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MECHANISMS AND FUNCTIONALITY OF MOBILE BRIDGES. CASE STUDY: BASCULE BRIDGE AT THE VIAREGGIO HARBOUR IN ITALY

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DEDICATION

*I dedicate this work to my late friend **Tefack Nafii** who inspired me.*

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LIST OF ABREVIATIONS AND SYMBOLS

ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AC	Alternating Current
AREMA	American Railway Engineering and Maintenance-of-way Association
BS	British Standard
CAD	Computer Aided Design
DC	Direct Current
DIN	Deutsches Institute fur Normung
EN	European Norm
FE	Finite Element
FEA	Finite Element Analysis
FRP	Fiber Reinforced Polymer
LM	Load Model
LRFD	Load Resistance Factor Design
NA	National Annex
OSD	Orthotropic Steel Deck
PLC	Programmable Logic Controller
SLS	Serviceability Limit State
TS	Tandem System
UDL	Uniformly Distributed Load
UK	United Kingdom
ULS	Ultimate Limit State

SYMBOLS

A	Area of cross section
Aa	Area of the steel section
$Aeff$	Effective area
Am	Surface area of the member per unit length
$Anet$	Net cross-sectional area
Av	Shear area of the steel profile
$cdir$	Directional factor
ce(z)	Exposure factor
$cseason$	Seasonal factor
$c(z)$	Orography factor
$c(z)$	Roughness factor
cpe	Pressure coefficient for the external pressure
cpi	Pressure coefficient for the internal pressure
E	Young modulus
ec	Edge distance of the column flange
Fc,d	Resistance of the column web in the compressed zone
fy	Yielding strength
fu	Ultimate tensile strength
Gk	Permanent loads
hw	Height of web
H	Height
Iv	Turbulence intensity
i	radius of gyration
iS	Influence width
kC	Correction factor
kT	Terrain factor
k1	Turbulence factor
lb	Length of beam

L_{cr}	Critical buckling length
$M_{b,d}$	Lateral torsional buckling resistance
$M_{c.Rd}$	Resisting moment of the beam
M_{cr}	Elastic critical moment for lateral torsional buckling
M_{Ed}	Design moment
$M_{el,d}$	Elastic resisting moment
$M_{pl,d}$	Plastic resisting moment
$N_{b,d}$	Buckling resistance
$N_{C,d}$	Resistance to axial compression
N_{Ed}	Design axial load
$N_{pl,d}$	Design plastic resistance of gross cross section
N_{SLS}	Serviceability limit state axial force solicitation
$N_{u,d}$	Design ultimate resistance of the net cross section at holes for fasteners
q_b	Basic velocity pressure
q_f,	Design fire load density
Q_k	Live loads
$q(z)$	Peak velocity pressure
r	Root radius
t	Time
t_f	Flange thickness
t_{wc}	Thickness of the column flange
t_p	Thickness of the plate
t_w	Web thickness
V	Volume of the member per unit area
V_b	Basic wind velocity
$v(z)$	Mean wind velocity

ABSTRACT

This thesis work aimed at performing a parametric analysis and a structural amelioration of a mobile bridge with an orthotropic steel deck (OSD) under constraints passing through a linear static analysis, using the computer program Midas civil. To achieve this objective, a state of art was done in order to have insights on mobile bridges, their types and their mechanisms and functionality, on how the structural analysis and design can be made. The case study used for the different analyses was a single-leaf bascule bridge with an orthotropic steel deck located in the harbour of Viareggio, in the maritime zone, Italy. This work started by creating models of the case study in both closed and opened configurations using Midas civil with adequate loads application and all the necessary load combinations, in order to get the different solicitations. These solicitations were used to statically verify the structure according to Eurocodes norms particularly EN 1993 - design of steel structures. A parametric analysis was then conducted by varying the angle of opening from 0 to 90 degrees at an interval of 15 degrees, the steel grade from S355 to S450 and operating mechanisms (from hydraulic to mechanical drive) simultaneously. Then, a structural amelioration of the bridge structure under constraints (strength, stresses and deflection) was conducted with all the verifications according to Eurocode 3 part 2 (EN 1993-2). The optimisation was made by varying the steel grade from S355 to S450. Results show that, after static linear and parametric analyses in closed and opened configurations, a reversal of solicitations from tension to compression of bending moment, shear and axial force is observed when the static scheme changes from a simply supported plate to a cantilevered or fixed plate. Equally, the maximum negative values of those solicitations are observed when the bascule bridge is opened at around 30 degrees with respect to the horizontal. After structural amelioration, verification of all the structural elements of the OSD was satisfactory with the bridge made up of grade S450 having a better structural capacity and a percentage gain in mass was assessed in order to understand if there is a possibility of cost amelioration, with a value of 17.17% gain in mass. Equally, a cost analysis with prices from the world market was performed and a gain of 562920.4 CFA francs is found when the structural amelioration is performed.

Keywords: mobile bridge, mechanisms, functionality, orthotropic steel deck, static linear analysis, parametric analysis, structural amelioration.

RESUME

Ce travail avait pour but de réaliser une étude paramétrique et une amélioration structurelle sous contraintes d'un pont mobile avec une plaque orthotrope en acier pour le tablier passant par une analyse statique linéaire, utilisant le programme informatique Midas civil. Pour atteindre cet objectif, un état de l'art a été fait afin d'avoir une vue d'ensemble sur les ponts mobiles, leurs types et leurs mécanismes et fonctionnalité, sur la façon dont l'analyse structurelle et la conception peuvent être faites. Le cas d'étude utilisé pour les différentes analyses était un pont basculant à un seul fléau avec un tablier en acier situé dans le port de Viareggio, dans la zone maritime, en Italie. Ce travail a commencé par la création de modèles du cas d'étude dans les configurations fermées et ouvertes en utilisant Midas civil avec toutes les charges appliquées et toutes les combinaisons de charges nécessaires, afin d'obtenir les différentes sollicitations. Ces sollicitations ont été utilisées pour vérifier statiquement la structure selon les normes Eurocodes particulièrement EN 1993 – calcul des structures en acier. Une analyse paramétrique a ensuite été menée en faisant varier simultanément l'angle d'ouverture de 0 à 90 degrés dans un intervalle de 15 degrés, la nuance d'acier de S355 a S450 et les mécanismes de fonctionnement. Ensuite, une amélioration structurelle de la structure du pont sous contraintes (résistance, contraintes et déflexion) a été réalisée avec toutes les vérifications selon l'eurocode 3 partie 2 (EN 1993-2). L'amélioration a été faite en faisant varier la nuance d'acier de S355 à S450. Les résultats après analyses statiques linéaires et paramétriques dans des configurations fermées et ouvertes montrent qu'une variation des sollicitations (moment de flexion, cisaillement et force axiale) quittant d'une valeur positive à une valeur négative est observée lorsque le schéma statique passe d'une plaque sur appuis simple à une plaque en porte-à-faux. De même, les valeurs négatives maximales de ces sollicitations sont observées lorsque le pont basculant est ouvert à environ 30 degrés par rapport à l'horizontale. Après l'amélioration structurelle, un pourcentage de gain en masse a été évalué afin de comprendre s'il existe une possibilité d'amélioration des coûts, avec une valeur de 17,17% de gain en masse. Également, une analyse des coûts avec les prix du marché mondial a été effectuée et un gain de 562920.4 CFA francs est constaté à l'issue de l'amélioration.

Mots clés: pont mobile, mécanismes, fonctionnalité, plaque orthotrope, analyse statique linéaire, analyse paramétrique, amélioration structurelle.

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GENERAL INTRODUCTION

Actual modern times are characterised by fast paced civilisation and development throughout the world and an increased rate of trade shares among cities and countries through various air and sea routes. With engineering evolution and consequent roads and railways progressive development, emerged the necessity for populations to perform crossings, not only to overcome geographical barriers but equally to save travelling times. Nevertheless, one principal problem faced has been the crossing of navigable waterways. As one of the key means of global transport, sea-river navigation could not be less important than the construction of the bridge that would affect its course. In view of ameliorating the transport process, it was then necessary to find a solution that would enable waterborne traffic without hindrance and at the same time their crossing. Because building bridges with necessary air draught for the passage of water vessels required a more elaborate work in terms of high inclination, mobile bridges became the most adequate and viable to this puzzle, in spite of all the challenges on their design.

From ancient times till today all throughout the world, conceiving, designing and implementing a mobile bridge has always been a line of complex and cumbersome procedures due to the need of expertise in the mechanical, electrical and civil engineering fields and due to high construction costs in the short-term and high maintenance costs in the long-term. As such the main objective of this work is to perform a parametric analysis and a structural amelioration of a mobile bridge with an orthotropic steel deck (OSD) under constraints passing through a linear static analysis, using the computer program Midas civil.

In order to accomplish this task, the fundamental theoretical background and concepts surrounding this work is described in the first chapter; notably a historical background, the mechanisms and functionality and the structural analysis of mobile bridges. The second chapter which is the methodology describes the techniques and methods used in modelling a mobile bridge and performing both the linear static and parametric analyses on Midas civil while respecting the codes prescriptions. It equally presents the algorithm used perform an efficient structural amelioration of a bridge structure. Finally, the third chapter which is the presentation and interpretation of results presents the site where the case study is found and the project. It equally presents the actions loading the mobile and the structural analysis results which are the solicitations, stresses, deflections and reactions. It ends by presenting the parametric analysis results, the structural elements verifications and the structural amelioration results assessed via Microsoft Excel software.

CHAPTER 1: MOBILE BRIDGES

Introduction

Mobile bridges, also known as movable bridges, represent an integral part of a country's transportation system and have proven to be an economical solution to the problem of how to carry a railroad line or highway across a navigable and active waterway. Thus, a mobile bridge refers to that type of bridge that changes position and configuration, vertically or horizontally, enabling both vehicular and waterborne traffic. Such bridges are unique and complex structures that integrate conventional structural components with mechanical, electrical and hydraulic systems. While there are many advantages associated with this type of bridge such as low approach spans, and low piers, there are also significant drawbacks and problems associated with their operation and performance leading to high rehabilitation and maintenance costs. This chapter is meant for an in-depth explanation of the theory underneath these concepts to facilitate understanding subsequent works. As such, the chapter starts by discussing the historical background of mobile bridges with the different tendencies throughout time. The next part talks about the types and features of mobile bridges, starting with the bascule type to the other types passing through the vertical lift and swing types. The following part discusses the mechanisms and kinematics of mobile bridges which greatly depend on the different mechanical components. The next section discusses the functionality of mobile bridges that depends on the machinery types, hydraulic and electrical systems. Then comes the part on the structural analysis and design of mobile bridges in which the design standards, constructive materials, mechanical models, loads, design criteria, bridge balance and counterweights are detailed. Finally, the part concerning control, inspection and maintenance of mobile bridges is discussed.

1.1. Historical background

Mobile bridges have been present since ancient times. Their origin and development through time are detailed starting from the early times to the last tendencies passing through modern times.

1.1.1. Early times

The oldest known movable bridge was built in the 2nd millennium BC in ancient Egypt and according to research the first movable bridges to be constructed have been in Ancient Egypt, circa 1855 BC, in the 12th Egyptian Dynasty. According to Edward H. Knight (1876), the primary allusion to movable bridges was made in Egyptian monuments, like palaces and temples, wherein illustrative drawings of those bridges around castles and fortified cities were made. During the reign of Ramsess II in 1355 BC, the use of so-called floating bridges at the Nile River became already mentioned (Hovey, 1926).

Otis Ellis Hovey stated that most likely around the third century and into the middle of the sixth century, the Chinese have used movable bridges in their channels, as they have developed an uncommon ability in engineering very early. Less than two centuries later, that is in 621 B.C., the earliest recorded Roman movable bridge was built in the Roman empire by Ancus Martius. It was a pile bridge and was later rebuilt as a wooden pile bridge, and some writers state that it had a draw span (Hovey, 1926). Pontoon bridges were probably and possibly the first and most common mobile bridges. These were frequently used in military missions and were built by piles of wood, with one or more small vessels, tied up together purposefully so that they could be moved or swung to allow a waterborne passage. An example of this type is the Darius boat bridge over the Bosphorus Thracian River in Turkey, which connects Europe to Asia, and the Xerxes Bridge over the Hellespont passage, about 480 B.C.; now called Dardanelles, in Turkey described in Figure 1.1. (Hovey, 1926).

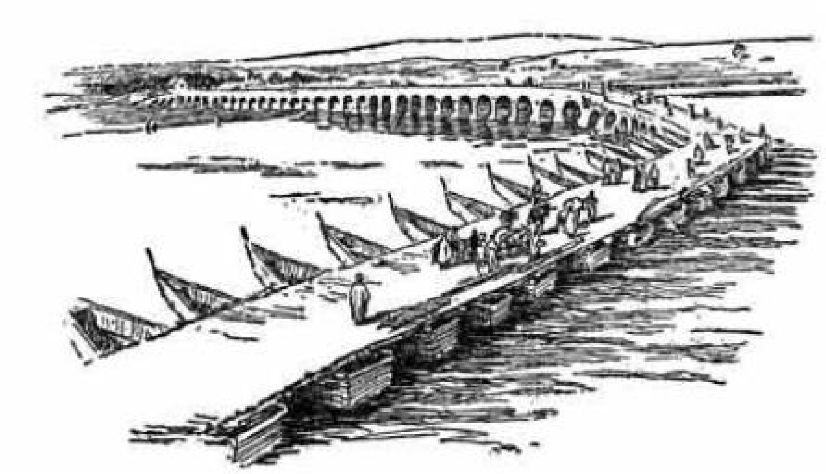


Figure 1.1. Sketch of Xerxes bridge (Anostructures, 2010)

The Romans developed the design and construction of fixed arch bridges very fast, however, it is difficult to trace the use of movable systems for several centuries after the start of the Christian era. Later, all through the Middle Ages, the drawbridge, became possibly the most common bridge, used as a passage in the closed configuration, however while in open position, as a protecting fortress barrier at medieval castles (Hovey, 1926.). The typical installation would be a lift movable bridge just outside the gate containing a platform made up of wood with a hinged articulation at edge of the gate. Permitting the platform to rotate about an axis on a trunnion, defining this type of bridge as the predecessor of bascule bridges. The raising of the platform can be seen in Figure 1.2., which was made possible via chains or ropes attached to a windlass over the entrance. Such bridges were sometimes counterweighted to ease this operation (Koglin, 2003).

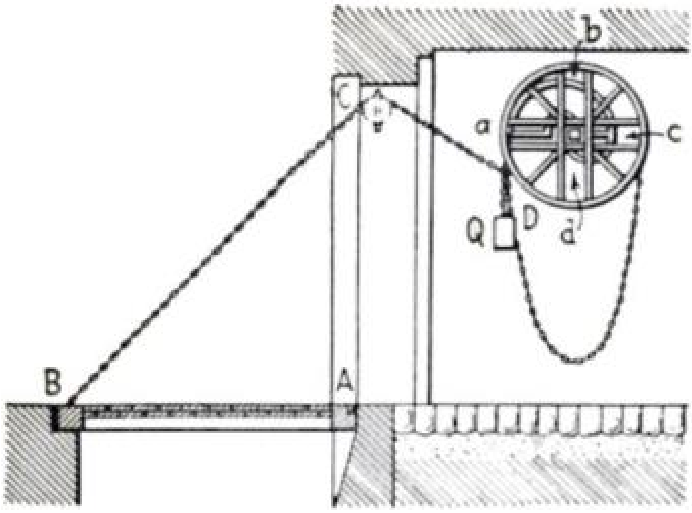


Figure 1.2. A typical drawbridge (Hovey, 1926)

The period from the fourteenth century to the middle of the sixteenth century represented the time of the Renaissance. The science of movable bridges was equally unforgotten and highly expressed during this era, as seen by the presence of several sketches on this subject drafted by one of the most amazing minds of that time, Leonardo Da Vinci. Many of these drawings have been safeguarded and were published in the 'Codice Atlantico' from which two have been selected. Figure 1.3. (left) from a sketch made 1500, represents an unequal-armed, or bob-tailed, center-bearing swing bridge, operated by hand winches, by means of ropes passed through snatch-blocks. While Figure 1.3.(right) represents a very well-conceived pontoon swing span, which enters a niche in the canal wall when open (Hovey, 1926).

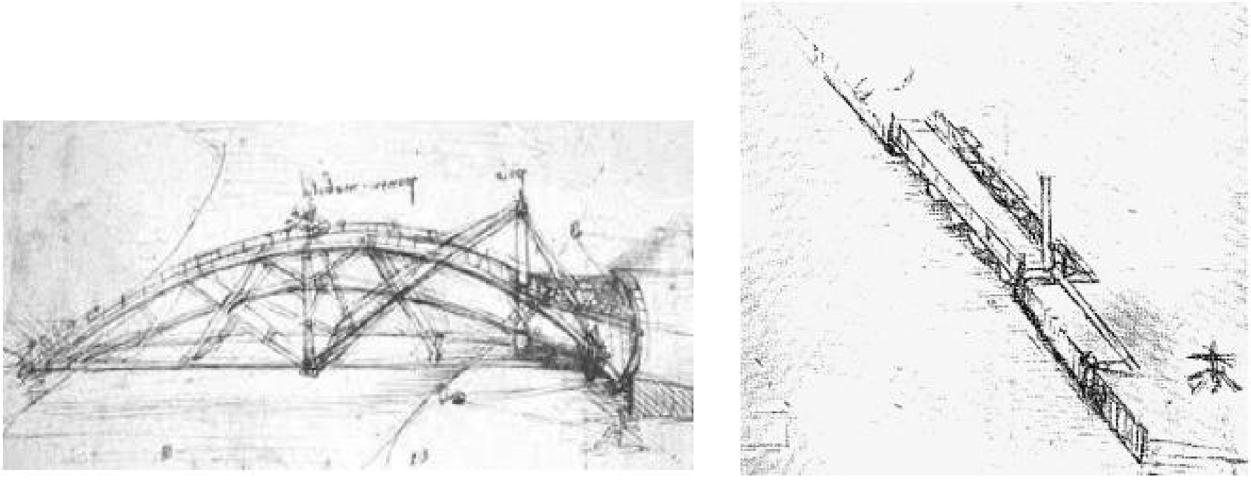


Figure 1.3. Sketches of Leonardo Da Vinci drawings (Hovey, 1926).

After the invention of the steam engine by James Watt in 1769 and the institution of the steam locomotive by George Stephenson, in 1829, the world of engineering experienced a particular boost due to the production of energy. Mobile bridges were not left aside and their conception, design and construction experienced a huge progress. In spite of the fact that the innovations referred previously, led to a full-size boom of movable bridges construction, the accompanied growth of the railroad industry, in the mid-nineteenth century and the lorry industry after world war I, led to a new beginning. The rate of increase of industrial growth has made it vital to discover a way to pass through rivers and active channels used by a variation of vessels of various sizes in order that the road and rail traffic could travel without permanently disrupting the flow of any kind of traffic. This became a specific hassle in increasing towns with vital navigable waterways, so movable bridges were the answer (Anostructures, 2010).

1.1.2. Modern times

Numerous medieval and early modern types of draw bridges (bascule type) were described by M. Gauthey. The oldest is shown in Figure 1.4., having a lever frame used for the raising operation pivoting at a more or less centered support, attached to the bridge end by chains. This type is usually counterweighted at the rear end of the frame with equilibrium achieved at the trunnion(support). Holland is a good example where such bridges have been built many years, with some of them still in use nowadays in Amsterdam (Hovey, 1926).

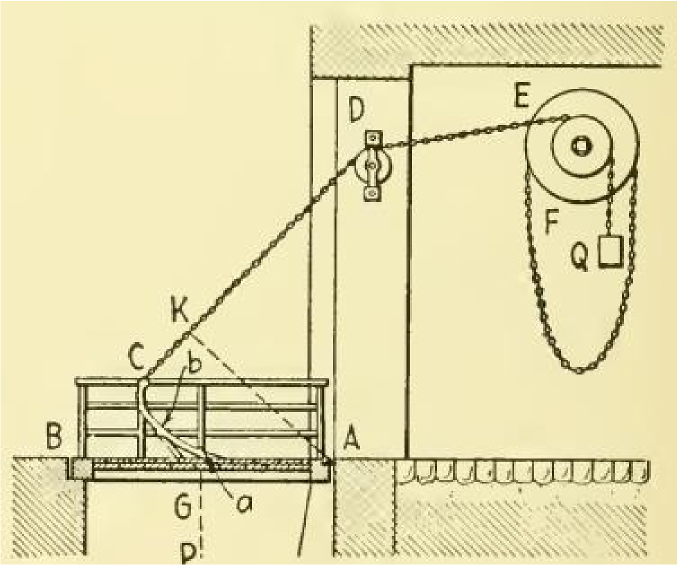


Figure 1.4. Early modern type bascule bridge (Hovey, 1926)

The construction of the Tower Bridge in London and Van Buren in Chicago back in the 1894 represents the oldest construction of a modern bascule bridge. A wide variety of bascule bridge designs had been improved and patented over the subsequent decades. Throughout this period, two types of bascules bridges prevailed, the trunnion bascule and the rolling lift bascule. The trunnion bascule type, derives from the medieval drawbridges and was developed by European military engineers in the early 18th century. JAL Waddell states in his work “Bridge Engineering” from 1926 that the bridge Michigan Avenue, in Buffalo, New York was the first major bascule bridge to be constructed. This type later evolved in the 19th and 20th centuries to two variations, the simple trunnion or "Chicago" and the multiple trunnion or Strauss (Hovey, 1926).

The first variation, which is the simple trunnion type, over which the Chicago Bascule Bridge Company had the official legal right was simply an amelioration of the counterweight’s mechanism. The second variation which is the multiple trunnions type was much more complex with trunnions observing a rectangular shape when the span was in a closed position and a parallelogram while in open position (Hovey, 1926). Another bridge type that evolved by adding an additional pulling back movement while conserving the natural upward-swinging motion is the rolling lift type. This was made possible by linking the span to a beam segment which inclines the span upward as it is pulled back in its pathway, simultaneously (Anostructures, 2010).

Scherzer and Rall had legal official rights on two variants of the rolling lift type at the end of the 19th century. Developed in 1893 by William Scherzer, American engineer, held twelve patents for various variations of this type of bridge from 1893 to 1921, which became the most well-known of all types until 1916. The Van Buren Street Bridge, located in Chicago, is the first model of this bridge, followed by many others, as this design was used in the replacement of many mobile bridges in England docks, such as Liverpool, Birkenhead and London (Anostructures, 2010).

The other variant to be designed was the Rall system, created and legalised by Theodor Rall in 1901. The Broadway bridge, in Portland, United States of America which is equally the largest span ever built of this type of system is one of the few bridges still existing of this form and perhaps most well-known (Anostructures, 2010).

Knowledge about the rotation of swing bridges on pivots with center bearings supporting devices was already noticeable at the beginning of the 17th century. However, at the beginning of the 19^{th} century, pivots with rim bearing supporting devices which were capable of supporting immense weights of large swing spans were developed by British engineers. Simplicity, reliability and economy of the center bearing type prevailed over the more complex design of the rim bearing type and was replaced by the centre bearing type by the third decade of the 20^{th} century (Anostructures, 2010).

As Otis Hovey cited in 1926, "when there are no restricting circumstances, a swing bridge is the simplest, best, and most economical type in first cost and maintenance". In the 20th century, George Hool, professor of Structural Engineering at the Wisconsin University, heartily supported the benefits of the bascule bridge in both its 1924 and 1943 editions of Movable Bridges and Long-Span Bridges. This preference was principally due to the fast opening of the bridge, causing the ship to pass through the channel more quickly.(Hool & Kinne, 1943) However, Hovey admitted that the bascule bridge was far advantageous when many parallel bridges had to be upright and when the waterways are too narrow (Hovey, 1926). Vertical lift bridges however experienced a little progress in their building until 1908. From this date on, and approximately in the next two decades, bridge engineers had a particular interest in this type of mobile bridges who built about seventy movable vertical lift bridges in America alone (Anostructures, 2010).

1.1.3. Last tendencies

With changes in available technology, new materials, increases in loads, and greater concern for safety, new types of movable bridges have occasionally been developed and older types have ceased to be used. During the last two decades, the mobile bridges technology has experienced changes in a very fast way, as a result of the introduction of hydraulic machinery and automatic controllers. As such, bridge operations became easier and safer using small machinery. Despite the great improvement of the constructive technical part, the architecture and aesthetics of some modern bridges dissatisfy (Anostructures, 2010).

Despite the fact that some of these bridges are not aesthetically pleasant, some projects have differentiated themselves by their fabulousness recently, reviving the hope with regards to the design of mobile bridges as presented in Figure 1.5. This has been possible principally due to the use of ultra-modern softwares available and revolutionary methods like the finite element method (Anostructures, 2010).

In recent times, kinematics has proven to play an extremely important role in the analysis and design of mobile bridges, which wasn't the case in the past. In present days, numerous simulation models for complicated kinematics mechanisms exist, easing the engineer's task and enabling a much better control of the structural system with the type of mechanical and elevation schemes (Anostructures, 2010).

Figure 1.5. Gateshead Millennium Bridge (Google Search, 2022)

1.1.4. Why are mobile bridges desirable

At many locations, a movable bridge is the only practical means of crossing a waterway. Movable bridges are appropriate for locations where a limited amount of land is available, particularly for urban locations where tall fixed spans would require graded approaches to provide clearance over a navigable waterway. The alternatives to mobile bridges that allow traffic to cross a waterway include ferries, aerial transporters, tunnels, and high-level bridges. Providing no crossing at all, so that a long detour is required, or closing the waterway to navigation and placing a simple, restrictive bridge across it are options that may be selected, sometimes unconsciously. Each of these alternatives has some drawbacks and should be studied before a decision on a crossing is made.

A movable bridge may be the least costly alternative to provide a means of crossing a navigable waterway. In a particular situation, such as a short span across a canal in an urban area, a movable bridge may be the only workable solution. It may be the most convenient option for the public, for instance, in an urban situation where a high level bridge would need very high piers and long approaches and require heavy vehicles to travel up steep grades. In many circumstances where steep grades are to be avoided, such as for railroads crossing navigable waterways between relatively low-lying embankments, a movable bridge can well be the only economically practical means of crossing the stream (Koglin, 2003).

1.2. Types and features of mobile bridges

From ancient times till nowadays, countries have developed many different forms of mobile bridges to carry a roadway and/or railway across a navigable waterway. And as said in the previous part, changes in technology, new materials, increases in loads, and greater concern for safety, new types of mobile bridges have in occasion been developed and old forms have stopped to be used. A proliferation of bridge types and variations of types produced was greatly observed by the late nineteenth and early twentieth centuries. Today there are three main types of movable bridge in common use in function of the motions of the movable spans which can be translation along an axis, rotation about an axis or the combination of translation or rotation. These are the bascule bridge, the vertical lift and the swing bridge. Uncommon types such as jack-knife, reticulated, retracting, transporting, folding, gyratory, and floating will be equally discussed in this section. As such, mobile bridges that are counterbalanced and open by pivoting about a horizontal axis should be called bascule bridges; mobile bridges that open by lifting, without rotating or translating horizontally, should be called vertical lift bridges and mobile bridges that open by pivoting about a vertical axis should be called swing bridges.

1.2.1. Bascule bridges

The term “bascule” originates from the French language and translates as ‘a balance’ (Waddell, 1916). Bascule bridges are related to medieval drawbridges that protected castles. The main principle upon which bascule bridges rely is that as long as one end of the span is raised, the other one has to be lowered. The term bascule is generally applied to any type that moves through a fixed or movable axis, and those that move through a circular segment of beam. This takes place by pivoting on a horizontal axis, at a certain angle with respect to the horizontal. The pivot should be close to its center of gravity so that the weight on one side can be balanced by the weight of the other side. The balance is usually not exact. If the bias is towards keeping the bridge closed, it is referred to as span heavy. If the bias is towards keeping the bridge open, or causing it to open, it is called counterweight heavy as shown in Figure 1.6. and Figure 1.7. (Koglin, 2003).

On the superstructure, the deck of a bascule bridge that moves is referred to as a “leaf” and these types of bridges may be made up of one or two leaves thus defining a single span, or two symmetrical ones, which when in the closed position, engage one another, ensuring that the two work together as one, causing their end deflection to be the same, when loaded (Hovey, 1926). bridges must be capable, large and resistant enough to overcome the friction of all the trunnions and joints and support the mass of the moving leaf, counterweights and the rest of the structural.

Figure 1.6. Bascule bridge (Florida Department of Transportation, 2018)

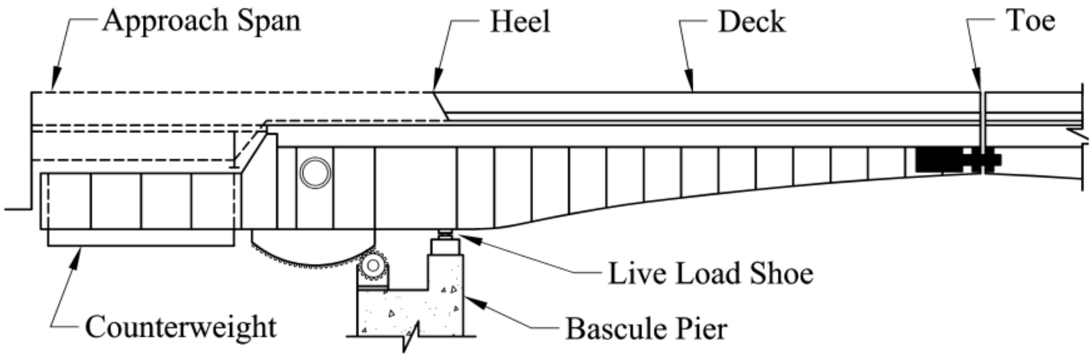


Figure 1.7. Bascule bridge leaf (Koglin, 2003)

Bascule bridges need not to be completely opened to allow the passage of small boats, this implies that the time for the operation is proportional to the sort and degree of opening and also to the boat type passing through the channel. One of the most important advantages of bascule bridges is the no limitation of air draught when in open position.

Bascule bridges are in general equipped with a counterweight, as such, the center of gravity of the leaf is shifted closer towards the pivot. This lowers the moment created by the leaf needed to be overcome by the operating system. Counterweights are made up of a steel case filled by with material depending on its weight, volume and cost. Concrete is a less expensive filling material has a low density, to increase its density, concrete is mixed with steel waste. In order to prevent and avoid uplift in closed position, a small support force is needed at the toe of the leaf that is the deck is a little bit overweighted with respect to the counterweight. This is made possible by the use of adjustable weights, with maximum allowable weight in function of the design code used; twenty-three kilograms (23 kg) for the Dutch code. These adjustable weights are equally used to balance the bridge during maintenance operations. Although counterweights reduce the required torque by the machinery by changing the center of gravity of the leaf, a larger torque will be required to start opening the bridge but also in order to stop the rotation of the bridge due to an increase in mass moment of inertia as shown by the equation below (Zantvliet,2015).

$$\boldsymbol{m}\ddot{u}_{\boldsymbol{z}}(\boldsymbol{t}) = \sum \boldsymbol{F}_{\boldsymbol{z}}(\boldsymbol{t}) \qquad (1.1)$$

1.2.1.1. Types of bascule bridges

Bascules bridges can be split into three main categories which are; the fixed trunnion type also known as the Chicago type, the rolling type and the Strauss type. Each of these types are patented by different authors and may be single or double leaf structure supporting roadway or railroad traffic. Single leaf structures are preferred for railroad traffic since it can be made to act as a simple span when closed and thus guarantees a greater rigidity.

a. Fixed trunnion bascule bridge

The fixed trunnion bascule bridge also known as the simple trunnion type was an evolution of Selby bridge (UK design) and is attributed to Mr. John Ericson and Mr. Edward Wilmann, who were Chicago city bridge engineers in early 1900s. The bridge superstructure is generally made up of a deck(orthotropic) and bascule girders having variable sections assuming a curvilinear shape. The main feature of this type of bridge is the use of a system of heavy counterweights mounted on a frame at the end of the span. This allows the reduction of the size of the mechanical power system components required to operate the bridge.

The simple trunnion is mainly characterised by the part of the leaf which extends over water and is longer than the rear part mounted on a frame. As the opening mechanism of the span is initiated, the counterweights and leaf weight are carried by the trunnions and the ends of the trunnions are supported in sliding or antifriction bearings which are attached to the piers as shown in Figure 1.8. (Anostructures, 2010).

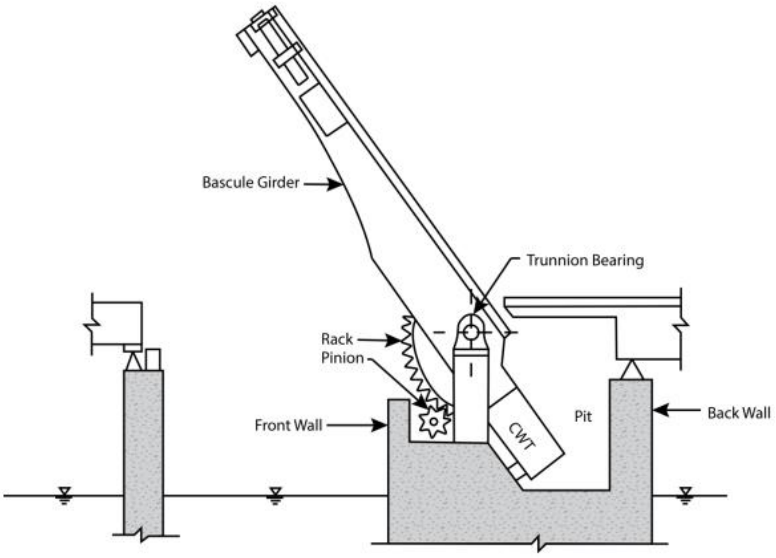


Figure 1.8. Single leaf simple trunnion bascule bridge (Florida Department of Transportation, 2018)

Also note that the tail or back end of the leaf reacts against the flanking span to stop the span and to resist uplift when there is traffic (live load) on the span. A lock bar mechanism (equally known as center lock) between the two leaves that transfers live load shear only allowing rotation, expansion and contraction to take place between the leaves as the live load moves from one leaf to the other as shown in Figure 1.9.

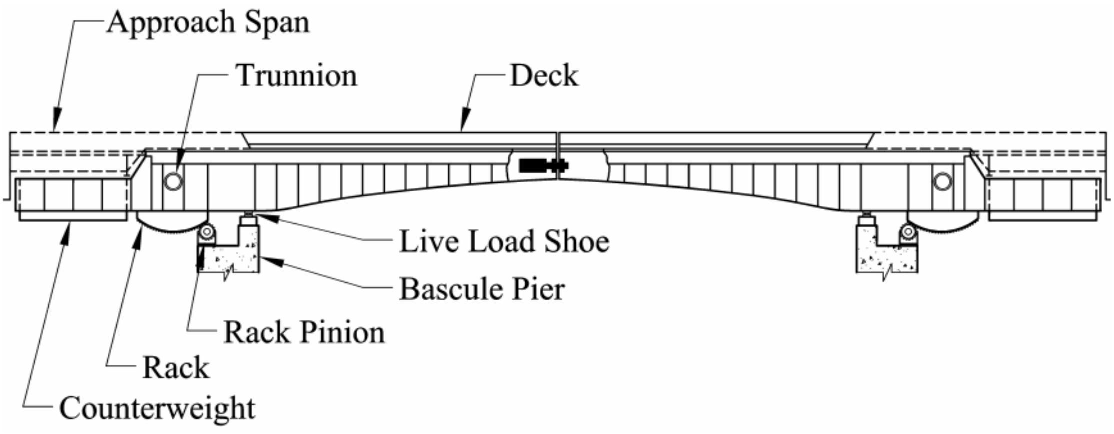


Figure 1.9. Double leaf simple trunnion bascule bridge (Koglin, 2003)

As the bridge mechanical operation begins, power is transmitted to pinions that rolls on curved racks in one direction to open the span and on the other to close it. The fact that the counterweight is placed outside of the pier so that it is exposed is advantageous to the pier since it minimises the width of the pier. Elastic bumpers help to absorb the shock. The Simple trunnion bascule is strong and simple in operation; hence it is one of the most used types of all. Nevertheless, it has a particular disadvantage regarding the use of deep pits when a medium or long span bridge close to water level is needed thus requiring large bascule piers. Immersion can accelerate deterioration of the steel and concrete components of the counterweight, particularly if the water is saline (Koglin, 2003).

b. Multiple trunnion bascule bridge

i. Rolling bascule type

In 1893, William Scherzer received a US patent for a rolling bascule bridge which became known as a rolling lift bridge, a term that is still used. However, another variation of the rolling lift type developed by Theodore Rall, is the Rall bascule bridge. Both sub-types will be developed subsequently.

(1) Scherzer Bascule Bridge

As said previously, rolling bascule bridges are generally referred as 'Scherzer Bascule bridges' due to their inventor and are characterised by a cylindrical curved part fixed to the bottom of the each of the bascule girders that rolls upon tracks (usually in the form of a heavy girder) when the bascule leaf rotates open or closed as shown by Figure 1.10. These curved parts are usually designed as "segmental girders". Sliding between the curved part treads and the running tracks is prevented by a teeth meshing that engages one another. Considering their large size, many Scherzer rolling bridges possess counterweights made of cast iron or another dense material to reduce the size and the cost of the structure. This leads to a reduction of wind resistance because it is possible to use a smaller diameter of tread and consequently smaller segmental girders (Koglin, 2003).

This type is particularly advantageous over other forms of movable bridges in making the navigation channel free for boats more quickly. Since it translates away from the channel as it rotates open, the angle necessary to provide the same clearance as other bridges is much smaller (Koglin, 2003).

Three common types of Scherzer bascules exist which are the deck double-leaf, the half through single –leaf and the through single-leaf.

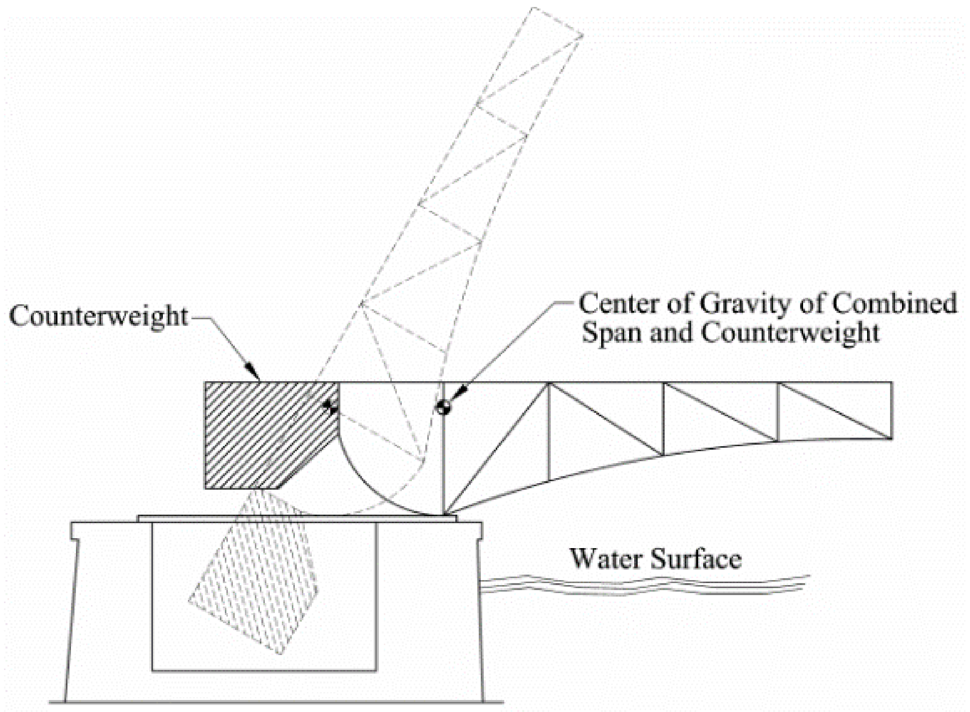


Figure 1.10. Scherzer rolling lift bascule bridge (Koglin, 2003)

(2) Rall Bascule Bridge

As stated above, a variation of the rolling lift type bascule bridge was developed by Theodore Rall, which combines rolling with longitudinal motion. The essential feature is a trunnion that is set inside a roller that moves along a track. By means of a linkage connecting the bascule girder to the pier, the movable span moves backward as it opens, in a controlled fashion, although not in the same manner and usually not quite as far as the Scherzer bascule. It can be seen in Figure 1.11. of the railway bridge over the Illinois River, at Peoria, Illinois.(Koglin, 2003)

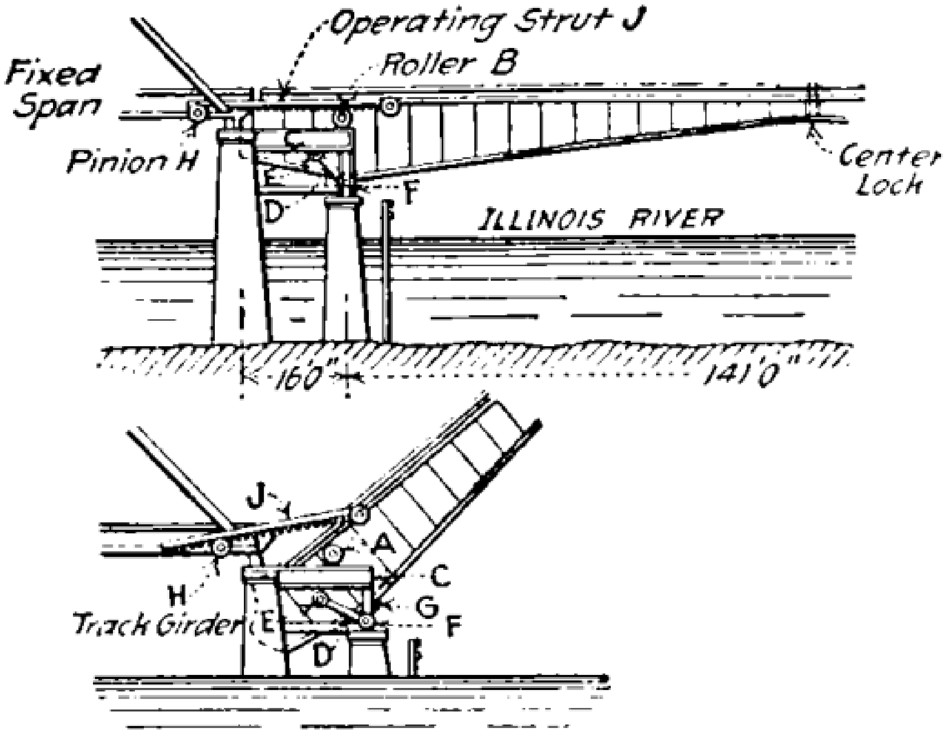


Figure 1.11. Rall bascule bridge, Illinois (Anostructures, 2010)

As mentioned by Otis E. Hovey in his book, “Movable Bridges-Volume 1”, one of the many features of the Rall type is that, when closed, it is possible to remove and replace the pivoted rollers A, as they are released of all loads. The retreating motion of the leaf allows a minimum span length to achieve a clear waterway, however the shifting of the centre of gravity by the rollers disturbs the foundation pressures. Also, the weight contact between the rollers and the tracks and the friction between them has to be taken into account, so it is necessary to perform the finest design.

j. Strauss bascule bridge

In the early twentieth century, Joseph Strauss developed economical bascule bridges at low elevations above the water to compete with the Scherzer rolling lift bascule. As such he was particularly involved and active in the development of the articulated counterweight type of bascule bridge and this bridge type is often called the "Strauss bascule" sometimes known as "multiple trunnions bascule". There have been more bascule bridges built from the Strauss designs than any other single type of bascule.

(1) Heel trunnion bascule bridge

The heel trunnion has a simple bridge span that is hinged at one end, on a horizontal axis at right angles to the roadway, and supported at the other end by an end lock. This form has a feature that distinguishes it from the other forms that is the presence of an overhead gyratory counterweight. The Figure 1.12. shows the overall operation of the Strauss heel-trunnion with the points D-E-B1-B2(trunnions) forming a parallelogram.

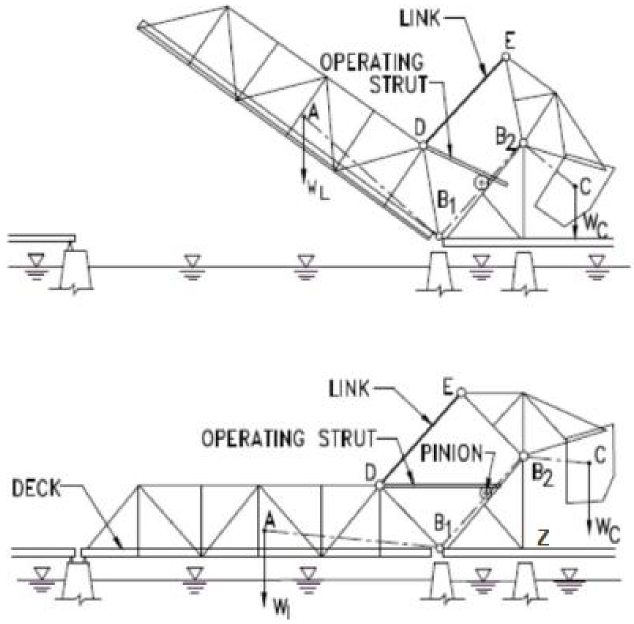


Figure 1.12. Strauss Heel-Trunnion (Berger et al., 2015a)

The presence of an operating strut that is articulated to the trusses at D ensures the motion of the leaf and extends with a rack engaging to a main pinion. When the bridge is in the closed position the strut which is heavy, upholds the span, but when it starts to rise, this act as cantilever, assisting the counterweight and keeps the span in an open position. During this movement, the trunnions that form a parallelogram fold up and the upper arm lowers, causing the counterweight to lower simultaneously. These points forming the parallelogram D-E-B1-B2, experience heavy stresses during this process. As shown by the Figure 1.11., this form has two piers that support the weight of the span and its counterweights (Berger et al., 2015a).

(2) Overhead Counterweight

The overhead counterweight Strauss type also known as the over deck counterweight can perhaps be described as an articulated counterweight type. This form has the distinguishing feature of having the counterweight placed above the road level, thus advantageous for locations that have the water level close to the road level as seen in Figure 1.13. This form is equally used when the pier cost has to be minimised since the weight of the span and its counterweights rest on one pier. Similar to the heel trunnion, some trunnions in this bridge form a parallelogram, which in this case, trunnions B-C-D-E. Nevertheless, the main inconvenience is the quantity of inter-moving parts needed and the hinged and swinging counterweights (Berger et al., 2015a).

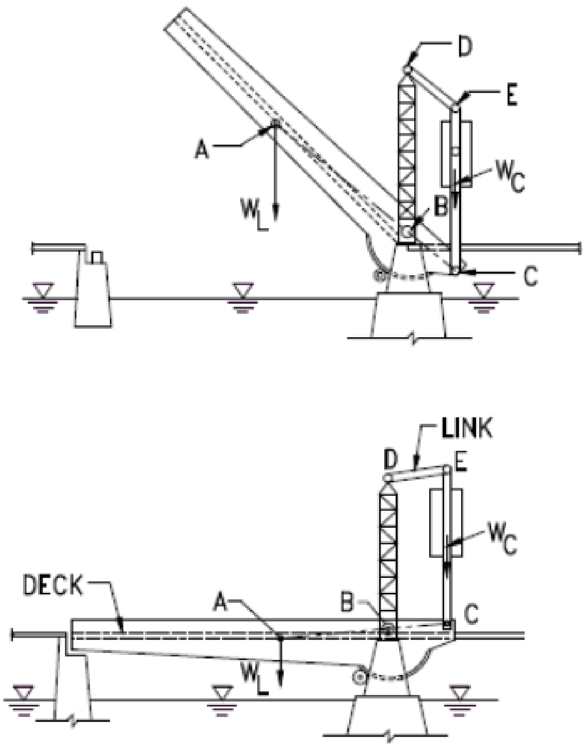


Figure 1.13. Strauss overhead counterweight type (Berger et al., 2015a)

(3) Underneath Counterweight

The principle of the underneath counterweight Strauss type (which is equally an articulated counterweight type) is the same as the one described above for the overhead counterweight, but the difference arises with the position of the counterweight and link which is located underneath the road level. This type is generally used when it exists enough clearance between high water level and road level. The operation of the underneath counterweight Strauss type is shown in Figure1.14. (Berger et al., 2015a).

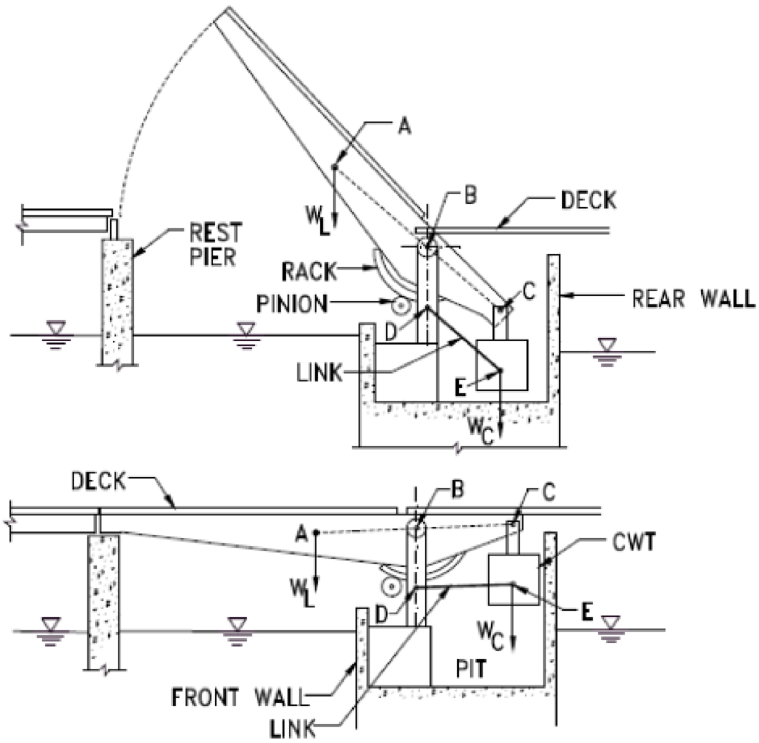


Figure 1.14. Strauss underneath counterweight type (Berger et al., 2015a)

k. Other types of bascule bridges

Several decades of early development led to more complicated versions of bascule bridges to avoid shortcomings, addressing particularly the need of low-level simple trunnion bascules for watertight pits, thus other types of bascule bridges were developed. Some were successfully introduced but others were easily put aside in favour of the most common types described above that is, the Belidor, the balance-beam, the roller bearing, the brown, the page, the semi-lift, the Dutch, the Non-counterweighted bascule.

1.2.2. Vertical lift bridges

Vertical lift span bridges consist of spans whose decks raise up vertically while in a horizontal position, parallel to the water line with the help of two or four towers on each side, providing sufficient clearance under the bottom of the span for vessels passing through the navigation channel. The towers are made up of rotating counterweight sheaves at their tops, with ropes on these sheaves that are connected at one end to the lift span and the other end to the counterweights. As the cables move upward through mechanical machines, the counterweights move subsequently downwards. The counterweights are placed internally or externally, ensuring balance of the system and minimizing the amount of power required for the lifting process. These bridges can have longer spans thus more economical in construction. Depending on the type of vessels, the span is lifted partially or totally thereby reducing the time for crossing the channel. Despite all advantages that this bridge entails, the main disadvantage is the restriction in height, limiting the air draught (Koglin, 2003).

Vertical-lift bridges can be made with different structural systems such as steel plate girder, steel truss, steel and concrete tower. The towers can be individual towers or framed as seen in Figure 1.15 (With, 2009).

Figure 1.15. Vertical lift bridge (Florida Department of Transportation, 2018)

1.2.2.1. Types of vertical lift bridges

Vertical lift bridges are normally labelled by the arrangement of the drive machinery. The drive machinery can be either mechanical or hydraulic with mechanical being the normal choice. Thus, four types of vertical lift bridges can be individuated that is, the span drive, the tower drive, the connected tower drive and the pit drive.

a. Span drive vertical lift bridge

Wire rope span drive vertical lift bridges have been built with the primary drive at mid-span transmitting power through long line shafts to the secondary speed reductions and hoist drums located at the ends of the span which helps in counterweighting the bridges. Thus, a span drive places all of the drive machinery in the center of the lift span and through drive shafts, operates a winch and hauling rope system to raise and lower the span as shown in Figure 1.16. (Chen & Duan, 2014).

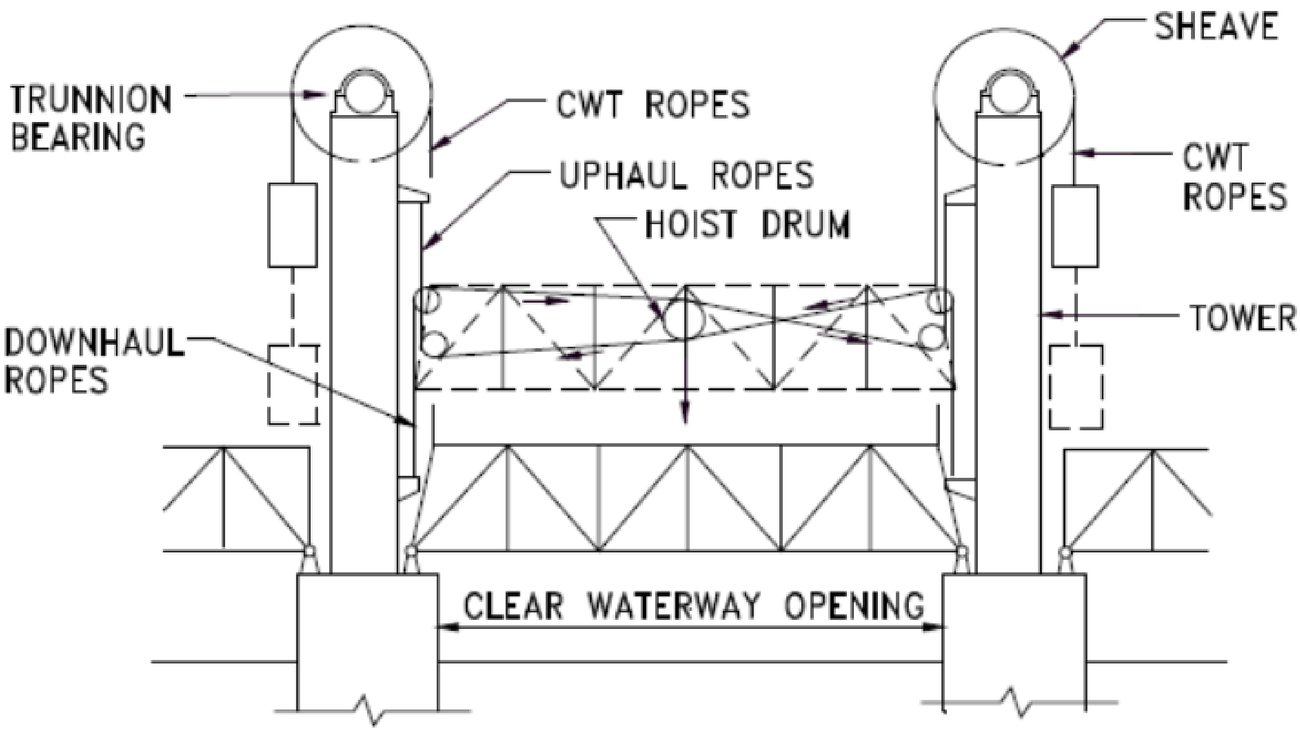


Figure 1.16. Span drive vertical lift bridge (Berger et al., 2015b)

b. Tower drive vertical lift bridge

In the tower drive vertical lift bridge shown in Figure 1.17., there is span drive machinery in each tower that rotates the counterweight sheaves. The forces necessary to raise the span are transmitted to the counterweight ropes by friction. Both ends of the lift span should raise and lower at the same rate so that the lift span remains horizontal and does not wedge itself between the towers during motion. There are various electrical/electronic means of controlling the drives in the two towers so that skew is kept within permissible limits (Ryall et al., 2003).

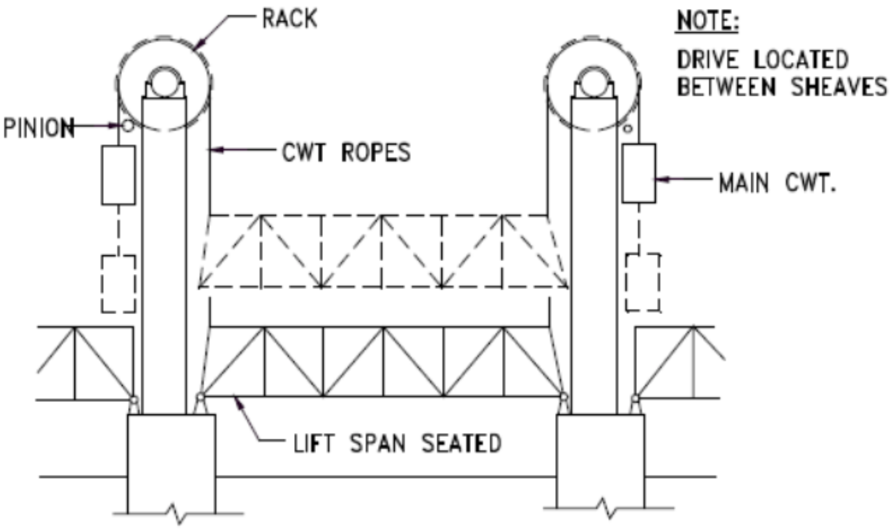


Figure 1.17. Tower drive vertical lift bridge (Berger et al., 2015b)

c. Connected tower drive vertical lift bridge

This type of bridge is most suitable for small spans, so there is no need for an auxiliary counterweights system. The machinery is mounted at the top of the span, on the structure connecting the towers, as can be seen in the Figure 1.18., but its positioning may vary depending on bridges. The mechanism works by means of a force received by the drive, which makes the pinions to rotate engaging the racks attached to the sheaves, causing them to rotate as well. These then transmit by friction to the ropes of the counterweights, thereby raising or lowering the span (Healy & Tilley, 2015).

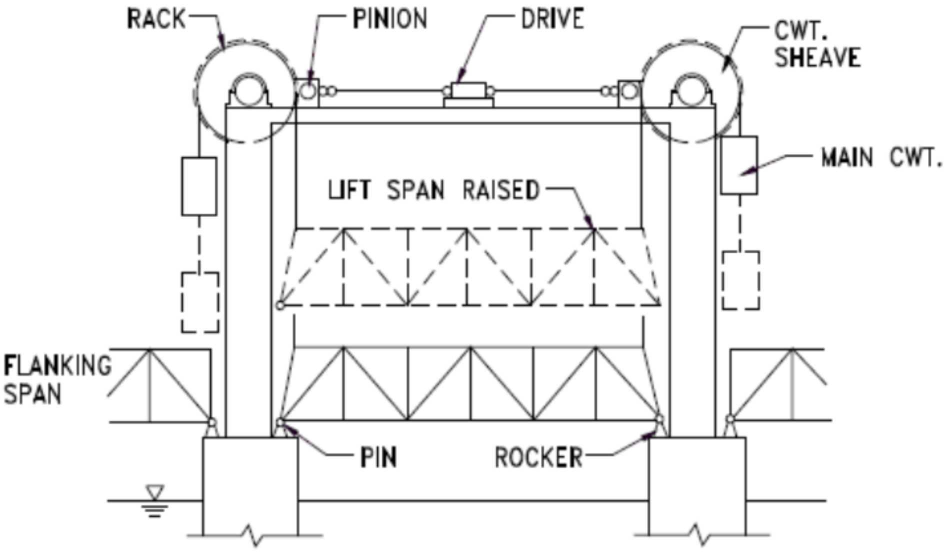


Figure 1.18. Connected tower drive vertical lift bridge (Berger et al., 2015b)

d. Pit drive vertical lift bridge

Also known as table lift bridges, they are characterized by the fact that the lifting mechanism is not visible when the bridge is in low position, it is used when there are aesthetic restrictions and for low lifting heights as seen in Figure 1.19. The tower supports are not required and the ascent is driven by hydraulic cylinders installed in wells, located inside the pillars. This mechanism is achieved through lifting posts that is fixed legs inside the pillars that extend and collect. These posts, guide the movable span during the movement and resist the horizontal forces applied to the span when it is in opened position (Ryall et al., 2003).

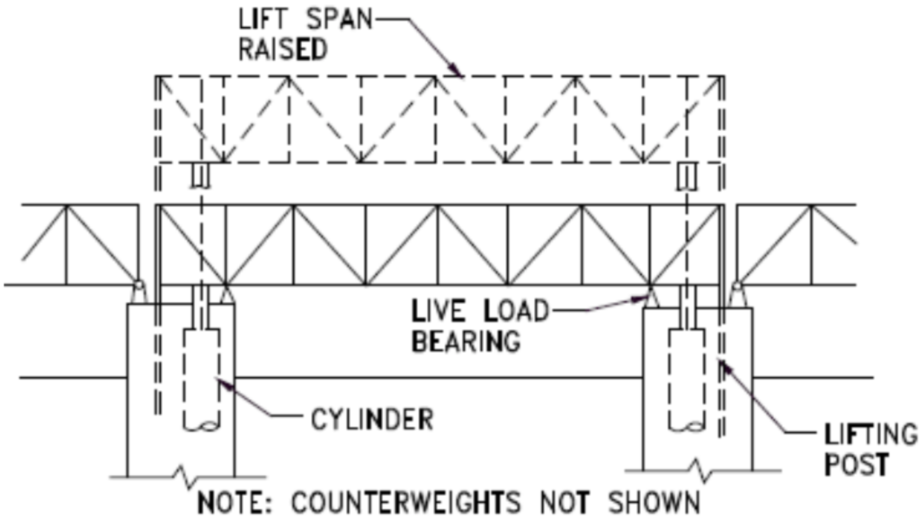


Figure 1.19. Pit drive vertical lift bridge (Berger et al., 2015b)

1.2.3. Swing bridges

Swing bridges are those who can provide a navigation channel by pivoting in a horizontal plane about a vertical axis, on a mechanical or hydraulic bearing mounted on a central pier called the pivot pier which carries the dead load of the span in open position. When the swing span is closed and carrying traffic, its ends are supported on rest piers. The pivot axis can be located at mid-length of the draw, so it is said to have equal-length arms or be symmetrical. Sometimes, the arms are not of equal length and the draw is termed unequal-armed or bobtailed and therefore requires counterweights at the ends of the shorter arms for balance. In open position there is no limit on air draught and the visual impact is considered minimum. The wind load is not as severe as on other types of bridges, so less power required and therefore more efficiency. Swing-span bridges can be made with different structural systems such as steel plate girder or steel truss. The principal characteristic noticed on these bridges is the necessity of having a big area to store the moving span when in open position. For a two-arms swing bridge, this has been sited in the middle of the navigation channel, thus reducing the length for navigation and making the maintenance task more inaccessible and difficult. Equally, the center pivot pier and fender system were often viewed as a significant impediment to navigation (Koglin, 2003).

Some examples of swing bridges are the El Ferdan swing bridge across the Suez Canal in Egypt shown in Figure 1.20., the George P. Coleman Bridge across the York River in the USA.

Figure 1.20. Swing bridge (Florida Department of Transportation, 2018)

1.2.3.1. Types of swing bridges

Swing bridges are categorised according to their type of bearing that is the center bearing, rim bearing, combined bearing, slewing-bearing and pontoon-supported swing bridges. The most common types are described here.

a. Center bearing swing bridges

The center bearing type shown in Figure 1.21. has a large, spherical bearing (pivot bearing) at the center of the span that supports all the dead load when the bridge is in the open position. The span is balanced so that its center of gravity is over the bearing, which receives the weight of the span through a heavy cross-girder and also keeps the span centered. A balance wheel system provides stability during rotation and in the open position on the perimeter of the pivot pier (Ryall et al., 2003).

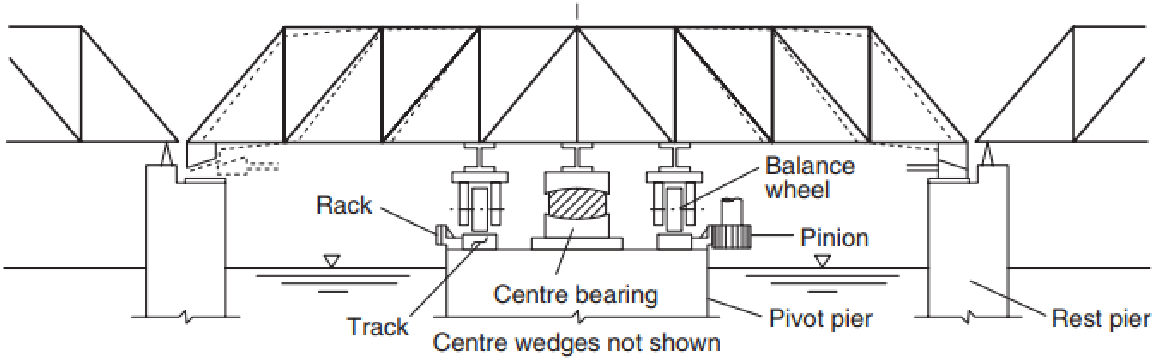


Figure 1.21. Center bearing swing bridge (Ryall et al., 2003)

b. Rim bearing swing bridges

They ride on a large number of tapered (conical) rollers positioned around the center of the span as shown in Figure 1.22. The rollers carry all the dead and live loads and provide stability during span operation. A center post is typically provided with radial members to keep the rim-bearing wheels centered. Each roller is mounted on a radial shaft and restrained from moving away from the center post. The weight of the span on the tapered rollers and sloping tracks prevents the rollers from moving toward the center bearing. The rollers are held in place both radially and circumferentially by a "cage" or a roller frame. Rim bearings are used for wide, heavily loaded swing bridges, such as those over the Harlem River in New York City, or long spans such as at El Ferdan (Ryall et al., 2003).

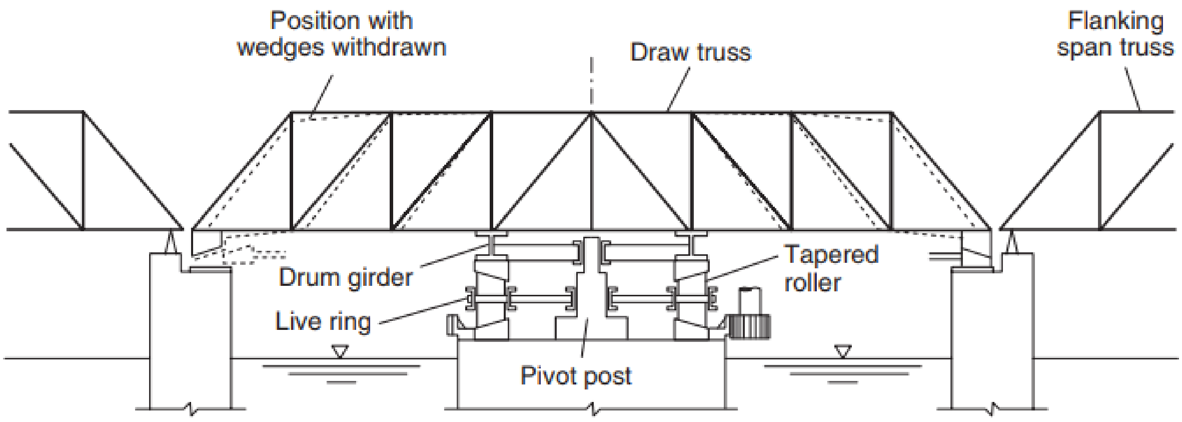


Figure 1.22. Rim bearing swing bridge (Ryall et al., 2003)

c. Combined bearing swing bridges

Combined bearing swing bridges are those equipped with both a centre bearing and a rim bearing as shown in Figure 1.23. Usually the centre bearing is mechanical, with either plain or antifriction rolling element, but hydraulic bearings can also be used. In this type of bridge, the dead load is shared by the centre and rim bearings, with the rim bearing usually supporting most of the dead load. The distribution of live load between the centre and rim bearing is a function of the transverse rigidity of the load distribution framing. Some designers find disadvantageous the uncertainty about load distribution. Jacking has been used to control the initial distribution of dead load between centre and rim bearings (Ryall et al., 2003).

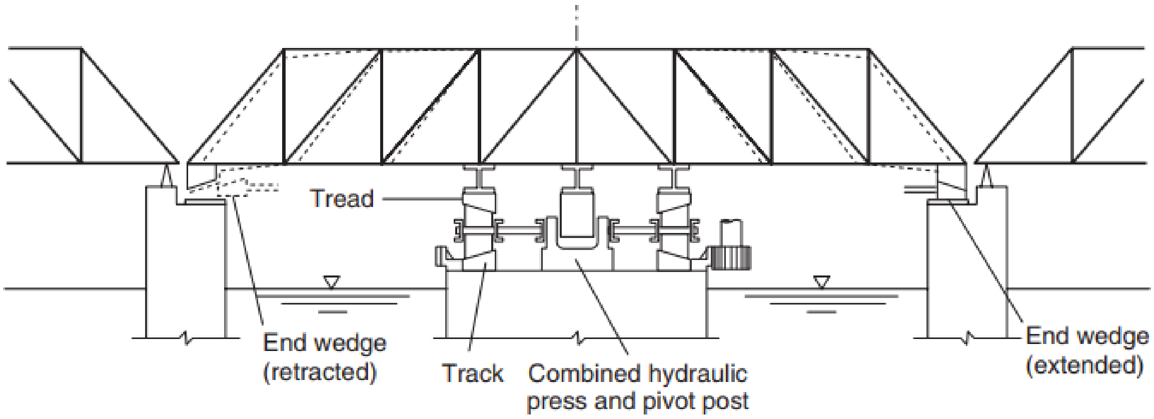


Figure 1.23. Combined bearing swing bridge (Ryall et al., 2003)

1.2.4. Other types of mobile bridges

There are many uncommon, variations, and novel types of movable bridge like the retractile bridge, the removable span, the transporter bridge. More recently, several imaginative movable pedestrian bridges have been built to unique designs such as the folding, the gyratory, the tilt, the curling bridges.

1.2.4.1. Retractile bridge

Also known as retractable bridges, retractile bridges operate by translating horizontally, or said more plainly, pulling back away from the navigable channel as seen in Figure 1.24. There are two types of retractile bridge: rolling and floating (Chen & Duan, 2014).

Figure 1.24. Retractile bridge (Google Search, 2022)

1.2.4.2. Removable span

As seen in Figure 1.25., these are “movable” bridges that are simple spans fitted for temporary removal. These may have a span length of about seven meters or less, but some are quite large and would require a rather heavy crane or other substantial equipment to move them out of the way (Koglin, 2003).

Figure 1.25. Removable span (Google Search, 2022)

1.2.4.3. Transporter bridge

Transporters were not really bridges, but actually had a more descriptive name, “aerial ferries.” These structures consisted of tall towers on each side of the navigation channel, connected by a trussed gantry or similar sort of “bridge” high enough to clear navigation as shown in Figure 1.26. (Koglin, 2003).

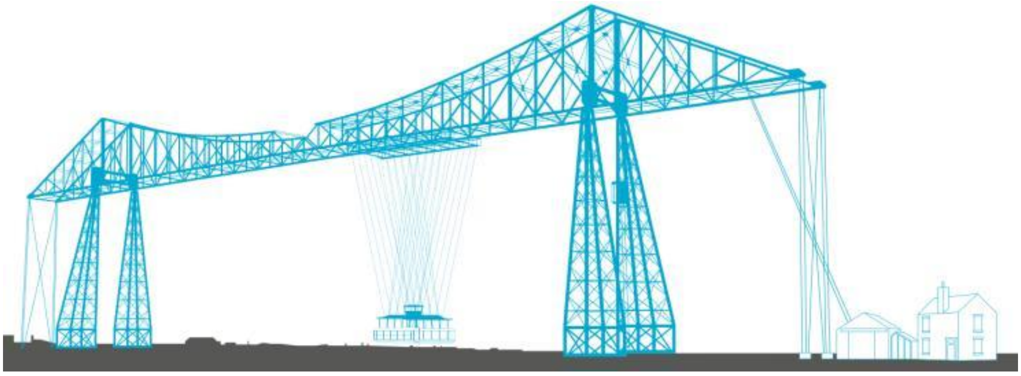


Figure 1.26. Transporter bridge (Google Search, 2022)

1.2.4.4. Folding bridge

Equally known as jack-knife bridge, they describe a bridge deck that folds about one or more transverse horizontal hinges in order to clear a waterway using lightweight members arranged in a balancing fashion so as to minimise the force necessary to operate them as seen in the Figure 1.27. (Ryall et al., 2003).

Figure 1.27. Folding bridge (Google Search, 2022)

1.2.4.5. Curling bridge

Also called rolling bridge, it consists of triangular sections hinged at the walkway level and connected above by two-part links that can be collapsed towards the deck by hydraulic cylinders mounted vertically between the sections as shown in Figure 1.28.

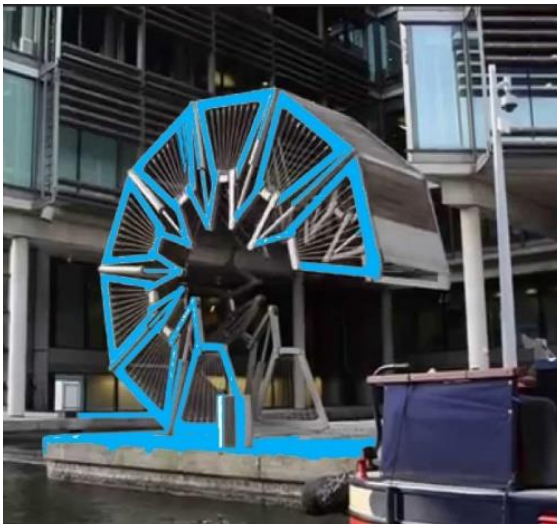


Figure 1.28. Curling bridge (Google Search, 2022)

1.2.4.6. Gyratory bridge

The operating principle is that a movable span rotates about a longitudinal axis, that is about an axis crossing the waterway. At both shores, there is a tower with a trunnion at an elevation of one-half the lift height, with the axis of the trunnions being collinear.

1.2.4.7. Tilt bridge

A tilt bridge is a type of movable bridge which rotates about fixed endpoints rather than lifting or bending, as with a drawbridge. Gateshead Millennium Bridge spanning the River Tyne in England, is a pedestrian bridge with two large hydraulic rams at each side that tilt the structure back allowing small watercraft to pass under as can be seen in Figure 1.29.

Figure 1.29. Tilt bridge (Google Search, 2022)

1.3. Kinematics and mechanisms of movable bridges

A mechanism is defined as a closed kinematic chain of linked structural elements which is connected to a rigid support. The degree of movement of any given mechanism is determined by the number of structural elements and the number of degrees of movement in the links. For a structure to be movable it must be turned into a mechanism by releasing at least one degree of movement. For each kinematic degree of movement, a drive must exist in order to control the mechanism. The movement of a bridge must be clearly defined and repeatable and, therefore, the number of drives always matches the number of degrees of movement. The number of kinematic degrees of freedom d for a given structure in space and in plane can be computed by the expressions 1.2 and 1.3.

$$d = 6.b - s - c \qquad (3D) \tag{1.2}$$

$$d = 3.b - s - c \qquad (2D) \tag{1.3}$$

where, b=number of connected bodies, s=number of support conditions, and c=number of connection conditions. During opening and closing, these degrees of movement are prescribed by the machinery driving the bridge. In the parked position the degrees of movement are constrained by a locking device or bearing, again turning the mechanism into a kinematically determined structure. The drive must carry the loads during movement and the constraint must carry the loads during traffic use of the bridge. Drive and constraint can be combined into one component or can be kept separate.

1.3.1. Kinematics of mobile bridges

Kinematic principles are best applied during the conceptual design phase of movable bridges when the mechanism for the guided movement is defined and also during the implementation phase when individual components have to be designed to accommodate the desired range of movement. The choice of the driving mechanism is also often influenced by kinematic consideration(Wallner&Pircher,2007). Each mobile bridge type defines its kinematic considerations.

1.3.1.1. Bascule-type bridges

The primary movement of a bascule bridge is a rotation about a horizontal axis as seen in Figure 1.30. Secondary movements, mostly vertical translations of the weight, were introduced.

In some cases, an attempt was made to provide better balance by guiding the weight along a curve. The trunnion-type bascule bridge has a fixed axis of rotation and a counterweight that balances the moving blade. The kinematic behaviour of the rolling lift bascule is different in that the location of the axis of rotation changes as the bridge opens (Wallner & Pircher, 2007).

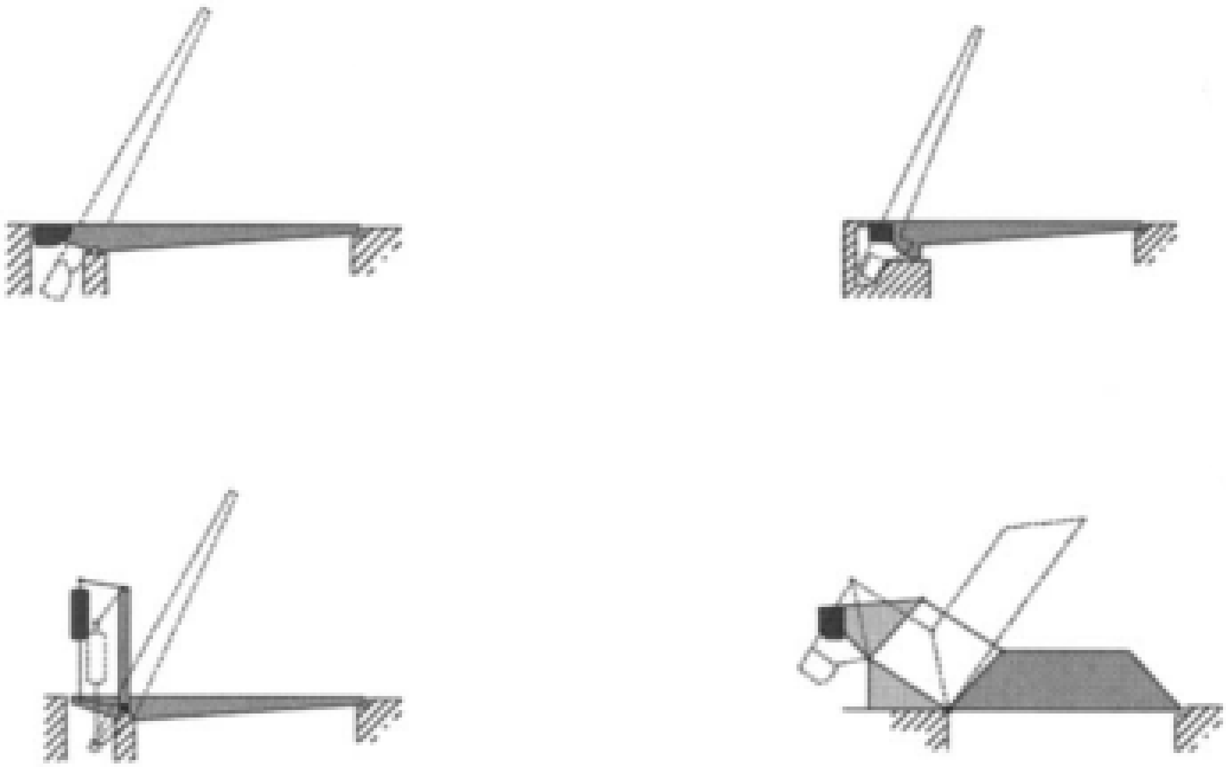


Figure 1.30. Kinematics of bascule bridges (Wallner & Pircher, 2007)

1.3.1.2. Vertical lift-type bridges

Generally, the vertical motion of the bridge deck is initiated at each of the four corners of the span. Synchronisation of the four individual degrees of movement in the corners of the bridge deck must be ensured in order to reduce this kinematic system to one single degree of movement. Avoiding small secondary rotations or sway motions of the bridge is a challenge for vertical lift bridges on a kinematic viewpoint, since it can potentially lead to jamming of the movable deck. To limit the opening movement to a pure vertical translation, pulley systems or other mechanical transmissions were first used and later on electronic control systems were inserted. A remarkable recent example is the Saltina bridge in Switzerland which opens automatically with the opening mechanism triggered by the water level which fills tanks acting as counterweights (Wallner & Pircher, 2007).

1.3.1.3. Swing-type bridges

The primary movement of swing bridges is clearly a rotation about one or two vertical axes. The Schwedler-type (rim bearing) mechanism consists of a pivot which carries the dead load while rollers carry any unbalance in the structure during movement, and a separate set of bearings which carry the live load and a portion of the dead load in the closed position. In order to shift from one load-bearing system to the other a secondary movement must be performed, namely a rotation about a horizontal axis as depicted in Figure 1.31. The pedestrian bridge at Limekiln Dock in London is an especially noteworthy example which uses the concept of cable-stayed system. Kinematically, this solution uses the concept of balanced bobtail swing bridges where the counterweight moves in a small circle around the mast and thus requires relatively little space (Wallner & Pircher, 2007).

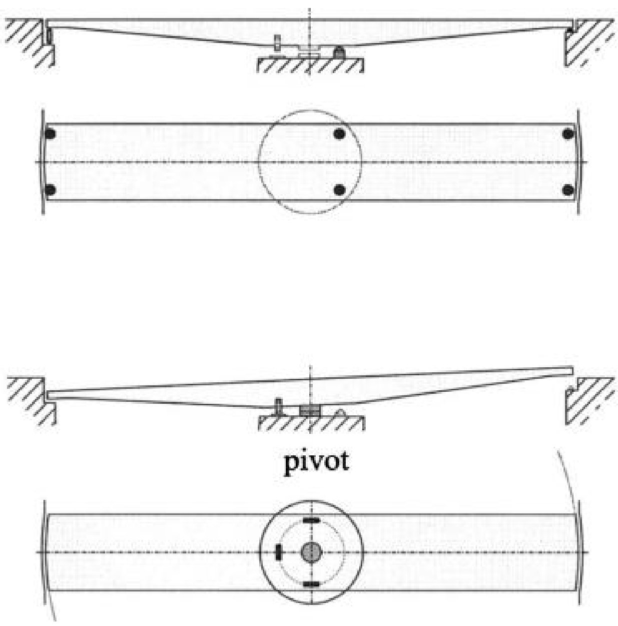


Figure 1.31. Secondary movement for swing bridges (Wallner & Pircher, 2007)

1.3.1.4. Other traditional systems

Other movable bridge systems which have been used historically like roll bridges, pontoons, propose kinematic systems that use a translation along a curved or straight line in the horizontal plane. These types of bridges have stopped to be built for general applications. However, they inspired the invention of incremental launching as a construction method where the bridge girder is moved incrementally in the longitudinal direction until the final position of the girder is reached (Wallner & Pircher, 2007).

1.3.1.5. New innovative systems

In recent years, some places in Europe have been marked by new and innovative systems for mobile bridges characterized by more involved modes of movement and fresh and innovative approaches towards the kinematics of such bridges.

a. Folding bridge

This pedestrian bridge in the Kiel, Germany will be considered. In its closed position this bridge resembles a cable-stayed system. The two “pylons” of this bridge can be rotated about their bases and in doing so lift the three spans of the bridge girder (which are connected with hinges) into a folded open position. The opening movement triggers three changes in the kinematic system with each of these systems having two kinematic degrees of movement. The closed position can be characterized as a simply supported beam with two hinges. The first phase of the movement is initiated by shortening the two stay cables. The final phase of the movement is driven by pulling on the diagonal cable of the bracing leading to the completely folded open position as depicted in Figure 1.32. (Wallner & Pircher, 2007).

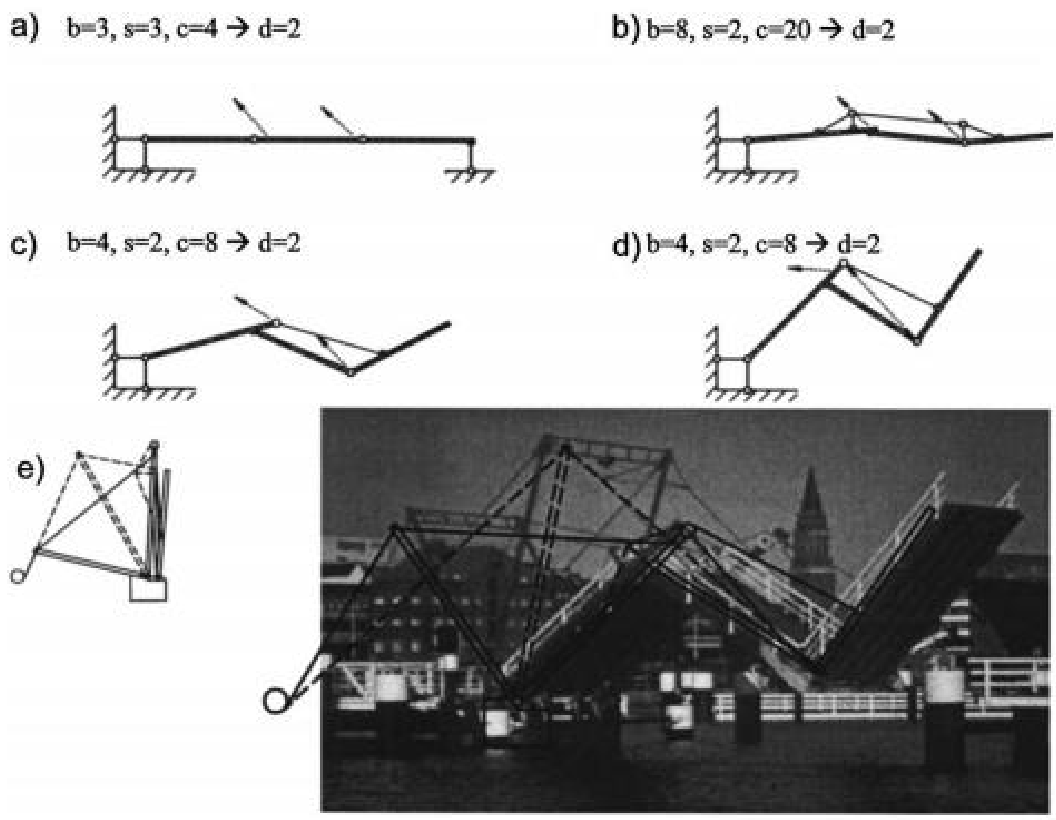


Figure 1.32. Opening process of the folding bridge (Wallner & Pircher, 2007)

b. Tilt bridge

Gateshead Millennium Bridge, Newcastle, U.K is a noteworthy example of this bridge type with primary movement, a horizontal rotation about an axis perpendicular to the river direction. It is a made up of two steel arches, one forming the deck, the other supporting it (equally serving as counterweight) and are connected in such a way that the structure can pivot about an axis through their joint base points as seen in Figure 1.33. The pinned supports allow for the rotation of the bridge which is driven by an eccentric hydraulic lever (Wallner & Pircher, 2007).

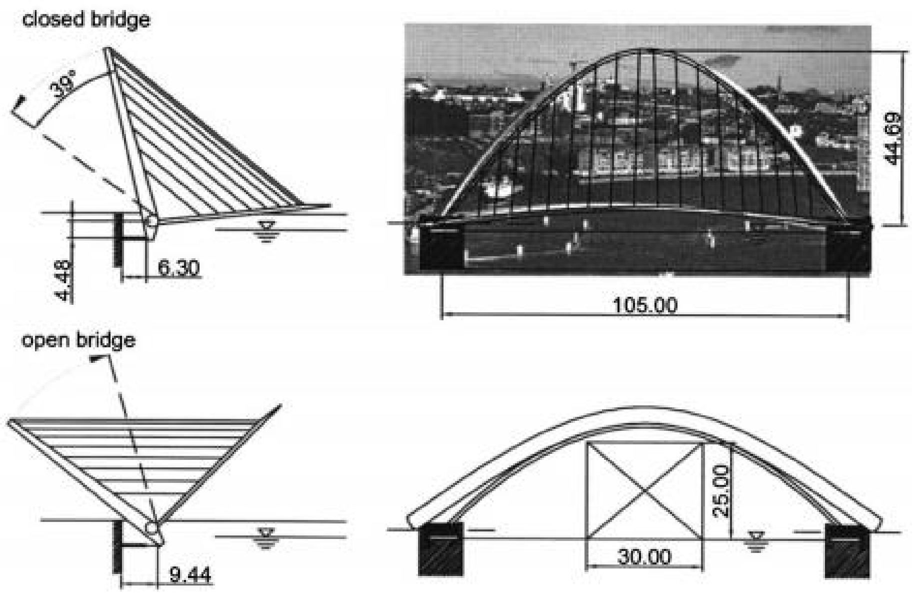


Figure 1.33. Millennium bridge (closed and open) (Wallner & Pircher, 2007)

c. Curling or rolling bridge

The amazing example in this case is the Paddington basin rolling bridge, U.K with a unique and new fashion. In its closed or unrolled configuration, the handrails act as the top beam of a simply supported truss. Elongation of vertical members (equipped with hydraulic rams) push sections of the handrail upwards causing the bridge to arch first and finally curling it to form its open position of a loop with the tip resting on the base. The first segment is kinematically anchored to the ground and each additional segment adds one kinematic degree of freedom and therefore requires one additional drive for the structure to work.

1.3.2. Mechanisms of mobile bridges

Operation of a mobile bridge at a certain location must be efficient when requested. Therefore the operating system should be reliable in order to ensure the fluidity of waterway traffic. Operating a mobile bridge is done in two ways, from the operator's control desk in the control room, that is the automatic (fully interlocked) and the manual (step by step control) operations. The operating system consists of the operating machinery and the operating mechanism. Currently the operation of mobile bridges lays on either a mechanical system or hydraulic system with auxiliary mechanical machinery. The mechanical system is advantageous in that, the prime mover drive can be in the form of electrical motors utilising modern sold state electronics to control speed and equalise torque instead of requiring complicated hydraulic systems; the speed remains constant throughout the operation of the bridge except for the acceleration and deceleration phases; there are potentially no hydraulic pipes in the machinery chambers, thus less space is required and the costs are reduce; larger opening angles are possible as the kinematic moment arms remain constant.

The hydraulic system is advantageous in that, the load in the hydraulic cylinders in still air conditions remains sensibly constant, and no reversal of load takes place within them during the whole lift operation; the system is very robust; can share load across a structure using even the simplest hydraulic circuit; less sensitive to water or flooding than a purely electrical solution.

The principal disadvantages are the need of specialised maintenance because is needed both knowledge about electrical and hydraulic machinery and the great environmental risk of oil spill.

Most movable bridges are powered by either electric-mechanical or hydraulic-mechanical drives with power driven pinions operating against racks or by hydraulic cylinders. Span drives consist of an arrangement of electrical, mechanical and hydraulic components that transfer the high speed, low-torque energy from the motors into the low-speed, high-torque motions of the two pinions. Drives may be electric motors, hydraulic equipment, or auxiliary drives. Either AC or DC power may be used to drive electric motors. Hydraulic equipment is usually either large actuating cylinders or hydraulic motors. Adequate pressure and volume of hydraulic fluid must be provided to the cylinder, or motor to power the opening and closing of the bridge. The fluid flow to the cylinder or motor is usually provided by an electrically operated and controlled hydraulic pump (Florida Department of Transportation, 2018).

The pinion of the span drive machinery engages with the rack on the movable span when opening and closing the span. The pinion is the last gear in the drive train. As it drives the motion of the span, it is commonly called the drive gear. The rack can be another gear, or a bar, or a track with teeth on one of its sides. The teeth of the rack are machined to match the teeth of a pinion and move the span. The three most common types of span drives are designated as Type 1, Type 2, and Type 3 Span drives.

Type 1 Span drives have all the outputs connected mechanically, as shown in Figure 1.34. The motor is connected to the primary speed reduction gearing, usually by a flexible coupling. This gearing is enclosed in a welded steel housing on most contemporary designs, but cast steel and various kinds of cast iron were and are used in manufacturing these speed reducers. Power is distributed from the primary reduction gearing to two (or more) sets of secondary reduction gearing located at the sides or ends of the movable span. Secondary speed reductions are often made with enclosed speed reducers as shown; however, open gearing is not uncommon. Some newer bridges may have a single large speed reducer, rather than smaller primary and secondary reducers (Florida Department of Transportation, 2018).

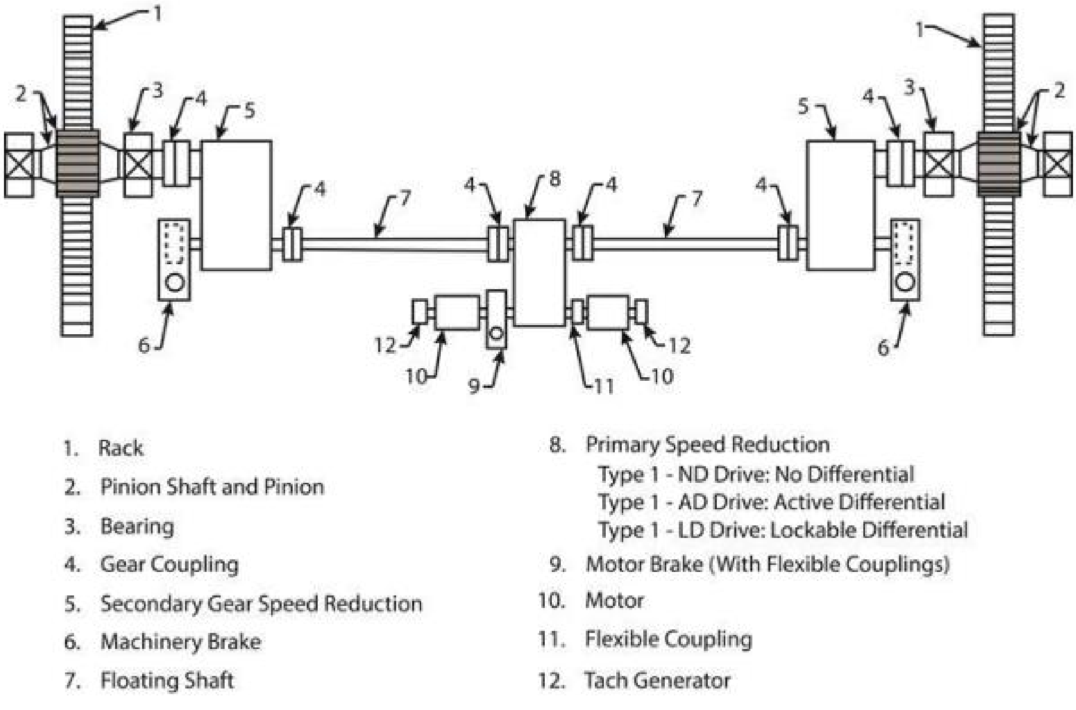


Figure 1.34. Type 1 span drive (Florida Department of Transportation, 2018)

A Type 2 span drive has two pinions, each powered by a separate hydraulic motor through a geared speed reduction, as shown Figure 1.35. Because the fluid input to the two motors comes from a common source, the torques applied by each pinion should be nearly equal, assuming that both sets of motors and gearing, and piping, etc., are alike. Open gear trains are also common.

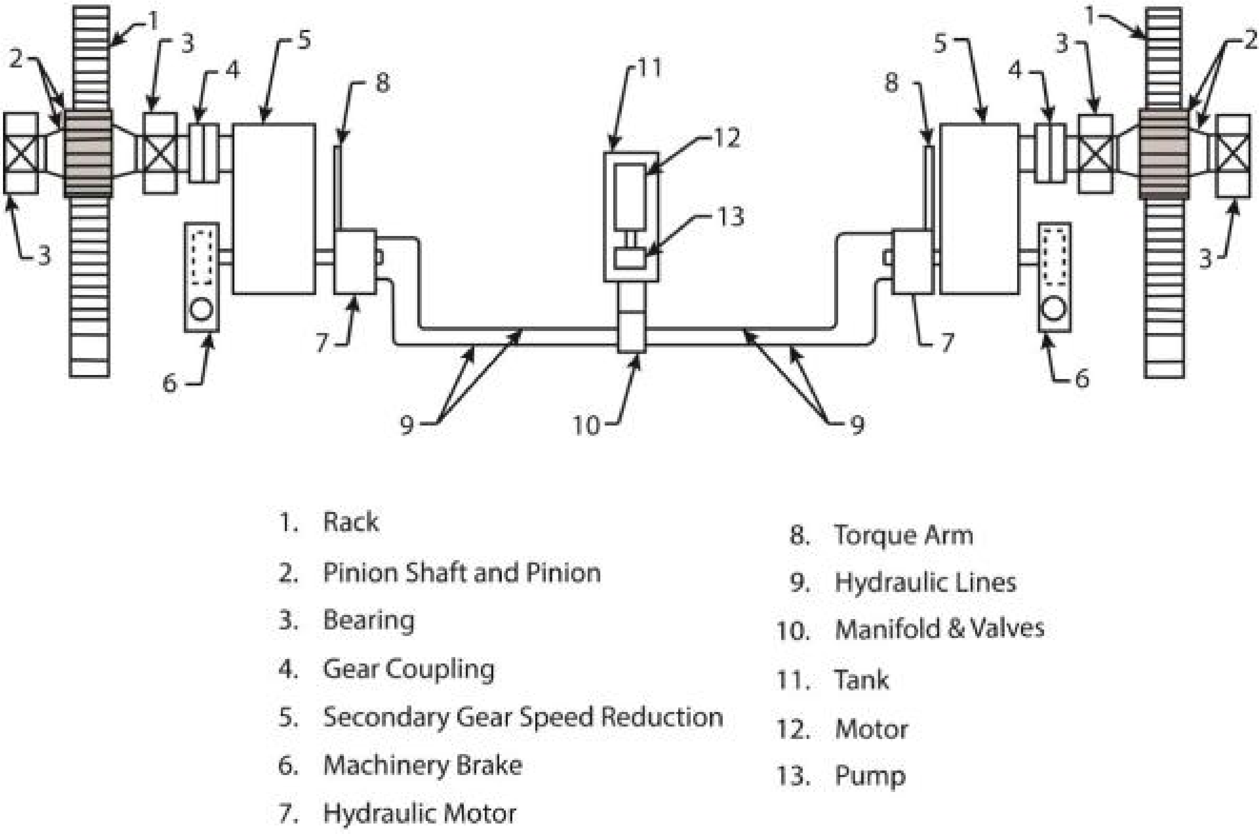


Figure 1.35. Type 2 span drive (Florida Department of Transportation, 2018)

Type 3 Span drives transmit power to the moving span by linear action, as depicted below in Figure 1.36. Type 3 drives have been installed on simple trunnion bascules, rolling lift bascules, swing bridges, and vertical lift bridges. Hydraulic cylinders are usually used to move the span, but chain, wire rope, and screw drives have also been used. The cylinder mountings shown are for a simple trunnion bascule or a pit vertical lift bridge.

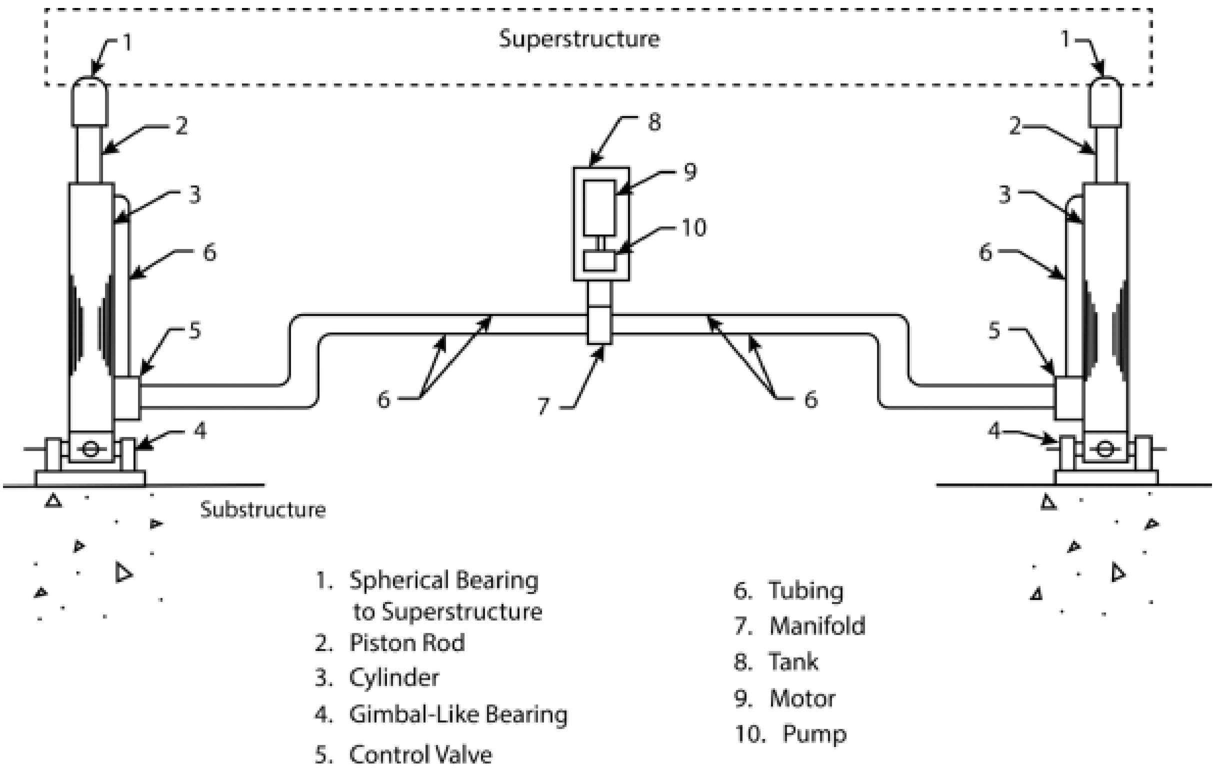


Figure 1.36. Type 3 span drive (Florida Department of Transportation, 2018)

1.3.2.1. Mechanisms of bascule bridges

The operating mechanisms are driven by the operating machinery. The electro-engine and the gearbox are the main components of the machinery. The outgoing shaft of the engine will rotate at a certain frequency (with about 1500 to 1800 rounds per minute) but with a low torque. However, to open the bridge, a high torque and a low rotational speed are required. To perform this, a gearbox reduces the rotational speed which automatically means an increase in torque, as denoted by equations 1.4 and 1.5;

$$\boldsymbol{P = \tau * \frac{\partial\theta}{\partial t} = \tau\omega} \tag{1.4}$$

$$\boldsymbol{\tau_{in}\omega_{in} = \tau_{out}\omega_{out}} \tag{1.5}$$

Today, engines can be regulated, with their speeds been increased in phases. The following phases can be identified; creep opening (20-30 sec.), full speed opening (30-70 sec.), creep stop opening (70-80 sec.), full speed closing (100-140 sec.) and creep closing (140-160 sec.).

Upon the opening of the bridge, all the mechanisms need to settle first, before the engine can increase to full speed. For example, the teeth of gears have a small space for motion. A slow rotational speed is used at start to make sure all the space for motions is cleared and full contact is made, this prevents damage. Care should be taken as regards the counterweights, since a full stop will create a large force on the mechanism as they want to keep their rotational speeds and large vibrations. Therefore, the speed is reduced the last few seconds of opening, to reduce the force. The last seconds of closing are also in slow speed such that the leaf should not bump at high speed on the supports at the resting pier. This can cause great damage to the operating mechanism, as there is an abrupt stop, and to the toe of the leaf itself. The mechanism for bascule bridges can be the gear rack, the push-pull rod or the Panama wheel one. The latter is a four-rod mechanism with two fixed pivots and two mobile pivots.

1.3.2.2. Mechanisms of vertical lift bridges

The machinery for a vertical lift bridge is mounted on the span or tower, provides a direct mechanical connection to raise and lower the span equally at all points, when the drive is actuated to avoid binding between the vertical lift span and its supporting towers and equally accommodate, the change in balance that occurs with a vertical lift span as the counterweight ropes pass over the sheaves on the tower tops. The motors, brakes, and reduction gearing are located at the center of the vertical lift span or on a frame at the towers' top. An enclosure mounted above the roadway, contains the operator's house and machinery room. Inside the machinery room are the drive motors and some reduction gearing in a frame, the brakes, and an emergency or auxiliary engine drive if the bridge is provided with one. The reduction gearing converts the high-speed, low-torque output of the drive motors or engine to low-speed, high-torque in horizontal shafting that lies transverse to the bridge axis. This shafting is extended from each side of the machinery room by means of floating shafts and couplings. Alongside the machinery room, on each side, near the main trusses, are additional machinery frames, one at each truss. These frames each contain two large rope drums, each with a large ring gear attached to it. The ring gears mesh with pinions mounted on the shafting that extends from the machinery room. Each drum of the Waddell type vertical lift bridge has four operating ropes attached to it, arranged so that as two ropes play off the drum, the other two are pulled in, so as to raise the movable span (Koglin, 2003).

1.3.2.3. Mechanisms of swing bridges

Machinery is more critical for swing bridges than for bascule or vertical lift spans, because mechanical drive components serve to hold live load supports in place, as is the case for all the end lifts and center wedges for a typical swing span. The swing span is mounted on the main pivot drum by means of considerable distribution girders which bear on the top rim of the steel drum. The swing span mechanism implemented consists of a number of components including: electric motors, gearing, rollers, racking, shafts and rail tracks. Dual electric motors are mounted in the centre of the span above the pivot and they drive motion through a number of gears and into a horizontal shaft. This shaft is fitted with a bevelled pinion that is keyed into a vertical shaft at either end. The vertical shaft has a lower end pinion which rotates on a fixed rack at the pier deck thus causing motion. The bearing of the drum on the pier is achieved by steel conical rollers which are contained between a track fixed to the underside of the drum and a second track fixed to the pier. As the drum rotates, the rollers allow for the smooth continuous bearing during motion. As the span returns to its closed position near the rest piers, a braking mechanism consisting of a ‘latch and catch’ is engaged. The latch component is essentially a bracket and wheel mounted on the swing span. The catch is a triangular piece with a centre void. As the span approaches the rest position, wheels on the latch transverse up the incline of the catch thus slowing motion before falling into the opening. Once the span is in its rest position and motion has been stopped by the latch and catch mechanism, four end lifts are raised to provide a firm bearing at the ends of the span. These end lifts are also operated by electric motors, gearing and shafts mounted on the swing span (Florida Department of Transportation, 2018).

1.4. Functionality of mobile bridges

Functionality of mobile bridges highly depends on the performance of the machinery, the hydraulic and electrical system.

1.4.1. Machinery system of mobile bridges

- Open Gearing: It transmits power from one shaft to another and alters the speed and torque output of the machinery.

- Speed Reducers Including Differentials (Closed Gearing): They have the same function as open gearing. In addition, the closed gearing is used as a torque equalizer, to match operating speeds and torques on various portions of the mechanical system.
- Shafts: They transmit all the required mechanical power from one part of the machinery system to another for opening and closing operations. Shaft condition is directly related to the structural integrity and proper functioning of the movable bridge.
- Couplings: They transmit power between the ends of shafts in line with one another. Some types of couplings can be used to compensate for slight imperfections in alignment between the shafts and they can be categorised into three groups that is rigid, flexible, and adjustable.
- Bearings: They provide support and help to maintain alignment of rotating shafts, trunnions, and pins.
- Brakes: Brakes on movable bridges are generally used as a “parking brake”. In other words, they are engaged when the span has stopped moving. Brakes can be of either the drum type or disc type, and can be released manually, electrically, or hydraulically.
- Air Buffers and Shock Absorbers: They are located between the span and the pier at points where impact may occur between the two that mechanically control the deceleration of the span at the fully closes and/ or fully open positions. Typically, the buffers are large pneumatic cylinders, or hydraulic energy absorbers.
- Span Locks: Span lock bars at the end of the span are engaged when the span is fully closed to prevent movement under live load. Span locks may also be provided at other locations on the span to hold the span in an open position against strong winds or to prevent movement from an intermediate position. They can be engaged either mechanically or hydraulically and typically consist of a forged steel lock bar that engages a receiving socket.
- Toe Locks: They transfer vehicular live load between the leaves of double-leaf bascule bridges when the structure is in the closed position. A toe lock typically consists of a forged steel lock bar that engages a receiving socket.
- Live Load Bearings and Strike Plates: Live load bearings and strike plates between the movable and fixed portions of the bridge are designed to bear most or all of the live load

when the bridge is carrying traffic. Live load bearings are provided at the end of the moving leaf truss/girder at the approach end.

- Wire ropes: These ropes are typically used to connect the vertical lift span to the counterweights located in the towers for operation on vertical lift bridges and are comprised of individual wire strands that are tightly wound together.
- Counterweight Sheaves: These are large diameter, angular-grooved wheels located at the top of the towers, over which the counterweight ropes are draped, supporting the entire weight of the span and counterweight during operation. These sheaves are supported by shafts and bearings.
- Operating Drums: Operating drums are large diameter, grooved drums to which the operating ropes are attached and are paid-out and taken-in during operation of the lift span. These drums are supported by shafts and bearings that are turned by the operating machinery.
- Span Guides: Span guides restrict lateral and longitudinal movement of the span during operation. Roller guides are commonly used as span guides.
- Counterweight Guides: Counterweight guides are used to limit lateral and longitudinal movement of the counterweights as they move within the towers. Sliding guides are frequently used as counterweight guides.
- Lifting Posts: The spans are guided vertically at each corner of the span by lifting posts. Typically, lifting posts are rigidly connected to the end floor beams and fascia girders.
- Equalizing Systems: Skewing of the lift span during raising and lowering is restrained (in both the longitudinal and transverse directions) by wire rope stabilizing systems.
- Tread Plates: Tread plates are mating steel plates with interlocking teeth and pockets on rolling bascule bridges and receive very high loads. As the bascule leaf travels along the lower tread plate, the teeth mate with the pockets to ensure proper alignment and travel during the opening of the leaf.
- Bumpers: They are normally used when there is a need for a deceleration of the moving span when electrical controls are not used.
- Balance wheels: When the end and centre supports are disengaged to allow the swing span to rotate, it is needed some kind of support that helps to stabilize the span and prevent tilting from loads, like the wind.

1.4.2. Hydraulic system of mobile bridges

Mobile bridges having hydraulic drive as the main drive usually possess a mechanical drive for the auxiliary machinery items such as span locks and wedges. Recently, hydraulic machinery has been introduced in mobile bridge design and has proven to be an effective solution, as the hydraulics can be closely matched to the power demands, which require good speed control over a wide range of power requirements. However, the systems also require a more specialized knowledge and maintenance practice than was traditionally the case with mechanical drives. Nonetheless, hydraulic machinery is often used for temporary operation such as for a temporary mobile bridge or when operating machinery is being replaced (Koglin, 2003).

Hydraulic drive system is made up of hydraulic cylinders that operate the mobile span. The typical design practice is to provide multiple cylinders so that one or more can be removed for maintenance while the span remains in operation. The cylinder end mounts incorporate spherical bearings to accommodate any misalignments. The hydraulic power unit, consisting of a reservoir, motors, pumps and control valves, is located between the cylinders. Typically, redundant motors and pumps are used and the valves can be hand operated if the control system fails. As movable bridges are located on waterways, the use of biodegradable hydraulic fluids is an option in case of a leak or spill (Koglin, 2003).

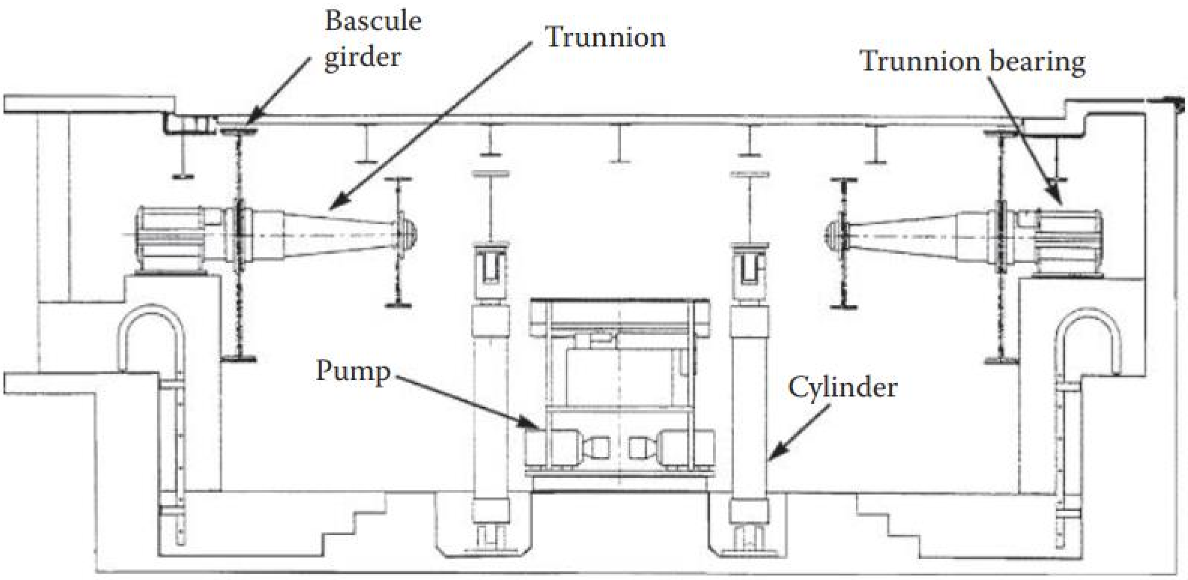


Figure 1.37. Section through a bascule pier showing hydraulic cylinders (Chen & Duan, 2014)

Open and Closed loop hydraulic circuits are both used on movable bridges. Open loop hydraulic circuits are the most common in movable bridge applications. Most hydraulic movable bridges are actuated by hydraulic cylinders that require large volumes of fluid and use differential flow rates in and out of the hydraulic cylinders. Open loop systems take up relatively large space due to the large volume of oil required. In open loop circuit applications, the pump draws fluid from a reservoir and pushes this fluid into the hydraulic system. After passing through the control valve circuitry and the actuator, the fluid returns to the storage reservoir. The reservoir typically is sized to hold at least three times the volume that can be displaced by the pump in one minute. Open loop systems move fluid in one direction only. Open loop pumps normally have a large diameter low pressure inlet port and a smaller high pressure outlet port. In a closed loop circuit design a single hydraulic pump is used to drive one or more hydraulic motors. The fluid that passes through the actuator is returned directly to the low-pressure side of the pump. The pump receives the same quantity of oil at the inlet as it is pumping through the outlet. Pressure, flow, and directional control are all achieved by the controlling elements of the pump (Florida Department of Transportation, 2018).

1.4.3. Electrical system of mobile bridges

It comprises four major groups of equipment, that is the power distribution equipment, the electrical machinery, the control system and the lighting systems.

The power distribution equipment includes the electric power sources, protective devices, and distribution equipment. The primary power source for movable bridges is a three-phase electric service with voltages of 120/240 volts, four-wire systems (for older bridges) and 277/480volts, four-wire systems. Protective devices include fuses and circuit breakers which provide overload and short circuit protection. Electrical devices are supplied through a raceway system, consisting of rigid metal conduit (housing electrical wires) and junction boxes (Florida Department of Transportation, 2018).

Electrical Machinery refers to electro-mechanical devices that operate the movable span and auxiliary devices such as locks, wedges, and traffic control equipment. The movable span is provided with one or more span motors (AC or DC type) that serve as the prime mover for the span and a motor controller which provides controlled motor speed and torque to ensure smooth movement of the movable span.

Drum controllers are used as motor control on older bridges, with primary drums using DC motors and secondary drum using AC motors. Limit switches regulate the limit of travel for machinery and provide an electrical signal to stop or change operation. Relays are low current switching devices that provide logical control of a bridge, using independently or concurrently a programmable logic controller (PLC) system (Florida Department of Transportation, 2018).

Bridges are often equipped with roadway lighting to illuminate the roadway for vehicular traffic. Warning gates with red flashing lights are used to warn approaching cars and vehicular traffic during bridge operation thus, alert the traffic that it must stop and indicate the vehicle exclusion area. Resistance gates are used to physically stop a vehicle that is either out of control or whose driver failed to realize the bridge was opening. Navigation lighting and signals are provided to guide and alert the channel water traffic, notifying the boat operator if the bridge is fully open or closed. They consist of alternating red and green lights mounted on the span (Florida Department of Transportation, 2018). Figure 1.38. describes the nature of the control panel and the motor control system.

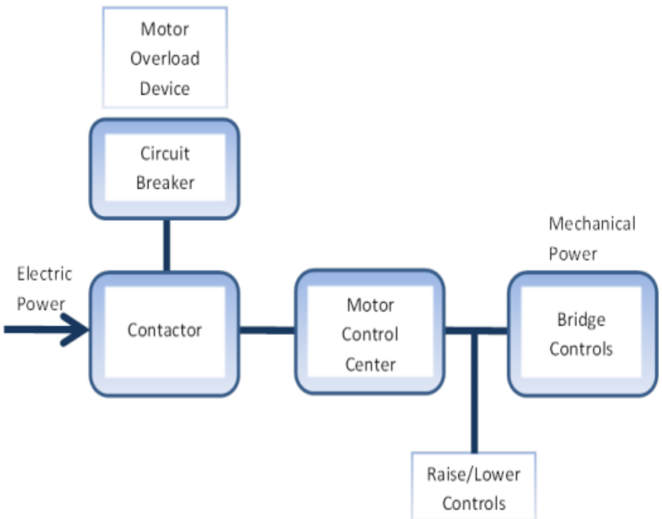


Figure 1.38. Control panel and motor control system (Florida Department of Transportation, 2018)

1.5. Structural analysis and design of mobile bridges

Structural analysis refers to a sequence of procedures used to determine the effects of design actions on structures. It can be split in two categories, that is the elastic analysis and the rigorous structural analysis.

In elastic analysis, individual members are assumed to remain elastic under the action of design loads at both the ultimate and serviceability limit states. This analysis type is divided into the first-order analysis and the second-order analysis. First-order elastic analysis does not take into account any changes in the geometry of the structure or changes to the effective stiffness of members due to compressive forces. Where the change in geometry is sufficient to modify the internal forces and moments, the changes are referred to as second-order effects. Second-order effects due to compressive axial strains in individual members that lead to joint displacements are commonly referred to as P-Δ effects; effects due to bending of individual members are referred to as P-δ effects. First-order elastic analysis is adequate for most bridge structures.

Rigorous structural analysis on the other hand act as an alternative to an elastic analysis and takes account of geometric deformations and non-linear behaviour of materials but would not be used for conventional bridge design.

There is equally member buckling analysis which is used to determine the effective length of compression members in bracing systems.

Finite Element Analysis (FEA) is a complex numerical method used to solve complicated problems which contain a number of variable inputs such as boundary conditions, applied loads and support types. FEA requires that the structure is broken up into smaller parts (or elements) which can be evaluated individually for a more accurate estimate of the solution. Structural design refers to making sure that appropriate sections of elements depending on materials are verified in functions of the stresses in those elements. The design procedure of a bridge must take into consideration several important factors in order to reach a best solution. These are choices of bridge systems, materials, dimensions, foundations, aesthetics, local landscape and environment. Structural designers are required to provide the most effective structural solution with maximum safety and minimum cost.

The structural analysis and design of a mobile bridge is different to that of fixed bridge in that, mechanisms must be considered in addition to the analysis as a fixed bridge, that is various configurations need to be considered.

1.5.1. Structural analysis

Structural Analysis is particularly important for structural engineers to ensure they completely understand the load paths and the impacts the loads have on their engineering design. As such there exist static and dynamic analysis with each type divided in linear and non-linear analyses.

1.5.1.1. Design standards and specifications

Currently specifications for mobile bridges in English are the American Association of State Highway and Transportation Officials (AASHTO) standards for highway mobile bridges, the American Railway Engineering and Maintenance-of-way Association (AREMA) standards for railway mobile bridges and the adapted codes from the Dutch (Nederlands Normaisatie Instituut, NEN) and the Germans (Deutsches Institute fur Normung, DIN). Although it is commonly a good practice to use the Eurocodes and/or design standards, these are incomplete in the detailed issues regarding the mechanical and electrical design (Anostructures, 2010).

1.5.1.2. Materials used for decking in mobile bridges

Structural and mechanical properties of materials are directly related to the durability and safety of the bridge, thus are to be considered carefully. The property considered in the choice of the material for a mobile bridge is the weight so as to reduce the counterweight and hence the cost. Other properties the materials are required to have include, hardness, ductility, tensile and compressive strengths, residual stress, resistance to corrosion, hydrogen embrittlement. In the past, the most common materials used for structural elements in movable bridges were wrought iron and steel. In the past few years some innovative movable bridges have been developed with aluminium and fibre-reinforced polymers (FRP).

1.5.1.3. Orthotropic steel deck

An orthotropic steel deck is composed by a steel plate with welded stiffeners in two perpendicular directions of the horizontal plane. The transversal stiffeners are known as cross beams or floor beams and the longitudinal stiffeners sometimes referred as ribs. The entire deck is supported by the main in the longitudinal direction as shown in Figure 1.39. Due to its outstanding structural properties and light weight, the steel plate deck system is adequate for the decking system of mobile bridges (American Institute of Steel Construction, 1963).

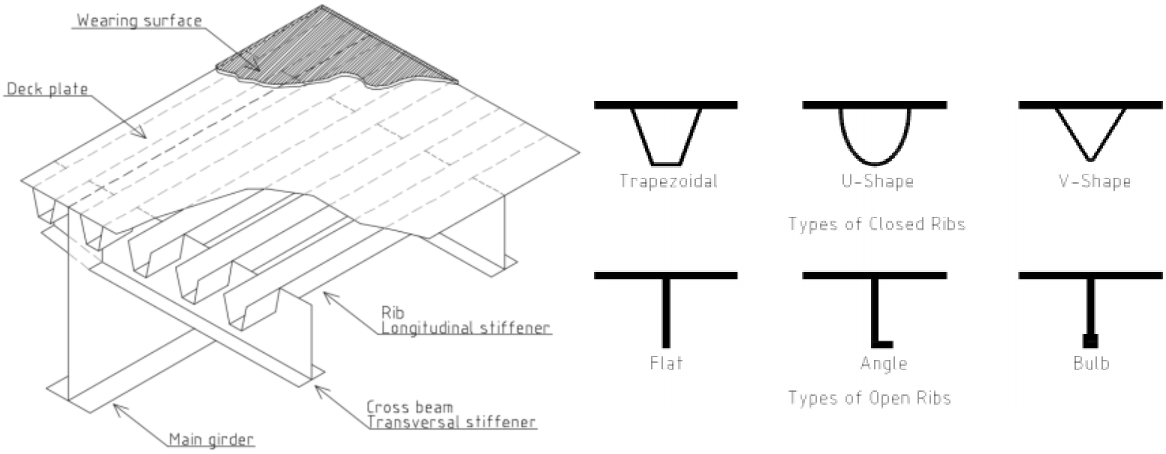


Figure 1.39. Orthotropic steel deck and types of ribs (Håkansson & Wallerman, 2015)

Two main types of orthotropic steel deck bridges exist, notably the box girder and plate girder bridges. Plate girder deck bridges are preferred for shorter spans due to simplicity in the design and construction. Box girder deck bridges on the other hand are ideal for long spans and curved spans due to their greater stability and resistance to twisting forces. Features of both types of deck are depicted in Figure 1.40.

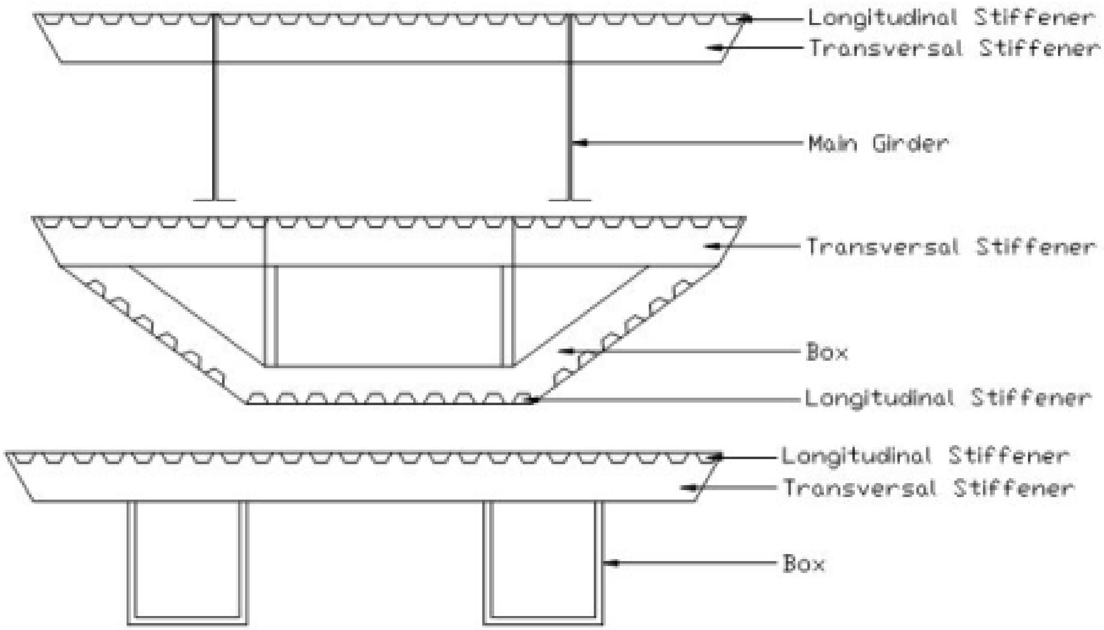


Figure 1.40. Types of OSDs (Håkansson & Wallerman, 2015)

The structural components making up an orthotropic steel deck are the wearing surface, deck plate, longitudinal stiffeners, transverse stiffeners and sometimes main girders. The wearing surface is generally made up of concrete, bitumen or polymer materials with the main purpose of providing skid resistance to the road users. It equally protects the deck against corrosion by water. Wheel loads are assumed to be dispersed at an angle of 45 degrees. The deck plate is a thin plate which transfers loads to the ribs and often serves as top flange for stiffeners and main girders. Longitudinal stiffeners provide support for the slender plate, increase the flexural rigidity of the cross section and help distribute loads to the transversal stiffeners. Two types of longitudinal stiffeners exist, that is the open and the closed ribs. Open ribs are less labour intensive and provide less structural stability. Closed ribs have high torsional rigidity and are advantageous. Transversal stiffeners are normally inverted T-sections and transfer loads to the main girders and equally provide support for the ribs (American Institute of Steel Construction, 1963).

Structurally, elements making up an orthotropic steel deck are linked in a complex way such that the system functions as one, with the structural elements fulfilling more than one function. As such, structural elements cannot be treated individually to assess the true response of the structure which is a very complex task. However, the deck can be divided into systems to ease the task, which can be analysed separately and later combined by linear superposition.

System 1 – Local deck plate deformation

System 1 consists of load transfer from the deck plate to the longitudinal ribs where transfer is done through deck plate deformation as shown in Figure 1.41. The local deformation of the deck plate from the wheel load results in transversal flexural stress in the deck and longitudinal stiffener as well as in the weld connecting them (Håkansson & Wallerman, 2015).

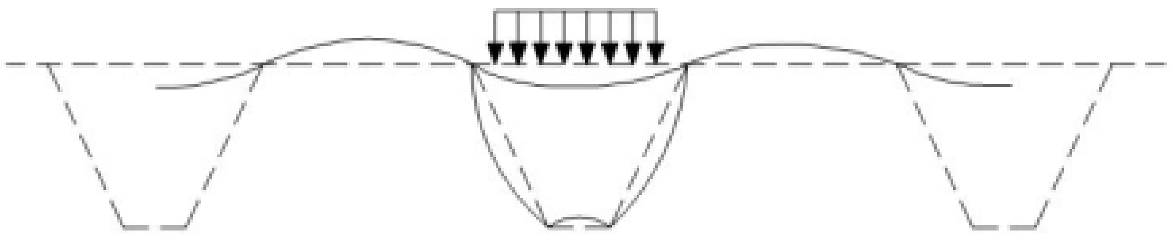


Figure 1.41. Structural system 1 (Håkansson & Wallerman, 2015)

System 2 – Panel deformation

Being the most complicated system to analyse, it requires the understanding of the two-way load distribution behaviour of the OSD panel when subjected to out-of-plane loading. The theory of elasticity of plates (plates loaded normally to the plane of the plate) remains a particular help in engineering solutions concerning the use of OSDs. This solution was founded in Huber's Equation given in expression 1.6.

$$\boldsymbol{D_x \frac{\partial^4 w}{\partial x^4} + 2H \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_y \frac{\partial^4 w}{\partial y^4} = p(x, y)} \tag{1.6}$$

This equation represents the static equilibrium of a plate of uniform thickness with orthogonal and torsional properties,

Where

Dx : plate flexural rigidity in the x-direction

Dy : plate flexural rigidity in the y-direction

H : effective torsional rigidity of the plate

p(x,y) : the loading at any point on the plate with coordinates of (x, y).

Solving expression 1.5 gives the moments Mx, My, Mxy, Dxy.

Since ribs share the same top flange, a phenomenon known as panel deformation is observed when a concentrated load is applied on the deck plate as shown in Figure 1.42.

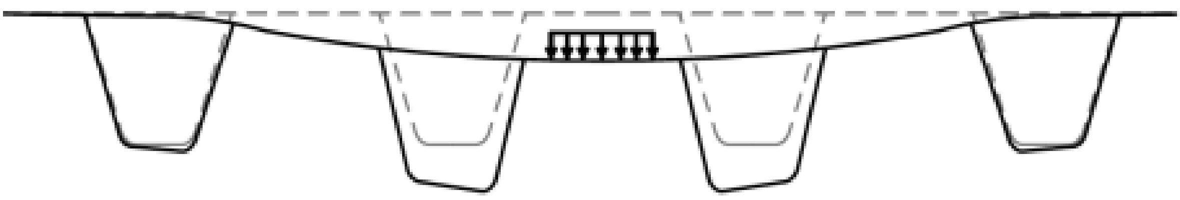


Figure 1.42. Structural system 2 (Håkansson & Wallerman, 2015)

System 3 – Rib longitudinal flexure

When ribs are loaded, the cross beams deflect since the ribs are continuous over the cross beams. To ensure this phenomenon, ribs are modelled as continuous over discrete flexible supports as depicted in Figure 1.40. In this model, cross beams are supposed simply supported between rigid main girders and will thus deflect upon loading.

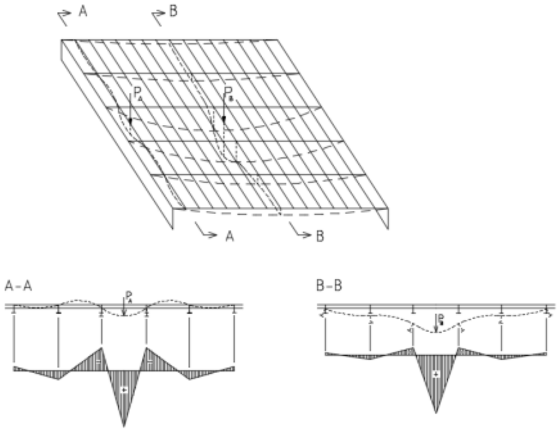


Figure 1.43. Structural system 3 (Håkansson & Wallerman, 2015)

System 4 – Floor beam in-plane flexure

At the intersection between floor beams and ribs, cut-outs are generally made as shown in Figure 1.44., this affects the geometry of the floor beam and makes it tough to assess in-plane stresses from bending and shear.

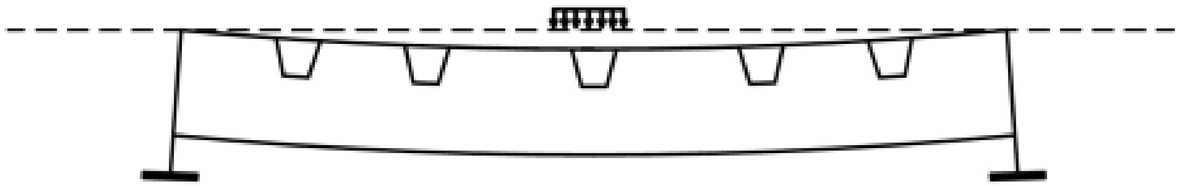


Figure 1.44. Structural system 4 (Håkansson & Wallerman, 2015)

System 5 – Floor beam distortion

Cut-outs are critical points where three different effects affecting local stresses in the floor beams occur. They include out-of-plane distortion from ribs bending, rib wall distortion due to shear forces and rib distortion due to uneven deflection as can be seen in Figure 1.45.

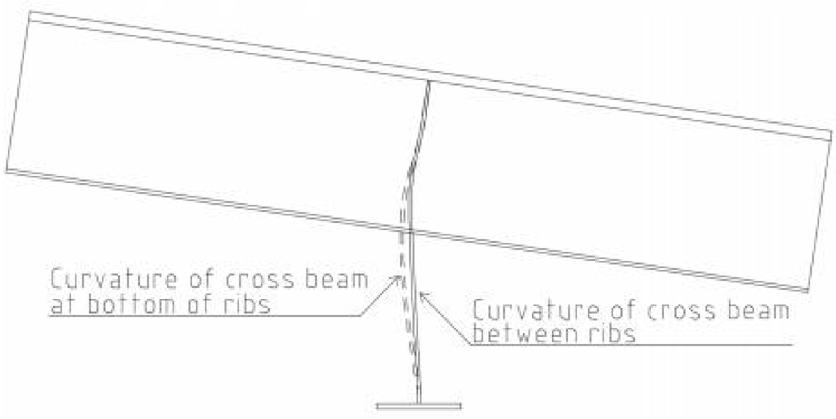


Figure 1.45. Structural system 5 (Håkansson & Wallerman, 2015)

System 6 – Rib distortion

In a closed-rib system, the rotation of the rib, when the wheel is at midspan and eccentric about the axis of the rib, causes the rib to twist about its center of rotation with consequent lateral displacement at midspan as shown in Figure 1.46.

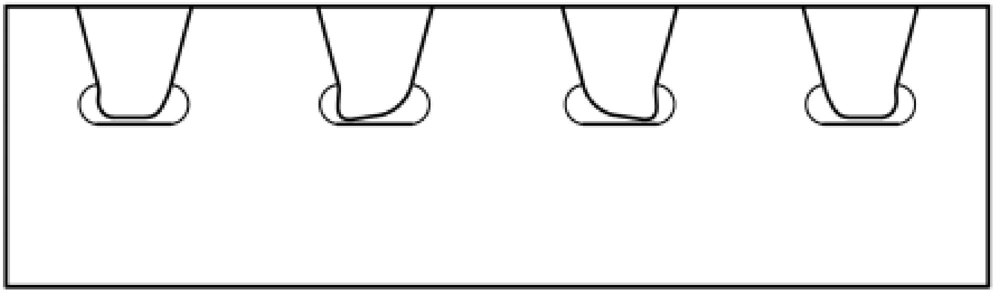


Figure 1.46. Structural system 6 (Håkansson & Wallerman, 2015)

System 7 – Global behaviour

This mechanism involves displacement and deformation of the primary girder plus orthotropic panel system as it spans between points of global support as seen in Figure 1.47. (Håkansson & Wallerman, 2015)

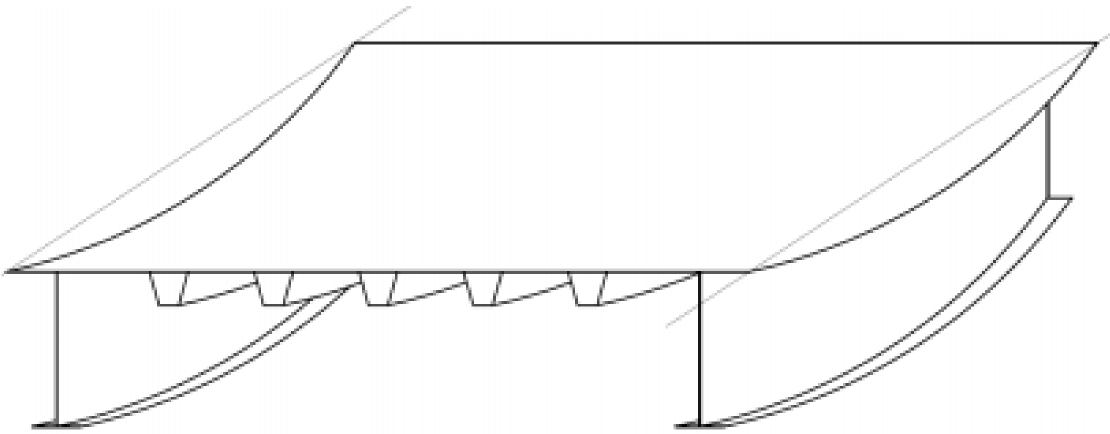


Figure 1.47. Structural system 7 (Håkansson & Wallerman, 2015)

1.5.1.4. Structural behaviour and load paths

The main structural members of the bridge condition the entire structure due to their own weight and aerodynamic unique characteristics. A good understanding of the load paths is needed, as well as the behaviour of support conditions and the interaction between the different moving elements of the movable bridge.

a- Bascule bridge

The main structural members of bascule bridges can be either trusses or girders. For purposes of simplified analysis, trunnion bascule bridges will be considered since they are mostly spread. For a single leaf bascule bridges the behaviour rolls up to a cantilever for dead loads and a simple span for live loads. In the case of double leaf bascule bridges, the behaviour rolls up to a cantilever for dead loads and for live loads depends on the type of connection made in the junction of the leaves. If the connection is made with shear locks which transfer shear forces, the girder behaves like an elastic propped cantilever and if is made with moment locks, that transfer both shear and bending moments, it behaves like a continuous girder, theoretically, because the deflections on the mid span reduce the effectiveness of the system (Anostructures, 2010).

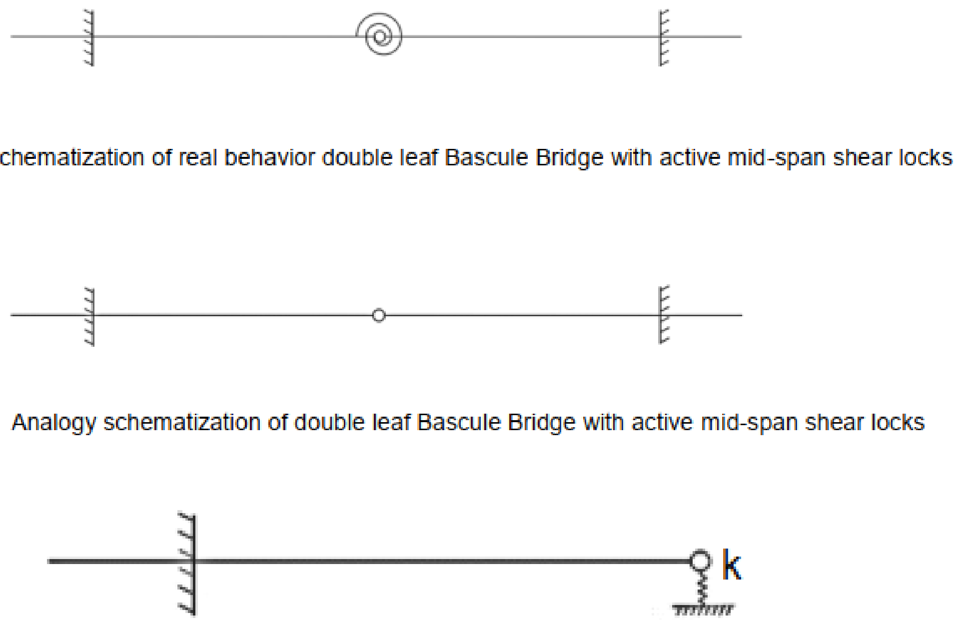


Figure 1.48. Bascule bridge structural systems (Anostructures, 2010)

b- Vertical lift bridge

The main structural members of the span of vertical lift bridges can be either trusses or girders and the main structural members of the lift towers can be trusses or reinforced concrete. The weight and both longitudinal and lateral loadings of the moving span are all supported by the lift towers and counterweights system. It is imperative that the towers offer a reasonable resistance so it can withstand all forces. The movement of each point where the span connects to the lift tower has to be synchronised in order to reduce the kinematic system to only one global movement, avoiding sway motions of the deck which is the most important aspect to consider in designing a vertical lift bridge (Anostructures, 2010).

c- Swing bridge

The main structural members of swing bridges can be also either trusses or girders. As the structural behaviour of the trusses or girders changes among the different positions of the bridge moving spans, the deflected shapes due to the self-weight are different. In open position, the weight of the span is supported by the centre bearing in centre bearing swing bridges, and by rollers in rim bearing swing bridges. In both cases the swing spans work as a double cantilever, balanced on the pivot point. In closed position, the swing span is supported by three points in centre bearing swing bridges, the centre bearing point and two rest piers, one at each side, and by four points in rim bearing swing bridges, two points in the rim bearing assembly and two rest piers. For purposes of a stiffer span and to restrain the compression chord, the common layout is the through truss which provides bracing between the two upper chords. Horizontal loading is transferred through the lateral bracing system as seen in Figure 1.49. (Anostructures, 2010).

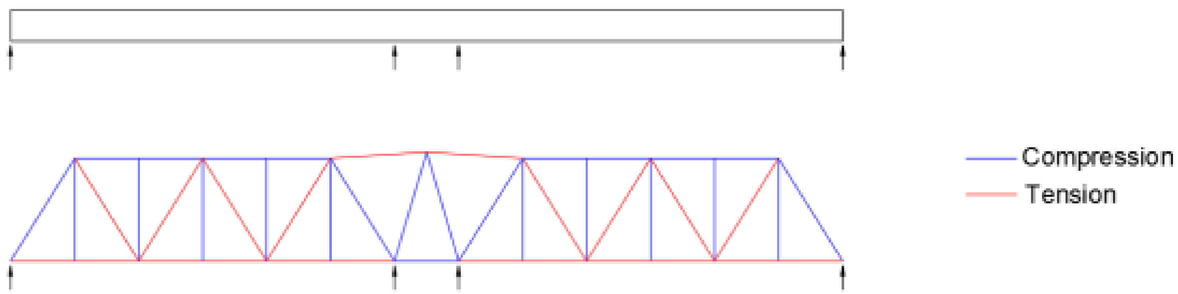


Figure 1.49. Swing bridge structural system (Anostructures, 2010)

1.5.1.5. Loads, load factors and load combinations

The loading adopted for bridge design is that specified in Eurocode 1: Actions on structures, which for bridges is quantified in part 2 (EN 1991 Part 2) and UK National Annex's to Eurocode. Nevertheless, for a movable bridge it should be taken into account the loads that are not stated in the Eurocodes.

Permanent loads

Self-weight is the basis of any design and in the case of movable bridges it has influence in the superstructure and in the operating mechanism. This is one of the loads that defines the required torque of the engine. This load is taken into account for the standard operation load of the operating mechanisms, according to the form of the superstructure. They include the loads of the structural, mechanical, hydraulic and electrical systems.

Wind load

Perhaps the most important load of all, wind causes different behaviour on the bridge when this is in open and closed position. Particularly for bascule bridges that, when raised, the entire deck area is subjected to the wind loads. Swing and vertical lift bridges are not so sensitive to this loading type.

Traffic load

Traffic load is one of the governing loads for the design of the superstructure and is designed according the Eurocodes for the closed position. For the operating mechanism this load can be neglected as there is no traffic present on the bridge during operation.

Temperature

The variation of temperatures affects directly the materials of a structure and it can cause displacements or stresses at the elements. Temperature is related with the geomorphology of each area.

Snow load

For movable bridges the combination of snow load and dead load can impose significant complications. It is designed according to the Eurocodes for the closed position.

Ship impact loads

The possible ship impact that can occur in the river will generate very large forces that would be severe to the design of the substructure. It is recommended that the movable spans are checked in the opened and closed position for a collision load from a small vessel as these may not sustain the impact forces.

Hydrodynamic loads

A hydrodynamic analysis has to be carried out in the river piers during mobile bridge design as these are subjected to river currents and tidal flows. Scour from tidal flow is a long-term effect and can become most critical than the ship impact loads, so a detailed design has to incorporate this effect.

Dynamic loads

The detailed design will have to consider the performance and dynamic loads of the bridge, due to the acceleration and deceleration of the operating mechanism. There are requirements for these loads in normal situation and in emergency case. The load introduced by this has no influence however on the force distribution between the two mechanisms.

Load factors and load combinations

Through the EN 1991-1-1 load combinations used in road bridges, these fundamental combinations of actions given by equation 1.7 are used to perform ULS safety verifications on shear and bending strength capacity of cross-sections

$$\boldsymbol{Ed} = \sum \boldsymbol{\gamma_G G_k} + \boldsymbol{\gamma_P P_k} + \boldsymbol{\gamma_Q Q_k} + \sum \boldsymbol{\gamma_Q \Psi_0 Q_k} \tag{1.7}$$

The factors applied to the loads are described in the tables below, in accordance to Tables NA. A2.1 and Tables NA. A2.4 of NA to BS EN 1990 (appended).

Actions	Ψ_0	Ψ_1	γ EQU	γ STR/GEO	γ STR/GEO
			(A)	(B)	(C)
Dead load	-	-	1.35	1.35	1.35
Traffic (gr1a or gr5) as leading	0.75	0.75	1.35	1.35	1.15
Traffic gr1 as accompanying	0.75	0.75	1.35	1.35	1.15
Wind as leading	1	1	1.7	1.7	1.45
Wind as accompanying	0.5	0.2	1.7	1.7	1.45
Snow as leading	0.8	1	1.5	1.5	1.3
Traffic (gr2), ψ already applied		-	1.35	1.35	1.15
Breaking		1	1.35	1.35	1.15

Equally, the load factors may depend on the different phases

ULS									
Phase	Self weight of steel	Self weight of slab	g2	Shrinkage	Temperature	Wind	Traffic		Predominant load
							Distributed	Tandem	
0	1.35	1.35/1.0	-	-	-	1.5	-	-	
1	From previous phase	From previous phase	1.35/1.0	-	1.5*0.6	1.5*0.6	1.35	1.35	Traffic
				-	1.5*0.6	1.5	1.35*0.4	1.35*0.75	Wind (loaded bridge)
				-	1.5*0.6	1.5	0	0	Wind (unloaded bridge)
				-	1.5	1.5*0.6	1.35*0.4	1.35*0.75	Temperature
2	From phase 0	From phase 0	1.35/1.0	1.2	From previous phase	From previous phase	From previous phase	From previous phase	Traffic
									Wind (loaded bridge)
									Wind (unloaded bridge)
									Temperature

Load combinations are considered both for the case in which the actions of the self-weight of the slab and g2 are active on the whole deck and both for the case of arrangement that maximizes the stresses in the section of interest (with maximum coefficients in unfavourable areas and minimum coefficients in favourable areas). In the case of mobile bridges, load combinations will be considered both in open and closed configurations. Having in mind that, in lifted(open) position, the dead load is favourable to the operation and it is then considered in the calculations with a favourable factor whereas wind governs as leading variable and there are no accompanying loads.

Here described are some load combinations given by the AASHTO Specifications, Load combinations for bascule and vertical lift bridge structures:

- Strength BV-I—Load combination related to structure in the open or closed position and dynamic effects of operating machinery.
- Strength BV-II—Load combination related to structure in any open position, dynamic effects of operating machinery, and wind.
- Strength BV-III—Load combination related to structure in the closed position, with live load and counterweight independently supported.

Load combinations for swing bridge structures:

- Strength S-I—Load combination related to structure in any open or closed position and dynamic effects of operating machinery.
- Strength S-II—Load combination related to live load on simple span configuration.
- Strength S-III—Load combination related to live load on continuous span configuration.
- Strength S-IV—Load combination related to structure in the open position, dynamic effects of operating machinery, and wind.
- Strength S-V—Load combination related to live load on a simple span configuration and wind.
- Strength S-VI—Load combination related to live load on a continuous span configuration and wind.

1.5.1.6. Structural analysis results

From a theoretical perspective, the primary goal of structural analysis is the computation of deformations, internal forces, stresses, support reactions, accelerations, and stability. In practice, structural analysis reveals the structural performance of the engineering design and ensures the soundness of structural integrity in design without dependence on direct testing. In other words, these results are used to verify a structure's fitness for use.

1.5.2. Structural design

It refers to a method or tool by which safe and economical specifications of a structure or a member of the structure sufficient to carry the load are found. In other words, finding out cross-sectional dimension, grade of material, amount of reinforcement, necessary to withstand the internal forces gotten from structural analysis.

1.5.2.1. Design criteria

The design of movable bridges requires much more effort than for the design of fixed bridges, for the reason that it must be taken into account the various position configurations of the leaf bridges and corresponding loads changes. So, it has to be considered two different approaches, when in closed position, which movable bridges are designed for the same design conditions and procedures as fixed bridges, and when in open position, which are designed following some specific conditions:

- inertia forces of the moving span due to acceleration and deceleration during the operation;
- frictional resistance of the machinery;
- malfunction and failure of the electro-mechanical devices;
- Impact of vessel.

In addition, there is a number of elements details and issues that have to be considered, such as the interaction of the structure and machinery, like locks, bearings and others (Anostructures, 2010).

1.5.2.2. Bridge balance and counterweights

Balance of mobile bridges is a key issue to be addressed and to answer this problem, all vertical lift and bascule bridges are counterweighted. It should be noted that wind loads can change to a direction that helps the lifting of the bridge bringing the moment close to zero at a particular angle of inclination. This brings a load reversal (from compressive to tensile load) in the lifting mechanism.

1.5.2.3. Superstructure

Superstructure represents the portion of the bridge above the piers that is the deck and its equipment, the girders and the bearings. A particular important matter in movable bridge decks is the reduction of self-weight, since it makes it easy for operation purposes of the lift machinery to be safe. As such, there are steel decks (open grid deck and exodermic and concrete filled grid deck), orthotropic steel decks, concrete decks (light-weight concrete), aluminium and FRP decks. For bascule bridges, the form of this steel orthotropic deck normally is of the deck span type, that is the main longitudinal members are underneath the deck. For swing bridges it is also possible an arrangement with a cable stayed main span. For lift bridges the solution adopted varies normally between trusses or tied arch main members. The major structural components used is steel with the purpose of minimising the weight. The surfacing used in moving spans has to be such that it minimises the weight, such as mastic asphalt or epoxy-bauxite (aluminium oxide).

For purposes of conception and initial sizing, some basic rules are acknowledged for economic strategy.

1.5.2.4. Substructure and foundation

The substructure includes the portion below the deck that is the pier, the abutments, the foundation (piles) and some counterweights. For vertical lift, swing and bascule bridges with overhead counterweight the common substructure is above the water level. For a bascule bridge with underneath counterweight, it is necessary a pit to house the counterweight when in open position. For swing and bascule bridges with overhead counterweight the common form of structural substructure would be a reinforced concrete slab above water level, supported on a raft of piles which can be driven or bored into the river bed. The material used for the substructure is reinforced concrete due to his extremely high capability of suffering high compressive stresses before any type of cracking.

Based on the information obtained by the investigations, deep foundations are likely to be suitable. Deep foundation options include a deep raft or a piled solution.

1.5.2.5. Detailing

Structural detailing can be defined as the process by which the design structural system is drafted in a form that a builder can use for construction for easy establishment. It is understood as determining the form of and the shaping and finishing of structural members and their connections. As such, for mobile bridges, it means all the detailed plans (site, structural, mechanical, hydraulic, electrical) are needed.

1.6. Control and Maintenance of mobile bridges

The greatest risk about a mobile bridge structure is the potential hazard and danger that this can bring to the various users (pedestrian, vehicle or navigational vessels). Other than the safety of the structure itself, requirements for monitoring, operation, functional safety and maintenance of the bridge and connection with the surroundings and the road are really key factors that must be considered as well.

1.6.1. Control

Table 1. 1. Control of different equipment (Koglin, 2003)

Control of different equipment making up mobile bridges are displayed in Table 1.1.

EQUIPMENT	DESCRIPTION
Safety gates and barriers	• Safety gates are of two types of gates: a warning gate and a physical barrier; • Each approach has to be provided with gates and extend across the full width of the bridge to stop access; • Powered electrically, however provision for manual operation is required, in case of malfunction or power cuts.
Traffic signals and signs	• Traffic signals have to be close to each type of safety gates; • Two types of traffic signals can be used: three colour signals and multiple red signals in vertical array, according to traffic control regulations.
Navigation signals and signs	• Navigation signals and signs have to be sited on each side of the bridge spans and have to possess suitable access; • Have to be in conform to the navigation authorities.
Lighting	• Red signal lights have to be provided on the safety gates and work simultaneously in all the operation; • Navigation lights have to be provided on each side of the bridge spans, requiring waterproofing and impact toughness.
Bells and warning devices	• Bells and lights have to be provided along with the gates and work simultaneously in all the operation; • Warning bells or gongs have to be provided with the traffic signals according with the type of operation (manual or electronical).

Control has to do with supervising the running of a process or checking the results of a survey or experiment. For newly constructed or rehabilitated movable bridges, the predominant control system in use is the programmable logic controller (PLC). This is a computer-based system that has been adapted from other industrial type applications. The PLC offers the ability to automate the operation of a bridge (Chen & Duan, 2014).

An important aspect of control not to be neglected is the notion of traffic control. Traffic control for movable railroad bridges involves interlocking the road signal system with the bridge operating controls. All the equipment described in the Table 1.1. have to follow specific and detailed requirements. These requirements classify the type of equipment and arrangements depending the type of bridge.

1.6.2. Maintenance

The proper performance of maintenance for a movable bridge includes inspection, testing, cleaning, adjustment, lubrication, and minor repairs and replacements. There are three types of maintenance in general use, and maintenance of most movable bridges is based on one of these types, or a combination thereof:

Repair Maintenance: maintenance staffs for bridges spend most of their time answering calls for emergency repairs. The rest of their time is spent waiting for these calls. Their immediate supervisors have found that being unavailable to respond immediately to emergency calls is the last position they want to be in, so, maintainers are not sent to do normal maintenance where they may be unavailable to respond to emergency calls. This type of maintenance is referred to as component failure maintenance in AASHTO Movable Bridge Inspection, Evaluation, and Maintenance Manual.

Normal Maintenance: maintenance staffs proceed to each bridge periodically, on a fixed schedule, and perform what is sometimes referred to as preventive maintenance. The maintenance crew lubricates all components according to the maintenance charts prepared for the bridge. Normal maintenance plus inspection maintenance is considered preventive maintenance in the AASHTO manual.

Inspection Maintenance: inspectors proceed to each bridge periodically, on a fixed schedule, and go over the entire bridge, checking each component on an inspection list, using inspection and testing methods included in a maintenance manual. They immediately make repairs and adjustments as needed, as called for in the manual.

1.7. Risk and safety of mobile bridges

Risk and safety analysis on mobile bridges need not to be forgotten so as to limit material and lives losses.

1.7.1. Risk

Research identified the risks associated with managing movable bridge operations, recognizing that the risks are the same whether operating locally or remotely; however, where the tender is stationed impacts how these risks are managed. Typical risks associated with movable bridge operations include:

• Life safety risk to navigation, vehicular (motorized and non-motorized, inclusive) and pedestrian users and bridge maintenance personnel during bridge operations

• Risk of delays to bridge users due to bridge inoperability or malfunction

• Risk of facility damage due to fire or unauthorized access.

Implementation of remote bridge operations inherently introduces the need for additional means to mitigate these risks when compared to local bridge operation. A remote tender must have the same abilities of a local tender in order to safely manage risk, otherwise the potential for increased incidents may occur such as:

• Increased risk of safety-related incidents to navigation, vehicular and pedestrian users due to:

- Reduction in tender visibility of the bridge and its users

- Reduction in the tender's ability to communicate with bridge users (e.g. flag signalling with mariners)

- Reduction in the ability to detect potential hazards or incidents at the bridge

• Increased risk of delays to bridge users due to bridge inoperability potentially caused by:

- Introduction of additional control equipment required to operate remotely

- Introduction of a communication link between the remote operating site and the bridge

- Reduction in the ability to detect potential maintenance needs at the bridge

- Unauthorized access/vandalism

- Delayed detection of smoke or fire conditions

• Increased risk of unauthorized operations due to introduction of remote operating equipment and communication links, such as cyber-attack (Chen & Duan, 2014).

1.7.2. Safety

Warning signs, hazard identification beacons, traffic signals, signal bells and gongs, gates and barriers, and other safety devices shall be provided for the protection of pedestrian and vehicular traffic. These shall be designed to be operative prior to the opening of the movable span and until the span has again been completely closed.

Vertical and horizontal clearances of movable bridges, in the closed and open positions, shall be subject to the approval of regulatory authorities having jurisdiction over waterway navigation.

The superstructure and substructure of movable bridges adjacent to a navigable channel shall be protected against damage from waterway traffic by suitable fenders, dolphins, or other protective devices.

Two types of gates should be provided on each approach roadway to a movable span bridge: a warning gate and a physical barrier, that is, resistance gate. They should extend across the full width of undivided roadways.

Where a pedestrian walkway is provided, either a separate gate shall be provided to block access to the span walkway, or the traffic gates may be designed to also serve this purpose.

Traffic signals shall be provided on all movable span bridges, other than manually-operated bridges. Warning bells or gongs should be provided to supplement the traffic signals. For the manually-operated spans, standard stop signs supplemented by red flags or lights may be used.

Stairways, platforms, and walks with railings and toe plates shall be provided to give safe access to the operator's house, machinery, trunnions, counterweights, lights, bridge seats, and all points requiring lubrication or other servicing.

The contract documents shall require all moving machinery parts to be painted Federal Safety colour. This shall include shafts, couplings, sides of open gears, brake wheels, crank arms, and other moving parts as applicable. The working surfaces of these parts shall not be painted (Chen & Duan, 2014).

Conclusion

The aim of this chapter was to discuss and understand the basic concepts and theories underpinning the mechanism and functionality of mobile bridges. As such the historical background, the types and features, the kinematics and mechanisms, the functionality, the structural analysis and design and the maintenance of mobile bridges were discussed. The first section described the historical background of mobile bridges between different era; moving from ancient times to modern times and last tendencies and equally stressing on the need of a mobile bridge. The second section presented the types and features of mobile bridges, starting from bascule bridges to swing bridges and passing through vertical lift bridges. Other uncommon types of mobile bridges were equally presented in this section. The third section focused on the kinematics and mechanisms of mobile bridges, stressing on the importance of kinematics in the conception of such bridges and describing the operating mechanism that helps to move the bridge in function of the bridge type. The fourth section showed how the functionality of mobile bridges is strictly related to their mechanical, hydraulic and electrical systems with the description of the latter systems. The fifth section dealt with the structural analysis and design of mobile bridges. The structural analysis sub-section detailed the different design standards specification and codes, described the structural behaviour and load paths of each bridge type, defined the load types, load factors and load combinations in function of the bridge type and explained the types of analysis performed and the expected results. The structural design sub-section on the other hand talked on the design criteria, followed by the bridge balance and counterweights which is a particular important aspect to consider in the design, described the superstructure, substructure and foundation design without forgetting the detailing aspect. The last section emphasised on the control and maintenance of mobile bridges describing the different types of maintenance done such bridges and when to be done. The two following chapters (chapters 2 and 3) will focus on the analysis of a single span bascule bridge as proposed by the case study that is subjected to wind load in particular in open condition using a computer aided program and the possible solutions that can be proposed in order to optimise such structures.

CHAPTER 2: METHODOLOGY

Introduction

National standards and specifications do not actually propose an analytical method to perform a successful and effective structural amelioration under constraints of a mobile bridge passing via a parametric analysis, thus finite element (FE) formulations can accurately be used to study the effects of various influential parameters on the ultimate strength, stresses and deflection of such bridges. In this line, this chapter aims at giving the methods used for performing a linear static analysis and a study of the most influential parameters affecting the strength, stresses and deflection of structural elements in a mobile bridge so as to ameliorate the structure. So, the chapter begins with a general site recognition done by documentary research. It is then followed by a data collection that will enable the modelling of the bascule bridge with an orthotropic steel deck. The next section defines the codes and action conditions necessary to perform the linear static analysis and equally the parametric analysis leading to the amelioration of the structure respecting constraints. The aim of this chapter is to show how influential parameters affect the linear static analysis and verification procedure of an existing mobile bridge in compliance with Eurocode 3, with the modelling and structural analysis procedure performed using the computer program Midas civil.

2.1. General site recognition

Based on documentary research of the site to be studied, the recognition of the site will be done. It allows to have the knowledge of the physical parameters like the geographical location, the climate, and the hydrology and on the other hand the socio-economic parameters.

2.2. Data collection

The data collected are the technical ones, notably the structural plans and data taking into consideration the properties of the material used on site.

2.2.1. Structural data

Structural data contains the structural plans that show the disposition of the different views and structural elements of the mobile bridge and their geometrical dimensions as shown by Figure 2.1. and described by Table 2.1. They are extracted using the software AutoCAD. These data constitute structural details and contain sections of structural elements like girders, cross-beams, longitudinal ribs and deck of the bridge.

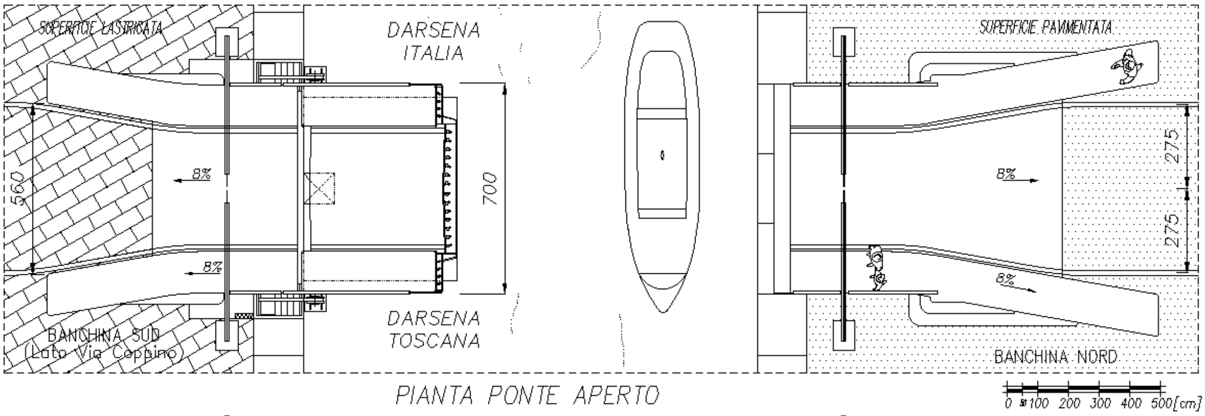
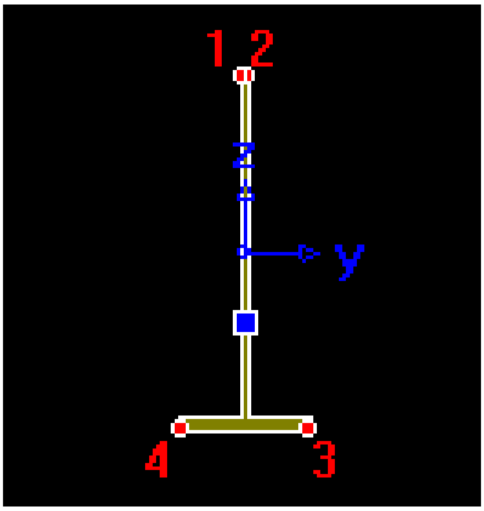


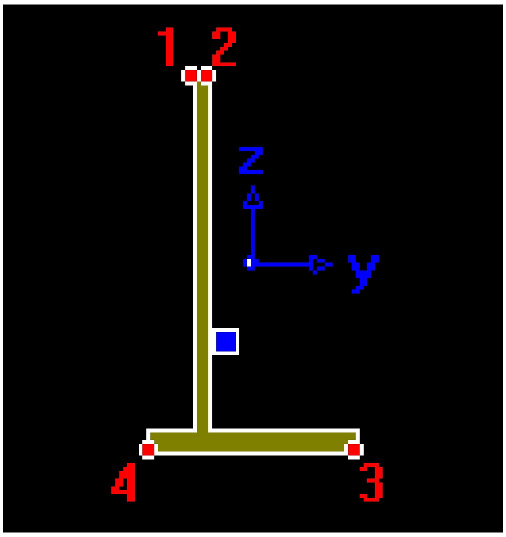
Figure 2.1. Plan view of the mobile bridge in opened state

Table 2.1. Structural elements section properties

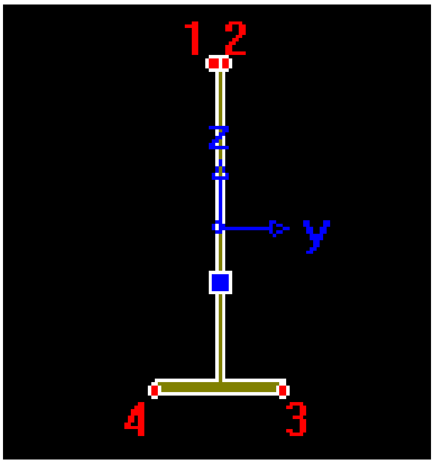
Main girder roadway		
bottom flange length(mm)	bfl	300
bottom flange thickness(mm)	bft	30
web length(mm)	wl	800
web thickness(mm)	wt	15
area(mm2)	A	21000
moment of inertia around y(mm4)	Iyy	1.53E+09
moment of inertia around z(mm4)	Izz	6.77E+07
y-coordinate of centroid(mm)	yG	150
z-coordinate of centroid(mm)	zG	252.1
section modulus around y(mm3)	Wyy	1.02E+07
section modulus around z(mm3)	Wzz	2.69E+05



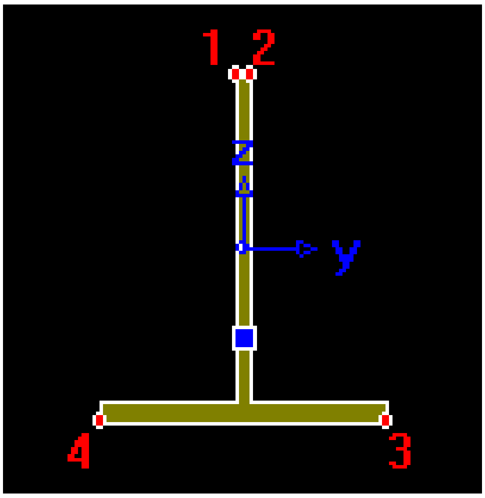
Main girder walkway		
bottom flange length(mm)	bfl	140
bottom flange thickness(mm)	bft	15
web length(mm)	wl	240
web thickness(mm)	wt	10
area(mm2)	A	4500
moment of inertia around y(mm4)	Iyy	2.98E+07
moment of inertia around z(mm4)	Izz	4.82E+06
y-coordinate of centroid(mm)	yG	51.3
z-coordinate of centroid(mm)	zG	75.5
section modulus around y(mm3)	Wyy	5.81E+05
section modulus around z(mm3)	Wzz	6.38E+04



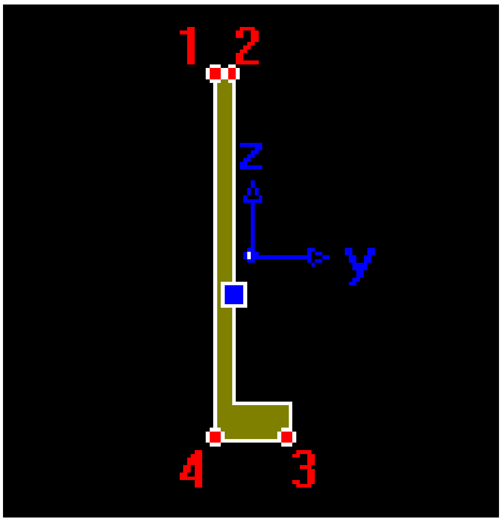
Cross beam roadway		
bottom flange length(mm)	bfl	210
bottom flange thickness(mm)	bft	20
web length(mm)	wl	518
web thickness(mm)	wt	14
area(mm2)	A	11452
moment of inertia around y(mm4)	Iyy	3.55E+08
moment of inertia around z(mm4)	Izz	1.56E+07
y-coordinate of centroid(mm)	yG	105
z-coordinate of centroid(mm)	zG	180.3
section modulus around y(mm3)	Wyy	3.38E+06
section modulus around z(mm3)	Wzz	8.65E+04



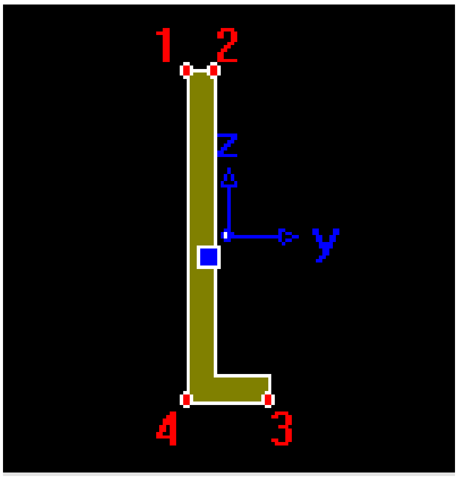
Cross beam walkway		
bottom flange length(mm)	bfl	210
bottom flange thickness(mm)	bft	15
web length(mm)	wl	240
web thickness(mm)	wt	10
area(mm2)	A	5550
moment of inertia around y(mm4)	Iyy	3.37E+07
moment of inertia around z(mm4)	Izz	1.16E+07
y-coordinate of centroid(mm)	yG	105
z-coordinate of centroid(mm)	zG	62.6
section modulus around y(mm3)	Wyy	3.21E+05
section modulus around z(mm3)	Wzz	1.85E+05



Longitudinal rib roadway		
bottom flange length(mm)	bfl	30
bottom flange thickness(mm)	bft	20
web length(mm)	wl	200
web thickness(mm)	wt	10
area(mm2)	A	2600
moment of inertia around y(mm4)	Iyy	1.04E+07
moment of inertia around z(mm4)	Izz	2.46E+05
y-coordinate of centroid(mm)	yG	9.62
z-coordinate of centroid(mm)	zG	79.2
section modulus around y(mm3)	Wyy	1.08E+06
section modulus around z(mm3)	Wzz	3.11E+03



Longitudinal rib walkway		
bottom flange length(mm)	bfl	20
bottom flange thickness(mm)	bft	10
web length(mm)	wl	120
web thickness(mm)	wt	10
area(mm2)	A	1400
moment of inertia around y(mm4)	Iyy	1.96E+06
moment of inertia around z(mm4)	Izz	5.52E+04
y-coordinate of centroid(mm)	yG	7.14
z-coordinate of centroid(mm)	zG	52.1
section modulus around y(mm3)	Wyy	2.75E+05
section modulus around z(mm3)	Wzz	1.06E+03



2.2.2. Materials characterisation

These data characterise the different materials that were used for the implementation of the structure. Some of them include the structural resistances, Young's modulus, etc as seen in Table 2.2. A good knowledge of the material properties will help to obtain the resisting forces and moments. The material properties will be those for the steel members.

Table 2.2. Characteristics of steel S355

Designation	**S 355 (NF EN 10025-2)**		Units
Characteristic Ultimate Strength	fuk	490	MPa
Characteristic Yield Strength	fyk	355	MPa
Elastic Modulus	Es	210000	MPa
ULS Safety Factor	gs	1.15	
Design Yield Strength	fyd	308.695652	MPa
Density	ρ	7850	kg/m3
Unit weight	γ	78.5	kN/m3
Shear modulus	G	80769.2308	MPa
Poisson's ratio in elastic range	ν	0.3	
Coefficient of linear thermal expansion	α	0.000012	°K^{-1}

2.3. Codes and action conditions

Actions are one of the most important things to be determined scrupulously. For this reason, the choice of design codes and standards to be implemented is equally very important. As such, this section deals with the different codes used for mobile bridges and the actions they consider. This study will focus on three main types of actions on mobile bridges. These actions are permanent, variable and accidental actions.

2.3.1. Codes

Depending on the location of the mobile bridge and government accepted standards, design codes are used for the definition of loads and their calculation. European and American codes will be used in this study and are reported in Table 2.3.

Table 2.3. Standards and codes

NORMS	TITLE
EN1990_E_2002	Basis of structural design
EN1991-1-1_E_2002	Actions on structures: General actions
EN1991-1-3_E_2003	Actions on structures: Snow loads
EN1991-1-4_E_2005	Actions on structures: Wind actions
EN1991-1-5_E_2003	Actions on structures: Thermal actions
EN1991-1-7_E_2006	Actions on structures: Accidental actions
EN1991-2_E_2003	Actions on structures: Traffic loads on bridges
EN1993-2_E_2006	Design of steel structures: Steel bridges
AASHTO LRFD part 2.4.2.	Bridge-type specific provisions

2.3.2. Permanent actions

Also known as static or dead loads, these are actions that act on the whole nominal life of the structure with a negligible variation of their intensity in time. These include the self-weight of the structural elements and the self-weight of the non-structural elements present in the nominal life of the structure but which do not take part in the load bearing mechanism.

2.3.3. Variable actions

These are actions on structures which have a consequent variation in magnitude with time. These actions include wind, traffic, snow, temperature, dynamic actions.

2.3.3.1. Wind action

Perhaps the most important load of all, wind causes different behaviour on the bridge when this is in open and closed position. In closed position it is treated like in the case of fixed bridges and differently while in open position. Particularly for bascule bridges that, when raised, the entire deck area is subjected to the wind loads. Wind load is anticipated to be critical when the bridge is opened. This will govern the size of the cylinders that will be used to operate the bridge. While the bridge is closed three different type of winds which are vertical, transverse and longitudinal, are considered in accordance to EN1991-1-4 & NA. The response of a structure to high wind pressures depends not only upon the geographical location and proximity of other obstructions to airflow but also upon the characteristics of the structure like the size, shape and dynamic properties of the structure. The general expression of a wind force Fw acting on a structure or structural member is given by expression 2.1

$$\boldsymbol{Fw = c_s c_d c_f q_p(z_e) A_{ref}} \qquad (2.1)$$

Where

Cs.Cd is the structural factor (equal to 1.0 when no dynamic response procedure is needed);

Cf is the force coefficient for the deck, the rectangular and the cylindrical pier;

$q_p(z_e)$ is the peak velocity pressure at reference height ze, which is usually taken as the height z above ground of the center of gravity of the structure subjected to wind action;

Aref is the reference area of the structure.

a. Basic wind velocity

The basic wind velocity, v_b is defined as a function of wind direction and time of year at 10 m above ground of terrain category II as shown in the expression 2.2. The fundamental value of the basic wind velocity is the characteristic 10 minutes mean wind velocity having the probability p of an annual exceedance. It is determined by multiplying the basic wind velocity by the probability factor calculated as shown by expression 2.3.

$$\boldsymbol{v_b = c_{dir} c_{season} v_{b,0}} \qquad (2.2)$$

Where

c_{dir} is the directional factor taken from the National Annex (the recommended value is 1.0);

c_{season} is the season factor taken from the National Annex (the recommended value is 1.0);

$v_{b,0}$ is the fundamental value of the basic wind velocity.

$$\boldsymbol{c_{prob} = \left(\frac{1-K\ln(-\ln(1-p))}{1-K\ln(-\ln 0.98)}\right)^n} \qquad (2.3)$$

Where

K is the shape parameter taken from the National Annex (the recommended value is 0.2);

n is the exponent taken from the National Annex (the recommended value is 0.5).

b. Mean wind velocity

The mean wind velocity at a height z above the terrain depends on the terrain roughness and orography and on the basic wind velocity as seen in the equation 2.4.

$$v_m(z) = c_r(z)c_o(z)v_b \tag{2.4}$$

Where

Cr(z) is the roughness factor taken from the National Annex;

Co(z) is the orography factor taken from the National Annex (the recommended value is 1.0).

The recommended procedure for the determination of the roughness factor at height z is based on a logarithmic velocity profile as shown in expression 2.5;

$$c_r(z) = \begin{cases} k_r \, ln\left(\frac{z}{z_0}\right) \;\; for\; z_{min} \leq z \leq z_{max} \\ c_r(z_{min}) \;\; for\; z \leq z_{min} \end{cases} \tag{2.5}$$

Where

z_0 is the roughness length that is obtained from Table 2.4.

k_r is the terrain factor which is calculated using expression 2.6

$$k_r = 0.19\left(\frac{z_0}{z_{0,II}}\right)^{0.07} \tag{2.6}$$

Table 2.4. Table of terrain categories and terrain parameters (CEN, 2005)

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.			

$$I_v(z) = \begin{cases} \frac{k_l}{c_0(z)ln(\frac{z}{z_0})} \;\; for\; z_{min} \le z \le z_{max} \\ I_v(z_{min}) \;\; for\; z \le z_{min} \end{cases} \tag{2.7}$$

Where

k_l is the turbulence factor taken from the National Annex (the recommended value is 1.0).

d. Peak velocity pressure

The peak velocity pressure at height z includes mean and short-term velocity fluctuations as seen in expression 2.8.

$$q_p(z) = \frac{1}{2}\rho v^2{}_m(z)[1 + 7I_v(z)] \tag{2.8}$$

Where

ρ is the air density taken from the National Annex (the recommended value is 1.25 kg/m3).

The exposure factor $c_e(z)$ is given by expression 2.9 and qb by expression 2.10

$$c_e(z) = \frac{q_p(z)}{q_b} \tag{2.9}$$

Where

$$q_b = \frac{1}{2}\rho v^2{}_m(z) \tag{2.10}$$

e. Force coefficients

$C_{f,x}$ is the force coefficient in X-direction (parallel to the width and perpendicular to the span). The recommended value for normal bridges is 1.3;

$C_{f,z}$ is the force coefficient in Z-direction (along the span). The recommended value is ±0,9.

$C_{f,y}$ is the force coefficient in Y-direction (perpendicular to the deck). The recommended values are 25%$C_{f,x}$ for plated bridges and 50%$C_{f,x}$ for truss bridges.

2.3.3.2. Traffic actions

Traffic loads are to be considered when the bridge is in closed configuration, because in open configuration, the vehicular traffic is halted. For traffic loads, traffic load models and group of loads are to be considered. The traffic load models are divided in vertical and horizontal forces. For vertical forces, there is LM1, LM2, LM3 and LM4 while for horizontal forces, braking and acceleration, centrifugal and transverse forces exist. Group of loads on the other hand include, gr1a, gr1b, gr2, gr3, gr4 and gr5 with characteristic, frequent and quasi-permanent values. Loads due to the road traffic, consisting of cars, lorries and special vehicles, give rise to vertical and horizontal, static and dynamic forces. The width w_l of notional lanes on a carriageway and the greatest possible whole (integer) number n_l of such lanes on this carriageway are defined in Table 2.5.

Table 2.5. Number and widths of notional lanes (Eurocode 1, 2003)

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
$5{,}4 \text{ m} \le w < 6 \text{ m}$	$n_1 = 2$	$\frac{w}{2}$	0
$6 \text{ m} \le w$	$n_1 = Int\left(\frac{w}{3}\right)$	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.			

The load models for vertical loads represent the following traffic effects:

- Load model 1 (LM1)

  Concentrated and uniformly distributed loads, which cover most of the effects of the traffic of lorries and cars. This model should be used for general and local verifications.

- Load model 2 (LM2)

A single axle load applied on specific tyre contact areas which covers the dynamic effects on the normal traffic on short structural members.

- Load model 3 (LM3)

A set of assemblies of axle loads representing special vehicles (for example for industrial transport) which can travel on routes permitted for abnormal loads. It is intended for general and local verifications.

- Load model 4 (LM4)

A crowd loading, intended only for general verifications, consisting of a uniformly distributed load (which includes dynamic amplification), typically equal to 5kN/m^2.

Load Model 1 consists of two partial systems described in Table 2.6.:

- Double-axle concentrated loads (tandem system: TS): each axle having the weight $\alpha_Q Q_k$, where α_Q are adjustment factors.

Uniformly distributed loads (UDL system): having as weight per square metre of notional lane $\alpha_q q_k$, where α_q are adjustment factors

Table 2.6. LM1: characteristic values and figure of application of LM1 (Eurocode 1, 2003)

Location	Tandem system *TS*	*UDL* system
	Axle loads Q_{ik} (kN)	q_{ik} (or q_{ik}) (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

Figure 2.2. shows how the LM1 can be applied for the analysis of bridges

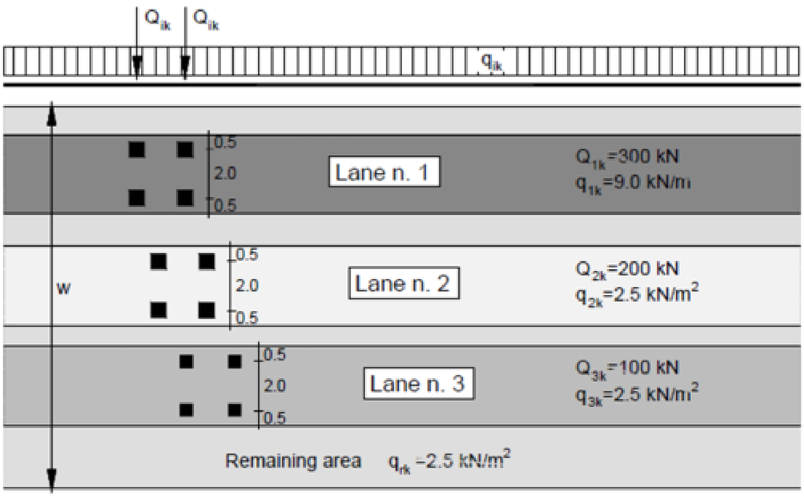


Figure 2. 2. Application of LM1 (Eurocode 1, 2003)

For the characteristic value of horizontal loads, only braking and acceleration forces will be considered. A braking or acceleration force, Q_{lk}, shall be taken as a longitudinal force acting at the surfacing level of the carriageway. The characteristic value should be calculated as a fraction of the total maximum vertical loads corresponding to LM1 and applied on lane number 1, as seen in the expression 2.11.

$$Q_{lk} = 0,6\alpha_{Q1}(2Q_{1k}) + 0,1\alpha_{q1}q_{1k}w_1L \quad (2.11)$$

with $180\alpha_{Q1}KN \leq Q_{lk} \leq 900KN$ as shown in Figure 2.2., $\alpha_{Q1} = \alpha_{q1} = 1$

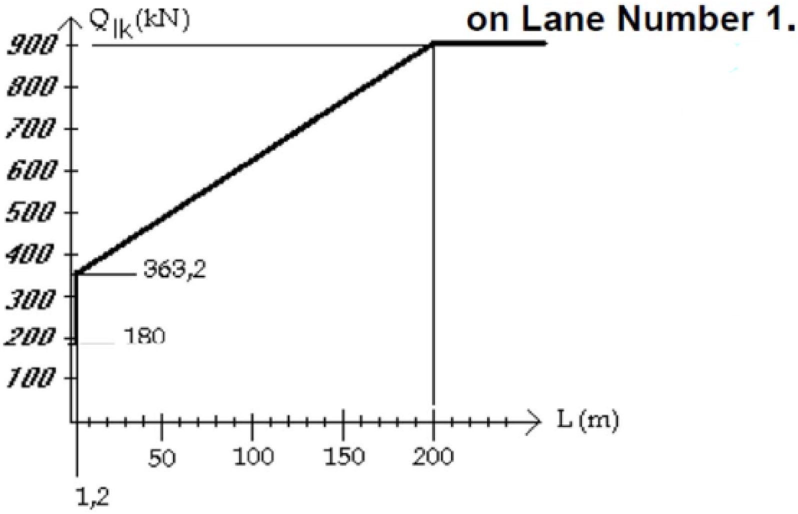


Figure 2.3. Graph to show the braking force's interval (Eurocode 1, 2003)

Equally, Q_{1k} can be gotten from the expressions 2.12

$$\boldsymbol{Q_{1k}} = \begin{cases} \boldsymbol{180 + 2,7L \; for \; 0 \leq L \leq 1,2m} \\ \boldsymbol{360 + 2,7L \; for \; L > 1,2m} \end{cases} \tag{2.12}$$

with L = length of the deck or of the part of the deck under consideration.

The simultaneity of the loading systems (vertical loads, horizontal forces, loads for footways) should be taken into account by considering the groups of loads defined in Table 2.7.

Table 2.7. Assessment of groups of traffic loads (Eurocode 1, 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	Centrifugal and transverse forces	Uniformly Distributed load
Groups of Loads	gr1a	Characteristic values				a	a	Combination value b
	gr1b		Characteristic value					
	gr2	Frequent values b				Characteristic value	Characteristic value	
	gr3 d							Characteristic value c
	Gr4				Characteristic value			Characteristic value b
	Gr5	See annex A		Characteristic value				
	Dominant component action (designated as component associated with the group)							

[a] May be defined in the National Annex.
[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.

For this work, group loads gr1a (LM1+combination value of pedestrian load on footways or cycle tracks) and gr2 (characteristic values of horizontal forces, frequent values of LM1) will be taken into account.

2.3.3.3. Snow actions

For movable bridges the combination of snow load and dead load can impose significant complications. It is designed according to the Eurocode 1 for the closed position. As such the relationship used to determine the design value of the snow load is given in expression 2.13.

$$s_d = \gamma_Q s_k \tag{2.13}$$

Where S_k is the characteristic snow load on the ground in kN/m^2 of the concerned area gotten from expression 2.14. Italy is found in the Mediterranean region of Europe as such, EN1991-1-3 gives the formula to calculate the characteristic value of snow load as

$$s_k = (0,498Z - 0,209)\left[1 + (\frac{A}{452})^2\right] \tag{2.14}$$

Where A is the site altitude above sea level in metres and Z is the zone number given on the map depicted in Figure 2.4. For the region of interest Z=2.

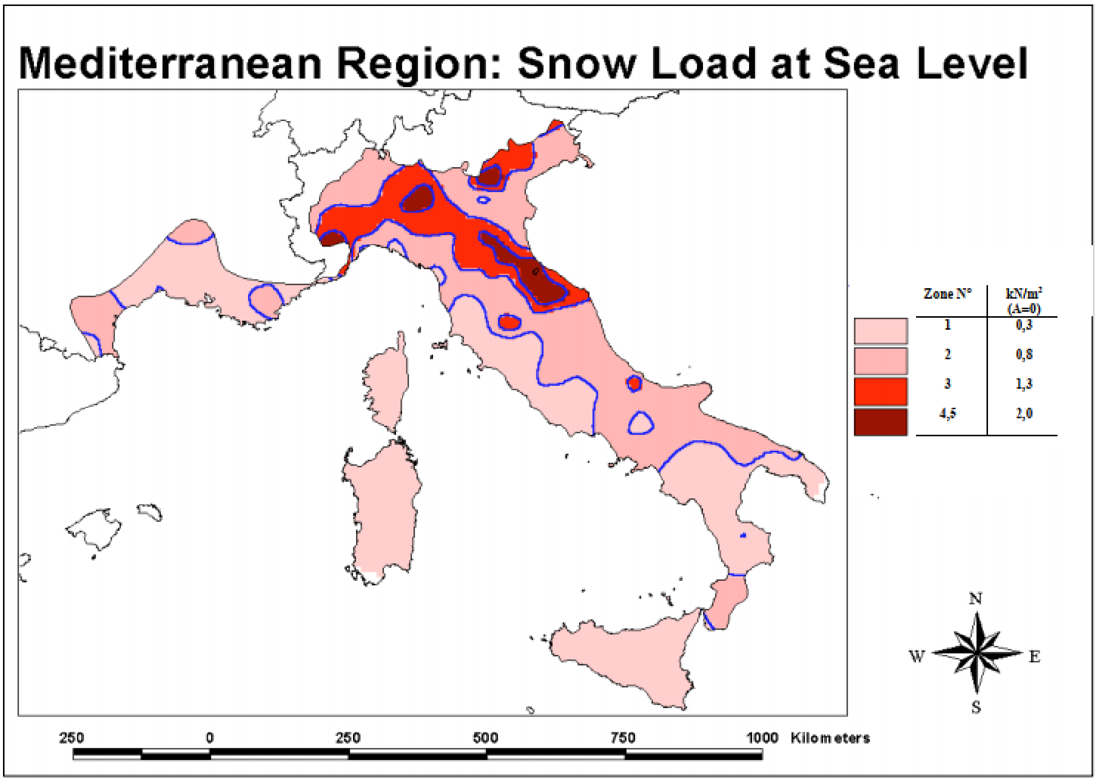


Figure 2.4. Map of climatic Mediterranean region (EN1991, 2003)

2.3.3.4. Temperature

The action of temperature increase or decrease on steel structures is characterised by expansion or contraction respectively. Consequently, for bridges, thermal variations can be uniform or differential. In this study, the uniform thermal variation will be considered since the movable bridge is entirely made up of steel. The allocated value according to EN1991 is $\pm 18^0$C. As such, the expansion or contraction value will be necessary to determine the size of thermal end joints; allowing for free deformation of the deck thus additional stresses. These values can be obtained as prescribed by the equation 2.15.

$$\Delta L = L_{TOT} . \Delta T . \alpha \tag{2.15}$$

Where,

L_{TOT} is the bridge length, α is the coefficient of thermal expansion of the material considered, ΔT is the uniform thermal variation and ΔL is the expansion or contraction.

2.3.3.5. Dynamic action

The analysis will have to consider the performance and dynamics loads of the bridge, due to the acceleration and deceleration of the traffic loads. There are requirements for these loads in normal situation and in emergency case. The load introduced by this has however a reduced influence on the force distribution between the two mechanisms.

2.3.4. Accidental actions

These loads are usually of short duration but of significant magnitude, that are unlikely to occur on a given structure during the design working life. An accidental action can be expected in many cases to cause severe consequences unless appropriate measures are taken. These loads include fire, explosions, impact from water vessels etc. In this work, the accidental load that will be taking into consideration is the ship impact load. The possible ship impact that can occur in this part of the river will generate very large forces that would be severe to the analysis of the substructure. It is recommended that the movable span is checked in the opened and closed position for a collision load from a small vessel as these may not sustain the impact forces. According to EN1991-1-1-7, accidental actions due to collisions from ships should be determined taking account of type of waterway, flood conditions, draught of vessels and type of structures. The action due to impact should be represented by two mutually exclusive forces that is a frontal force Fdx and a lateral force with a component Fdy acting perpendicularly to the frontal impact force and a friction component FR parallel to Fdx. Expression 2.16 helps calculate the value of FR as follows

$$F_R = \mu F_{dy} \tag{2.16}$$

Where, μ is the friction coefficient (the recommended value is 0,4).

The height of application of the impact force and the impact area may be defined in the National Annex. In the absence of detailed information, the force may be applied at a height of 1,5 m above the relevant water level. The impact area should be equal to b x h (bpier is the width of the obstacle in the waterway).

For frontal impact, b = bpier and h = 0,5m

For lateral impact, b = 0,5m and h = 1,0m.

2.3.5. Limit states

A structure is 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 limit states).

2.3.5.1. Ultimate limit states

According to EN 1990 (2002), ultimate limit states (ULS) correspond to the loss of structural capacity of the whole structure or one of its fundamental elements (for example structural collapse). It concerns the safety of users and/or the safety of the structure. The loss of structural capacity includes:

- Loss of equilibrium of the whole structure or one of its fundamental parts;
- Excessive displacements or deformations;
- Reaching of the maximum strength capacity of parts of structures, joints, foundations;
- Reaching of the maximum strength capacity of the entire structure;
- Reaching of failure mechanisms in the soils.

2.3.5.2. Serviceability limit states

Serviceability limit state (SLS) is the inability of the structure to meet the specify service requirement. This includes mainly:

- Functioning of the structure or structural members under normal use;
- Comfort of users;
- The appearance of the construction works.

2.3.6. Load combinations

A load combination defines a set of values used for the verification of the structural reliability of a structure for a limit state under the simultaneous influence of different loads. For the verification of the structure at ultimate limit state, the fundamental combination and the accidental combination will be used, with expressions 2.17 and 2.18 describing each combination.

ULS1(fundamental combination): $\sum \gamma_G G_k + \gamma_P P_k + \gamma_{Q,1} Q_{k,1} + \sum \gamma_{Q,i} \Psi_0 Q_{k,i}$ (2.17)

ULS2(accidental combination): $\sum G_k + P_k + A_d + \left(\Psi_{1,1} or \Psi_{2,1}\right) Q_{k,1} + \sum \Psi_{2,i} Q_{k,i}$ (2.18)

Where:

G_k are the permanent loads

$Q_{k,1}$ is the leading variable load

$Q_{k,i}$ are the accompanying variable loads

A_d is the accidental load

$\gamma_{i,j}$ is the safety factor for permanent and variable loads

$\Psi_{i,j}$ are the combination coefficients

The choice between Ψ1,1 or Ψ2,1 should be related to the relevant accidental design situation (impact, fire or survival after an accidental event or situation).

A load envelope is obtained from ULS combinations to have the most unfavourable condition for an element.

For the verification of the structure at serviceability limit state, the load combinations used are given in the expressions 2.19, 2.20, 2.21.

SLS1(characteristic combination): $\sum \boldsymbol{G_k} + \boldsymbol{P} + \boldsymbol{Q_{k,1}} + \sum \boldsymbol{\Psi_{0,i} Q_{k,i}}$ (2.19)

SLS2(frequent combination): $\sum \boldsymbol{G_k} + \boldsymbol{P} + \boldsymbol{\Psi_{1,1} Q_{k,1}} + \sum \boldsymbol{\Psi_{2,i} Q_{k,i}}$ (2.20)

SLS3(quasi-permanent combination): $\sum \boldsymbol{G_k} + \boldsymbol{P} + \sum \boldsymbol{\Psi_{2,i} Q_{k,i}}$ (2.21)

There are other combinations resulting from traffic loads and due to the fact that the bridge has a closed and an opened configuration. Notably, in lifted position (opened position) wind governs as leading variable load and there are no accompanying loads.

2.4. Structural analysis

As said in chapter one, structural analysis defines a sequence of procedures used to determine the effects of design actions on structures. The analysis type to be performed for the assessment of the mobile bridge is the linear static analysis in both opened and closed configurations. In this analysis type, a linear relation holds between applied forces and displacements and it is applicable to structural problems where stresses remain in the linear elastic range of the used material. In closed configuration, the mobile bridge will be analysed like a fixed bridge with the necessary support and constraint conditions. In opened configuration, opening angles will be varied and the bridge will be analysed using adequate constraints. To ease structural analysis, numerical programs equipped with finite element analysis are used in the engineering world, thus will consequently be used.

2.4.1. Numerical static analysis tools

Numerical modelling in civil engineering is used as a tool that eases engineering work in the evaluation of the behaviour of structures. In line with this, the software used for the structural analysis of the bridge will be Midas civil.

2.4.1.1. Presentation of the numerical software

Midas civil is an adequate program used for the analysis of bridge structures since it uses the concept of finite element modelling and analysis.

Midas civil

Midas civil is a bridge design and analysis software that combines powerful pre-and post-processing features with an extremely fast solver, which makes bridge modelling and analysis simple, quick, and effective. It enables creation of nodes and elements as if drawings were made using the major functions of CAD programs. It provides linear and nonlinear structural analysis capabilities and is capable of handling different types of analysis notably construction stage analysis, heat of hydration analysis, moving load analysis, modal analysis and many more. The program's efficient analysis algorithms yield exceptional versatility and accurate results appropriate for practical design applications. Midas civil also provides results for different structures like reinforced concrete structures, metallic frame structures and other types that are compatible with MS Excel, which enables the user to review all analysis and design results systematically.

2.4.1.2. Analysis procedure

Modelling procedure on structural analysis computer software is generally similar. It begins with the definition of material and section properties. Defining material properties means talking of the material that makes up the structure, in our case steel and defining its Young' modulus, poisson's ratio, shear modulus and others. On the other hand, defining section properties deals with the definition of geometry of structural and non-structural elements. After defining the materials and sections, load cases and load combinations are defined. The different loads acting on the bridge structure are assessed by hand calculations and the values are implemented on the model. Load combinations at ULS and SLS are defined in opened and closed configurations as discussed in section 2.3.6. After this, constraints and support conditions are ideally defined. The bridge structure will be loaded and a static linear analysis will be done to obtain the solicitations and deformations. Parameters will then be defined to perform a parametric analysis so as to optimise the sections of structural elements in compliance with the norms defined.

2.4.2. Analysis of the bridge in closed configuration

In closed configuration, a mobile bridge is analysed as a simply supported bridge, since the leading variable action is the traffic. As such, the single span bascule bridge in closed configuration is brought to a simply supported beam with varying constraint conditions. As such the orthotropic deck will be modelled taking into account support conditions.

2.4.2.1. Preliminary calculations

Initial calculations are done so as to perform a preliminary design. Preliminary design is done on the basis of standards and norms. In this case, the obtained data, that is geometrical and material, will serve as preliminary design for the analysis.

2.4.2.2. Structural models

Constraint and support conditions on a structural element greatly define the nature of the internal forces acting in the element. As such adequate support and constraint conditions must be chosen in order to perform the structural analysis of the bridge in closed configuration. As said above, a mobile bridge in closed configuration acts as a simply supported bridge. Thus, the first support condition is a simply supported span or plate. Others constraint conditions, like a double hinged span and a fixed span are considered. Figure 2.5. describes the support condition that will be used for the analysis. Equally a propped cantilever beam model can be used for the analysis of a closed single span bascule bridge.

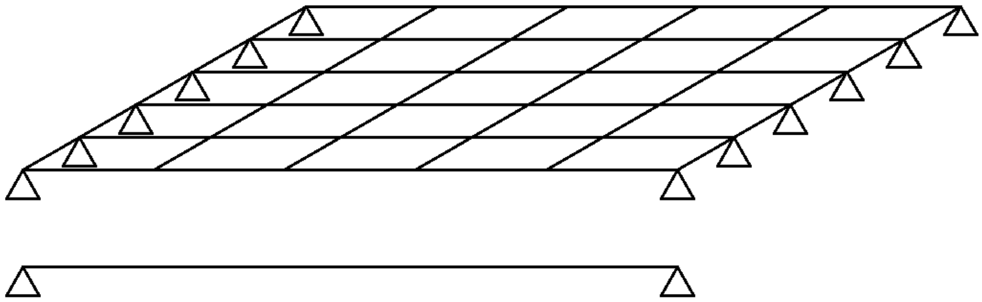


Figure 2.5. Bridge in closed configuration

2.4.3. Analysis of the bridge in opened configuration

In this case, the bridge is opened at a certain angle with respect to the water level and equilibrium is maintained by a shear and a couple at the rotating or trunnion end. In trunnion bascules, it is important that the trunnion reactions be thoroughly analysed. The reactions for the design of the hydraulic cylinders are equally very important and not to be forgotten. During the movement of the span the dead-load stresses are constantly changing, and in the case of some members they may pass through a maximum greater than those existing when the leaf is either closed or open. The leading variable action in such configuration is the wind action, so varying the opening angles will help define a chart of internal load reversals. Thus, defining the adequate structural models is a very important part in the analysis of a mobile bridge in the opened configuration.

2.4.3.1. Structural models

In opened configuration, the static scheme used for the analysis of a mobile bridge is that of a guided built-in support at the base and/or the presence of a simple support along the length of the span. The guided built-in support is a special fixed support which ensures the equilibrium of the leaf in the opened position. The simple support along the length represents one which will be used to design the hydraulic cylinders. The use of the built-in support equally defines the presence of moment as internal force at the rotating end; a stress that will be used to design the latter end. Figure 2.6. shows the static scheme that will be used for the analysis of the mobile bridge in the opened state.

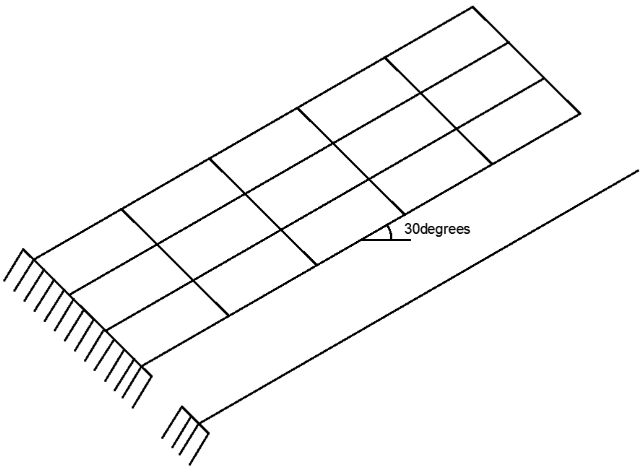


Figure 2.6. Bridge in open configuration

2.5. Parametric analysis and structural amelioration

Also known as a sensitivity analysis, a parametric is the study of the influence of different geometric or physical parameters or both on the solution of a problem. Many different parameters may influence the solicitations and even ultimate strength of structural elements in a mobile bridge. During their studies, researchers have focused on the angle of opening, the operating mechanism, the loading conditions, the type of steel used, as they are the most influential ones. Based on the finite-element model that will be made and simulations performed, a parametric analysis will systematically be conducted to analyse the interested properties of the structural elements making up the mobile bridge.

Structural amelioration on the other hand, is concerned with finding a better shape and size of a structure or of its parts which has minimum weight/volume or minimum manufacturing cost, satisfying all design constraints. This has the benefits of saving resources as well as improving the safety and efficiency of the structure.

2.5.1. Parametric analysis

The parameters under study are the most influential ones as said in the introductory part of section 2.5. For this case, the parameters that will be used are the opening angle, the drive mechanism and the steel type.

The opening angles with respect to the horizontal water level will be varied, starting from the closed configuration and incrementing by 15 degrees each time and solicitations will be studied to assess load reversals. Analysis will equally be done at 72 degrees which is the actual maximum opening angle.

The actual drive mechanism uses two hydraulic cylinders; thus, this mechanism will be changed to a mechanical and analysis performed to assess differences in solicitations and stresses.

Changing the tensile strength of the steel used leads to a change of section hence a change in the weight of structural elements. Steel type S450 will be used and analysis performed.

2.5.2. Structural amelioration

Amelioration here, will be done on a geometrical and inertial point of view. The solicitations and stresses from the linear static analysis and parametric analysis will be used to check on a high possibility of geometrical and inertial properties amelioration.

2.5.3. Amelioration constraints

The system as a whole is subjected to strength requirements, stresses and deflection constraints. Deflection in function of the configuration is set by the different Eurocodes.

2.5.4. Amelioration algorithm

The first step consists in setting up the amelioration parameters as input. They include, geometry, loading, deflection requirements, design code, section database effective length of each member calculated as shown in Annex B3. The second step is making sure strength requirements are satisfied. It is done according to the design code, with all load cases and load combinations considered. The third step is concerned with reducing stresses and deflections with stresses and deflection constraints tested at critical nodes as defined earlier. The fourth step deals with adjusting member sections. Members are replaced simultaneously by suitable sections from a user defined database. The fifth step is to check out that the strength, stresses and deflection constraints are once more verified. Figure 2.7. describes the amelioration algorithm.

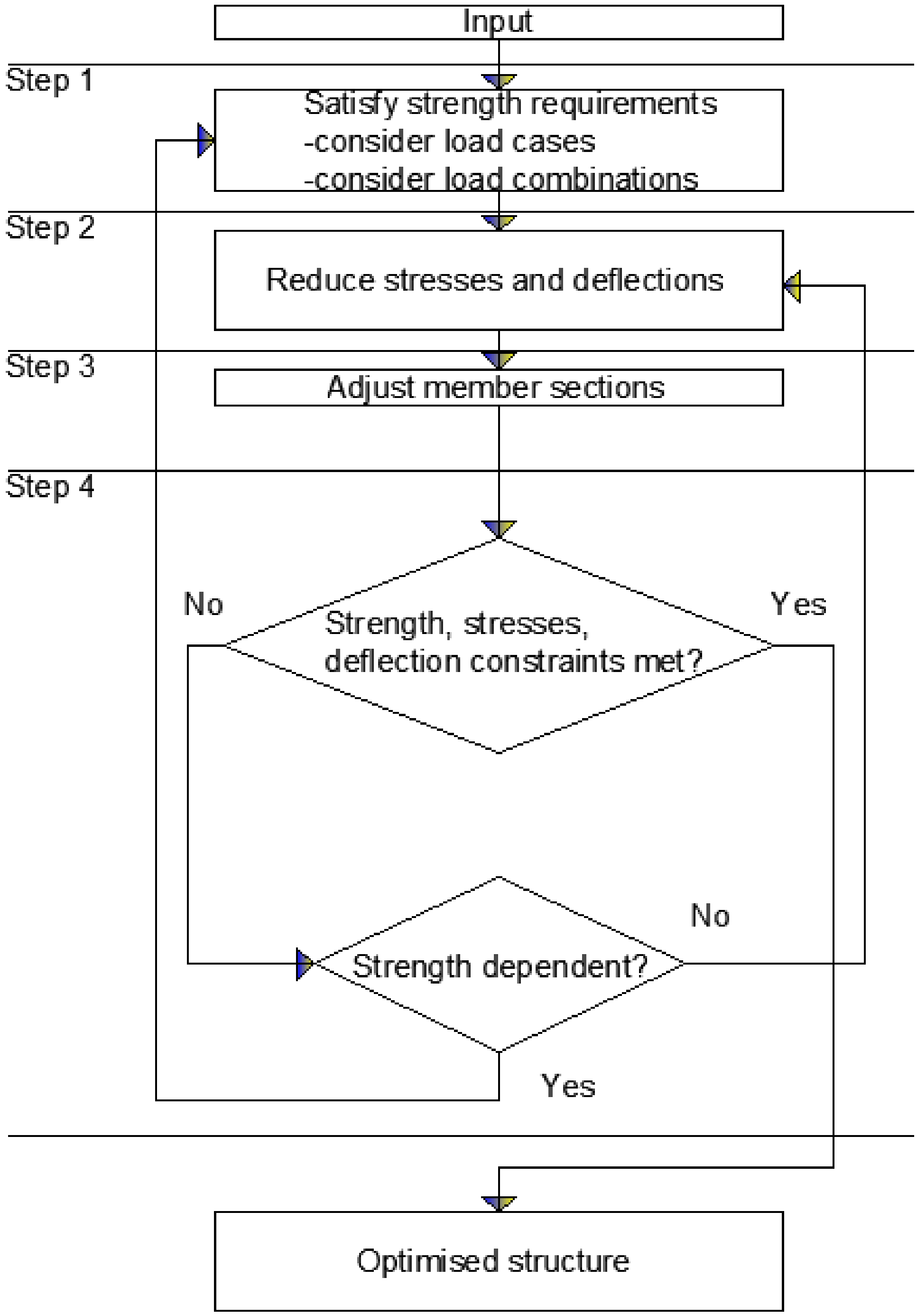


Figure 2.7. Amelioration algorithm

For strength requirements to be satisfied, classification of section, bending, shear and axial verifications of structural elements need to be done.

2.5.4.1. Classification of sections

The classification of a section depends on geometric characteristics. The sections of the members to be design are going to be classified as class 1, 2, 3, or 4. Eurocode 3 defines these classes as follows and are depicted in Annex B:

- 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 fibre 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.

Effective width models for direct stresses, resistance models for shear buckling and buckling due to transverse loads as well as interactions between these models for determining the resistance of uniform members at the ultimate limit state will be used.

2.5.4.2. Members in flexure

The ULS design verification procedure of the members in bending will take into considerations uniaxial bending, shear resistance and lateral torsional buckling.

a. Uniaxial bending

Member verification for uniaxial bending should be performed as prescribed by equation 2.22

$$\boldsymbol{\eta_1} = \frac{N_{Ed}}{\frac{f_y A_{eff}}{\gamma_{M0}}} + \frac{M_{Ed} + N_{Ed} e_N}{\frac{f_y W_{eff}}{\gamma_{M0}}} \leq \mathbf{1.0} \tag{2.22}$$

Where :

A_{eff} is the effective cross-section area;

e_N is the shift in the position of the neutral axis;

M_{Ed} is the design bending moment;

N_{Ed} is the design axial force;

f_y is the design axial force;

W_{eff} is the effective elastic section modulus;

γ_{M0} is the partial factor.

The main girder uses the deck plate as its top flange. Having established the effective cross section of the main girder, the main girder can be analysed for bending by calculating the normal stress using Navier's equation, which is compared to the yield strength. As for the main girders the stiffeners need to be checked for bending. Navier's formula is given by expression 2.23

$$\sigma_z = \frac{N}{A} + \frac{M}{I} y \tag{2.23}$$

Deflections are equal verified. This is done by taking the deflection values from the software analysis of the numerical model and comparing them to the limiting values prescribed by the Eurocode 3.

Stress verifications at SLS are equally needed, to make sure that the stresses in element are less than the values prescribed by the codes. Expression 2.24 gives the prescription to be followed

$$\sigma_{Ed,ser} \leq \frac{f_y}{\gamma_{M,ser}} \tag{2.24}$$

Where the recommended value for $\gamma_{M,ser}$ according to the EN1993-2 is 1.00.

b. Shear resistance

For plates with an unstiffened web, resistance to shear buckling should be checked according to expression 2.25 and transverse stiffeners should be provided at supports if

$$\frac{h_w}{t} \geq \frac{72}{\eta} \varepsilon \quad where\ \varepsilon = \sqrt{\frac{235}{f_y[N/mm^2]}} \tag{2.25}$$

Where

h_w is the height of the web;

t is the thickness of the web.

According to EN 1993-1-5 (2006), the value $\eta = 1.20$ is recommended for steel grades up to and including S460. For higher steel grades $\eta = 1.00$ is recommended.

As such, for unstiffened or stiffened webs the design resistance for shear should be taken as given by expression 2.26

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_w t}{\sqrt{3}\gamma_{M1}} \tag{2.26}$$

in which $V_{bf,Rd}$ is the contribution from the flange and the contribution from the web is given by expression 2.27

$$\boldsymbol{V_{bw,Rd}} = \frac{\chi_w f_{yw} h_w t}{\sqrt{3}\gamma_{M1}} \tag{2.27}$$

χ_w which represents the contribution of the web to the shear buckling resistance should be obtained from Table 2.8.

Table 2.8. Assessment of shear buckling factor (Eurocode 3, 2003)

	Rigid end post	Non-rigid end post
$\bar{\lambda}_w < 0{,}83/\eta$	η	η
$0{,}83/\eta \le \bar{\lambda}_w < 1{,}08$	$0{,}83/\bar{\lambda}_w$	$0{,}83/\bar{\lambda}_w$
$\bar{\lambda}_w \ge 1{,}08$	$1{,}37/(0{,}7+\bar{\lambda}_w)$	$0{,}83/\bar{\lambda}_w$

with the slenderness parameter $\overline{\lambda_w}$ calculated from expression 2.28

$$\overline{\boldsymbol{\lambda_w}} = \frac{h_w}{37.4\, t\, \varepsilon \sqrt{k_\tau}} \quad \text{where } k_\tau \text{ is taken from Annex A of EN1993-1-5} \tag{2.28}$$

When the flange resistance is not completely utilized in resisting the bending moment (M_{Ed} < $M_{f,Rd}$) the contribution from the flanges should be obtained as prescribed by expression 2.29

$$\boldsymbol{V_{bf,Rd}} = \frac{b_f t_f^2 f_{yf}}{c\gamma_{M1}}\left[1 - \left(\frac{M_{Ed}}{M_{f.Rd}}\right)^2\right] \tag{2.29}$$

With the coefficient c obtained from the expression 2.30

$$\boldsymbol{c = a\left(0.25 + \frac{1.6\, b_f\, t_f^2\, f_{yf}}{t\, h_w^2\, f_{yw}}\right)} \tag{2.30}$$

b_f and t_f are taken for the flange which provides the least axial resistance, equally b_f being taken as not larger than $15\varepsilon t_f$ on each side of the web.

The moment of resistance of the cross section consisting of the effective area of the flanges $M_{f,Rd}$ is obtained by equation 2.31

$$\boldsymbol{M_{f,Rd}} = \frac{M_{f,k}}{\gamma_{M0}} \tag{2.31}$$

When an axial force N_{Ed} is present, the value of $M_{f,Rd}$ should be reduced by multiplying it by the factor given by expression 2.32

$$\left(1-\frac{N_{Ed}}{\frac{(A_{f1}+A_{f2})f_{yf}}{\gamma_{M0}}}\right) \tag{2.32}$$

where A_{f1} and A_{f2} are the areas of the top and bottom flanges respectively.
The verification should then be performed as prescribed by expression 2.33

$$\eta_3=\frac{V_{Ed}}{V_{b,Rd}}\leq 1.0 \tag{2.33}$$

where V_{Ed} is the design shear force.

2.5.4.3. Members in compression

For members in compression, the plate is verified at ULS for buckling resistance. One of the main issues when designing steel structures is buckling. When a plate is slender, there is a risk of buckling to occur. The post-buckling or post-critical behaviour of a plate is described by a considerable load carrying capacity after buckling. So for verification, the stresses in the plate should be compared to the critical stress, which for a plate that is supported on two edges is calculated like that of column as shown by equation 2.34

$$\sigma_{cr}=\frac{\pi^2 E}{12(1-\upsilon^2)(\frac{a}{t})^2} \tag{2.34}$$

Where
a = critical length of the plate
t = thickness of the plate
The elastic critical plate buckling stress of the equivalent orthotropic plate is obtained from expression 2.35

$$\sigma_{cr,p}=k_{\sigma,p}\sigma_E \text{ where } \sigma_E=\frac{\pi^2 E t^2}{12(1-\upsilon^2)b^2}=190000\left(\frac{t}{b}\right)^2 \tag{2.35}$$

Where
$k_{\sigma,p}$ is the buckling coefficient according to orthotropic plate theory with the stiffeners smeared over the plate which is obtained from Annex A.1 of EN1993-1-5;
b is total width of the bridge.
Check for web breathing must equally be considered via the formulation given by expression 2.36

$$\frac{bw}{tw} \leq 3.0 + 4.0 * L \leq 300 \tag{2.36}$$

Where

L is the length of the span

In the effective width method, the bending moment and shear forces are mainly carried by one part of the cross section (flange or web). Thus, the critical buckling stress is set equal to the yield strength.

For the deformability indices, it is checked that the deformations induced by the permanent loads are contained within the value of L/300, even if they provide suitable workshop countermounts and that the deformations for the mobile loads are contained within the limits of L/500.

Conclusion

A successful structural amelioration passes through a thorough parametric study and the correct implementation of the necessary verification procedures described in the chapter. The aim of this chapter was to give detailed step by step procedures used all through the thesis work. Accordingly, the chapter started by recognising the site through physical and geographical parameters. Subsequently, the procedure for collecting data, assessing codes and standards used in this study, the different actions and actions combinations that will be used in the structural verification of the case study and presenting the different analysis tools that were used for the work was detailed. After that, the different analysis steps made using ideal modelling tools in order to statically verify the structure, analyse the behaviour of a mobile bridge with an orthotropic steel deck under wind action as a leading variable action. Later on, the steps for a parametric analysis followed by a structural amelioration by varying the strength of steel were presented. The chapter ended with the assessment of the mass percentage after amelioration.

CHAPTER 3: PRESENTATION AND INTERPRETATION OF RESULTS

Introduction

The third chapter presents the results of the step-by-step procedure detailed in chapter two. It begins presenting the results of the research on the site where the case study is found, the data collected and the results of the linear static analysis of the elements of the OSD for the mobile bridge in closed and opened configurations. Subsequently, it presents the results of the parametric analysis on structural elements by varying the angle of opening and operating mechanisms, leading to the presentation of the geometric and inertial amelioration results.

3.1. General site presentation

The mobile bridge to be studied is found in the harbour of the city of Viareggio in Italy. This part is reserved to a general presentation of Viareggio, presenting its physical and socio-economical parameters, which can influence the design of the structure.

3.1.1. Physical parameters

Here, parameters like the geographical location, the climate, the hydrology and the topography of the city will be presented.

3.1.1.1. Geographical location

Viareggio, a seaside city in northern Tuscany, Italy, on the coast of the Tyrrhenian Sea with geographical coordinates, latitude 43° 51′ 57.85″ N and longitude 10° 14′ 59.35″ E. The case study is located in the ship industrial zone particularly in the port of Viareggio as shown in Figure 3.1.

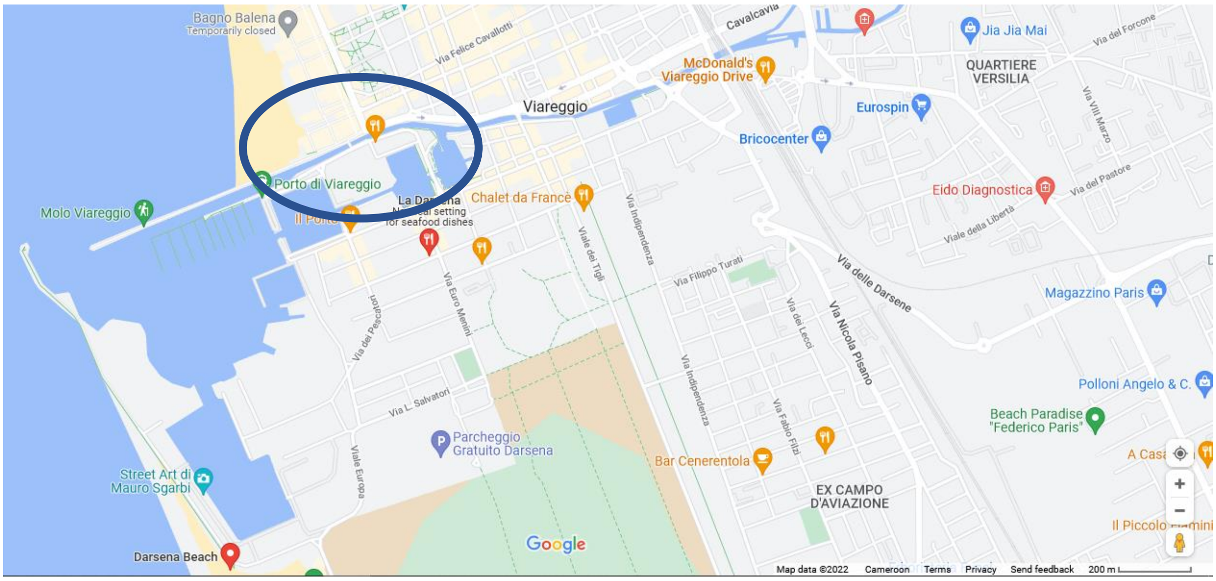


Figure 3.1. Geographical location of Viareggio, Italy (Google Search, 2022)

3.1.1.2. Climate

The city of Viareggio is found in the Mediterranean climatic region of Europe, characterised by hot and dry summers, while winters are mild and wet. The climate is characterised by high levels of humidity (between 60 and 80% of relative humidity in the summer months) and a yearly rainfall off 900 to 1,000 millimetres (35 to 39 in) as a result of the proximity of the Apuanian Alps to the coast. During winter months (Dec-Feb), high temperatures can vary between 14–15 °C (57–59 °F) and 4–5 °C (39–41 °F), while night time temperatures can reach below 0 °C (32 °F). In the summer (Jun-Aug), temperatures peak around 31–33 °C (88–91 °F). The very high summer humidity (average 70%) and low cloud cover can mean that the heat index temperature is 20 °C (68 °F) or higher than the air temperature. These data can be gotten online from the "Weather Channel".

3.1.1.3. Topography and hydrology

The entire area of Viareggio extends over the coastal flooding plain of Versilia. Located on the Ligurian Sea (although traditionally considered to face the Tyrrhenian Sea), it has 10 kilometres of sandy beaches, of which 6 kilometres are managed by private beach resorts and the remaining 4 kilometres are public (most of the public beach is part of the Parco Naturale Regionale Migliarino-San Rossore-Massaciuccoli. Viareggio borders the local municipalities of Camaiore, Massarosa and Vecchiano) (PI). The municipal area comprises the Lake of Massaciuccoli and several canals, the most important ones of which are known as Burlamacca, Farabola, Fossa dell'Abate (bordering the municipality of Camaiore), and Fosso Le Quindici

3.1.2. Socio-economical parameters

Viareggio is known as a seaside resort as well as being the home of the famous carnival of Viareggio. This section is concerned with the description of parameters like population, economy, culture and transportation.

3.1.2.1. Population

The population of Viareggio in 2022 is estimated at about 60 936 while in 1981 it was at about 58 263. However, a population drop is observed from 2017(estimated at about 62 343) to 2022 due to the demographic crisis with decrease rate of 2,26%. These population estimates and projections come from the latest revision of the Viareggio commune. Figure 3.2. shows the population variations in the city of Viareggio.

Name	Province	Population Census 1981-10-25	Population Census 1991-10-20	Population Census 2001-10-21	Population Census 2011-10-09	Population Estimate 2022-01-01
Viareggio	Lucca	58,263	57,514	61,103	61,857	60,936

Viareggio

60,936 Population [2022] – *Estimate*

32.53 km² Area

1,873/km² Population Density [2022]

-0.15% Annual Population Change [2011 → 2022]

Viareggio: comune in northern Tuscany, Italy – Elevation: 2 m – Official Website – Local dialing code: 0584 – Postal code: 55049 – Licence plate code: LU

Figure 3.2. Population variation in the city of Viareggio (Google Search, 2022.)

3.1.2.2. Economy and culture

The primary sectors of Viareggio's economy are tourism, commerce and services, include fishing and floriculture (the flowers of Versilia). The city also houses prolific shipyards. At the beginning of the 19th century the craftsmen from Viareggio used to build small fishing vessels along the banks of the Burlamacca canal. As the century moved on, however, this small shipbuilding activity prospered until it became an internationally acclaimed centre. Nowadays, Viareggio represents the main luxury yachts producer city in the world thanks to the presence in the territory of shipyards like Azimut Benetti, Codecasa, Fipa, Rossinavi, Perini navi and many others.

The Carnival of Viareggio was established in 1873, while the now ever-present papier mâché – used to build the floats featured during its parades – was first introduced in 1925. The official masks of the Carnival are Burlamacco and Ondina, drawn for the first time in 1930.

3.1.2.3. Transportation

Viareggio can be reached by car from the A11 (Firenze-Mare) motorway, via the A11/"Bretella" Lucca-Viareggio link road or A12 (Genoa-Rosignano) motorway. Viareggio railway station is located near the city center, with 60 daily trains running along the Rome–Pisa–La Spezia–Genoa line, and the line to Florence, as well as international trains. The nearest airport is Pisa's "Galileo Galilei" international airport, just 20 kilometres (12 mi) south of Viareggio's city center. Florence's "Amerigo Vespucci" airport is 95 kilometres (59 mi) to the east.

Ports and marinas can equally be found in Viareggio, with two extensions to today's Burlamacca canal built in 1577. It is on its banks that the first maritime activities developed. In 1938 the Marina of the Empire was built, which was followed in the 1970s by the Marina of Viareggio (also known as the New Marina), the Marina of the Madonnina, and the new lighthouse.

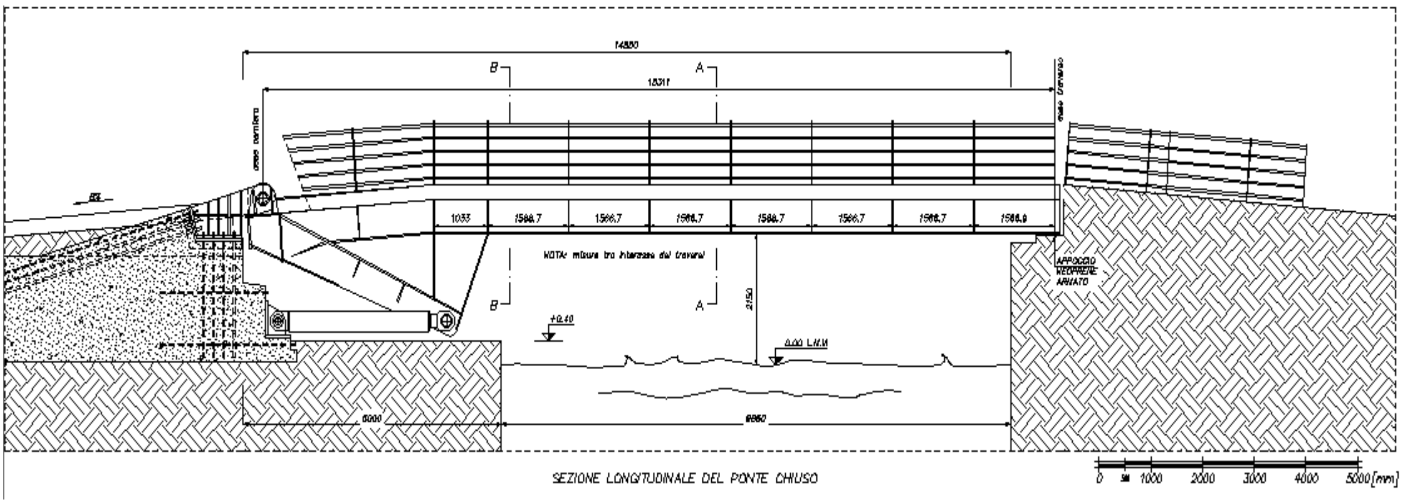
3.2. Presentation of the project

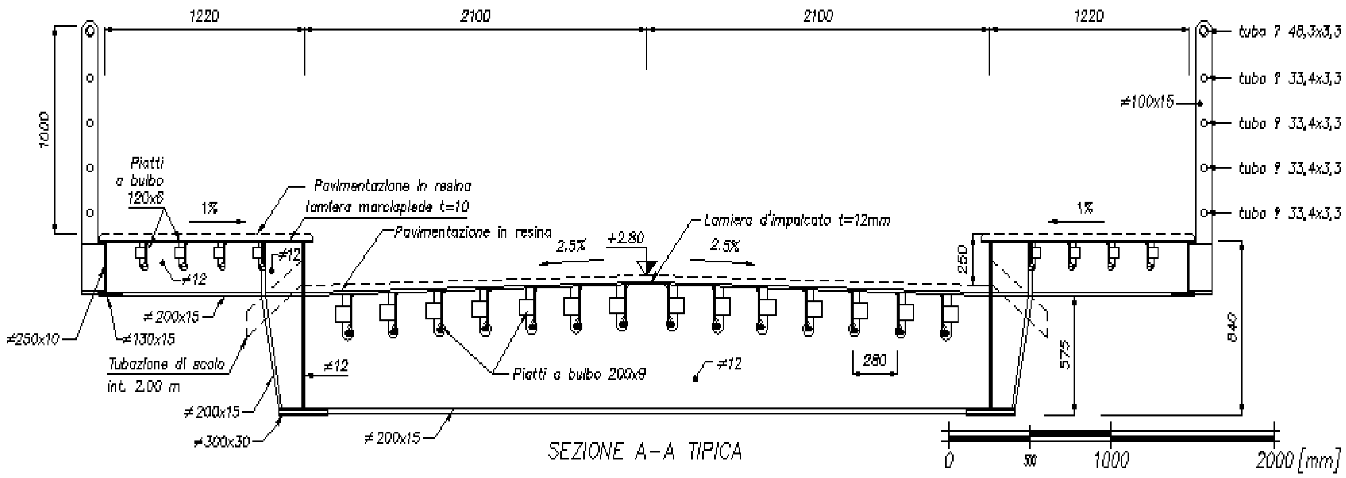
The project consists of a bascule bridge with a fixed counterweight and a lifting mechanism based on hydraulic pistons; in the harbour Viareggio built due to the construction of new docking quays and the enlargement of work and manoeuvre basins. The structure realized has a net span between the two quayside basements of 14.85 metres. The bascule bridge, with single bay, consists of a 4.20 metre roadway flanked by two walkways, each of 1.20 metres.

3.2.1. Geometrical and structural data presentation

The single span bascule bridge consists of an open-style steel section with the rotation axis located above and externally to the deck; this permits an increased active movement arm together with the advantage of not needing a lifting pit. The transversal section is compact, typical of metal bridges with orthotropic decking and features high efficiency in terms of material use. Figure 3.3. shows the different views of the bridge.

The decking is a 12 mm sheeting stiffened longitudinally with bulb plates at a distance of 280 mm, and transversally by steel ribs with inverted T-section at intervals of 1500 mm. For the decking, protection consisted in roadway paving in epoxy resin typical of light metallic structures. The two main girders with a height of about 850mm are placed at distance of 4200mm and are made up of steel with an inverted T-section with deck acting as the top flange. The counterweight block consists of concrete reinforced both with normal steel bars for reinforced concrete and with prestressed Dywidag bars, 32 mm in diameter. The lifting arrangement features two double-acting pistons, with the cylinder liner in Fe52.2 internally lapped, with rod of chromium plated steel C40 with double crossed chromium plating and stainless-steel coating.





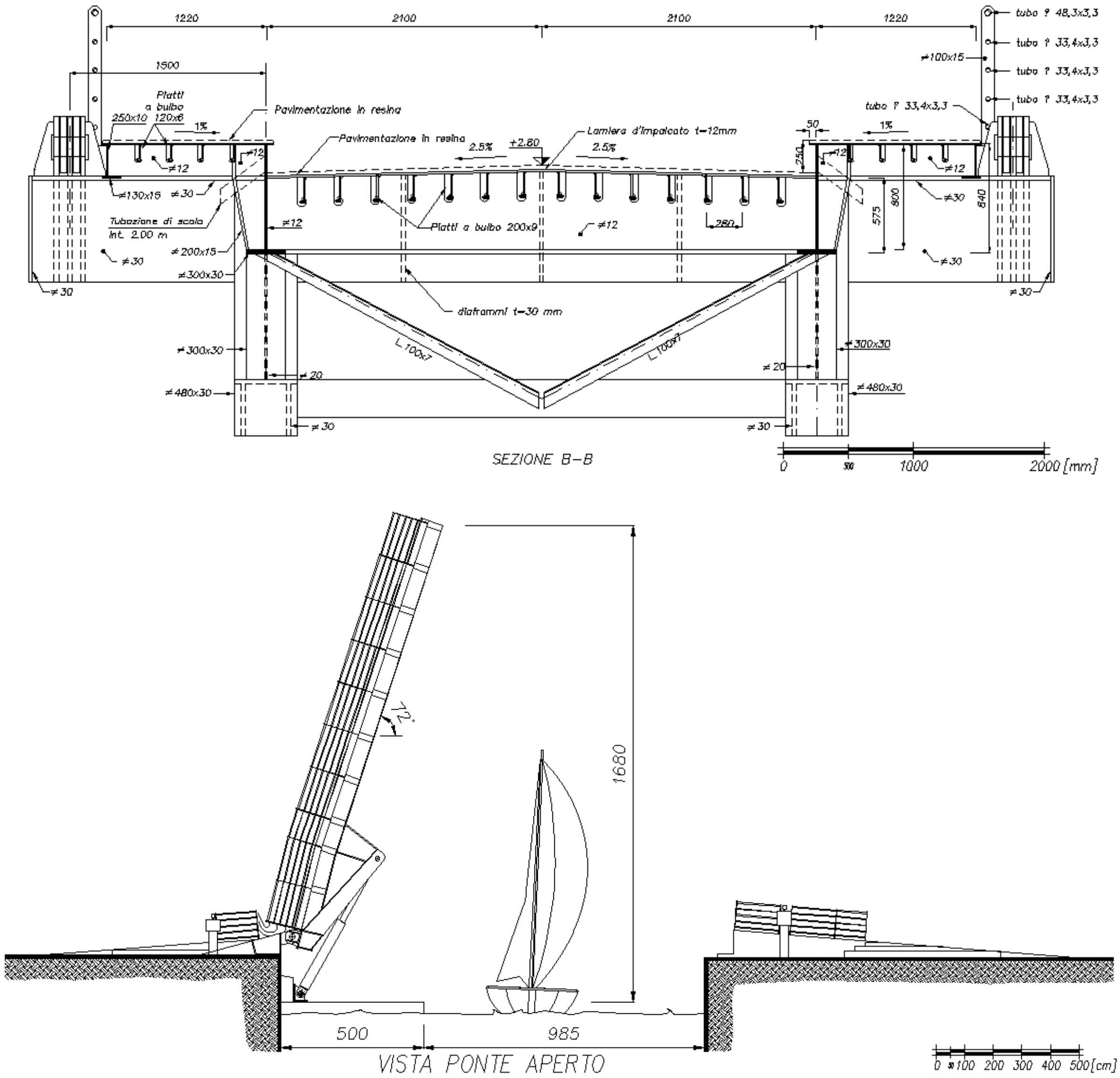


Figure 3.3. Different views of the Viareggio mobile bridge

3.2.2. Characteristics of materials

The structure is made of steel as the main material with a S355 grade and will be optimised using S450 as presented in Table 3.1. for the structural elements.

Table 3.1. Characteristics of different steel grades used

Designation	S 355 (NF EN 10022)		Units
Characteristic Ultimate Strength	f_{uk}	490	MPa
Characteristic Yield Strength	f_{yk}	355	MPa
Elastic Modulus	E_s	210000	MPa
ULS Safety Factor	γ_s	1.15	
Design Yield Strength	f_{yd}	308.695652	MPa
Density	ρ	7850	kg/m^3
Unit weight	γ	78.5	kN/m^3
Shear modulus	G	80769.2308	MPa
Poisson's ratio in elastic range	ν	0.3	
Coefficient of linear thermal expansion	α	0.000012	$°K^{-1}$

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	210000	MPa
ULS Safety Factor	γ_s	1.15	
Design Yield Strength	f_{yd}	382.6086957	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	$°K^{-1}$

3.3. Actions determination

The mobile bridge allows both vehicular and waterborne traffic. For this bridge, the traffic class is class three so the adequate coefficients need to be applied. All the actions were determined as described in section 2.3.

3.3.1. Permanent actions

The different permanent actions present on the bridge include the self-weight of the structural members, the additional self-weight due to other components like the operating mechanism and the self-weight of non-structural components like the guardrails, the roadway and walkway pavements. Table 3.2. presents the different values for permanent actions.

Table 3.2. Permanent actions

PERMANENT ACTIONS	VALUES	UNITS
Self-weight of structural elements	Software	-
Additional self-weight	0.9	kN/m
Self-weight of non-structural elements		
Guardrails	0.5	kN/m
Road and walkway pavements	0.9	kN/m

3.3.2. Variable actions

The different variable actions present on the bridge include the wind, traffic, snow and temperature actions. Table 3.3. presents the different values for variable actions.

Table 3.3. Variable actions

VARIABLE ACTIONS	VALUES	UNITS
Wind action		
Closed configuration transversal	11.9	kN/m
Opened configuration	2	kN/m^2
Traffic action		
Vertical		
Uniformly Distributed Load lane 1	9	kN/m^2
Uniformly Distributed Load Remaining Area	2.5	kN/m^2
Tandem System	300	
Crowd pressure	5	kN/m^2
Horizontal		
Braking Action	5	kN/m^2
Snow action	0.8	kN/m^2
Temperature action		
Temperature rise	18	ºC/m
Temperature fall	-18	ºC/m

3.4. Analysis results

Upon the modelling of the structure with Midas civil software and determination and application of actions while respecting the different codes and specifications, the different quantities are to be assessed for the result of the analysis in both closed and opened configurations. These quantities are the following, solicitations (bending moment, shear and axial forces), stresses, reactions and displacements in the main structural elements of the OSD.

3.4.1. Analysis of the results in closed configurations

The quantities defined in section 3.4 are presented taking into accounts the load combinations defined in section 2.3.6. most importantly the envelop which gives the worst conditions on structural elements.

3.4.1.1. Solicitations

Bending moments, shear and axial forces in different structural elements are presented in Table 3.4.

Table 3.4. Solicitations in structural elements of the OSD and shown in Figure 3.4.

Load combinations	Solicitations in structural elements main girder roadway		
	bending moment(kNm)	shear force(kN)	axial force(kN)
Fundamental combination ULS	575.8	387.5	2161.5
Fundamental combination SLS	416.4	285.9	1594.5

Load combinations	Solicitations in structural elements main girder walkway		
	bending moment(kNm)	shear force(kN)	axial force(kN)
Fundamental combination ULS	12.4	15.1	196
Fundamental combination SLS	8.7	10.7	140.6

Load combinations	Solicitations in structural elements longitudinal rib roadway		
	bending moment(kNm)	shear force(kN)	axial force(kN)
Fundamental combination ULS	6.3	7.3	82.2
Fundamental combination SLS	4.6	5.3	58.8

Load combinations	Solicitations in structural elements longitudinal rib walkway		
	bending moment(kNm)	shear force(kN)	axial force(kN)
Fundamental combination ULS	1.2	-1.6	46.2
Fundamental combination SLS	0.9	-1.1	33.7

Load combinations	Solicitations in structural elements cross beam roadway		
	bending moment(kNm)	shear force(kN)	axial force(kN)
Fundamental combination ULS	27.8	-142.3	153
Fundamental combination SLS	20.5	-105.4	112.8

Load combinations	Solicitations in structural elements cross beam walkway		
	bending moment(kNm)	shear force(kN)	axial force(kN)
Fundamental combination ULS	-2.7	7.95	-23.9
Fundamental combination SLS	-2.02	5.86	-17.6

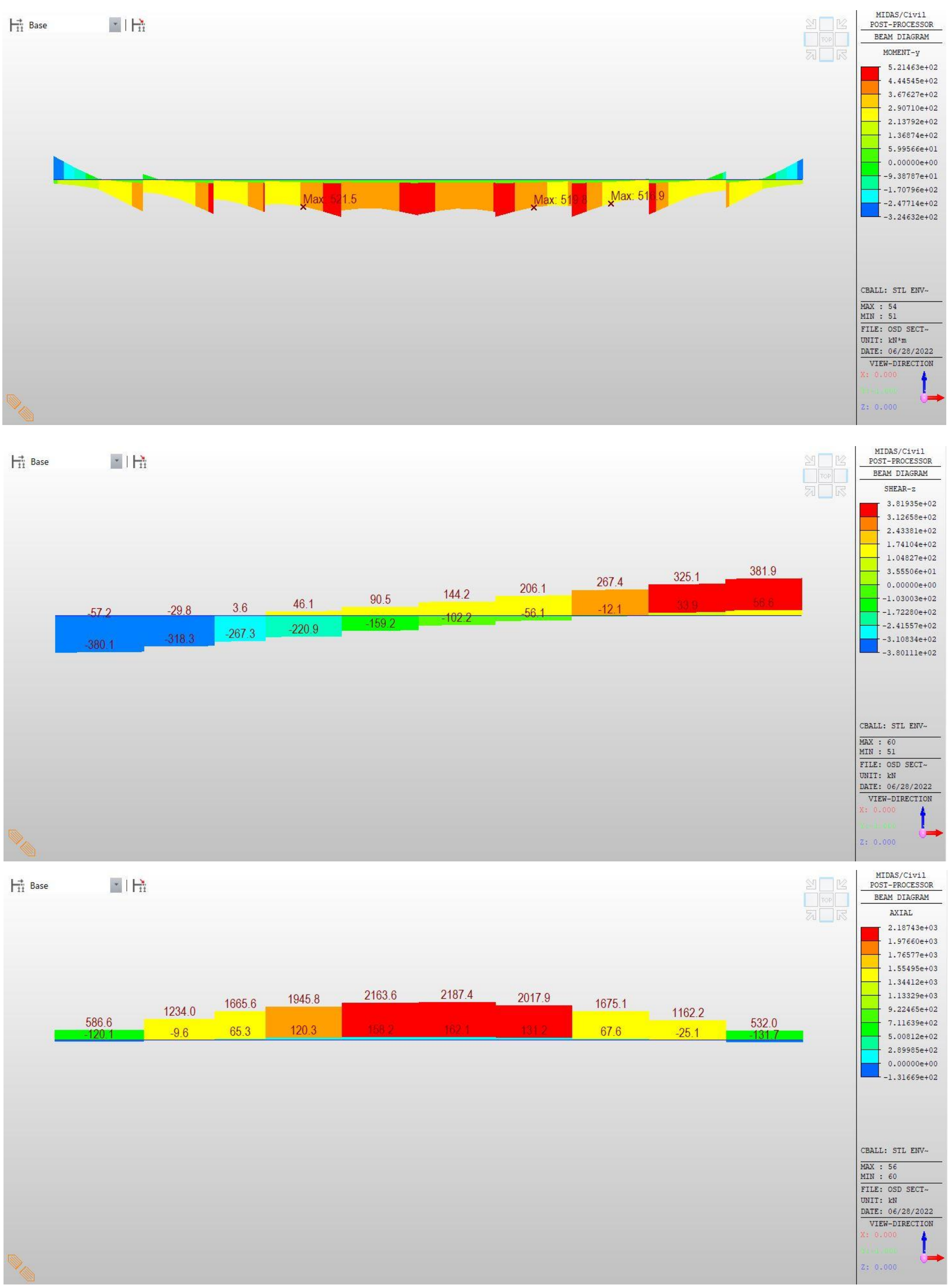


Figure 3.4. Bending moment, shear force and axial force envelopes at ULS for main girder

3.4.1.2. Stresses

Maximum and minimum stresses in different structural elements of OSD are displayed in Table 3.5. and depicted in Figure 3.5.

Table 3.5. Stresses in structural elements of the OSD

Load combinations	Stresses in structural elements	
	main girder roadway	
	Maximum stress(MPa)	Minimum stress(MPa)
Fundamental combination ULS	180.69	-166.6
Fundamental combination SLS	132.82	-118.68

Load combinations	Stresses in structural elements	
	main girder walkway	
	Maximum stress(MPa)	Minimum stress(MPa)
Fundamental combination ULS	90.7	53
Fundamental combination SLS	64.6	37.7

Load combinations	Stresses in structural elements	
	longitudinal rib roadway	
	Maximum stress(MPa)	Minimum stress(MPa)
Fundamental combination ULS	70.6	-14.9
Fundamental combination SLS	52.2	-10.6

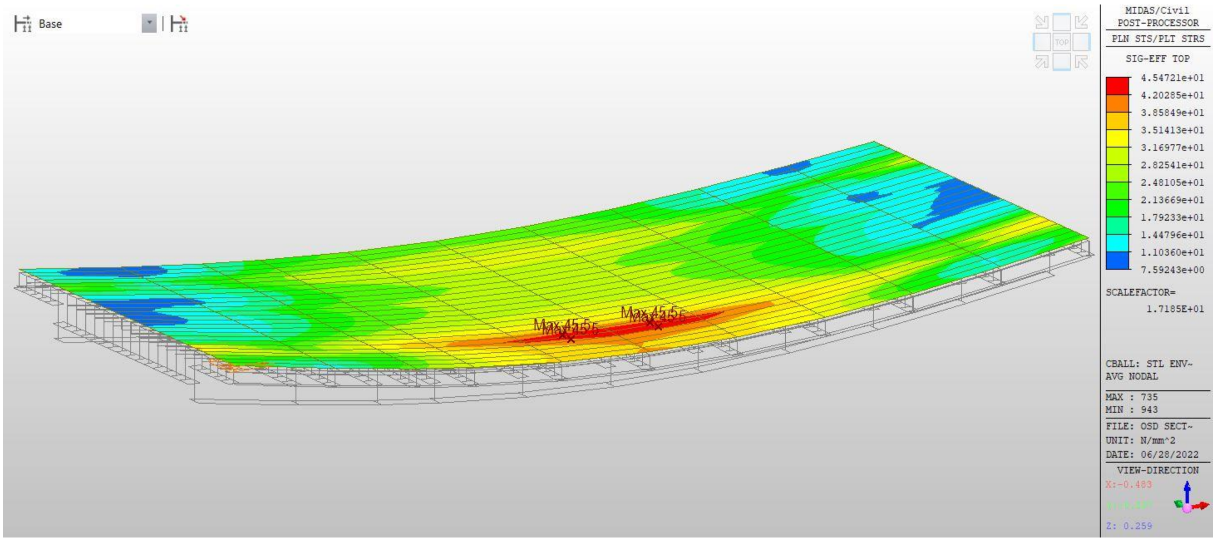
Load combinations	Stresses in structural elements	
	longitudinal rib walkway	
	Maximum stress(MPa)	Minimum stress(MPa)
Fundamental combination ULS	57.5	-35.6
Fundamental combination SLS	41.8	-25.2

Load combinations	Stresses in structural elements	
	cross beam roadway	
	Maximum stress(MPa)	Minimum stress(MPa)
Fundamental combination ULS	55.6	-56.4
Fundamental combination SLS	40.8	-41.3

Load combinations	Stresses in structural elements	
	cross beam walkway	
	Maximum stress(MPa)	Minimum stress(MPa)
Fundamental combination ULS	46.5	-91.6
Fundamental combination SLS	34	-66.9

Load combinations	Stresses in structural elements	
	Deck roadway	
	Maximum stress(MPa)	Minimum stress(MPa)
Fundamental combination ULS	1.98	-2.96
Fundamental combination SLS	1.35	-2.09

Load combinations	Stresses in structural elements	
	Deck walkway((Smax))	
	Maximum stress(MPa)	Minimum stress(MPa)
Fundamental combination ULS	6.33	-2.96
Fundamental combination SLS	4.23	-2.09



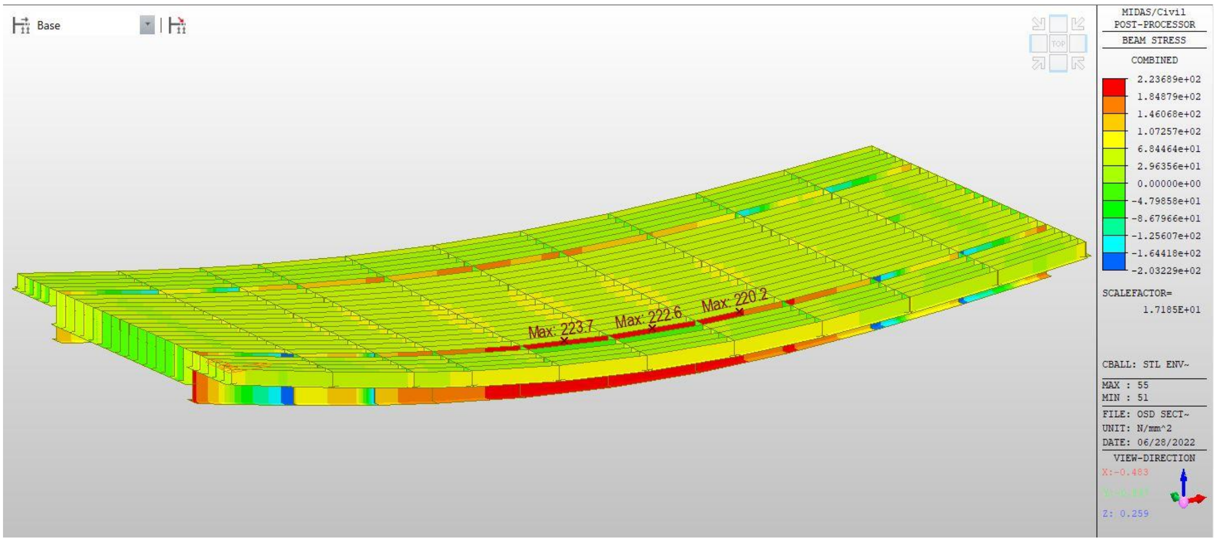


Figure 3.5. Stresses in plate and beam elements

3.4.1.3. Reactions

Reactions in the three directions (X,Y,Z) are presented for different structural elements in Table 3.6. and seen in Figure 3.6.

Table 3.6. Reactions in structural elements of the OSD

Load combinations	Reactions in structural elements main girder roadway		
	Fx(kN)	Fy(kN)	Fz(kN)
Fundamental combination ULS	-432.93	-20.89	397.02
Fundamental combination SLS	-318.31	-15.06	292.27

Load combinations	Reactions in structural elements main girder walkway		
	Fx(kN)	Fy(kN)	Fz(kN)
Fundamental combination ULS	-98.39	-54.16	11.71
Fundamental combination SLS	-71.41	-39.4	8.2

Load combinations	Reactions in structural elements longitudinal rib roadway		
	Fx(kN)	Fy(kN)	Fz(kN)
Fundamental combination ULS	-226.93	78.97	117.06
Fundamental combination SLS	-166.98	57.73	86.37

Load combinations	Reactions in structural elements longitudinal rib walkway		
	Fx(kN)	Fy(kN)	Fz(kN)
Fundamental combination ULS	-156.91	-51.45	3.55
Fundamental combination SLS	-115.22	-37.66	2.57

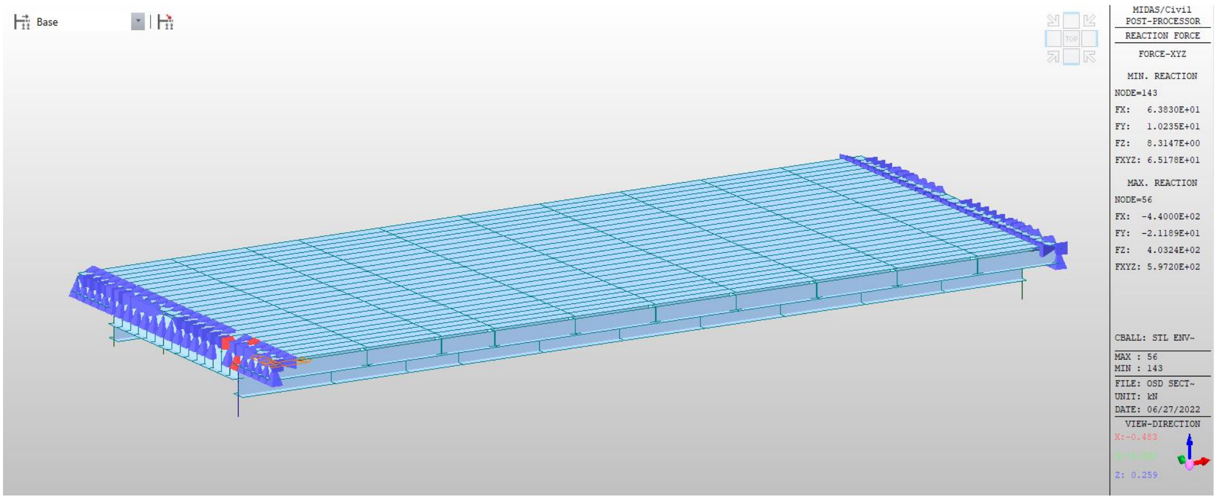


Figure 3.6. Reactions of bascule at ULS with highest values on main girders

3.4.1.4. Displacements

The maximum displacement is assessed and presented in Table 3.7. and depicted in Figure 3.7.

Table 3.7. Global displacement of the OSD bridge

Load combinations	Global bridge displacement		
	maximum displacement		
	Ux(mm)	Uy(mm)	Uz(mm)
Fundamental combination ULS	0	0	-38
Fundamental combination SLS	0	0	-27

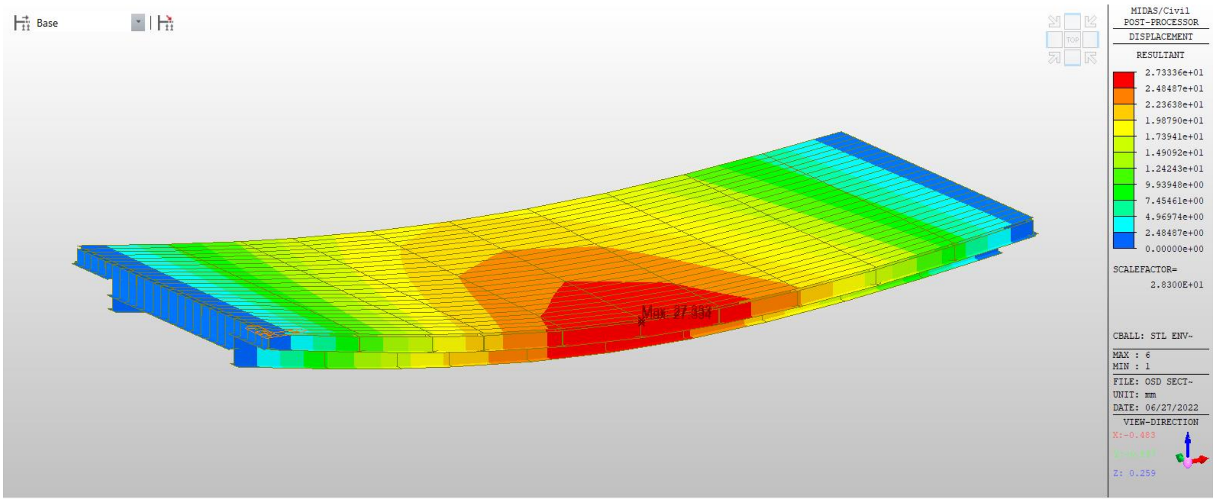


Figure 3.7. Deflection of bascule bridge

3.4.2. Analysis results in opened configuration

In opened configuration, the bridge will be analysed at the maximum opening angle which is 72 degrees with respect to the water level, taking into account hydraulic drive as operating mechanism. In this case only the main girders of roadway and the decks are assessed since they have the largest values of solicitations.

3.4.2.1. Solicitations

Bending moments, shear and axial forces in the main girder are presented in Table 3.8. and shown in Figure 3.8.

Table 3.8. Solicitations in main girder

Load combinations	Solicitations in structural elements		
	main girder roadway		
	bending moment(kNm)	shear force(kN)	axial force(kN)
Fundamental combination ULS	-523.73	-191.3	-1946.8
Fundamental combination SLS	-363.3	-132.7	-1358.8

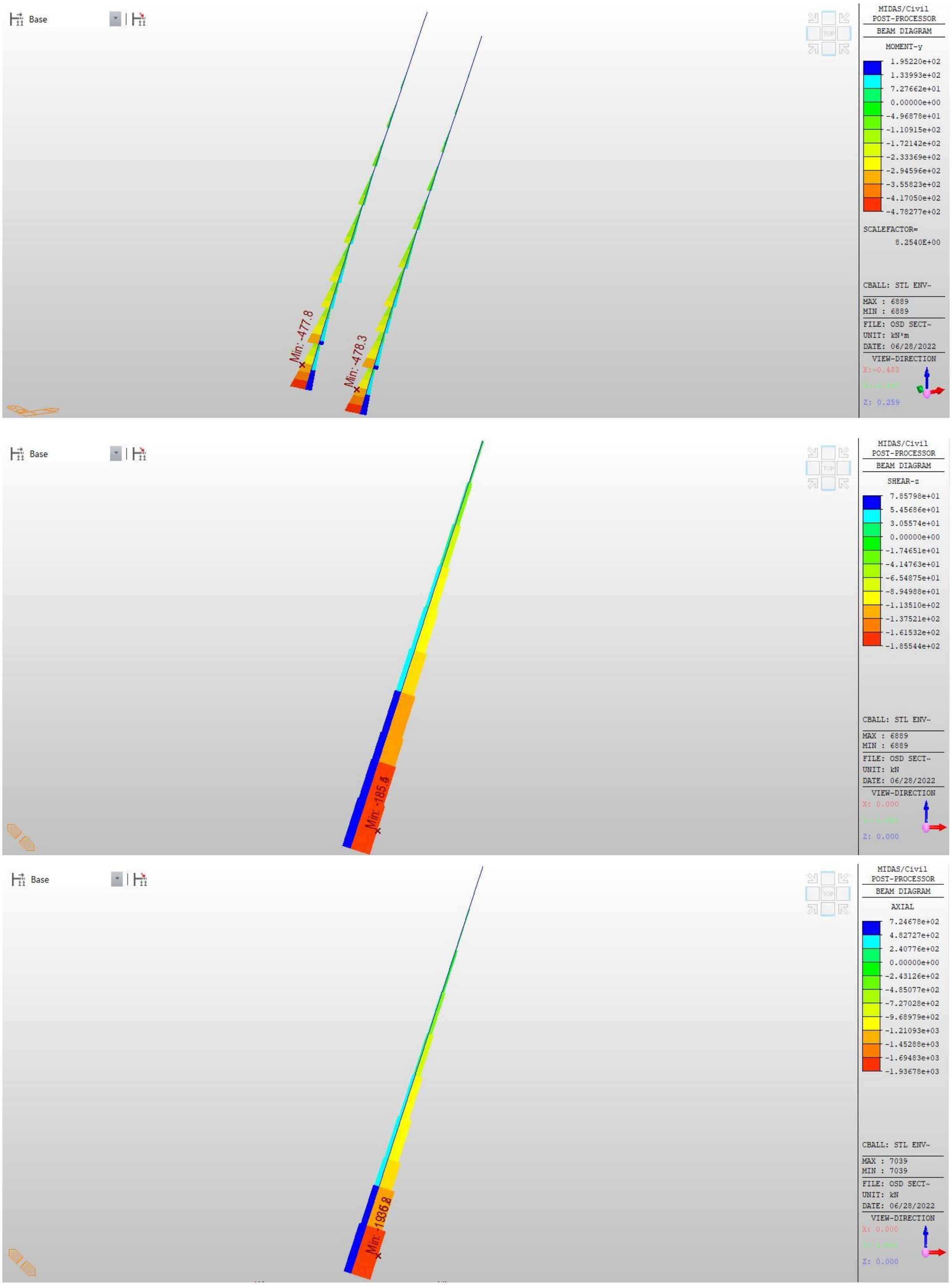


Figure 3.8. Bending moment, shear force and axial force envelopes at ULS for main girder in opened state

3.4.2.2. Stresses

Maximum and minimum stresses in different structural elements of OSD are displayed in Table 3.9. and depicted in Figure 3.9.

Table 3.9. Stresses in main girder and deck

Load combinations	Stresses in structural elements	
	main girder roadway	
	Maximum stress(MPa)	Minimum stress(MPa)
Fundamental combination ULS	-181.1	-4.7
Fundamental combination SLS	-125.8	-3.3

Load combinations	Stresses in structural elements	
	Deck roadway	
	Maximum stress(MPa)	Minimum stress(MPa)
Fundamental combination ULS	51.94	1.25
Fundamental combination SLS	35.91	0.87

Load combinations	Stresses in structural elements	
	Deck walkway((Smax))	
	Maximum stress(MPa)	Minimum stress(MPa)
Fundamental combination ULS	51.94	0.51
Fundamental combination SLS	35.9	0.36

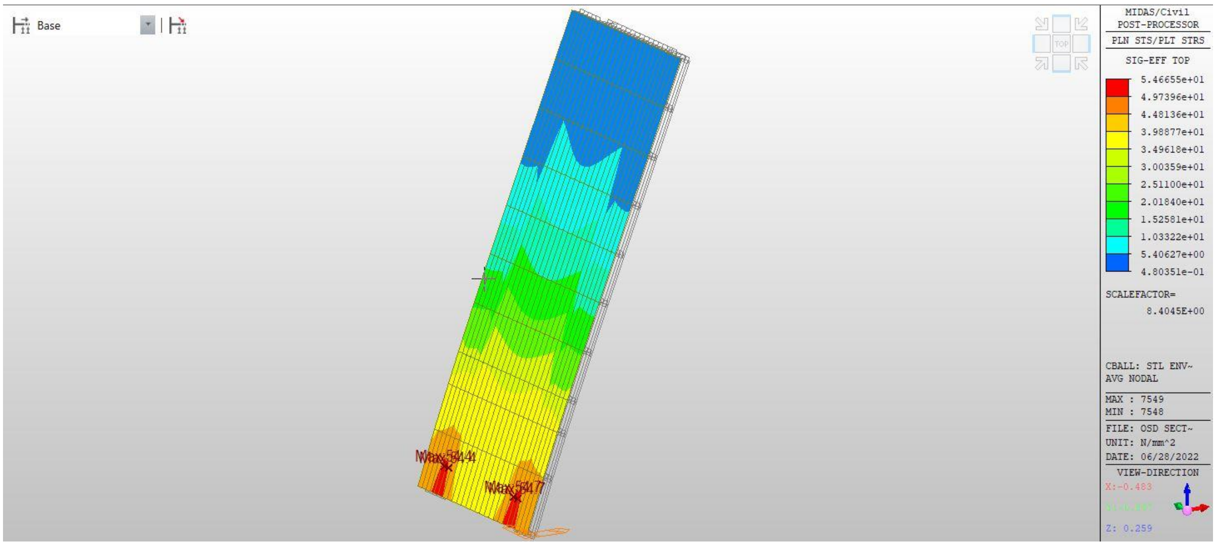


Figure 3.9. Stresses in bridge at opened state

3.4.2.3. Reactions

Reactions in the three directions (X,Y,Z) are presented in Table 3.10. for the main girder and shown in Figure 3.10.

Table 3.10. Reactions of main girder

Load combinations	Reactions in structural elements		
	main girder roadway		
	Fx(kN)	Fy(kN)	Fz(kN)
Fundamental combination ULS	409.1	30.2	1880.7
Fundamental combination SLS	285	20.9	1308.2

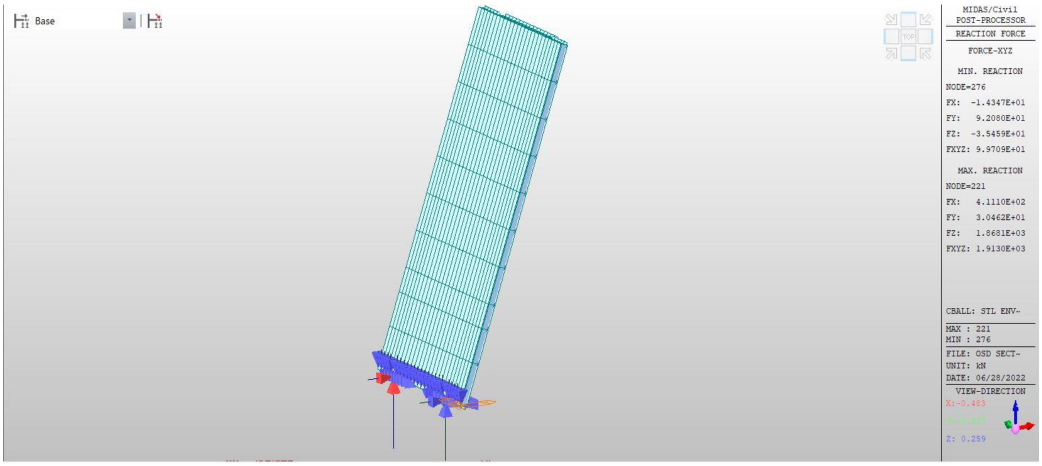


Figure 3.10. Reactions in bridge at opened with maximum values on main girders

3.4.2.4. Displacements

The maximum displacement of the top nodes is assessed and presented in Table 3.11. and seen in Figure 3.11.

Table 3.11. Top nodes displacement

Load combinations	Displacements in structural elements		
	maximum displacement of top nodes		
	Ux(mm)	Uy(mm)	Uz(mm)
Fundamental combination ULS	69.71	0.15	-72.92
Fundamental combination SLS	48.38	0.1	-50.62

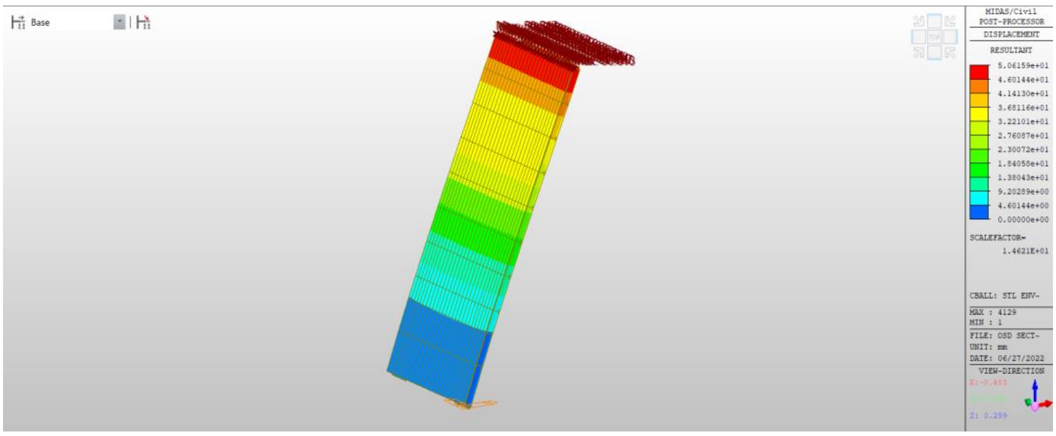


Figure 3.11. Displacements in bridge at opened state

3.5. Parametric analysis results, verifications and amelioration results

A sensitivity analysis equally known as a parametric analysis was carried on the mobile bridge with parameters angle of opening coupled with operating and varying the steel type (using S450). Results are here assessed in terms of solicitations, stresses, reactions and displacements and are displayed in both tabular and graphical forms. A verification was equally made following the prescriptions of section 2.5.4. Finally results from the amelioration of the structure are equally presented.

3.5.1. Parametric analysis results

The parameters considered are as said in section 2.5.1. So, the opening angles with an increment of 15 each time is coupled with operation mechanism (hydraulic and mechanical drive). As shown by graphs below a mechanical drive stresses more the bridge structure than the hydraulic drive. Also, analysis of the ameliorated structure is presented with the use of S450.

3.5.1.1. Hydraulic span drive

Solicitations, stresses, reactions and displacements of the bridge with the hydraulic drive are assessed and shown in Table 3.12.

Table 3.12. Solicitations, stresses, displacements and reactions in main girder with different opening angles using a hydraulic span drive

15 degrees									
M(kNm)		T(kN)		N(kN)		Stress(MPa)		Displacement(mm)	
ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS
-672.2	-490.8	-248.7	-181.8	-2416.9	-1766.1	-228.6	-166.98	89	65

Reactions(kN)					
ULS			SLS		
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)
2220.21	-41.7	853.35	1622.92	-30.36	623.73

30 degrees									
M(kNm)		T(kN)		N(kN)		Stress(MPa)		Displacement(mm)	
ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS
-697.7	-503.55	-257.5	-185.94	-2529	-1826.8	-238.26	-172.03	97.02	69.99

Reactions(kN)					
ULS			SLS		
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)
2016.9	42.6	1462.9	1457.3	30.68	1056.9

45 degrees									
M(kNm)		**T(kN)**		**N(kN)**		**Stress(MPa)**		**Displacement(mm)**	
ULS	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	SLS
-601.7	-428.2	-234.97	-167.3	-2403.4	-1712.1	-218.5	-158.3	110.56	78.67
Reactions(kN)									
ULS			SLS						
Fx(kN)	FykN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)				
1499.1	40.2	1833.3	1068.3	28.5	1306.2				

60 degrees									
M(kNm)		**T(kN)**		**N(kN)**		**Stress(MPa)**		**Displacement(mm)**	
ULS	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**
-547.7	-400.6	-222.8	-156.8	-2240.3	-1579.1	-209.3	-147.4	84.57	59.52
Reactions(kN)									
ULS			SLS						
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)				
906.5	35.9	2017.8	639.4	25.2	1422.5				

75 degrees									
M(kNm)		**T(kN)**		**N(kN)**		**Stress(MPa)**		**Displacement(mm)**	
ULS	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**
-498.1	-344.4	-181.8	-125.6	-1859.4	-1287.7	-172.8	-119.5	69.43	47.98
Reactions(kN)									
ULS			SLS						
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)				
297.1	28.6	1814.9	206.3	19.7	1257.2				

90 degrees									
M(kNm)		**T(kN)**		**N(kN)**		**Stress(MPa)**		**Displacement(mm)**	
ULS	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**
-359	-239.8	-128.8	-85.9	-1351.2	-908.2	-125.2	-83.9	49.79	33.32
Reactions(kN)									
ULS			SLS						
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)				
-129.6	-26.5	1334.8	-86.4	-17.8	888.9				

3.5.1.2. Mechanical span drive

Solicitations, stresses, reactions and displacements of the bridge with the mechanical drive are assessed and presented in Table 3.13.

Table 3.13. Solicitations, stresses, displacements and reactions in main girder with different opening angles using a mechanical span drive

15 degrees									
M(kNm)		**T(kN)**		**N(kN)**		**Stress(MPa)**		**Displacement(mm)**	
ULS	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**
-702.3	-513.1	-260.5	-190.5	-2525.9	-1846.9	-238.9	-174.6	97.55	71.23

Reactions(kN)					
ULS			SLS		
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)
2320.6	43.6	892.4	1697.2	31.8	652.6

30 degrees									
M(kNm)		**T(kN)**		**N(kN)**		**Stress(MPa)**		**Displacement(mm)**	
ULS	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**
-725	-523.8	-268	-193.7	-2629.5	-1901.3	-247.6	-179	100.79	72.79

Reactions(kN)					
ULS			SLS		
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)
2097.2	44.3	1521.4	1516.8	31.9	1100.3

45 degrees									
M(kNm)		**T(kN)**		**N(kN)**		**Stress(MPa)**		**Displacement(mm)**	
ULS	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**
-698.3	**-498.7**	**-257.2**	**-183.8**	**-2554**	**-1826.2**	**-239.5**	**-171.2**	**97.17**	**69.39**

Reactions(kN)					
ULS			SLS		
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)
1589.2	42	1954.4	1136.7	29.9	1397.7

60 degrees									
M(kNm)		**T(kN)**		**N(kN)**		**Stress(MPa)**		**Displacement(mm)**	
ULS	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**
-624.1	-439.7	-228.9	-161.3	-2304.4	-1626.6	-215.1	-151.7	86.92	61.26

Reactions(KN)					
ULS			SLS		
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)
932.8	36.9	2075.5	658.8	25.9	1465.2

75 degrees									
M(kNm)		**T(kN)**		**N(kN)**		**Stress(MPa)**		**Displacement(mm)**	
ULS	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**
-507.9	-351.2	-185.2	-128	-1898.1	-1316.4	-176.2	-122	70.8	49

Reactions(kN)

ULS			SLS		
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)
303.9	29.1	1852.5	211.3	20.1	1285

90 degrees									
M(kNm)		T(kN)		N(kN)		Stress(MPa)		Displacement(mm)	
ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS
-360.1	-240.7	-128.8	-85.9	-1361.8	-916.1	-125.9	-84.4	50.08	33.54
Reactions(kN)									
ULS			SLS						
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)				
-129.6	-26.5	1344.9	-86.4	-17.8	896.3				

3.5.1.3. Analysis of the optimised structure with the hydraulic drive using S450

Solicitations, stresses, reactions and displacements of the ameliorated bridge with the hydraulic drive are assessed and displayed in Table 3.14.

Table 3.14. Solicitations, stresses, displacements and reactions in the optimised main girder with different opening angles using a hydraulic span drive

15 degrees									
M(kNm)		T(kN)		N(kN)		Stress(MPa)		Displacement(mm)	
ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS
-581.6	-424.1	-228.6	-166.9	-2288.7	-1670.3	-260.7	-190.2	106.71	77.77
Reactions(kN)									
ULS			SLS						
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)				
2102.3	39.7	800.8	1534.8	28.9	584.6				

30 degrees									
M(kNm)		T(kN)		N(kN)		Stress(MPa)		Displacement(mm)	
ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS
-611.5	-440.4	-239.6	-172.7	-2423.7	-1747	-275.2	-198.3	112.26	80.82
Reactions(kN)									
ULS			SLS						
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)				
1934.9	41.1	1394.8	1395	29.5	1005.6				

45 degrees									
M(kNm)		T(kN)		N(kN)		Stress(MPa)		Displacement(mm)	
ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS	ULS	SLS
-599.7	-426.7	-234.3	-166.8	-2393.5	-1704.8	-270.9	-192.8	110.154	78.37
Reactions(kN)									
ULS			SLS						
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)				
1492.7	39.7	1825.9	1063.5	28.2	1300.7				

60 degrees									
M(kNm)		**T(kN)**		**N(kN)**		**Stress(MPa)**		**Displacement(mm)**	
ULS	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**
-547.1	-383.9	-213	-149.5	-2200.2	-1546.3	-248.1	-174.2	100.55	70.57

Reactions(kN)					
ULS			SLS		
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)
894.5	35.6	1977.3	629	24.9	1389.9

75 degrees									
M(kNm)		**T(kN)**		**N(kN)**		**Stress(MPa)**		**Displacement(mm)**	
ULS	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**
-457.5	-315.2	-177.3	-122.1	-1857.1	-1282.6	-208.5	-143.8	84.13	58

Reactions(KN)					
ULS			SLS		
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)
300.6	29	1809.9	208.1	19.9	1250.3

90 degrees									
M(kNm)		**T(kN)**		**N(kN)**		**Stress(MPa)**		**Displacement(mm)**	
ULS	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**
-339.9	-227	-129.6	-86.4	-1387.3	-931.4	-155.5	-104.2	62.3	41.68

Reactions(kN)					
ULS			SLS		
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)
-129.6	-26.5	1334.8	-86.4	-17.8	888.9

Closed configuration									
M(kNm)		**T(kN)**		**N(kN)**		**Stress(MPa)**		**Displacement(mm)**	
ULS	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**	**ULS**	**SLS**
521.5	378.5	381.9	281.7	2187.4	1613.4	223.7	164.4	42.73	30.7

Reactions(kN)					
ULS			SLS		
Fx(kN)	Fy(kN)	Fz(kN)	Fx(kN)	Fy(kN)	Fz(kN)
-410.2	-21.2	391.6	-301.6	-15.2	288.3

3.5.1.4. Graphs

On the graph of Figure 3.4. to Figure 3.7., the reversal of solicitations and stresses (from tensile to compressive) is observed as the bridge leaves from a closed configuration (0 degree) to the opened one (15 degrees), explaining the change of static condition from a simply supported plate to a cantilevered or fixed plate for the three cases. For the solicitations (bending moment, shear force and axial force), the negative values increase till 15 degrees attaining a maximum value at 30 degrees opening then drop till 90 degrees opening. The maximum values are observed at 30.

Figure 3.8. and 3.9. represent the reactions in the X and Z directions respectively on the main girders. For the X-direction, the reversal occurs at two points (from 0 to 15 degrees and from 75 to 90 degrees) with a maximum value observed at 19 degrees. For the Z-directions, all the values are positive with a maximum value seen at 60 degrees.

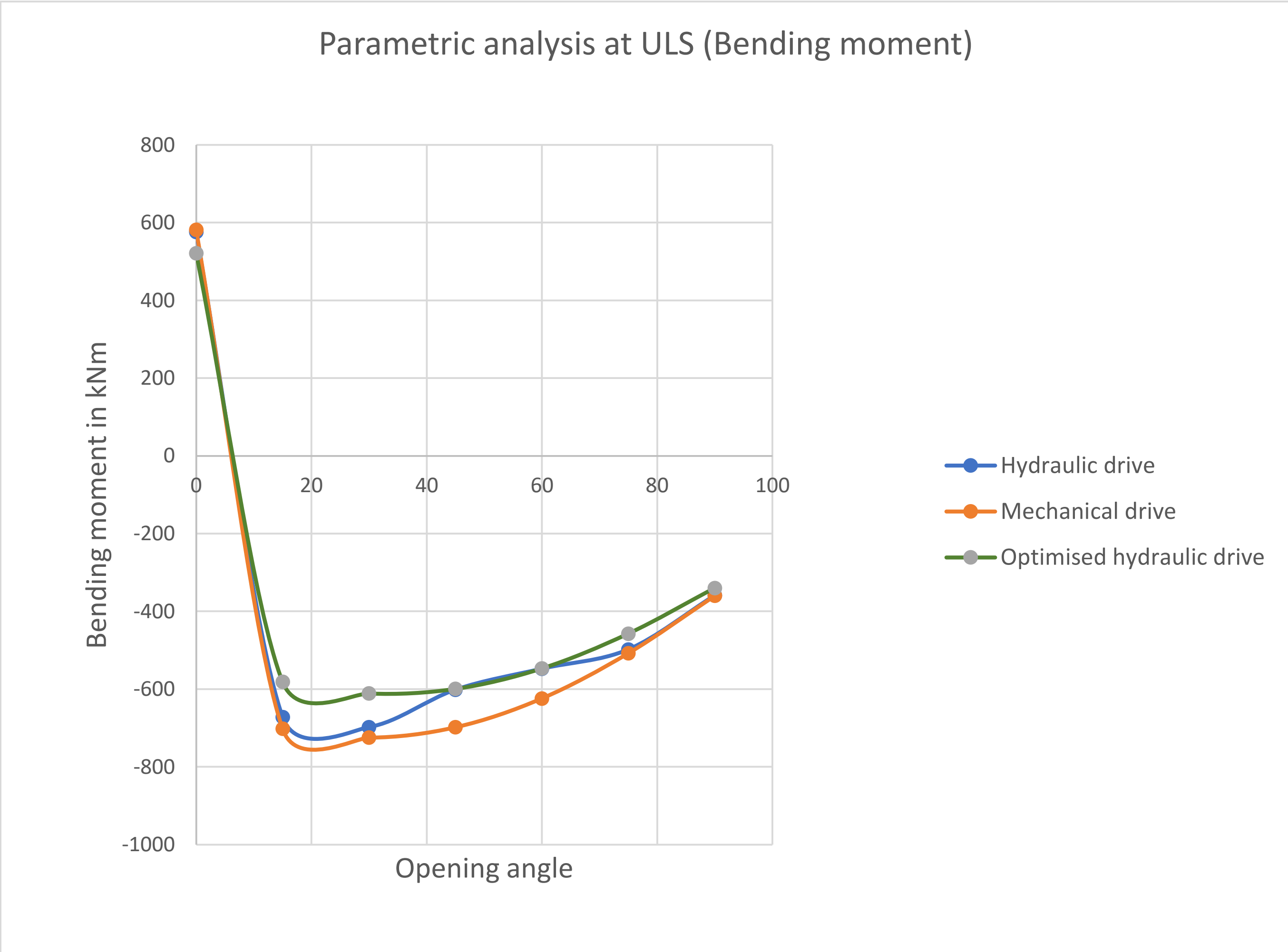


Figure 3.12. Variation of bending moment at ULS for hydraulic drive, mechanical drive and ameliorated hydraulic drive.

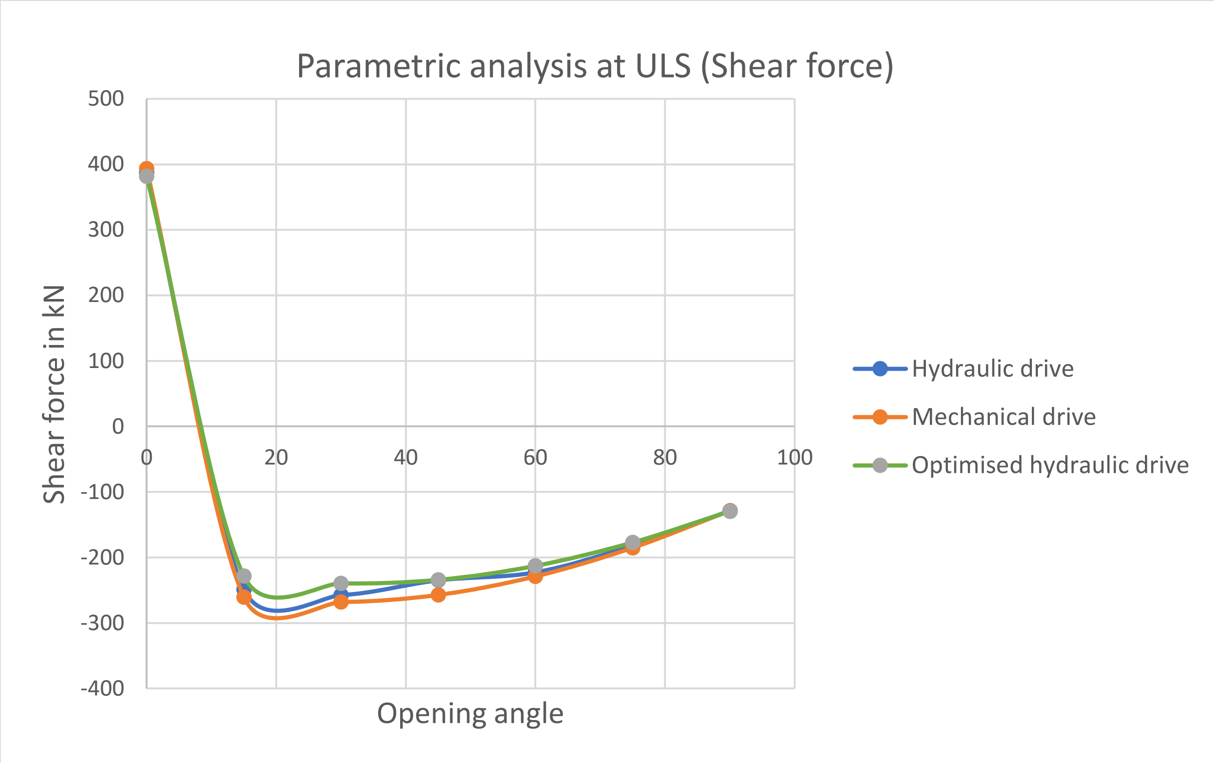


Figure 3.13. Variation of shear force at ULS for hydraulic drive, mechanical drive and ameliorated hydraulic drive.

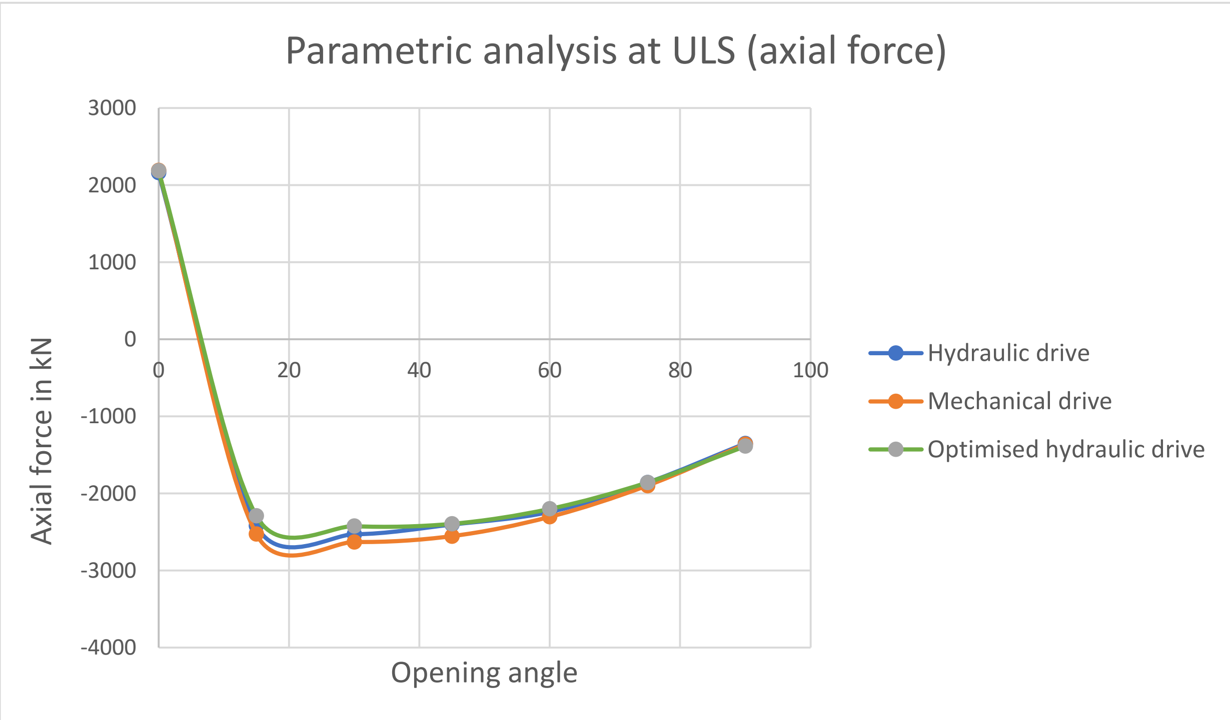


Figure 3.14. Variation of axial force at ULS for hydraulic drive, mechanical drive and ameliorated hydraulic drive.

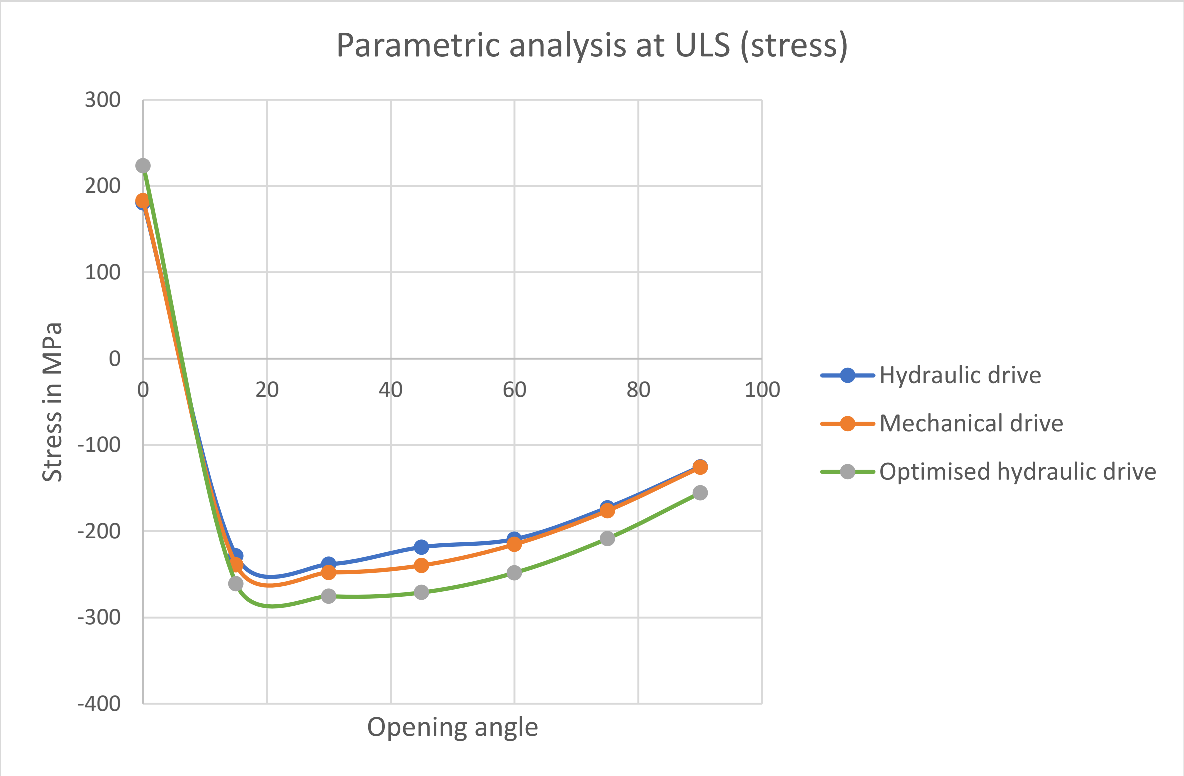


Figure 3.15. Variation of stresses at ULS for hydraulic drive, mechanical drive and ameliorated hydraulic drive.

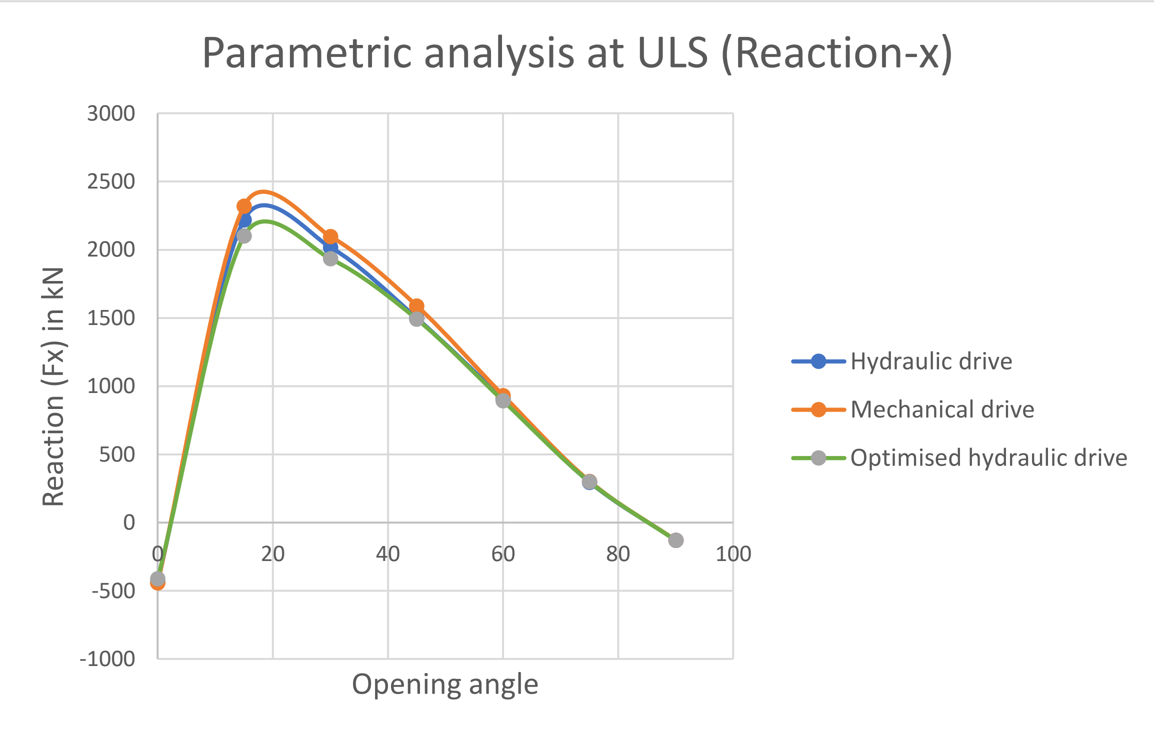


Figure 3.16. Variation of reactions (Fx) at ULS for hydraulic drive, mechanical drive and ameliorated hydraulic drive.

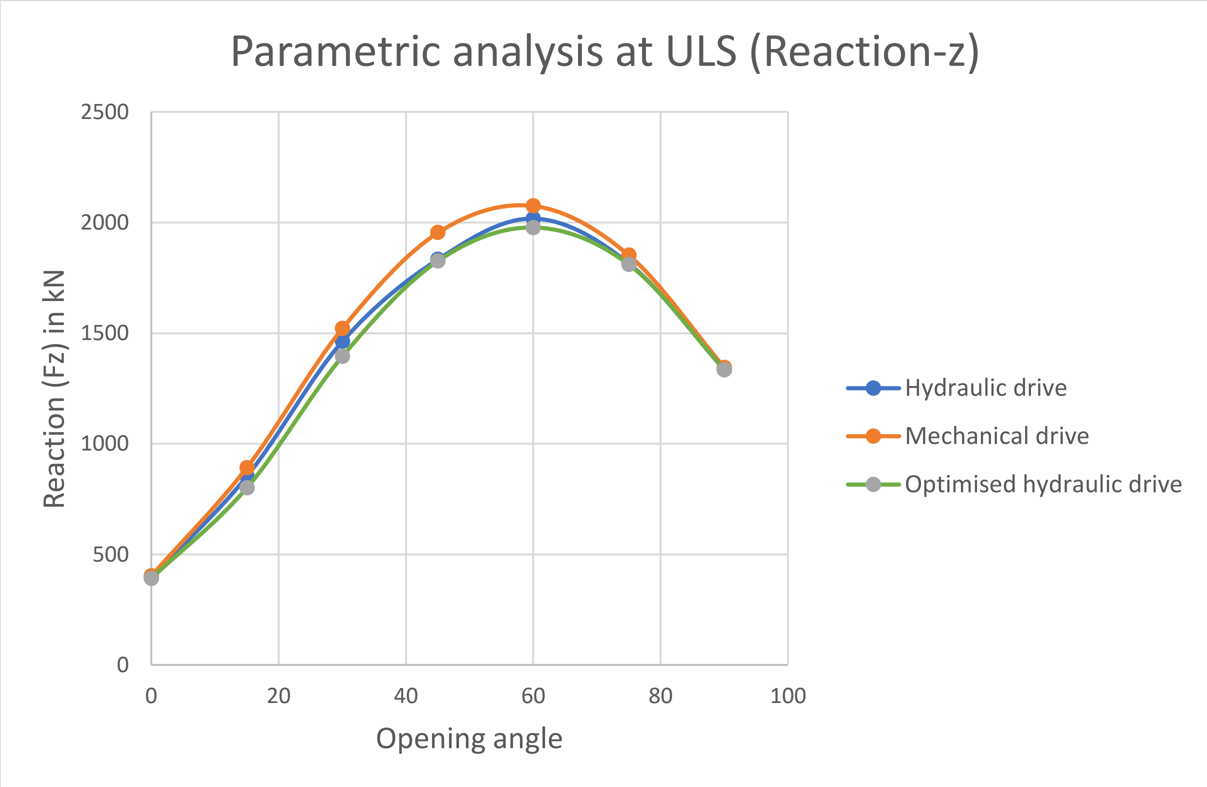


Figure 3.17. Variation of reactions (Fz) at ULS for hydraulic drive, mechanical drive and ameliorated hydraulic drive.

3.5.2. Verifications

After performing the different types of analyses, the bascule bridge made up of an OSD has to be checked according to the expressions given in section 2.5.4 extracted from Eurocode 3 part 1.5. Checks are performed on the structure in place with the hydraulic drive considering the most severe loading conditions (envelope curve). The main girder, plate, cross beams and longitudinal ribs are check at ULS and global deflection of the bridge structure in closed configuration is checked at SLS. After optimisation by changing the steel type (S450) and geometrical properties, the bridge structure is checked, using the expression of section 2.5.4. The checks are performed on structural elements of the OSD at ULS and a deflection check is equally done.

3.5.2.1. Bridge in place with hydraulic drive

Bending, shear, tensile and strength checks are done on the main girders, cross beams and longitudinal ribs with the results shown in Table 3.15. with deflection in Table 3.16.

Table 3.15. Verification of structural elements at ULS with hydraulic drive

Main girder roadway			
Uniaxial bending		**Verification**	
η1	0.2082676	< 1	OK
Stresses		Verification	
σz	64.2913041	< fy	OK
σ in element	238.26	< fy	OK
Shear resistance		Verification	
η3	0.095	< 1	OK
Buckling		Verification	
σcr	98.6	> σz	OK
σ(plate)	41.7	< fy	OK

Cross beam roadway			
Uniaxial bending		**Verification**	
η1	0.004	< 1	OK
Stresses		Verification	
σz	31.7	< fy	OK
Shear resistance		Verification	
η3	0.005	< 1	OK
Tensile resistance		Verification	
η2	0.012	< 1	OK

Longitudinal rib roadway			
Uniaxial bending		**Verification**	
η1	0.016	< 1	OK
Stresses		Verification	
σz	70.6	< fy	OK
Shear resistance		Verification	
η3	0.29	< 1	OK
Tensile resistance		Verification	
η2	0.09	< 1	OK

Table 3.16. Deflection verification at SLS

Deflection for self-weight		Verification
Value at SLS(mm)	5.4	OK
L/300(mm)	50.9	

Deflection for moving load		Verification
Value at SLS(mm)	11.5	OK
L/500(mm)	30.5	

3.5.2.2. Ameliorated bridge with hydraulic drive

Bending, shear, tensile and strength checks are done on ameliorated main girders, cross beams and longitudinal ribs with the results shown in Table 3.17.

Table 3.17. Verification of ameliorated structural elements at ULS with hydraulic drive

Main girder roadway			
Uniaxial bending		**Verification**	
η1	0.14	< 1	OK
Stresses		Verification	
σz(Navier)	60.0899	< fy	OK
σ in element	275.2	< fy	OK
Shear resistance		Verification	
η3	0.11	< 1	OK
Buckling		Verification	
σcr	98.6	> σz	OK
σ(plate)	45.5	< fy	OK

Cross beam roadway			
Uniaxial bending		**Verification**	
η1	0.07	< 1	OK
Stresses		Verification	
σz	35.5	< fy	OK
Shear resistance		Verification	
η3	0.004	< 1	OK
Tensile resistance		Verification	
η2	0.007	< 1	OK

Longitudinal rib roadway			
Uniaxial bending		**Verification**	
η1	0.76	< 1	OK
Stresses		Verification	
σz	90.4	< fy	OK
Shear resistance		Verification	
η3	0.0087	< 1	OK
Tensile resistance		Verification	
η2	0.022	< 1	OK

SLS verifications are presented in Table 3.18., 3.19., 3.20. and 3.21.

Table 3.18. Deflection verification at SLS

Deflection for self-weight		Verification
Value at SLS(mm)	5.4	OK
L/300(mm)	50.9	

Deflection for moving load		Verification
Value at SLS(mm)	11.5	OK
L/500(mm)	30.5	

Table 3. 19. Deflection verification of the ameliorated bridge at SLS

Deflection for self-weight		Verification
Value at SLS(mm)	4.8	OK
L/300(mm)	50.9	

Deflection for moving load		Verification
Value at SLS(mm)	10.6	OK
L/500(mm)	30.5	

Table 3.20. Stress verification at SLS

Stress verification at ULS		
$\sigma_{Ed,ser}$	-125.8	OK
fy	360	

Table 3.21. Web breathing verification

b/t	53.33	$b/t \leq 30 + 4{,}0\, L \quad \leq 300$ for road bridges
3.0+4.0*L	64.04	

3.5.3. Amelioration results

The variation of area between the structure in place using S355 and the ameliorated structure using S450 is assessed. A percentage gain is then calculated which corresponds to an equivalent percentage gain in mass of the structure. Table 3.22. shows the assessment of area variation and the percentage gain in area and thus in mass.

Table 3.22. Amelioration results

	Number	Area before (S355)(mm^2)	Area after (S450)(mm^2)	Percentage gain (%)
Main girder	2	42000	33450	20.36
Cross beam				
Roadway	9	103068	81270	21.15
Walkway	9	49950	40590	18.74
Longitudinal Rib				
Roadway	14	36400	31500	13.46
Walkway	8	12000	10360	13.67
Plate	1	50400	46200	8.33
Sum		293818	243370	17.17

Upon amelioration a mass percentage gain of 17.17% is observed.

In order to perform a cost analysis, mild steel plate of grades S355 and S450 prices per ton were assessed on the actual world market from the producers and suppliers "Made-in-China". Table 3.23. presents the different prices in CFA francs noting that 1US=644CFA francs

Table 3.23. Prices per ton of steel grades.

Steel grade (Mild steel plate)	Price per ton in CFA francs
S355	386400
S450	450800

After performing the cost analysis, a consequent gain in price is seen when the structural steel grade is changed from S355 to S450 as shown in Table 3.24.

Table 3.24. Cost analysis

Mass	Value(ton)	Price/ton(CFA)	Total price(CFA)
Initial mass with S355	43.269	386400	16719141.6
Final mass with S450	35.839	450800	16156221.2
Gain in price			562920.4

Figure 3.18. Comparative analysis between unameliorated and ameliorated bascule bridges

Conclusion

The main objective of this chapter was to present the results of the structural amelioration of a bascule bridge undergoing a series of parametric analyses varying the opening angles and the operating mechanisms, notably hydraulic or mechanical drive. To begin a linear static structural analysis was performed on the different elements of the orthotropic steel deck and verification were in compliance with Eurocode 3. Equally actions and actions combinations were chosen and defined ideally in both opened and closed configurations describing either waterborne or vehicular traffic as prioritising. Then, the mobile bridge was ameliorated by changing the characteristic yield strength of the steel from 355 MPa to 450 MPa. This led to the possibility of a change in sectional properties of all the structural elements and thus a variation of bending moment, shear force, axial force, stresses, deflections and reactions in those elements, with a reduction of values while leaving from a normal structural steel (355 MPa) to a high strength steel (450 MPa). A percentage gain of mass up to about 17% was observed while changing the steel type, thus a more economical solution. All these results were made possible due to ease of a dynamic modelling of the structure on Midas civil software assigning correctly restraints at support ends and constraints adequate between structural elements. Also, Microsoft Excel enabled the easy assessment of different results and the elaboration of different curves explaining the variation of solicitations, stresses, deflections and reactions between models with a hydraulic drive, a mechanical drive and an ameliorated model with a hydraulic drive.

GENERAL CONCLUSION

The main objective of this thesis work was to perform a parametric analysis and a structural amelioration of a mobile bridge with an orthotropic steel deck (OSD) under constraints passing through a linear static analysis, using the computer program Midas civil. To accomplish this, a single span bascule steel bridge was used for the study. This study started with a linear static analysis of the mobile bridge in compliance with Eurocodes and the parametric analysis were made in the software Midas civil in order to ameliorate the bridge under constraints using MS Excel software. The work was partitioned in three chapters, the first chapter was a state of art on mobile bridges, types, their mechanisms and functionality, how structural analysis and design can be made and review on orthotropic steel decks as they are used as superstructure for these bridge structures. The second chapter was the methodology, where the modelling procedure of the mobile bridge was explained stressing on adequate support conditions in function of the configuration (closed or opened). Later on, the linear static analysis steps were explained focusing on actions definitions and load combinations definitions in both configurations. Then the procedure of the sensitivity analysis was described varying the angle of opening and the operating mechanism simultaneously and steel type. This led to the procedure of the structural amelioration of the mobile bridge under constraints; notably strength, stresses and deflections. These procedures were implemented with the help of the finite element software Midas civil. In the third chapter, the results of the analyses were presented. Upon static linear and parametric analyses in closed and opened configurations, a reversal of solicitations (bending moment, shear and axial force) is observed when the static scheme changes from a simply supported plate to a cantilevered or fixed plate.

More specifically, the bending moment leaves from 575.8kNm in closed configuration to -672.2kNm in opened configuration at 15 degrees. The maximum negative values of those solicitations at ULS are observed when the bascule bridge is opened at around 30 degrees with respect to the horizontal. At this opened state in the most stressed main girder, the bending moment at the fixed end is observed to be -697.7kNm, the shear force has a value of -257.5kN, the axial force value is -2529kN and the stress is -238.3MPa. Leaving from the structure in place to the ameliorated structure, solicitations, deflections and reactions at ULS experienced a general reduction due to a reduction in sectional properties in the optimised structure that is a bending moment of -611.5kNm, a shear force of -239.6kN and an axial force of -2423.7kN. The stresses on the other hand experienced a general increase in their values again due to reduction of sectional properties with a value at ULS of -275.2MPa.

Verifications in all the structural elements of the OSD in all different cases with different drive mechanisms and in compliance with Eurocode 3 part 2 were all satisfying and were presented in tables of section 3.5.2. Structural optimisation was performed by varying from S355 to S450 with static linear analysis results and the verifications of the structural elements satisfying. After optimisation, the total cross-sectional area left from 293818mm^2 (before amelioration) to 243370mm^2, thus a percentage gain in total volume and total mass of 17.17%. A cost analysis with prices from the actual world market confirm a gain of 562920.4 CFA francs after amelioration.

Having in mind the complex nature related to the conception and analysis of mobile bridges, further studies can be carried out to extend this study in the case of vertical lift and swing bridges; in the structural analysis and design of a bascule bridge under dynamic actions in order to take into account kinematics; to understand the behaviour of the operating mechanism due to acceleration and deceleration of the mobile bridge; to perform a structural amelioration of operating mechanisms regarding the bridge opening time in order to take into account interaction between structural systems.

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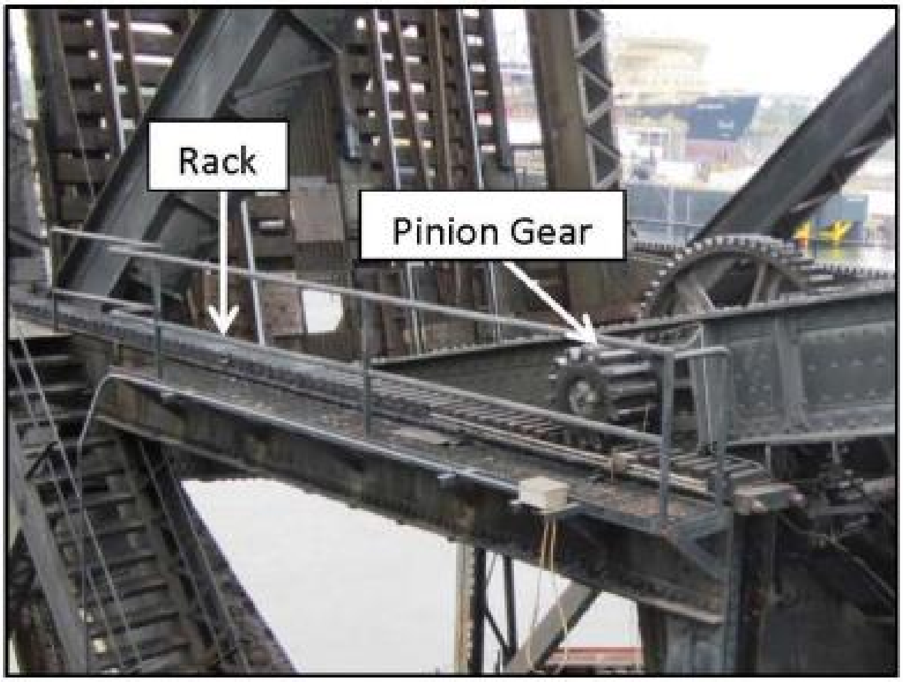
ANNEX

ANNEX A

Annex A1. Bascule bridge presentation



Annex A2. Mechanical drive

Annex A3. Hydraulic drive

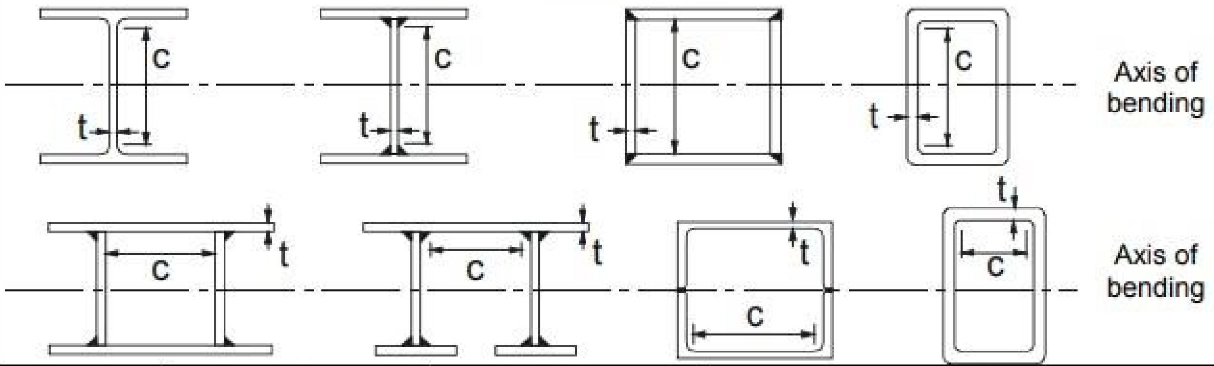
Annex A4. Motor

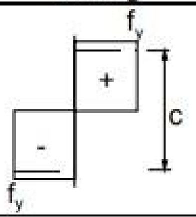
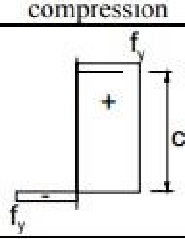
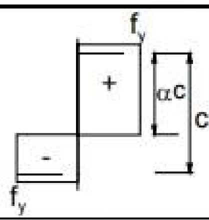
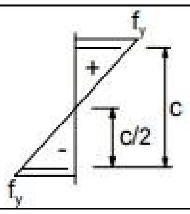
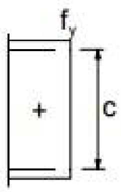
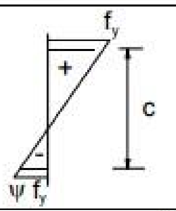
ANNEX B

ANNEX B1. Steel cross section's classification

Table 5.2 (sheet 1 of 3): Maximum width-to-thickness ratios for compression parts

Internal compression parts



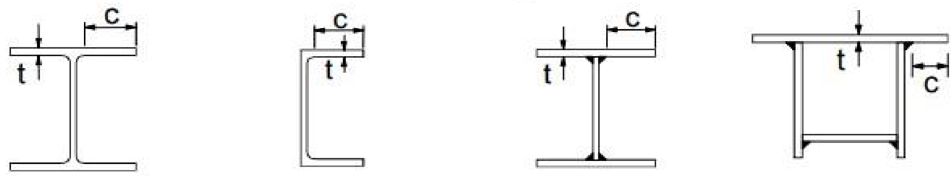
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{36\varepsilon}{\alpha}$
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

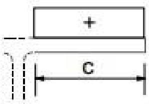
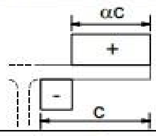
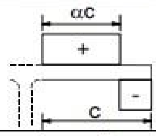
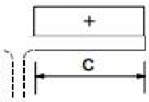
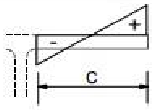
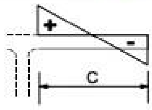
*) $\psi \le -1$ applies where either the compression stress $\sigma \le f_y$ or the tensile strain $\varepsilon_y > f_y/E$

Table 5.2 (sheet 2 of 3): Maximum width-to-thickness ratios for compression parts

Outstand flanges



Rolled sections | Welded sections

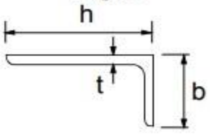
Class	Part subject to compression	Part subject to bending and compression	
		Tip in compression	Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \le 9\varepsilon$	$c/t \le \frac{9\varepsilon}{\alpha}$	$c/t \le \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \le 10\varepsilon$	$c/t \le \frac{10\varepsilon}{\alpha}$	$c/t \le \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$
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

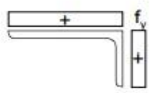
Table 5.2 (sheet 3 of 3): Maximum width-to-thickness ratios for compression parts

Angles

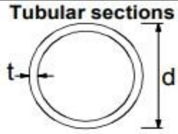
Refer also to "Outstand flanges" (see sheet 2 of 3)



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

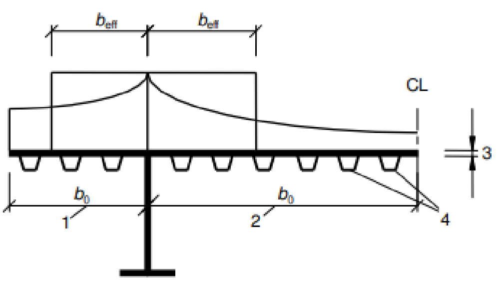
Annex B2. Classification of sections

Structural elements	Classification
Main girder roadway	2
Main girder walkway	1
Cross beam roadway	1
Cross beam walkway	1
Longitudinal ribs roadway	1
Longitudinal ribs walkway	1

Annex B3. Effective width calculation

Effective width under shear lag beff is calculated as prescribed by the expression beff = β b0, with the method of calculation hereby described.

Structural elements	Effective length (mm)
Main girder roadway	2988
Cross beam roadway	1500
Cross beam walkway	1500
Longitudinal rib roadway	280
Longitudinal rib walkway	240



1 for flange outstand
2 for internal flange
3 plate thickness t
4 stiffeners with $A_{s\ell} = \sum A_{s\ell i}$

Table 3.1: Effective[s] width factor β

κ	Verification	β – value
$\kappa \leq 0{,}02$		$\beta = 1{,}0$
$0{,}02 < \kappa \leq 0{,}70$	sagging bending	$\beta = \beta_1 = \dfrac{1}{1+6{,}4\,\kappa^2}$
	hogging bending	$\beta = \beta_2 = \dfrac{1}{1+6{,}0\left(\kappa - \dfrac{1}{2500\,\kappa}\right)+1{,}6\,\kappa^2}$
$> 0{,}70$	sagging bending	$\beta = \beta_1 = \dfrac{1}{5{,}9\,\kappa}$
	hogging bending	$\beta = \beta_2 = \dfrac{1}{8{,}6\,\kappa}$
all κ	end support	$\beta_0 = (0{,}55 + 0{,}025 / \kappa)\,\beta_1$, but $\beta_0 < \beta_1$
all κ	Cantilever	$\beta = \beta_2$ at support and at the end

$\kappa = \alpha_0\, b_0 / L_e$ with $\alpha_0 = \sqrt{1 + \dfrac{A_{s\ell}}{b_0 t}}$

in which $A_{s\ell}$ is the area of all longitudinal stiffeners within the width b_0 and other symbols are as defined in Figure 3.1 and Figure 3.2.