.

From the previous results, the execution plans are established. Figures 3.18, 3.19 and 3.20 show details about lateral and internal supports and the connection between the deck and top chord respectively.

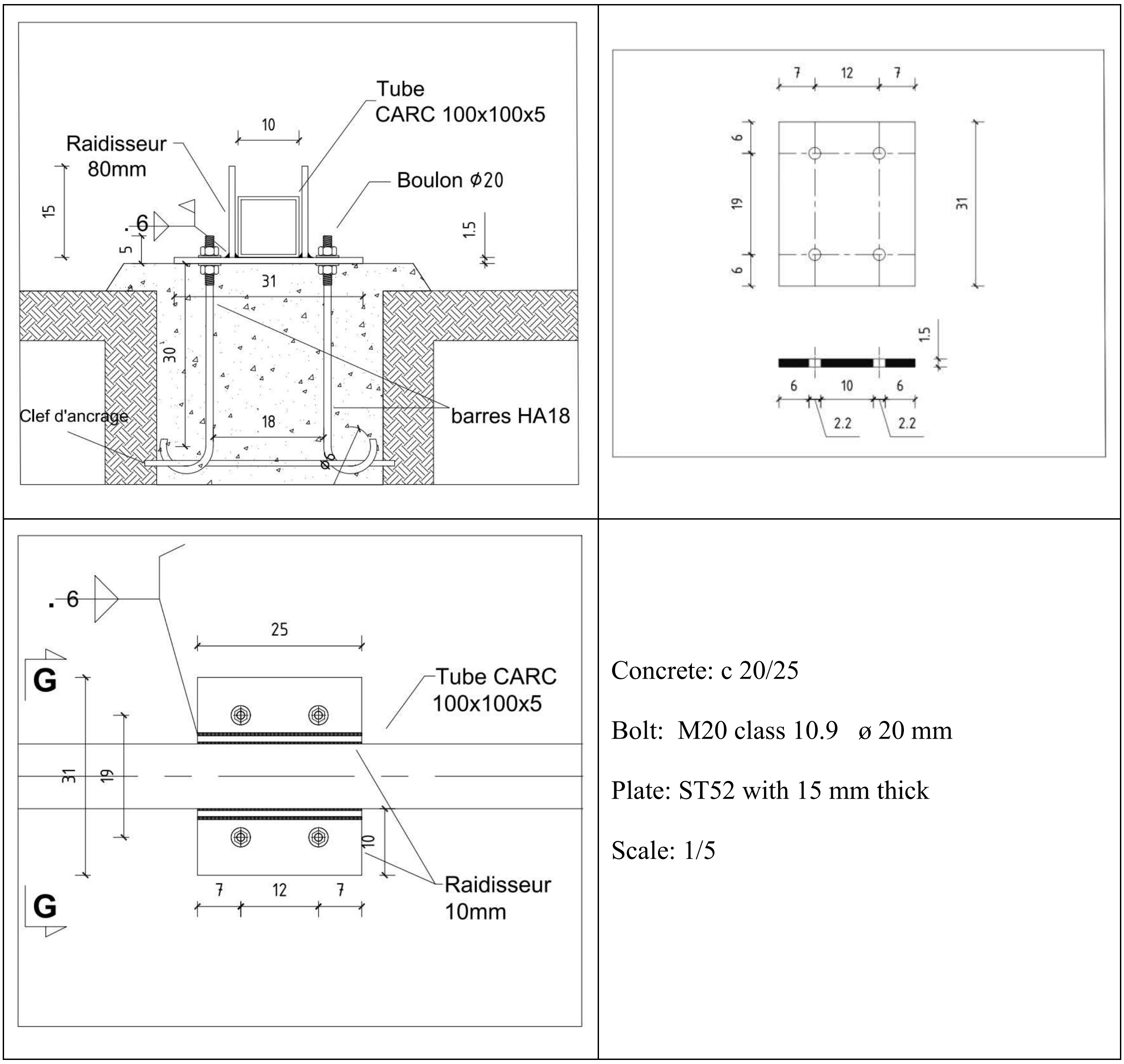
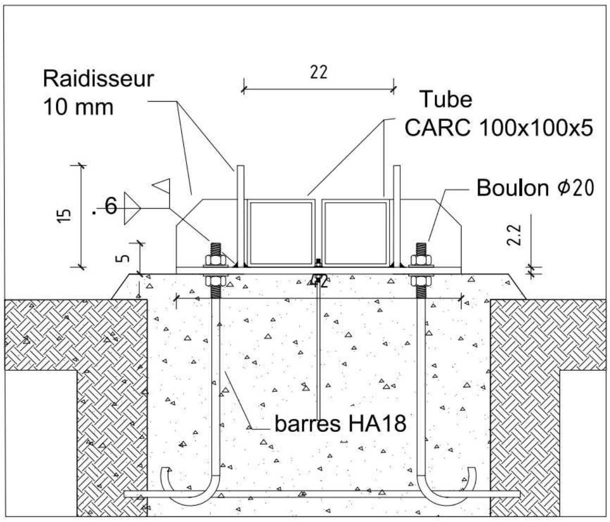


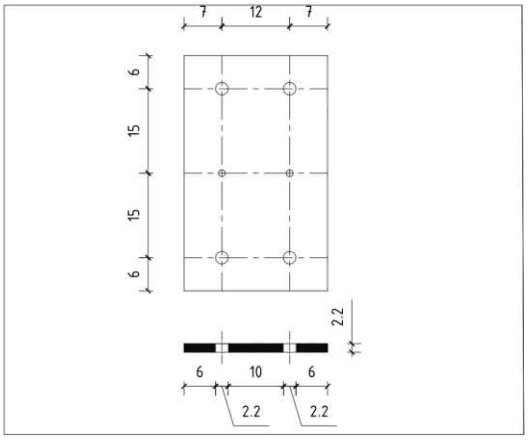
Figure 3.18. Details B (lateral supports)

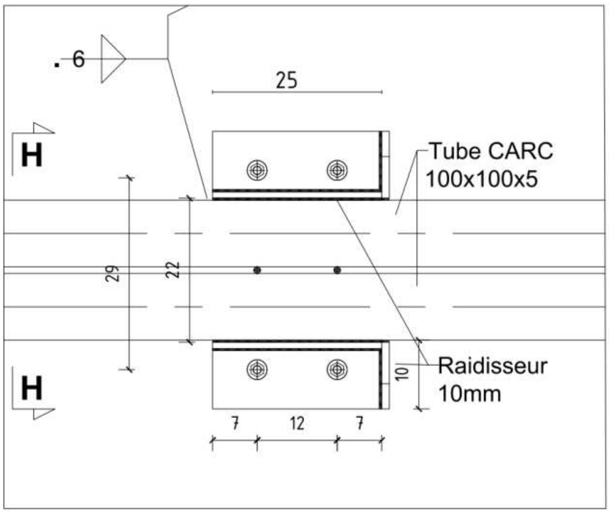
With respect to the Eurocode prescriptions, the distance range between bolts depending on the thickness of the plate, for a 15 mm plate are given in Table 3.21.

Table 3.21. Distances between bolts and from edges for 15 mm of thickness

	min	**max**
e_1	26.4	100
e_2	26.4	100
p_1	48.4	200
p_2	52.8	200







Concrete: c 20/25

Bolt: M20 class 10.9 ø 20 mm

Plate: ST52 with 22 mm thick

Scale: 1/5

Figure 3.19. Details E (internal supports)

With respect to the Eurocode prescriptions, the distance range between bolts depending on the thickness of the plate, for a 22 mm plate is given in Table 3.22.

Table 3.22. Distances between bolts and from edges for 22 mm of thickness

	Min	**Max**
e_1	26.4	128
e_2	26.4	128
p_1	48.4	200
p_2	52.8	200

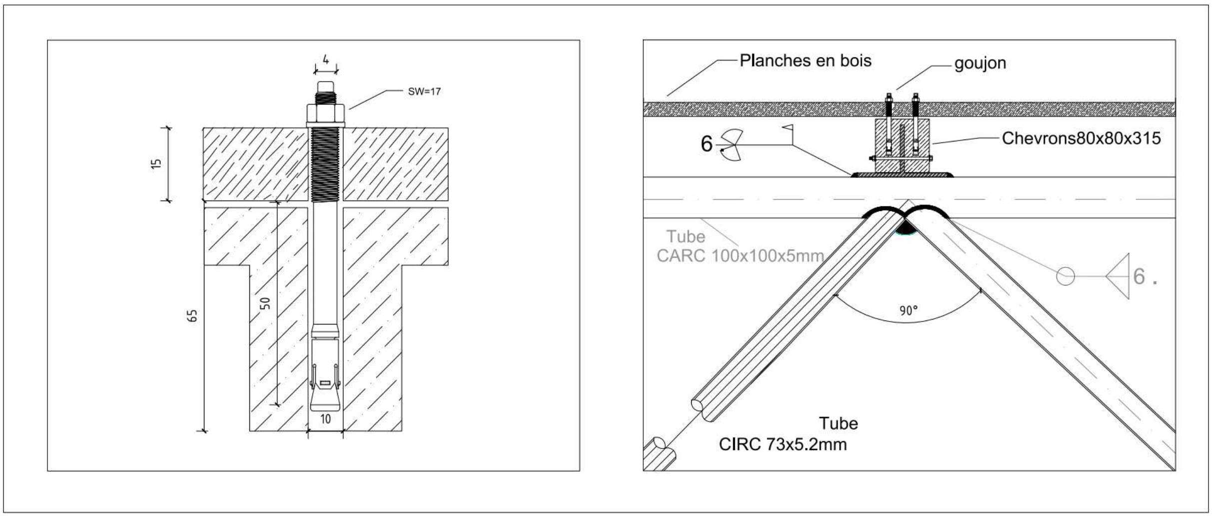


Figure 3.20. Details G (junction of deck with top chord)

b. Verification at Service ability limit state

At this stage, for the verification, it is more useful to consider the global deflection of the bridge and not the deflection of local elements. As given in Figure 3.21, the global deflection is equal to 2.7 cm.

Figure 3.21. Deflection of bridge in Midas

Considering the Eurocode prescription, $\frac{L}{250} = 6.7\ cm\ > 2.7\ cm$. So, the verification for deflection is verified.

3.4. Structural improvement

A parametric study will be carried out on our model considering different steel grades with varying sections of elements.

3.4.1. Case Study

The analyses were done with high strength structural steel, particularly S 450 (NF EN 10025-2) and S690Q/QL/QLI (NF EN 10025-6).

3.4.1.1. Ultimate Limit State verifications.

a. Verification at Ultimate Limit State

Stresses depend on section geometrical characteristics. Verification will be done for each section.

i. Geometrical characteristics

The geometric dimensions obtained are the results of a long iterative reduction process. The constraints to the reduction process are stress verifications and stability control. Both local and global stability have to be checked because the smaller the section, the greater the failure mechanisms due to instability.

- **Top and bottom chord elements**

On the top and bottom chords, a rectangular CARC tube of different steel grades with the different characteristics presented in Tables 3.23, 3.24, 3.25 and 3.26 is used.

Table 3.23. Top and bottom chord element goemetry S450

Top and Bottom Chord: CARC 70X70X4 mm		
Width of Steel Tube (mm)	a	70
Thickness of Steel tube (mm)	s_1	4
Height of Steel (mm)	h_{steel}	70
Cross Sectional Area of Steel (mm^2)	A_s	1,056
Height of Steel Centroid (mm)	y_G	35
Resistance Modulus of Steel (mm^3)	W	21,984.9
Inertia (mm^4)	I_{yy}	769,472

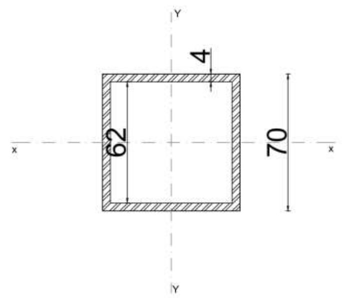


Figure 3.22. CARC 70x70x4mm

Table 3.24. Top and bottom chord element goemetry S690

Top and Bottom Chord: **CARC 60X60X4 mm**		
Width of Steel Tube (mm)	A	60
Thickness of Steel tube (mm)	s_1	4
Height of Steel (mm)	h_{steel}	60
Cross Sectional Area of Steel (mm²)	A_s	896
Height of Steel Centroid (mm)	y_G	30
Resistance Modulus of Steel (mm³)	W	15,689.9
Inertia (mm⁴)	I_{yy}	470,698.7

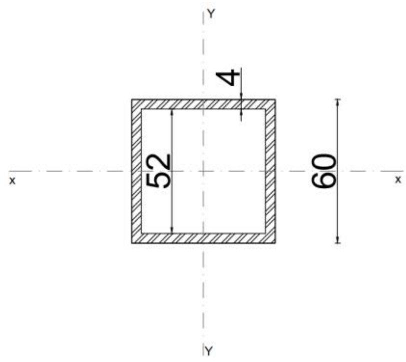


Figure 3.23. CARC 60x60x4mm

- **Top and bottom crossing tube.**

The CIRC characteristics are given in table 3.25.

Table 3.25. Crossing tube element goemetry for S450 and S690QL

Crossing Tube: CIRC 45X3 mm		
Diameter of Steel Tube (mm)	D	45
Thickness of Steel Lower Flange (mm)	s_1	3
Cross Sectional Area of Steel (mm²)	A_s	395.64
Height of Steel Centroid (mm)	y_G	22.5
Resistance Modulus of Inf Steel (mm³)	W	3,897.1
Inertia (mm⁴)	I_{yy}	87,683.7

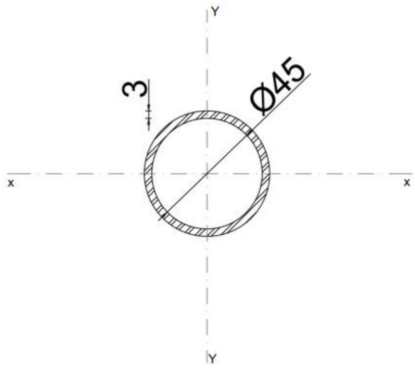


Figure 3.24. CIRC 45x3 mm

- Diagonal element

For the diagonal elements, CIRC was used, with its characteristics given in Table 3.26.

Table 3.26. Diagonal element goemetry for s450 and S690QL

Diagonal Element: CIRC 40x3 mm		
Diameter of Steel Tube (mm)	D	40
Thickness of Steel Lower Flange (mm)	s_1	3
Cross Sectional Area of Steel (mm^2)	A_s	348.54
Height of Steel Centroid (mm)	y_G	20
Resistance Modulus of Inf Steel (mm^3)	W	3,001.8
Inertia (mm^4)	I_{yy}	60,036.1

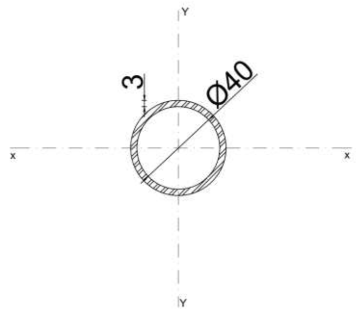


Figure 3.25. CIRC 40x3mm

ii. Stresses Verification

According to the Navier formula given in the previous chapter, the verification of the section in terms of stress has been performed in an excel sheet. Results of stresses in elements have to be less than the design yield strength of S450 (f_{yd}= 419.047 MPa) and S690 (f_{yd}= 657.142 MPa). Table 3.27 gives us the verifications.

- **For STEEL: S 450 (NF EN 10025-2)**

Table 3.27. Stresses verification for S450 elements

	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	$s_{Steel,Sup}$	$s_{Steel,Inf}$
Envelope of the ULS combinations	1.17	2.62	-319.62	**-249.45**	**-355.88**
Verification	OK				
Top Chord of internal frame					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	$s_{Steel,Sup}$	$s_{Steel,Inf}$

Envelope of the ULS combinations	0.9	1.14	-280.54	**-224.72**	**-306.60**
Verification	OK				
Bottom Chord of external frame					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	**$s_{Steel,Sup}$**	**$s_{Steel,Inf}$**
Envelope of the ULS combinations	1.17	1.63	110.13	**157.50**	**51.07**
Verification	OK				
Bottom Chord of internal frame					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	**$s_{Steel,Sup}$**	**$s_{Steel,Inf}$**
Envelope of the ULS combinations	**1.01**	**2.12**	**120.30**	**159.49**	**68.34**
Verification	OK				
Crossing Elements Top					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	**$s_{Steel,Sup}$**	**$s_{Steel,Inf}$**
Envelope of the ULS combinations	0.25	1.67	20.10	**114.95**	**-13.34**
Verification	OK				
Transversal Element bottom					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	**$s_{Steel,Sup}$**	**$s_{Steel,Inf}$**

Envelope of the ULS combinations	**0.80**	**0.31**	**-12.30**	**174.19**	**-236.37**
Verification	OK				
Diagonal Elements in Tension					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	**$s_{Steel,Sup}$**	**$s_{Steel,Inf}$**
Envelope of the ULS combinations			26.17	**75.08**	**23.63**
Verification	OK				
Diagonal Elements in Compression					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	**$s_{Steel,Sup}$**	**$s_{Steel,Inf}$**
Envelope of the ULS combinations			-45.17	**-129.59**	**-40.80**
Verification	OK				
Brace Elements in Tension					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	**$s_{Steel,Sup}$**	**$s_{Steel,Inf}$**
Envelope of the ULS combinations			21.34	**61.22**	**19.27**
Verification	OK				
Brace Elements in compression					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	**$s_{Steel,Sup}$**	**$s_{Steel,Inf}$**

Envelope of the ULS combinations			-32.72	**-93.87**	**-29.55**
Verification	OK				

- **For steel S690Q/QL/QLI (NF EN 10025-3)**

Top Chord of external frame					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	**$s_{Steel,Sup}$**	**$s_{Steel,Inf}$**
Envelope of the ULS combinations	1.17	2.62	-319.62	**-282.14**	**-431.28**
Verification	OK				

Top Chord of internal frame					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	**$s_{Steel,Sup}$**	**$s_{Steel,Inf}$**
envelope of the ULS combinations	0.90	1.14	-280.54	**-255.74**	**-370.46**
Verification	OK				

Bottom Chord of external frame					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	**$s_{Steel,Sup}$**	**$s_{Steel,Inf}$**
Envelope of the ULS combinations	1.17	1.63	110.13	**197.48**	**48.34**

Verification	OK				
Bottom Chord of internal frame					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	$s_{Steel,Sup}$	$s_{Steel,Inf}$
Envelope of the ULS combinations	**1.01**	**2.12**	**120.30**	**198.12**	**70.40**
Verification	OK				
Crossing Elements Top					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	$s_{Steel,Sup}$	$s_{Steel,Inf}$
Envelope of the ULS combinations	0.25	1.67	20.10	**114.95**	**-13.34**
Verification	OK				
Transversal Element bottom					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	$s_{Steel,Sup}$	$s_{Steel,Inf}$
Envelope of the ULS combinations	**0.80**	**0.31**	**-12.30**	**174.19**	**-236.37**
Verification	OK				
Diagonal Elements in Tension					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	$s_{Steel,Sup}$	$s_{Steel,Inf}$
Envelope of the			26.17	**75.08**	**75.08**

ULS combinations					
Verification	OK				
Diagonal Elements in Compression					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	$s_{Steel,Sup}$	$s_{Steel,Inf}$
Envelope of the ULS combinations			-45.17	**-129.59**	**-129.59**
Verification	OK				
Brace Elements in Tension					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	$s_{Steel,Sup}$	$s_{Steel,Inf}$
Envelope of the ULS combinations			21.34	**61.22**	**61.22**
Verification	OK				
Brace Elements in compression					
	Maximum Design Internal reactions			**Stresses**	
Combination	**M (kNm)**	**T (kN)**	**N (kN)**	$s_{Steel,Sup}$	$s_{Steel,Inf}$
Envelope of the ULS combinations			-32.72	**-93.87**	**-93.87**
Verification	OK				

iii. Resistance to buckling

The verification of elements in terms of resistance to buckling has been performed in an excel sheet and is given in Table 3.28 and 3.29.

- For S 450,

Table 3.28. Buckling reistance for improved section S450

Top chord Element CARC 70X70X4mm		
Compression Force (kN)	$N_{ed,\,max}$	319.6
Reduce slenderness	$\bar{\lambda}$	0.7
Effort axial critique	N_{cr}	813,684.1
Imperfection factor	α	0.2
value to determine the reduction factor X	ϕ	0.8
Buckling factor	χ	0.8
Buckling resistance (kN)	$N_{b,Rd}$	333.7
The Buckling Resistance Nb,Rd > Ned, No Buckling		

Crossing Tube: CIRC 45X3 mm		
Compression Force (kN)	$N_{ed,\,max}$	12.3
Reduce slenderness	$\bar{\lambda}$	3.1
Critical axial force	N_{cr}	17,747.5
Imperfection factor	α	0.2
value to determine the reduction factor X	ϕ	5.5
Buckling factor	χ	0.1
Buckling resistance (kN)	$N_{b,Rd}$	15.1
The Buckling Resistance Nb,Rd > Ned, No Buckling		

Diagonal Tube : CIRC 40x3 mm		
Compression Force (kN)	$N_{ed,\,max}$	45.2
Reduce slenderness	$\bar{\lambda}$	1.6
Critical axial force	N_{cr}	64,495.1
Imperfection factor	α	0.2
value to determine the reduction factor X	ϕ	1.8
Buckling factor	χ	0.4
Buckling resistance (kN)	$N_{b,Rd}$	49.2
The Buckling Resistance Nb,Rd > Ned, No Buckling		

- **For S690Q/QL/QLI**

Table 3.29. Buckling reistance for improved section S690

Top chord Element CARC 60X60X4mm		
Compression Force (kN)	$N_{ed, max}$	319.6
Reduce slenderness	$\bar{\lambda}$	1.1
Critical axial force	N_{cr}	497,743.9
Imperfection factor	α	0.2
value to determine the reduction factor X	ϕ	1.2
Buckling factor	χ	6
Buckling resistance (kN)	$N_{b,Rd}$	323.6
The Buckling Resistance Nb,Rd > Ned,No buckling		

Crossing Tube: CIRC 45x3 mm		
Compression Force (kN)	$N_{ed, max}$	12.3
Reduce slenderness	$\bar{\lambda}$	3.9
Critical axial force	N_{cr}	17,747.5
Imperfection factor	α	0.2
value to determine the rcduction factor X	ϕ	8.2
Buckling factor	χ	0.1
Buckling resistance (kN)	$N_{b,Rd}$	15.3
The Buckling Resistance Nb,Rd > Ned, No buckling		

Diagonal Tube : CIRC 40x3 mm		
Compression Force (kN)	$N_{ed, max}$	45.2
Reduce slenderness	$\bar{\lambda}$	1.9
Critical axial force	N_{cr}	64,495.1
Imperfection factor	α	0.2
value to determine the reduction factor X	ϕ	2.5
Buckling factor	χ	0.5
Buckling resistance (kN)	$N_{b,Rd}$	51.8
The Buckling Resistance Nb,Rd > Ned,No buckling		

3.4.1.2. Serviceability limit state

Considering the Eurocode prescription, $\frac{L}{250} = 6.7\ cm\ > 2.3\ cm$. So, the verification for deflection is verified.

3.4.2. Total weight comparison.

Here are the calculations of the weights of the different elements that constitute our temporary bridge as well as the total weight of the bridge according to the grade of steel and the sections considered.

a. Comparison for each group of elements.

Table 3.30 shows the differences in weight for each group of considered structures.

Table 3.30. Percentage of mass reduction

GROUP 1: Top and Bottom Chord						
Steel	**CARC 100x100x5 mm**		**CARC 70x70x4 mm**		**CARC 60x60x4 mm**	
	S 235 (NF EN 10025-2)		**S 450 (NF EN 10025-2)**		**S690Q/QL/QLI (NF EN 10025-3)**	
	Value	**Unit**	**Value**	**Unit**	**Value**	**Unit**
Total Linear	211.100	**m**	211.079	**m**	211.079	**m**
Section Area	0.002	**m^2**	0.001	**m^2**	0.001	**m^2**
Total Volume	0.401	**m^3**	0.223	**m^3**	0.189	**m^3**
Density	7,850	**kg/m^3**	7,850	**kg/m^3**	7,850	**kg/m^3**
Total Weight	3,148.249	**kg**	1,749.763	**kg**	1,484.648	**kg**
% of reduced			**44.421**		**52.842**	
Group 2: Crossing Tube						
Steel	**CIRC 100x4 mm**		**CIRC45x3 mm**		**CIRC 45X3 mm**	
	S 235 (NF EN 10025-2)		**S 450 (NF EN 10025-2)**		**S690Q/QL/QLI (NF EN 10025-3)**	
	Value	**Unit**	**Value**	**Unit**	**Value**	**Unit**

Total Linear	90.450	**m**	90.450	**m**	90.450	**m**
Section Area	0.001	**m²**	0.001	**m²**	0.001	**m²**
Total Volume	0.109	**m³**	0.036	**m³**	0.035	**m³**
Density	7,850	**kg/m³**	7,850	**kg/m³**	7,850	**kg/m³**
Total Weight	856.128	**kg**	280.917	**kg**	280.917	**kg**
% of reduced			**67.1875**		**67.1875**	

Group 3: Diagonal Element in Longitudinal Direction

Steel	**CIRC 73x5.2 mm (Diagonal)**		**CIRC40x3 mm (Diagonal)**		**CIRC 40x3 mm**	
	S 235 (NF EN 10025-2)		**S 450 (NF EN 10025-2)**		**S690Q/QL/QLI (NF EN 10025-3)**	
	Value	**Unit**	**Value**	**Unit**	**Value**	**Unit**
Total Linear	154.432	**m**	154.432	**m**	154.432	**m**
Section Area	0.001	**m²**	0.001	**m²**	0.001	**m²**
Total Volume	0.170	**m³**	0.053	**m³**	0.053	**m³**
Density	7,850	**kg/m³**	7,850	**kg/m³**	7,850	**kg/m³**
Total Weight	1,342.053	**kg**	422.532	**kg**	422.532	**kg**
% of reduced			**68.51599728**		**68.51599728**	

Group 4: Diagonal Element in Transversal Direction

Steel	**CIRC 73x5.2 mm (Diagonal)**		**CIRC50x4 mm (Diagonal)**		**CARC 65x65x4 mm**	
	S 235 (NF EN 10025-2)		**S 450 (NF EN 10025-2)**		**S690Q/QL/QLI (NF EN 10025-3)**	
	Value	**Unit**	**Value**	**Unit**	**Value**	**Unit**
Total Linear	198.640	**m**	198.640	**m**	198.640	**m**
Section Area	0.001	**m²**	0.001	**m²**	0.001	**m²**
Total Volume	0.220	**m³**	0.069	**m³**	0.069	**m³**
Density	7,850	**kg/m³**	7,850	**kg/m³**	7,850	**kg/m³**
Total Weight	1,726.231	**kg**	543.486	**kg**	543.486	**kg**
% of reduced			**68.515**		**68.515**	

From these previous data, Figure 3.26 shows graphically the variation of the weight.

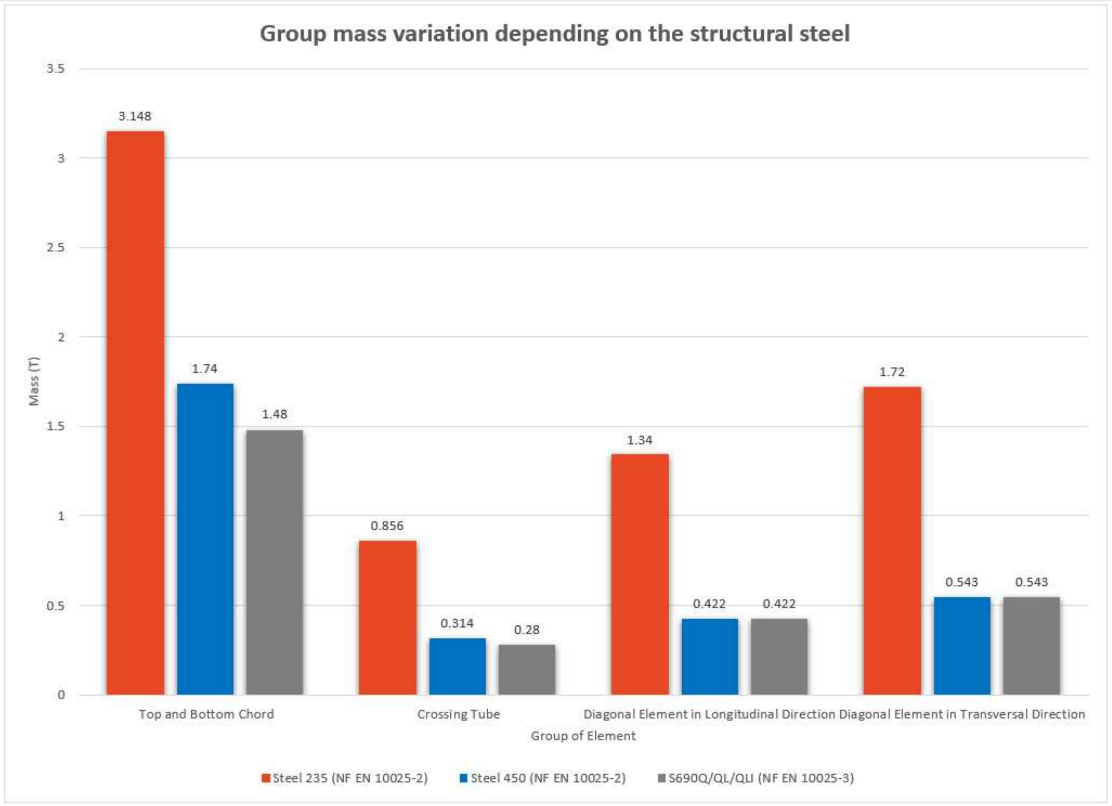


Figure 3.26. Group weight variation depending on the structural steel

b. The whole structure.

After having determined separately the weight loss for each group of elements, it is now necessary to bring out an overall evaluation in order to have a total reduction in weight of the structure. This will help, after further analysis, to determine the most favorable situation. Table 3.31 and Figure 3.27 present effectively the variation in mass depending on the steel grade. The final total weight is obtained by doing the sum of the weight of the steel grade and the Dabema wood wearing course.

Table 3.31. Percentage of weight reduction

Materials	Steel						Wood	
	S 235 (NF EN 10025-2)		S 450 (NF EN 10025-2)		S690Q/QL/QLI (NF EN 10025-3)		DABEMA	
	Value	Unit	Value	Unit	Value	Unit	Value	Unit
Total Linear	654.601	m	654.601	m	654.601	m	16.500	m
Section Area	0.006	m^2	0.003	m^2	0.002	m^2	0.335	m^2
Total Volume	0.900	m^3	0.381	m^3	0.347	m^3	5.527	m^3
Density	7850	kg/m^3	7850	kg/m^3	7850	kg/m^3	700.00	kg/m^3
Weight	**7,072.663**	kg	**2,996.700**	kg	**2,731.584**	kg	**3,869.250**	kg
Total Weight	**10,941.913**		**6,865.950**		**6,600.834**			
Total % weight reduced			**37.250**		**39.673**			

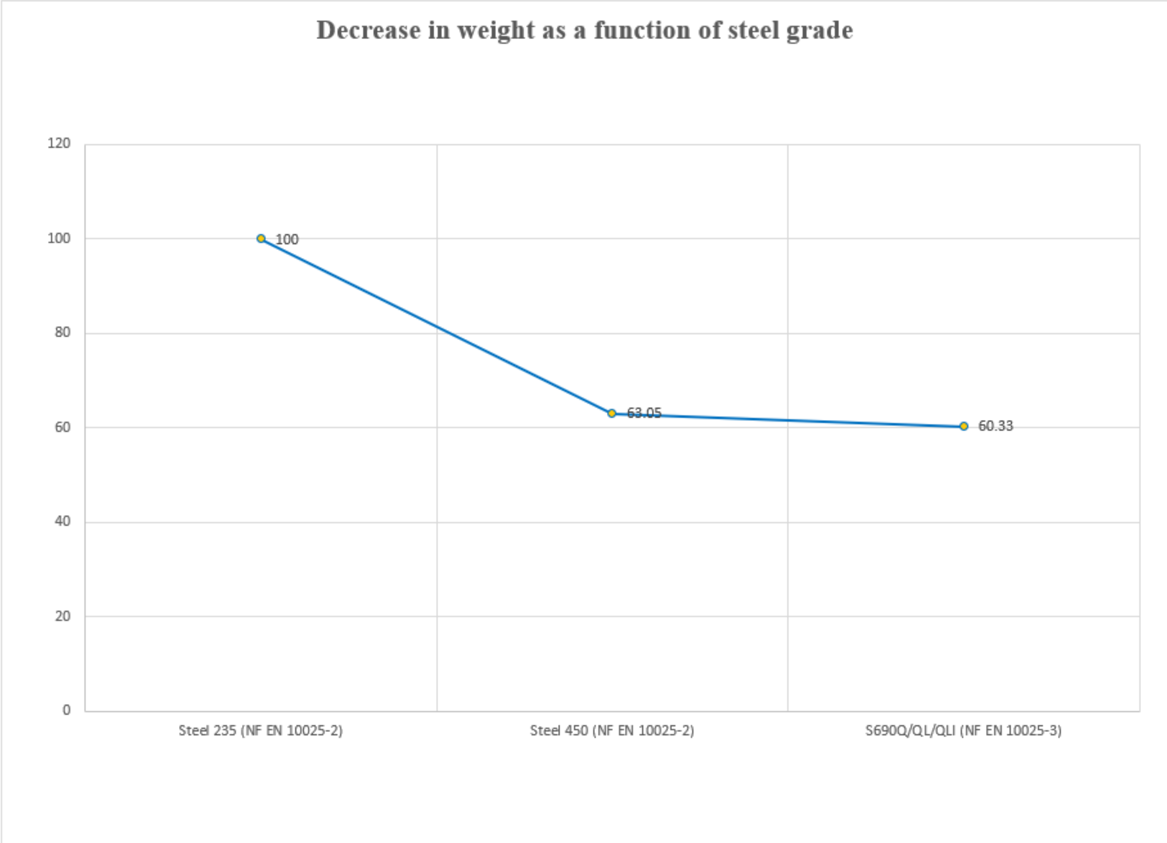


Figure 3.27. Decrease in weight as a function of steel grade

From the above data, it is observed that the variation in weight is very significant when going from conventional steel S235 to high strength steel S450 (38.26% reduction), but this reduction is not as significant when going from S450 grade to S690QL (Less than 3% reduction).

3.4.3. Cost comparison

The prices of steel profiles on the market depend on several parameters, such as ductility class, structural class, and treatment methods, such as galvanized steel. For our study, the price was based on the price per tone of the square and round tube steel grades S235Jr, S460M, and S690QL proposed by the company "Made-in-China manufacturers and suppliers" on Wednesday, June 15, 2022.The following amounts will be considered: 255,300 CFA/Ton for S235jr, 394,050 CFA/Ton for S450 and 499,500 CFA/Ton for galvanized S690QL, and 197,250/Ton for Dabema wood. Table 3.32 presents the total costs of the structure according to the structural steel classes, and Figure 3.28 the percentage cost increase in function of steel grade. The final total cost is the sum of the cost of the steel grade plus the cost of the wood.

Table 3.32. Percentage of total cost variation

Material	S 235 (NF EN 10025-2)	S 450 (NF EN 10025-2)	S690Q/QL/QLI (NF EN 10025-3)	DABEMA
Total weight (kg)	7,072.6	6,865.9	6,600.8	3,869.2
Cost/Ton (CFA)	255,300	394,050	499,500	194,250
Total cost (CFA)	2,557,252.79	3,457,129.45	4,048,718.53	
% Of variation in cost		**35.19**	**58.32**	

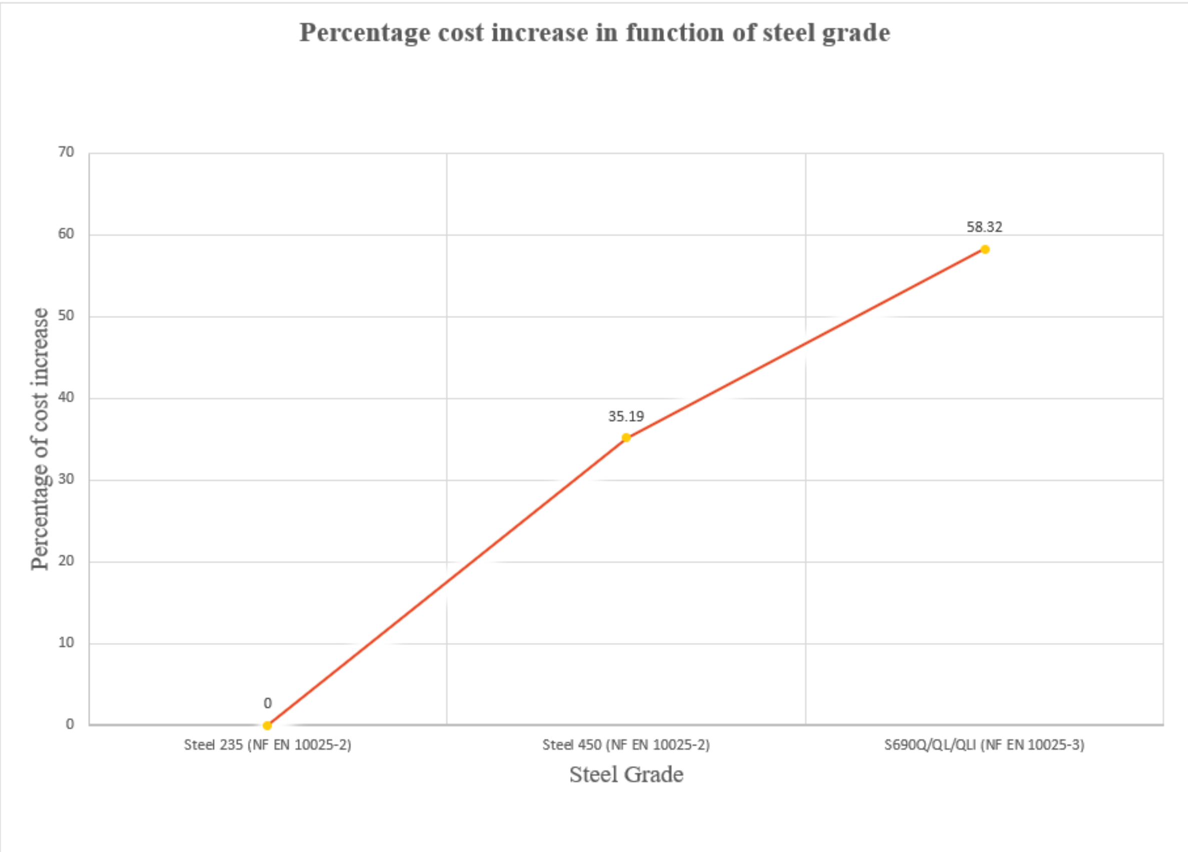


Figure 3.28. Percentage cost increase in function of steel grade

From the previous results, it is observed that when we use the S450 steel as improvement steel for the same load capacity, there is an overall reduction in weight of 37.25 % for a variation in costs of + 35 %, while with the S690QL, a total reduction in weight of 39.67 % for a variation in costs of 58.32% is observed. For all those reasons, we can see it is more convenient to use the S450 as steel grade for the improvement.

CONCLUSION

To conclude, the results of the methodology of Chapter 2 have been developed and applied to the case study which was a temporary bridge in Nachtigal hydroelectric power dam project in Nachtigal-Cameroon. After a general presentation of the site based on its geographical location, its surface water hydrology, its climate and its socio-economic activities, a presentation of the project was followed through its geometry and characteristics of the materials used. The structure was modeled on Midas/Civil 2019 and SAP2000 version 22. A static analysis of the structure was done in order to ensure that our structure is stable. The solicitations were obtained and a static analysis has been performed. After ensuring that the structure is appropriately designed, sensitivity analysis was performed. The goal was to find out the reduction in weight by the variation of the sections of elements and the utilization of HSS. Different HSSs were used, one with characteristic yield strength equal to 440 MPa (S450), and another with characteristic yield strength equal to 690 MPa (S690QL). The results obtained from the analysis revealed that the weight of the bridge decreases considerably by around 40% when using HSS. The stability problems encountered during the analysis (locally or globally) are mostly those of buckling and deflection, which are in most cases a limit for the HSS to be used. Since the structural improvement problem is not separated from the economic problem, it was observed that the cost of improvement with the steel class S690 is almost double the original price i.e., 58.32%, for a total reduction in weight of 39.67%. For the steel class S450, it was observed an increase in the total cost of the structure by 35% for a weight reduction of 37.25%. so S450 was therefore agreed for an inprovement that is both structurally and economically advantageous.

GENERAL CONCLUSION

Coming to the end of the study entitled "Design strategies and erection performances of temporary bridge.", it is paramount to recall that the main aim of this work was to improve using high strength steel the weight of a modular temporary bridge structure, in order to ease its transportation to the different usage zones during the construction project of the hydroelectric power dam of Nachtigal-Cameroon. To achieve this objective, a review of the concept of temporary bridge construction, the design requirements and mechanisms of implementation of temporary bridges were presented in chapter 1, the methodology to be used was presented in chapter 2 and the results presentation and interpretation were done in chapter 3. The structure was modelled in Midas/Civil 2019 and SAP2000 version 22 software, the solicitations were obtained and a static analysis was done. After ensuring that our structure was suitably designed, the sensitivity analysis was performed. The goal was to find out the reduction in weight by the variation of the sections of elements and the utilization of HSS. Different HSSs were used, one with characteristic yield strength equal to 440 MPa (S450), and another with characteristic yield strength equal to 690 MPa (S690QL).

The improvement problems not being dissociated from the economic problems, it is observed that the improvement with the S690 steel class costs almost twice as much, i.e., 58.32% compared to the initial price, for a reduction in total weight of the bridge of 39.67%. On the other hand, with the S450 steel class it is obtained an increase in the global cost of the structure of 35% for an almost similar reduction in weight, i.e., 37.25%. so S450 was therefore agreed for an improvement that is both structurally and economically advantageous.

The subject matter is very broad and it was necessary to limit the scope of research for this work. Another obstacle was the absence of rules for grades higher than S690 in Eurocode 3 because the latest publication, Part 1-12 extends the design rules up to S700. In order to improve this work, the following suggestions can be made for future studies:

- Use of a steel grade higher than S690 to perform the checks;
- Analysis of the structure during the lifting phase;
- Study the impact of changes in the shape and geometry of the elements

BIBLIOGRAPHY

AASHTO. (2007). AASHTO LRFD Bridge Design Specifications: SI Unit 4th Edition 2007. In *American Association of State Highway and Transportation Officials*.

Aida, N., Reveiz, F., Officer, C. I., Expert, T., Mbianyor, B., Compliance, S., Bella-corbin, A., Safeguards, C., Officer, C., Durowoju, R., Diallo, K., & Shonibare, W. (2017). *PROJECT : NATCHIGAL HYDROPOWER PROJECT COUNTRY :* 1–26.

Army, D. of the. (1986). *B a i l e y b r i d g e*. *5*, 373.

Army, U. (2010). *Weapon SyStems 2010*. 1–371.

Arthur Ward Buckingham. (1911). *Methods of bridge erection*. 116.

Asghar, S. (2021). *Designing a Truss Bridge*. *January*. https://doi.org/10.13140/RG.2.2.12015.05282

Bois, D. D. U., & Grume, D. D. E. L. A. (n.d.). *Description du bois*. 2–5.

Bouassida, Y., Bouchon, E., Crespo, P., Croce, P., Davaine, L., Denton, S., Feldmann, M., Frank, R., Hanswille, G., Hensen, W., Kolias, B., Malakatas, N., Mancini, G., Ortega, M., Raoul, J., Sedlacek, G., Tsionis, G., Athanasopoulou, E. A., Poljansek, M., & Pinto, A. (2012). Bridge design to eurocodes. In *JRC, European Commission* (Vol. 1, Issue 1). https://doi.org/10.2788/82360

BRIDGE ENGINEERING. (n.d.).

British Standards Institution (BSI). (2019). *BSI Standards Publication Execution of steel structures and aluminium structures*.

Brown, D. (1996). *Kühne Konstruktionen über Flüsse*.

Collin, P., & Johansson, B. (2005). *No Title*. 12. https://doi.org/4b6d0850-0ad9-11dc-9854-000ea68e967b

Cookson, M. D., & Stirk, P. M. R. (2019). *済無No Title No Title No Title*.

EN 1991-1-4. (2011). *1*(2005).

EN 1991-1-5. (2011). *1*(2005).

EN 1991-2. (2003). *1*(2005).

En, B. S. (2005a). *Eurocode 3 : Design of steel structures —*. *3*.

En, B. S. (2005b). *Eurocode 3 : Design of steel structures —*. *3*(1).

Env, X. P. (2000). *Eurocode 1 : Bases de calcul et actions sur les structures et document d ' application nationale*. 1–5.

Env, X. P., & Pa, B. (1997). *Eurocode 1 : Bases de calcul et actions sur les structures*.

Gerbo, E. J., Casias, C. M., Thrall, A. P., & Zoli, T. P. (2016). New Bridge Forms Composed of Modular Bridge Panels. *Journal of Bridge Engineering*, *21*(4), 04015084. https://doi.org/10.1061/(asce)be.1943-5592.0000871

index @ www.estrepublicain.fr. (n.d.). https://www.estrepublicain.fr/

index @ www.lindependant.fr. (n.d.). https://www.lindependant.fr/

Iv, I. I. I. I. I. (1991). *Eurocodes Afrique • Africa Eurocodes Eurocodes Afrique • Africa Eurocodes*. 10–11.

Joiner, C. J. H. (2006). One More River to Cross: The Story of British Military Bridging. In 1990 Pen and Sword (Ed.), *Pen & Sword Books Ltd* (illustrée).

Mabey-Johnson-Bridge-Smartbook-version-2-pdf @ fr.scribd.com. (n.d.). https://fr.scribd.com/document/410482862/Mabey-Johnson-Bridge-Smartbook-version-2-pdf#sidebar

Matiere, N., Ung, Q. H., & Nicolaudie, P. A. (2018). Unibridge®: A new concept in prefabricated modular bridge. *Lecture Notes in Civil Engineering*, *8*, 981–987. https://doi.org/10.1007/978-981-10-6713-6_98

Riaz, F., Ahmad, R., Alam, K., & Abid, A. S. (2013). *Design Optimization of Modular Bridge Structure*. *328*, 970–974. https://doi.org/10.4028/www.scientific.net/AMM.328.970

Russell, B. R., & Thrall, A. P. (2013). Portable and Rapidly Deployable Bridges: Historical Perspective and Recent Technology Developments. *Journal of Bridge Engineering*, *18*(10), 1074–1085. https://doi.org/10.1061/(asce)be.1943-5592.0000454

Schumacher, A. (2017). *Modern Tubular Truss Bridges Modern Tubular Truss Bridges 2 . Examples of Welded CHS Truss Bridges*. *January 2002*. https://doi.org/10.2749/222137802796337332

Shahid, Z. (2020). *Design of a Truss Bridge to support 500g load. December*. https://doi.org/10.13140/RG.2.2.32897.28007

Taly, N. (1997). *[Book] toaz.info-design-of-modern-highway-bridgestalypdf-pr_8176a1cfd0bc10180a42261b08372221*.

Taylor, P., & Séquin, C. H. (2013). *Computer-Aided Design and Applications CAD Tools for Aesthetic Engineering. January 2015*, 37–41. https://doi.org/10.1080/16864360.2004.10738271

Tecchio, E. G. (2020). *Degree Course on Civil Engineering Design of Bridges*. www.dicea.unipd.it

Tecchio, G. (n.d.). *Module I : Basis of design and execution of bridges Terminology , typologies , constitutive*.

Uc, V. (2011). *Computers in Industry Structural optimization with CADO method for a three-dimensional sheet-metal vehicle body. 62*, 78–85. https://doi.org/10.1016/j.compind.2010.06.001

Weldegiorgis, F., & Dhungana, A. R. A. J. (2020). *Parametric design and optimization of steel and Development of a workflow for design and*.

Worenz, O. (2018). *Temporary bridge constructions and its implementation in a civil and military context. January*.

Yang, M. C., & Angeles, L. (2005). *and design outcome. 26*, 649–669. https://doi.org/10.1016/j.destud.2005.04.005

Zhu, Q.-X., Wang, H., Mao, J.-X., Wan, H.-P., Zheng, W.-Z., & Zhang, Y.-M. (2020). Investigation of Temperature Effects on Steel-Truss Bridge Based on Long-Term Monitoring Data: Case Study. *Journal of Bridge Engineering*, *25*(9), 05020007. https://doi.org/10.1061/(asce)be.1943-5592.0001593

Zierhofer, F. (2011). *Die Notwendigkeit des österreichischen militärischen Behelfsbrückenbaus in In- und Auslandseinsätzen.TherMilAk. Wiener Neustadt: 2011* (TherMilAk).

WEBOGRAPHY

https://www.huayaosteel.cn/products_list/953001447931150336.html?gclid=Cj0KCQjw8O-VBhCpARIsACMvVLNooOEppxb1VDwi7lvMqp8nJs5JPRw65x1BnglgQwhqZ5w4Z5DCZEAaAvpwEALw_wcB.html

https://www.made-in-china.com/products-search/hot-china-products/Steel_Plate_Price.html

https://www.mabeybridge.com/products?f.Category|categoryNames=Bridging.html

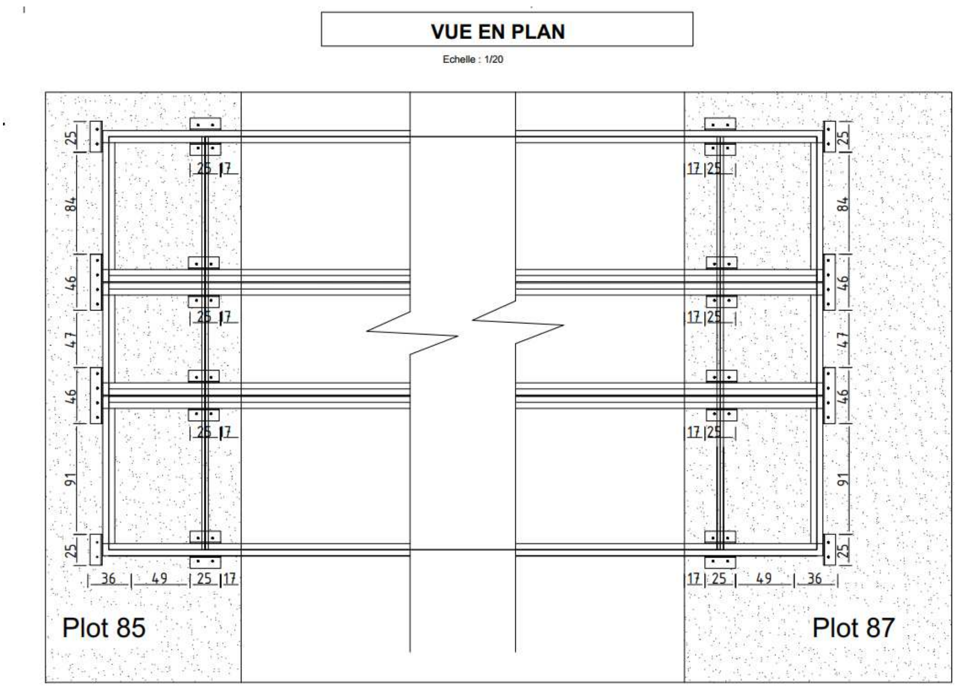
https://www.fs.fed.us/t-d/pubs/htmlpubs/htm06232824/page04.html

ANNEXES

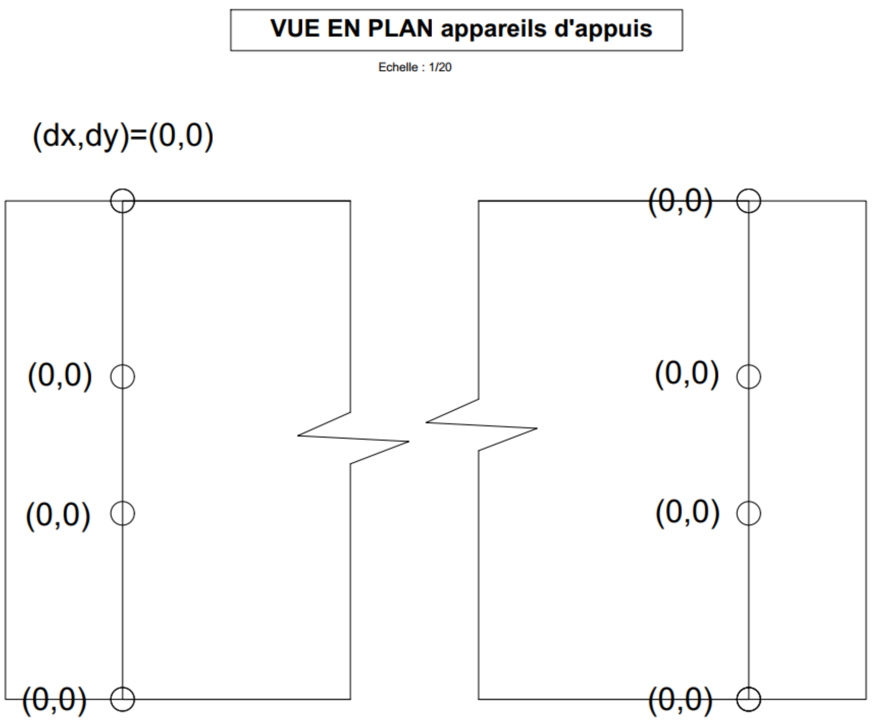
```
+==================================================================+
               |  MIDAS(Modeling, Integrated Design & Analysis
Software)    |
               |  MIDAS/Civil - Load Combinations
|
               |                                            (c)SINCE
1989  |

+==================================================================+
               |  MIDAS Information Technology Co.,Ltd.        (MIDAS
IT)  |
               |  MIDAS/Civil  Version 8.7.5
|

+==================================================================+

-----------------------------
DESIGN TYPE : General
-----------------------------

LIST OF LOAD COMBINATIONS
========================================================================
======================
NUM  NAME          ACTIVE             TYPE
               LOADCASE(FACTOR) +          LOADCASE(FACTOR) +
LOADCASE(FACTOR)
========================================================================
======================
1    gLCB1         Active             Add
            Self Weight( 1.350) +          GuardRail( 1.350) +
Slab Load( 1.350)
 +         Windloadcons( 1.500)
------------------------------------------------------------------------
----------------------
2    gLCB2         Active             Add
            Self Weight( 1.350) +          GuardRail( 1.350) +
Slab Load( 1.350)
 +         Windloadcons(-1.500)
------------------------------------------------------------------------
----------------------
3    gLCB3         Active             Add
            Self Weight( 1.350) +          GuardRail( 1.350) +
Slab Load( 1.350)
 +       Thermal Action( 1.500)
------------------------------------------------------------------------
----------------------
4    gLCB4         Active             Add
            Self Weight( 1.350) +          GuardRail( 1.350) +
Slab Load( 1.350)
 +       Thermal Action(-1.500)
126  STL ENV_STR   Active             Envelope
```

```
                gLCB90( 1.000) +               gLCB91( 1.000) +
gLCB92( 1.000)
 +              gLCB93( 1.000) +               gLCB94( 1.000) +
gLCB95( 1.000)
 +              gLCB96( 1.000) +               gLCB97( 1.000) +
gLCB98( 1.000)
 +              gLCB99( 1.000) +              gLCB100( 1.000) +
gLCB101( 1.000)
 +             gLCB102( 1.000) +              gLCB103( 1.000) +
gLCB104( 1.000)
 +             gLCB105( 1.000)
-----------------------------------------------------------------------
----------------------
127  STL ENV_SER   Active             Envelope
               gLCB106( 1.000) +              gLCB107( 1.000) +
gLCB108( 1.000)
 +             gLCB109( 1.000) +              gLCB110( 1.000) +
gLCB111( 1.000)
 +             gLCB112( 1.000) +              gLCB113( 1.000) +
gLCB114( 1.000)
 +             gLCB115( 1.000) +              gLCB116( 1.000) +
gLCB117( 1.000)
 +             gLCB118( 1.000) +              gLCB119( 1.000) +
gLCB120( 1.000)
 +             gLCB121( 1.000) +              gLCB122( 1.000) +
gLCB123( 1.000)
 +             gLCB124( 1.000)
```

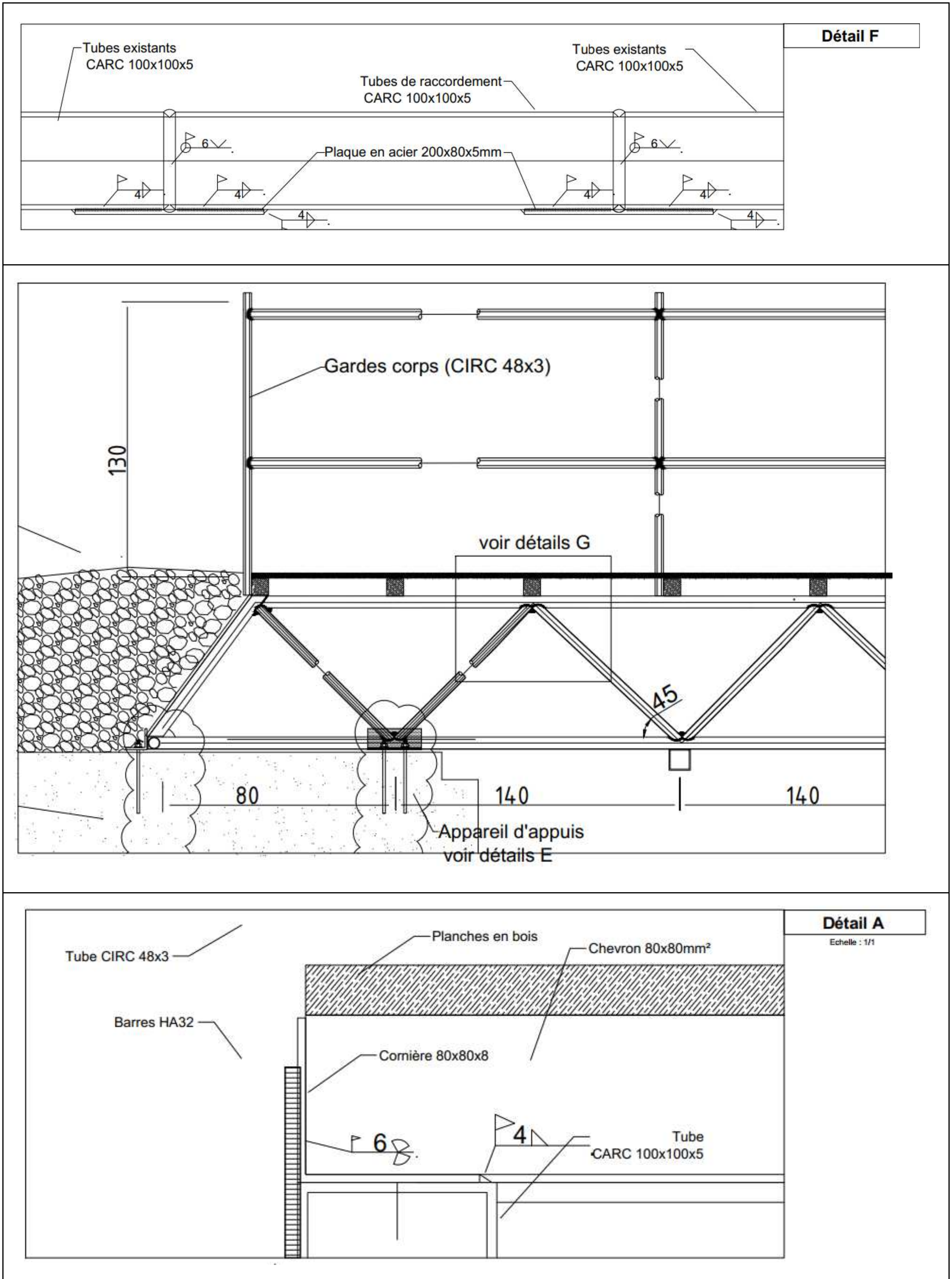
Annex 1. Part of load combination in Midas



Annex 1. Plane view of the structure



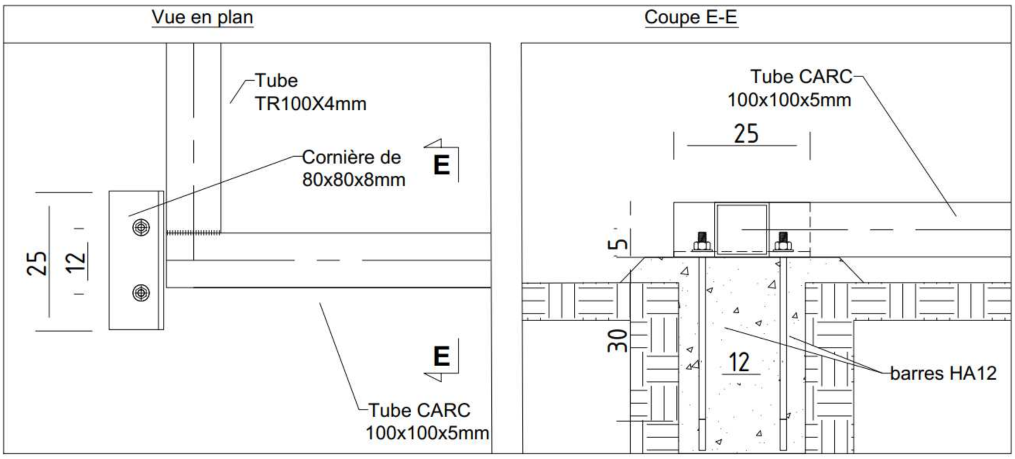
Annex 2. Plane view of bearing



Annexe. 3 Details of bridge

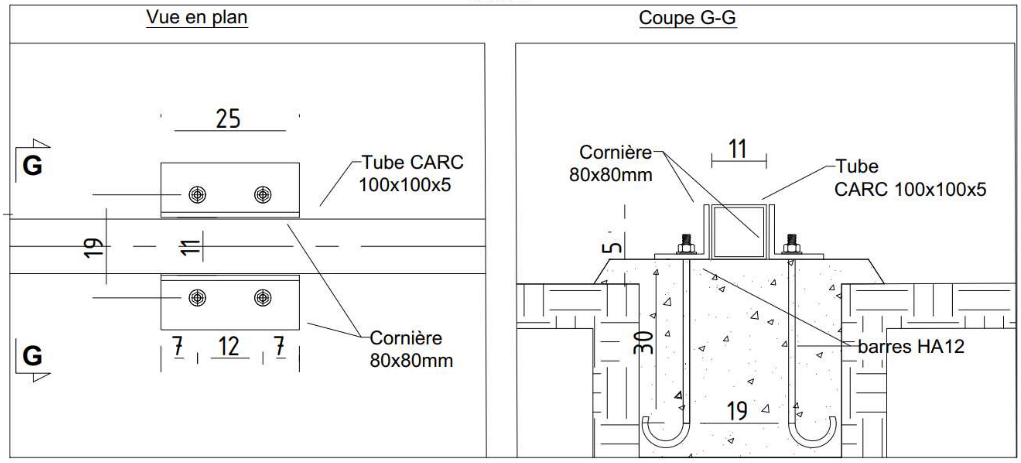
Détail B

Echelle : 1/5



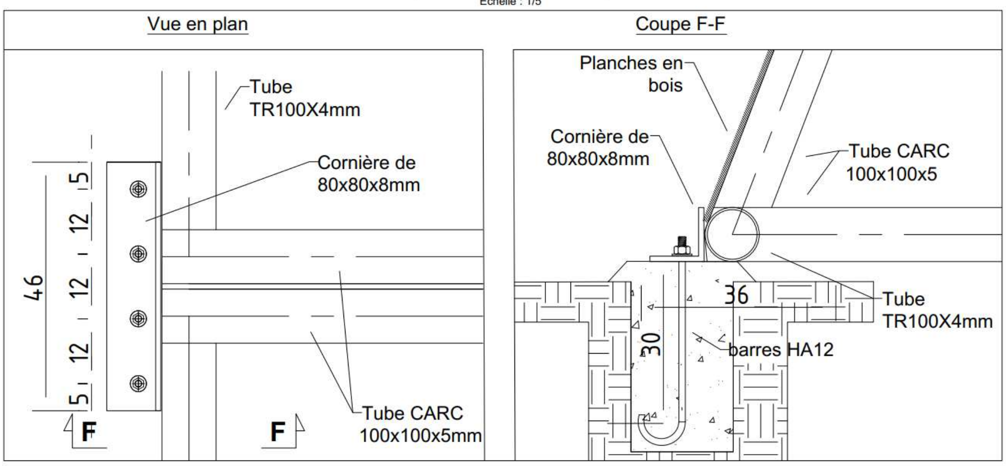
Détail C

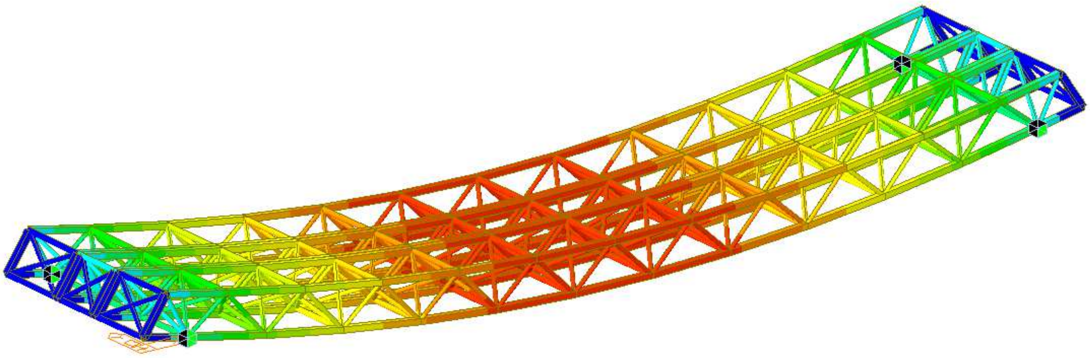
Echelle : 1/5



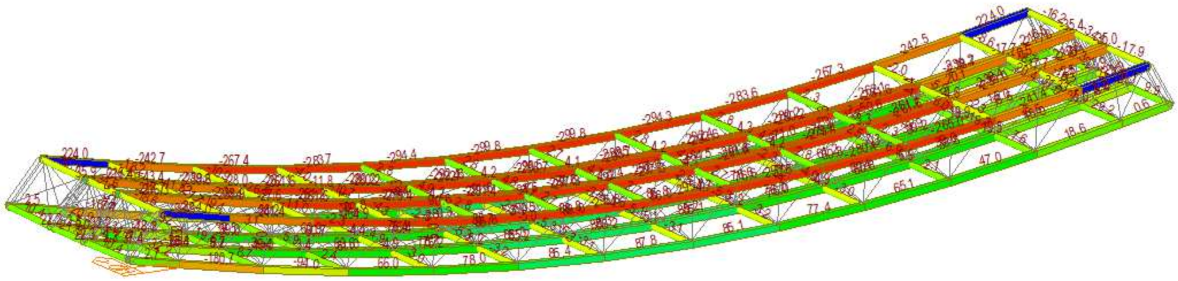
Détail D

Echelle : 1/5

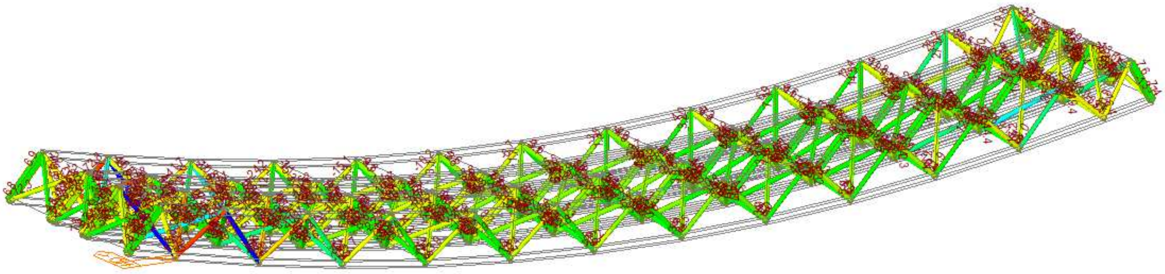




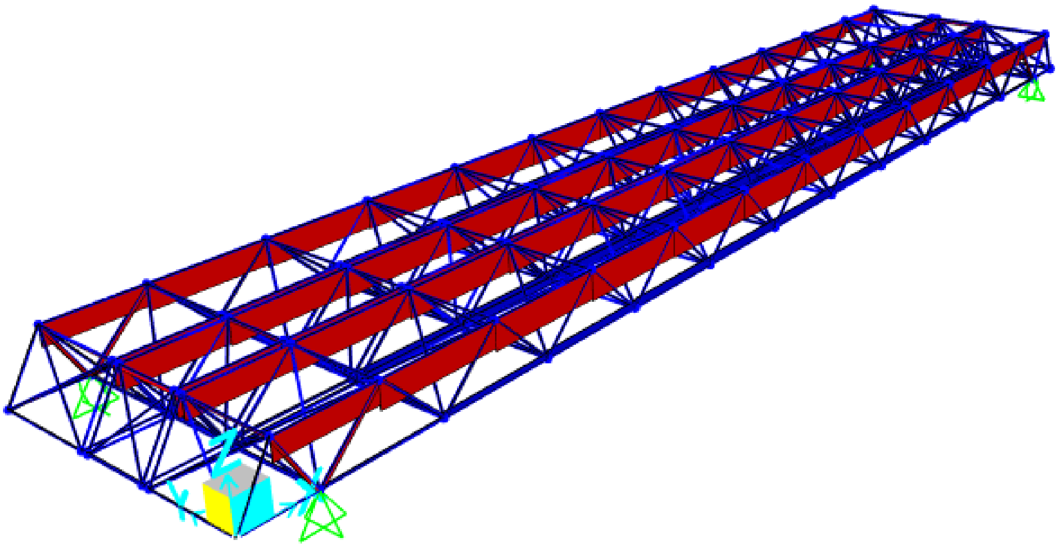
Annexe 5. Deflection of the entire bridge



Annex 6. Normal force on beam elements



Annex 7. Normal force on truss elements



Annex 8. Normal force on beam elements SAP 2000

Annex 9. Render of the bridge with ArchiCAD 22 and Lumion 8.5