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Analysis of the construction process of a cable-stayed bridge

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Abstract

In the cable-stayed bridge design, one of the stages that it has to be most carefully calculated is the construction process, where the highest stresses are reached. This construction process can be affront different ways depending on the calculation method used, since it increases, decreases or takes into account some efforts created during the erection course.

The purpose of this thesis is to show how does the construction process of a cable-stayed bridge evolves. The efforts created during each stage of the raising of this structure are continuously changing becoming the cable-stayed bridge a highly static indeterminate structure.

Some of the existing calculation methods and approaches will be presented, as it also will several construction methods to have a general vision of the possibilities there are when a cable-stayed bridge has to be designed.

What is going to be seen is the calculation process of the bridge by means of MIDAS/Civil Software, a commercial structure calculation program which is going to guide us through its tutorial to know how does this software works and how does the bridge must be calculated.

Due the tensioning process is always a very expensive operation, what MIDAS is going to try is to calculate those tensioning forces by means of a Final Stage method, this is to say, by knowing the OCS (Objective Completion Stage), to apply them in a second calculation process that will be carried with the objective of just one tensioning operation per stay. Furthermore, that calculation process will be done by means of the Forward analysis, which follows the natural sequence of the erection of the bridge, oppositely the Backward Analysis, which calculates the bridge from the end to the beginning.

The definition of the OSS (Objective Service Stage) for each stage given a load hypothesis will be exposed to compare and have a guideline for when the real construction process is carried out. It also will be presented the axial, shear and bending moments diagram for each stage.

Al this data will be analysed at the end of the thesis extracting some conclusions about the results of cable-stayed bridge's construction process. There will be two analysis: one referred to the construction method, and a second one of the evolutions of each element of the structure.

Keywords: Cable-stayed bridge, Forward analysis, Lack of Fit Force, MIDAS/Civil Software, Cantilever method, Objective Service Stage.

Resumen

En el diseño de puentes atirantados, una de las etapas que tiene que estar más cuidadosamente calculada es el proceso de construcción, donde se alcanzan los mayores esfuerzos. Ese proceso de construcción se puede afrontar de diferentes maneras dependiendo del método de cálculo usado, ya que este tiene puede incrementar, disminuir o tener en cuenta algunos de los esfuerzos creados durante el proceso de erección.

Esta tesis pretende mostrar cómo evoluciona el proceso constructivo de un puente atirantado. Los esfuerzos creados durante cada etapa del levantamiento de la estructura cambian continuamente transformando el puente en una estructura estáticamente muy indeterminada.

Algunos de los métodos de cálculo y aproximaciones existentes se van a presentar, así como algunos métodos constructivos para tener una visión general de las posibilidades que hay a la hora de diseñar un puente atirantado.

Lo que se va a ver es el proceso de cálculo de un puente usando Midas/Civil Software, un programa de cálculo que va a guiarnos a través de su tutorial para conocer cómo funciona este software y cómo se debe calcular un puente atirantado.

Debido a que el proceso de tensionado siempre es una operación muy cara, lo que MIDAS intentará es calcular esas fuerzas de tensión mediante un método de Etapa Final, es decir, conociendo el OCS (Objective Completion Stage), para aplicarlas en un segundo proceso de cálculo que se llevará a cabo con el objetivo de realizar una sola operación de tensado por cable. Además, ese proceso de cálculo se realizará mediante el Análisis Forward, que sigue la secuencia natural de la construcción del puente, opuestamente el Análisis Backward, que calcula el puente desde el final hasta el principio.

La definición de la OSS (Objective Service Stage) para cada etapa dada una hipótesis de carga se expondrá para comparar y tener una guía para cuando se lleve a cabo el proceso de construcción real. También se presentará el diagrama de momentos axiales, de corte y de flexión para cada etapa.

Todos estos datos serán analizados al final de la tesis, extrayendo algunas conclusiones sobre los resultados del proceso de construcción del puente atirantado. Habrá dos análisis: uno referido al método de construcción y otro de las evoluciones de cada elemento de la estructura.

Palabras Clave: Puente atirantado, Forward Análisis, Lack of Fit Force, MIDAS/Civil Software, método de construcción en voladizo, Estado de Servicio Objetivo

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Chapter 1 – Introduction and Objectives

Background

Nowadays, the evolution of engineering and the eagerness of constructing higher, longer and larger structures requires an increase of wit not only on design and calculation, but also when the idea incorporated into a plan must be carried to reality. That is why more and more construction methods appear with which some designs can be brought and years ago they were limited by their construction method.

The bridges capture very well that evolution which, after all, is also the evolution of structural engineering because it takes us from rudimentary bridges built with a trunk dropped over a stream over 2000 years ago, to engineering wonders that challenge the forces of nature and physics today.

Today, there are many types of bridges that have been appearing over the years: arch bridges, beam bridges, suspension bridges, gantry bridges, suspension bridges, floating bridges ... and each of them requires a specific construction method, or more than one depending on the type of bridge and the environment where it is built.

The advancement of technology allows to be adapted better and better to the terrain under construction either from more specific machinery, from more advanced materials or from increasingly developed design programs that manage to capture behaviors more adapted to reality, such as the way in which environmental factors affect the structure, or how it reacts to certain events whose probability of happening is relatively low. All this makes it possible to build safer structures with a longer useful life and that, after all, is what we are looking for.

Although the technology of 10 years ago is already obsolete for the processes for which it was conceived due to the rapid growth that we suffer, technological advances continue to arrive to cover the needs that appear from the theoretical processes that require them. It must be considered that apart from all existing practical constructive methods, the ones that are currently only theory and those that are improved versions of those currently used, must be added.

Hence the need to continue researching and studying the new construction processes that appear, comparing them with other construction techniques in order to optimize the method we use to create the structures of tomorrow.

Objectives

The objectives of this thesis look for reveal the behaviour of a cable-stayed bridge under construction, stage by stage, taking a look on the differences between calculate a cable-stayed bridge with the *Final Stage* analysis in comparation with the same bridge calculated with the *Forward Stage* analysis to see the importance of a good-choosing method to analyse the construction process of this type of bridges and the influence this election has on them.

Main Objectives

For the construction of a cable-stayed bridge there is several things that must be taken into account. In order to achieve efficiency, a deep study of each part or element necessary for the construction must be done. The quality of materials, modern technologies for machinery or the good qualification of workers are some to achieve the success.

On the other hand, before the construction process there are a lot of things that must be worked, and on which the construction method will depend on. These are the things this thesis will have as a target to show.

The first objective for the successful construction of any structure is the importance of a good bridge design. This work will expose the different calculation methods with its functions, the parameters they take into account and the procedure suited.

After a good design, the analysis of the results becomes a required task to understand how does the structure work and in order to know the necessary actuations that must be done to ensure a safe construction process.

As a third target, this thesis will look for the knowledge that will allow to choose the rightful construction method as the optimum for each type of bridge, and in which situations it will be able to be applied. This is to say that the properties of each construction method will be presented to know the ambit of its application. The importance of choosing a good method is directly related with the composition of the bridge, since if the bridge is built only with concrete, phenomena as creep or shrinkage will have to be taken into account. By the way, in this tutorial those phenomena will not be taken into account.

It will also be an objective to make us realise that the construction process is most of times when the structure experiments the highest stresses and not during its service stage.

And obviously, the discovering of the world of cable-stayed bridges and its design methods will be also a target wanted to achieve in this thesis.

Specific objectives

The specific objectives are coming from the main ones, since there is some aspects must be taken into account from each one of those. It is true that in what this thesis wants to deep is the construction process of a cable-stayed bridge, but there are some of the parts from the construction methods that need to be explained to know how the calculation process uses the different systems to reach the better choosing of the construction procedures in the design of those structures.

The explanation of each calculation method will be exposed and some examples of bridges constructed that way will be presented in order to better understand how do they work. The three calculation methods announced on the abstract will be developed in a way the main differences between them can be noticed.

On the other hand, what will this thesis focus in, is in the construction process of a cable-stayed bridge by means of a deep analysis of each stage of the construction process. It will be necessary to mention the evolution stage to stage to see the importance of the order in the construction process.

The behaviour of all the parts composing the cable-stayed bridge will be also studied. Each element of the bridge has different properties so it is being waited that the way they react to the same efforts has different paths, and it is important to analyse how do the structure affront these different reactions from each part of the cable-stayed bridge.

It must be taken into account that this thesis will work with a non-constructed bridge, since what it is analysed is a bridge proposed by MIDAS/Civil Software tutorial's. In any case, the purpose of the tutorial is not only to learn how to use this software, that it also is, but the way to extract the results of the calculation to be evaluated.

Apart from the realisation of the tutorial, another specific target will be the development in the learning of structures behaviour, which will require the understanding of the way that the bridge reacts to all imposed efforts and balances itself making the materials resist those forces.

Even if the material characteristics are not deeply studied since its properties are just introduced like the software tells to do it, it is necessary to realise that every material has a lot of parameters that must be controlled to avoid its collapse. Thus the analysis of the efforts suffered in each stage will can be used as a guide to know where and when the different types of materials must be used to resist all the construction process.

Methodology

The steps that are going to be followed during this thesis to achieve the targets presented previously, are outlined on the table below.

Step	Description
1	Explanation of construction methods and analysis algorithms
2	Design of a cable-stayed bridge by means of MIDAS/Civil Software following the tutorial that the program offers.
3	Analysis of all the construction stages, one by one, commenting axial, shear and bending moments diagrams, and deformed shape.
4	Analysis of the evolution of the construction process and MIDAS/Civil Software obtained results.
5	Conclusions.

Chapter 2 – State of art

History, Facts and types of cable-stayed bridges

As it can be found on [1] and [2], a bridge is a structure which is built over some physical obstacles such as a body of water, valley or road, and its purpose is to provide crossing over that obstacle. It is built to be strong enough to safely support its own weight as well as the weight of anything that should pass over it. Bridges were and can be built out of different materials and in different designs, depending on its intended function, terrain where the bridge is built, the material used to make it, and the available funds.

The first bridges appeared in nature by themselves. A log could fall across a stream and form a natural bridge or stones could fall into a river from a nearby cliff. The simplest type of bridge is stepping stones, so this may have been one of the earliest types. When humans started building bridges, they built them in simple form out of cut wooden logs or planks, stones, with a simple support and crossbeam arrangement, sometimes with use of natural fibers woven together to hold materials. One of the oldest arch bridges from the Hellenistic era in existence is *Arkadiko Bridge* in the Peloponnese, Greece. It dates from the bronze age, 13th century BC and it was part of a former network of roads, designed to accommodate chariots, between the fort of Tiryns and town of Epidauros in the Peloponnese.

1. ARKADIKO BRIDGE (13TH CENTURY BC), PELOPONNESE, GREECE

Ancient Romans were the greatest bridge builders of ancient times. They built arch bridges and aqueducts that could stand in conditions that would damage or destroy earlier designs. Some of them still stand today. An example is the *Alcántara* Bridge, built over the river Tagus, in Spain. They also used cement which reduced the variation of strength found in natural stone and consisted of water, lime, sand and volcanic rock. Brick and mortar bridges were built after the Roman era, as the technology for cement was lost (then later rediscovered). Some of their most beautiful bridges were built over ravines while over rivers where no rock or island emerges from the water to carry the piers.

Indians also built bridges, whose are documented in their ancient text the *Arthashastra* which was written between 4th and 3rd century BC. […] They used plaited bamboo and iron chain as materials.

2. PICTURE OF AN ANCIENT INDIAN BRIDGE NEAR PENIPE, ECUADOR

Although large Chinese bridges of wodden construction existed at the time of the Warring States period, the oldest surviving stone bridge is the *Zhaozhou Bridge*. It was built from 595 to 605 AD during the Sui Dynasty. This bridge is also historically significant as It is the world's oldest stone segmental arch bridge built with open spandrels.

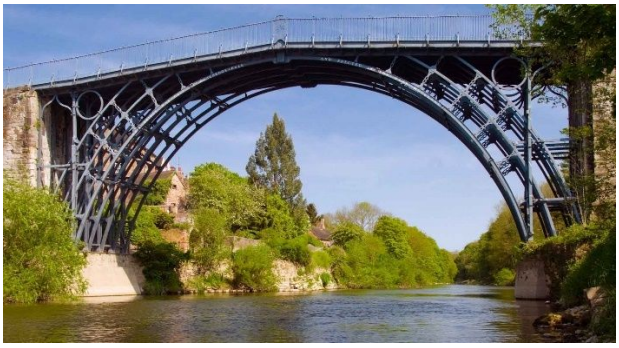
3. ZHAOZHOU BRIDGE (605 AD) IN ZHAO COUNTY, CHINA

Between 12th and 16th century many bridges were built with houses on them. They were the solution for limited accommodation in walled cities and only France had as many as 35.

4. VIEW OF THE ANCIENT PONT-AU-CHANGE, PARIS

Inca civilization in the Andes mountains of South America used rope bridges, a simple type of suspension bridge, just prior to European colonization in the 16th century. Hans Ulrich, Johannes Grubenmann, and others improved bridge-building in the 18th century. At the same time, Hubert Gautier wrote the first book on bridge engineering (1716).

A major breakthrough in bridge technology came with the erection of *The Iron Bridge*, built in Coalbrookdale, England in 1779. It was one of the engineering marvels of the time because it used cast iron for the first time.



5. THE IRON BRIDGE (1779), IN COALBROOKDALE, ENGLAND

Industrial revolution in the 19th century brings truss systems of wrought iron (an iron alloy with a very low carbon) but it did not have the tensile strength to carry the large weights. Enters steel, with its higher tensile strength, which replaces the iron and allows for much larger bridges. Gustave Eiffel, with his fresh ideas, was on of the first to use it.

In Canada and the U-S, numerous timber covered bridges were built in the late 1700s to the late 1800s, reminiscent of earlier designs in Germany and Switzerland. In later years, some were partly made of stone or metal but the trusses were usually still made of wood; in the U.S., there were three styles of trusses, the *Queen Post,* the *Burr* Arch and the *Town* Lattice. these structures sti

The first welded road bridge in the world was built by welding pioneer Stefan Bryla in 1927.

6. STEFAN BRYLA'S BRIDGE (1927), IN MAURZYCE, POLAND

With Industrial Revolution many different types of bridge appear and became possible because of technological advancements.

Cable-stayed-bridge is a structure formed, normally, by two pylons, a deck and several cables that connect the deck and the pylons. Is a structure similar to suspended bridge, that has also its towers and a deck held by cables, but in this case, the cables connect directly to the pylon and not to the suspender cables.

This kind of bridge is usually used to replace a cantilever bridge because of the necessity of longer spans but not as long as for a suspension bridge to be built, for economic reasons.

The first person to design a cable-stayed bridge was the venetian inventor Fausto Veranzio in 1595, who was also the first to design a modern suspension bridge. But it was not until 19th century when the first cable-stayed bridge was built and many early suspension bridges were cable-stayed like footbridge *Dryburgh Abbey Bridge*, *James Dredge's Victoria Bridge,* in Bath, England (built in 1836), *Albert Bridge* (built in 1872) and *Brooklyn Bridge* (built in 1883).

7. ALBERT BRIDGE (1872) IN LONDON, UK AND BROOKLYN BRIDGE (1883), IN BROOKLYN, NEW YORK

Concrete-decked cable-stayed bridge over the Donzère-Mondragon canal at Pierrelatte was designed by Albert Caquot in 1952 and was one of the first modern cable-stayed bridges but it had not a lot of influence on the posterior bridges, since there is no other that came after looked up to it. So it can be assumed, that the first modern cable-stayed bridge was the *Strömsund Bridge* designed by Franz Dischinger in 1955 in Sweden. This bridge was the beginning of the modern cable-stayed bridges construction.

8. DONZÈRE-MONDRAGON CANAL'S BRIDGE AT PIERRELATTE AND STRÖMSUND BRIDGE IN STRÖMSUND, SWEDEN.

There is many ways to build a cable-stayed bridge:

- **A side-spar cable-stayed bridge [3]**. This kind of cable-stayed bridge has only one tower and is supported only on one side, this is to say, the cable support does not span the roadway, rather being cantilevered from one side.
  The cable paths of this structures are aligned with the bridge centreline and only differs, structurally talking, in the way that the stresses are transferred through the tower to the foundation. However, this is not the only way to construct this kind of bridges. As in *Jerusalem Chords Bridge*, the deck can also be designed to be curved, so the tower could be offset and the bridge deck wrap around the spar in an arc.

9. JERUSALEM CHORDS BRIDGE, BUILT IN 2008 BY SANTIAGO CALATRAVA

- **Cantilever-spar cable-stayed bridge.** [4] explains that these bridges are a modern variation of the cable-stayed bridge and they are constructed with a single cantilever beam on one side of the section. Its mast is made to resist the flexion caused by the cables, since the cable forces of this bridge are not balanced by opposite cables and the bridge applies a large tipping force on its base. This design was initiated by the structural engineer Santiago Calatrava in 1992 with the Alamillo Bridge in Seville, Spain. The main skill of this bridge is that the angle of the bar away from the bridge and the weight distribution in the crossbar serve to reduce the tipping forces applied to the foot of the crossbar. On the other hand, the same engineer designed another bridge, El Puente de la Mujer (2002) in Buenos Aires, Argentina, where the mast extends to the deck with cable support and is counteracted with a structural tail.

10. PUENTE DEL ALAMILLO (1992), IN SEVILLE, SPAIN AND PUENTE DE LA MUJER (2002), IN BUENOS AIRES, ARGENTINA.

- **Multiple-span cable-stayed bridge.** Is a cable-stayed bridge with more than 3 spans. Is a bridge with a difficult design because the loads from the main spans are not anchored back near the end abutments by stays. This also makes structure less stiff so additional design solutions, like “cross-bracing” stays and stiff multi -legged frame towers, have to be applied. A “cross-bracing” stays bridge example is the *Ting Kau Bridge*, where this technique was used to stabilise the pylons. On the other hand, *Millau Viaduct* and *Mezcala Bridge* are an example of twin-legged frame towers, or *General Rafael Urdaneta Bridge*, where very stiff multi-legged frame towers were adopted.

11. TING KAU BRIDGE (1998), IN HONG KONG, CHINA

12. MILLAU VIADUCT (2004), IN MILLAU, FRANCE AND MEZCALA BRIDGE (1993), IN GUERRERO, MEXICO

13. GENERAL RAFAEL URDANETA BRIDGE (1962), IN MARACAIBO, VENEZUELA

- **Extradosed bridge.** As can be found on [5], an extradosed bridge employs a structure that combines the main elements of both a prestressed box girder bridge and a cable-stayed bridge. The main difference between other cable-stayed bridge and an extra-dosed bridge is that this kind of bridges uses much shorter stay-towers or pylons so they must have stiffer and stronger decks. Its cables are also connected to the deck further from the pylons given the stiffness of the deck. The extradosal bridge form is mostly suited to medium-length spans between 100 and 250 metres, and over fifty bridges had

been constructed around the world until now, but as the material costs are reasonably elevated for this kind of bridges, they have frequently been adopted when overall height, navigation clearance, or aesthetic requirements have made the cable-stayed or girder alternatives less feasible. The first extradosed bridges were the *Ganter Bridge* and *Sunniberg Bridge*, in Switzerland. Nowadays, we can find modern extradosed bridges like *Earthquake Memorial Bridge*, in Muzaffarabad, Pakistan, or *Harpe Bridge,* the first extradosed bridge in Scandinavia.

14.GANTER BRIDGE (1980), IN VALAIS, SWITZERLAND AND SUNNIBERG BRIDGE (1998), IN KLOSTERS, SWITZERLAND.

15. EARTHQUAKE MEMORIAL BRIDGE (2014), IN MUZAFFARABAD, PAKISTAN, AND HARPE BRIDGE (2016), IN NORWAY

The name comes from the word *extrados*, the exterior or upper curve of an arch, and refers to how the "stay cables" on an extradosed bridge are not considered as such in the design, but are instead treated as external prestressing tendons deviating upward from the deck. In this concept, they remain part of (and define the upper limit of) the main bridge superstructure.

- **Cable-Stayed cradle-system bridge.** This type of bridges is one of the newest variants. It has so called “cradle-system” which carries the strands within the stays from bridge deck to bridge deck. Each epoxy-coated steel strand is carried inside the cradle in a

2.54cm steel tube. These cables are continuous which means that this bridge has no anchorages in the pylons and its cables can be removed, inspected and replaced individually. The first two bridges built that way are *Penobscot Narrows Bridge*, in Prospect, Maine; and *Veterans' Glass City Skyway,* in Toledo, Ohio.

16 PENOBSCOT NARROWS BRIDGE (2006), IN PROSPECT, MAINE AND VETERANS' GLASS CITY SKYWAY (2007), IN TOLEDO, OHIO

Advantages and Disadvantages of Cable-Stayed Bridges

The advantages and disadvantages of cable-stayed bridge must involve more than consistency and cost. The needs of the actual span must be considered, due the increasing amount of traffic the bridges have to support and the potential disaster could impact to the structure on day. By the way, cable-stayed bridge are considered the most suitable option to cross any span, for this reason, the actual advantages and disadvantages of this kind of structures are presented.

Advantages

Cables-stayed bridge is a type of bridge that offers several advantages but also some disadvantages. Since the appearance of those structures, there has been a lot of improvements on the way is being designed and constructed. For instance, the time required for the complete construction of cable-stayed bridge is significantly lower than other structures, since it does not need the same levels of anchoring that you will find in alternative designs. There is also a decreasing of time because not many cables are needed to support the deck, unlike suspended bridges.

If cable-stayed bridge is directly compared with suspension bridge, because of the similar design, I can be said that the first one offers more strength to span a gap than the latter. cable-stayed bridge can handle more pressure on a consistent basis compared to the suspension design, and it allows the deck to have more resilience against wear and tear because there is greater rigidity in its construction.

Economics is also an advantage for cable-stayed bridge, since there are less elements to consider with this design, and the installation costs can be significantly lower because there are fewer manhours involved. A lot of designs are around 30% cheaper to construct than other options making cable-stayed bridge a very attractive choice.

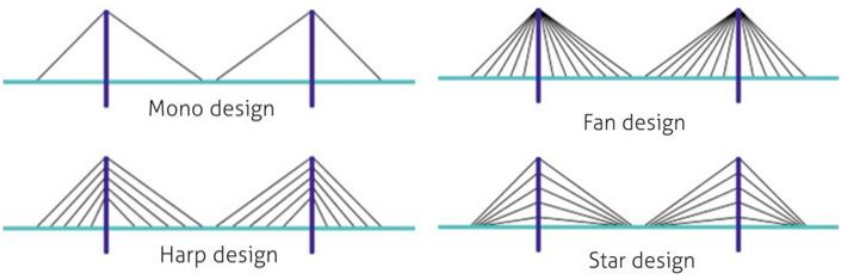
Referring to size-bridge, cable-stayed bridge can be constructed for almost any length. Although the span length is restricted because of its design, what makes cable-stayed bridge unique is that engineers can connect different spans together with the support pylons to create a bridge of almost indefinite length. For instance, Jiaxing-Shaoxing Sea Bridge [18] is considered the longest cable-stayed bridge in the world, with 2.680 m of main span, and 69.5 km of total length.

17. JIAXING-SHAOXING SEA BRIDGE, THE LONGEST BRIDGE IN THE WORLD

As it has been seen before, cable-stayed bridge has multiple design option: side-spar bridge, multi-spar, cantilever spar, extradosed, etc. Like this, engineers can choose among all those types to create the most adapted bridge to their conditions.

Another advantage that it must be taken into account, is that cable-stayed bridge is a self-supported structure, since the cables that are used to create consistency and stability provide the structure with the temporary and permanent supports it requires simultaneously. The more weight is added to one specific section, the more the cables help to displace the extra pressure throughout the remainder of the structure to prevent excessive stresses.

Some designs of cable-stayed bridge offer the possibility of a symmetrical design, and that brings more stability and strength. But in that symmetry, there are also different ways to place the cables. The four most recognized types are presented on the next figure.



18. CABLES DISTRIBUTION TYPES

Disadvantages

As every structure, cable-stayed bridge has also some disadvantages that must be commented. For instance, even if the bridge can have almost indefinite total length, it does not occur when span-size is considered. With all new technologies helping to calculate the stresses a cable-

stayed bridge can hold, the limit of size-span for a single span option reaches 1.100 m in length, which is not negligible at all.

Although a cable-stayed bridge can be placed almost everywhere, there are some specific places where it is not very suitable to built it. Consistently high winds environments does not work very well because of the rigidity that the cables provide for the overall structure.

The maintenance and repairing of a cable-stayed bridge can be also challenging because of the bundle areas for the support structures, which are placed in regions where a physical inspection can be hardly reachable.

Due the fact that most of cable-stayed bridge that are built today use a combination of concrete and steel to create a rigid structure and unless there are protections in place that maintain the quality of the metals used for the span, the support cables can be highly susceptible to corrosion and rust.

Construction Methods

There are many ways to construct a cable-stayed bridge, such as with temporary supports method, with cantilever method or launching girder.

To understand the way that cable-stayed bridges are constructed, a short explanation of the two construction methods, related with the purpose of this thesis, will be exposed. These methods are: temporally supports method and cantilever method.

Temporary Supports Method

The temporary supports erection method is the most used way of building cable-stayed bridge. In this technique the bridge deck is first built over a set of temporary and permanent supports. Then, during the tensioning process, the stays are successively placed and tensioned by the jacks and the deck is raised from the temporary supports.

The main advantage of the temporary supports erection method is that conventional construction techniques can be used and therefore, both, construction cost and period can be significantly reduced compared with its opposite erection technique, the cantilever method.

On the other side, this method has a huge disadvantage, and this is that it is very site-dependant because the temporary supports cannot always be placed, such as when the bridge has to cross a river or a very deep span.

19. SUTONG CABLE-STAYED BRIDGE - CONSTRUCTION PHASE

Cantilever Method

Once the limitations of the temporary supports method are presented, it is obvious that a new construction method must appear to fil this gap, and that is where the cantilever erection method had to be invented.

With this technique, the bridge is built by means of the placement of deck segments in cantilever. These segments are supported by stays located in the alternative sides of the pylon to keep the balance. Once the segments in cantilever are fixed and supported, the next segment is placed.

The main advantage of this method is that there is no site-limitation, just the way that the material and the machinery are placed on the pylon that will be used to start the construction.

20. RUSSKY ISLAND BRIDGE - PRECAST CANTILEVER METHOD

Calculation Algorithms

The studies that bring the realization of this thesis, are looking to show the evolution of stresses on construction process of a cable-stayed bridge, approaching the differences between the design of a cable-stayed bridge using the *Final Stage* method and the *Forward Stage* method.

It is important to say that for bridges design, and for any construction, a final load must be fixed as the target the structure will carry with once is built, and it has to be able to hold it without structural problems. This is why Objective Completion Stage (OCS) and Objective Service Stage (OSS) will be presented.

Objective Completion Stage (OCS)

As [11] and [12] expose, the Objective Completion Stage is the target scheme of forces or deflections that is defined according to the designer criterion.

This scheme is the one that must be achieved once the construction of the bridge is finished assuring that the structure will hold those stresses, this is to say, the bridge will be designed from those forces, using materials, lengths and heights according to them.

Objective Service Stage (OSS)

From the Objective Completion Stage, explained before, it can be extracted the Objective Service Stage, which, like is defined on [11], [12] and [13], is the target geometry and/or stress state in service, usually defined by the designer in early stages of design.

This Stage satisfies the stress distribution pursued by the designer in such a way that under certain load hypothesis, like Target Load, the stays present a given vector of forces $\{N^{OSS}\}$. This stage will be achieved when Target Load is applied into the structure.

As it is known, a cable-stayed bridge is a highly statically indeterminate structure, that means that balance, compatibility or stiffness equations are not enough to determinate unknown strains, such as the pretensions of the cables, whose appear during design and construction stages of cable-stayed bridge. Therefore, and like MIDAS/Civil Software shows on its tutorial, to find the initial pretension cable forces, the *Unknown Load Factor* method has to be applied.

It can also be said that the behaviour of this kind of bridges is nonlinear, so it makes you to face a continuously changing static scheme. However, there are some different methods to calculate the pretension of the stays like, for example, the *Backward Algorithm (BA),* the *Direct Algorithm (DA)*, the *Forward Algorithm (FA)*, the *Genetic Algorithm (GA)*, or the *Final Stage,* among others.

Those methods are going to be presented, explaining some properties of them.

Backward Algorithm (BA)

To define the *Backward Algorithm,* it could be said that it uses the disassembling of a cable-stayed bridge according to the opposite sequence of events which occur during its erection to calculate the stresses of the stays. Due its properties it is specifically based in peculiarities of cable-stayed bridges built with temporary supports. As [11] explains, its simplicity advantages the possibility to be reproduced by any structural code that permits the modelling of the pre-stresses of the stays by means of imposed strains or imposed temperature increments.

The changes on the structural system and the tensioning process are simulated by means of the stage superposition principle from a backward approach. To simulate the non-linear effects of the raising of the temporary supports during the tensioning process, a *Local Iterative Process (LIP)* is used. On that process, temporary supports are modelled as vertical boundaries which activation depends of vertical deformations on the girder, where the temporary supports are placed.

One of the main advantages of this procedure is that there is no need to use separate models to calculate the evolution of the stresses in the strands when strand by strand tensioning technique is used. Furthermore, the stays elongations when prestressed can be easily obtained when the stays are prestressed in a single operation or strand by strand

On the other hand, one of the main disadvantages of this procedure is that time-dependant phenomena, such as creep, shrinkage or cable relaxation, cannot be directly computed as the analysis is performed according to the reverse time direction.

The process of this algorithm is outlined as follows.

- In the initial stage, the self-weight of the structure, g1, is counterbalanced by a set of temporary and permanent supports. This way, vertical reactions in both supports are found.
- Then, during the tensioning process, the stays are successively placed and tensioned by the jacks and the deck is raised from the temporary supports. In these stages, the load g1, is counterbalanced by the non-raised supports and by the tensile forces introduced into the placed stays.
- When the tensioning process is completed, the final desired stage, **Objective Completion Stage (OCS)**, is achieved.
- This stage can be easily calculated from the **Objective Service Stage (OSS)**, once the target load is applied to the structure.

Direct Algorithm (DA)

On the other hand, when the *Direct Algorithm* is used, on [13] explains that it appears up the necessity of the application of the unstressed length of the stays concept into the modelling of the construction process of cable-stayed bridge, which is the main point of this algorithm, and it is normally used on the construction of cable-stayed bridge with prefabricated cables. The concept of *unstressed length* of the stays means that the elongation caused because of the difference of length between the stay measured horizontally without any axial stress or strain (even the selfweight) and the final position of this stay once is already on site, is equivalent of introducing an imposed strain. As it will be explained on next chapter, MIDAS/Civil Software solves this concept with Lack of Fit Force function. As this strain normally presents a value close to 0.003, the clear disadvantage of the measuring on site given the high sensitivity to geometrical tolerances converts this method into a rarely used technique.

Like *Backward* Algorithm, is specifically based on the peculiarities of the temporary supports construction method. The simulation follows a direct approach where next or previous stage information is required. On that approach, any stage must be analysed by an independent Finite Element Model.

Stage superposition principle is not applied on *Direct Algorithm*, so the time-dependant phenomena can not be easily computed and this procedure is limited, for now, to cable-stayed bridge with steel cables.

Cable forces are simulated with imposed strains or temperature variations. Direct Algorithm does not need separate models to calculate the evolution of the stresses on the stays, even when the strand by strand tensioning technique is used.

Non-linear effects of the raising from temporary supports are also simulated with a Local Iterative Process, but in this case, no auxiliary models are required, since the structural scheme of the Finite Element Model of the stage is directly updated.

To avoid the necessity of an Overall Iterative Process, the algorithm uses the cable's non-tensioned length.

Forward Algorithm (FA)

Unlike the method of Backward approach, the Forward approach method can easily include the calculation of the time-dependant phenomena such as creep, shrinkage or cable relaxation and that brings a reduction of the construction time, an increasement in safety and a better control of the construction process. To define this method, it could be said that this algorithm follows

the erection sequence to fulfil the purpose that constructions stages should be composed in a way that they are included and should be used to heck the structural safety. Using his method, stresses in construction stages, constructions sequence and possibility of construction can be known to choose an optimal construction method.

Like *Backward* and *Direct Algorithm*, [12] explains that *Forward Algorithm* is also specifically based for the peculiarities of the temporary support construction method. Changes on structural system and tensioning process are simulated by means of stages superposition principle from the forward approach. It also uses a Local Iterative Process to calculate the non-linearities of the raising of the temporary supports during the tensioning process and the forces on the cables are simulated by imposed strains on the stays, as *Backward Algorithm* does.

To ensure the Objective Completion Stage at the end of the tensioning process, a Global Iterative Process is used, which update the imposed strains to introduce them on the N last tensioning operations.

This algorithm is a useful tool to correct detected deflections on the tensioning process *in situ*. The temperature variations must be included on the analysis adding the effects to a suitable auxiliary model.

This procedure will be the one used on MIDAS/Civil Software tutorial for the design of a cable-stayed bridge.

Articles like [10] explains that depending on the erection method, the structural system of a cable-stayed bridge can be changed greatly and that change can cause an unstable state during construction. So construction stages should be composed in a way that they are included and used to check the structural safety. Through the *Forward Analysis*, stresses in construction stages, construction sequence an possibility of construction can be verified and an optimal construction method can be selected.

Chapter 3 – MIDAS/Civil Software Tutorial

Introduction

The purpose of this thesis is to calculate a standard three span continuous cable-stayed bridge to see the axial, shear and bending efforts on the structure during each stage of construction. Using MIDAS software a 40+110+40m span bridge will be projected, with two 60m-pylon (divided in two: 20m + 40m) and 4 stays on each one.

This is the purposed bridge that MIDAS software gives as a tutorial to learn how to use the program. However, a deep analysis is needed to show how does the construction process must be, this is to say, we're going to stop and analyse each stage to have a reference model to compare with the real construction stages.

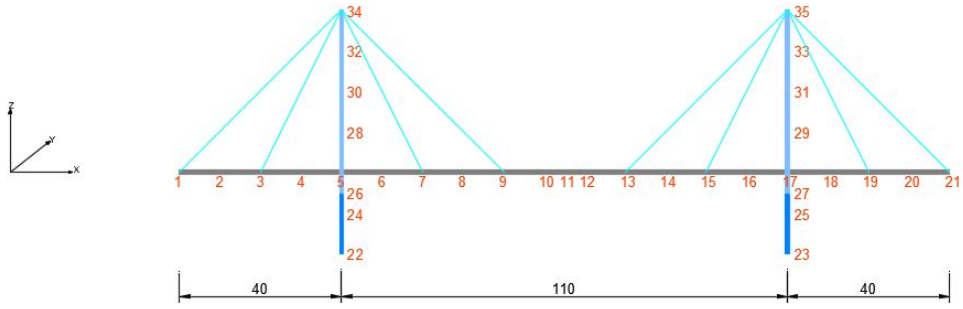
Material and Section Properties

First of all, the material, section and geometry properties must be introduced in the software, and to simplify the model and obtain a focused results, since the objective of this exercise is to evaluate the construction process, just four elements will be calculated: the girder, the cables, and the tower (which is going to be splat in two: lower and upper tower).

The given properties for the 4 elements are presented in the next table:

	Modulus of Elasticity (tonf/m^2)	Poisson's Ratio	Weight Density (tonf/m^3)	Area (m^2)	Ixx (m^4)	Iyy (m^4)	Izz (m^4)
Girder	$2.1x10^7$	0.3	7.85	0.8	15.0	1.0	15.0
Lower Tower	$2.5x10^6$	0.17	2.5	50	1000.0	500.0	500.0
Upper Tower	$2.1x10^7$	0.3	7.85	0.3	5.0	5.0	5.0
Cable	$1.57x10^7$	0.3	7.85	0.005	0.0	0.0	0.0

After the introducing of this data, the geometry is raised.



As told, we have got a cable-stayed bridge with three span (40+110+40), two towers (splat in two) and four cables for each tower. It is necessary to create a link between nodes 26 and 5, and the same link between 27 and 5 for a good connection between the two parts of both towers, so a movement limitation in the three axes must be given. The more preoccupant movement that is wanted to be avoid, is the movement on axis Y because is the weakest direction, thus that's why a resistance value of SDy = 100.000.000 tonf/m is given in that direction.

The other two directions are less preoccupant because of the geometry of the bridge. A force on axis X should push the whole structure, so not too much elevated resistance is needed. And it is even less worrying on Z direction.

But it is not only those elements that must be controlled. Any structure must have a boundary condition on its supports. In this case, the tower bases will have a fixed condition, that means all movements and rotations will be blocked. On the other hand, on the pier bases, movements on X axis and rotation on Y axis, will be allowed. […]

Unknown Load Factor function

After all these adjustments on the static scheme of the bridge, the most difficult part arises. In many calculation methods, there is normally more than one tensioning operation because of the relaxation of the first placed cables after the lasts are putted on site. This is to say, the tensioning force given by the jack to the last placed cable, makes the axial forces on the first placed cables are reduced making those cables working on a lower force as the one projected.

It is obvious to say that this lower force creates an imbalance that makes the structure fight against unwished efforts, so it is necessary to use the jack again to rebalance the tensions on the towers, but a second use makes the construction bridge more difficult and expensive. So

that is what is wanted to be avoid through the search of a prestress tension that equilibrates all the forces once the last cable is set.

As the tutorials says, the initial cable prestress, which is balanced with dead loads, is introduced to improve section forces in the main girder and tower, cable tensions and support reactions of the bridge. It requires many iterative calculations to obtain initial cable prestress forces because a cable-stayed bridge is a highly indeterminate structure. And there are no unique solutions for calculating cable prestress directly. Here is where the **Unknown Load Factor (ULF)** function in MIDAS/Civil comes to scene.

This function is based on an optimization technique, and it is used to calculate optimum load factors that satisfy specific boundary conditions for a structure. It can be used effectively for the calculation of initial cable prestress.

The procedure of calculating initial prestress for a cable-stayed bridge by Unknown Load Factor function is outlined in 6 steps here below:

Step 1: Cable-stayed bridge modelling

Step 2: Generate Load Conditions for Dead Loads for Main Girder and Unit Pretension Loads for Cables

Step 3: Input Dead Loads and Unit Loads

Step 4: Load Combinations for Dead Loads and Unit Loads

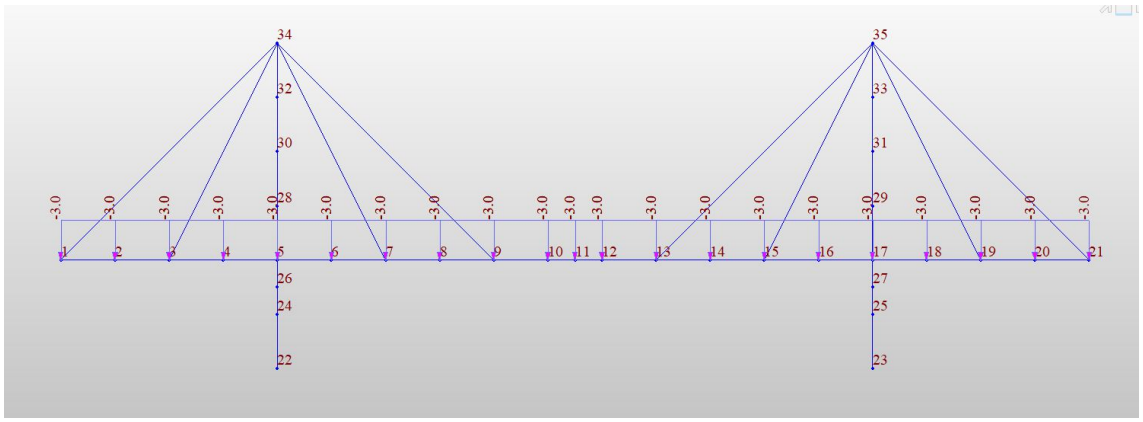
Step 5: Calculate unknown load factors using the Unknown Load Factor function

Step 6: Review Analysis Results and Calculate Initial Prestresses.

As the bridge studied is a symmetrical structure, the number of required unknown initial cable prestress values will be set at 4 and the loading conditions for self-weight, superimposed dead load and unit loads for cables to calculate initial prestresses for the dead load condition will be imputed.

The load conditions that are going to be used are: Self-weight, Additional Load, 4 Tensions load conditions (1 to 4), and Jack Up.

For Additional Load, the tutorial propose a 3 tonf applied on each node of the girder as it is seen on the next figure.



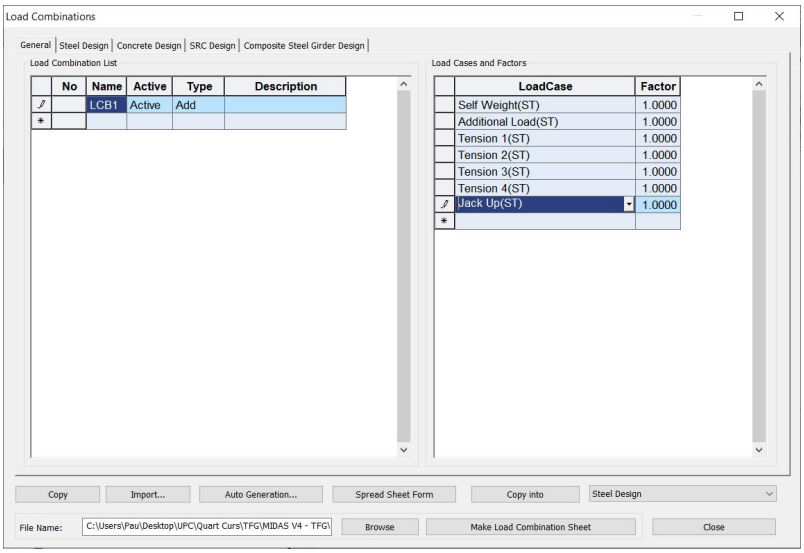
21. ADDITIONAL LOAD INPUTTED ON THE STRUCTURE

To finish with the first part of the tutorial, a vertical displacement value of 0.01m will be assigned to Jack Up Load Case.

After that, the static structural analysis for self-weight, superimposed dead loads, unit pretension loads for the cables and Jack Up loads will be performed.

Final Stage Analysis

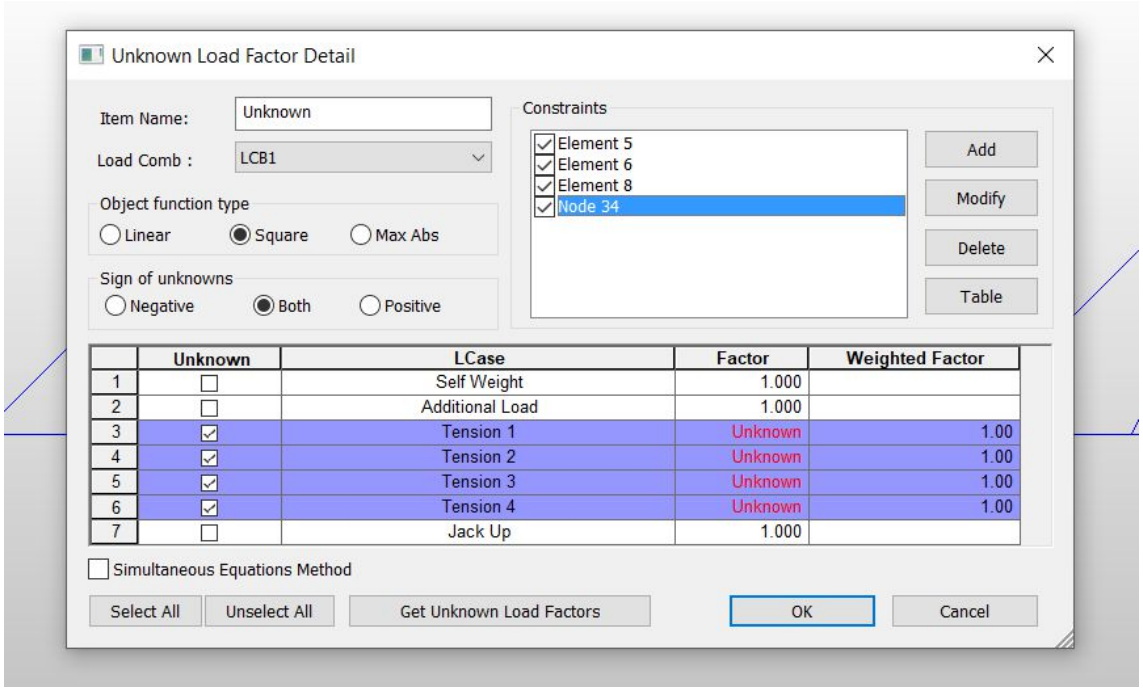
As a Final Stage Analysis, a load combination, using the same 4 loading conditions, will be generated under the name of **LCB 1.**



22. LOAD COMBINATION WINDOW FROM MIDAS/CIVIL SOFTWARE

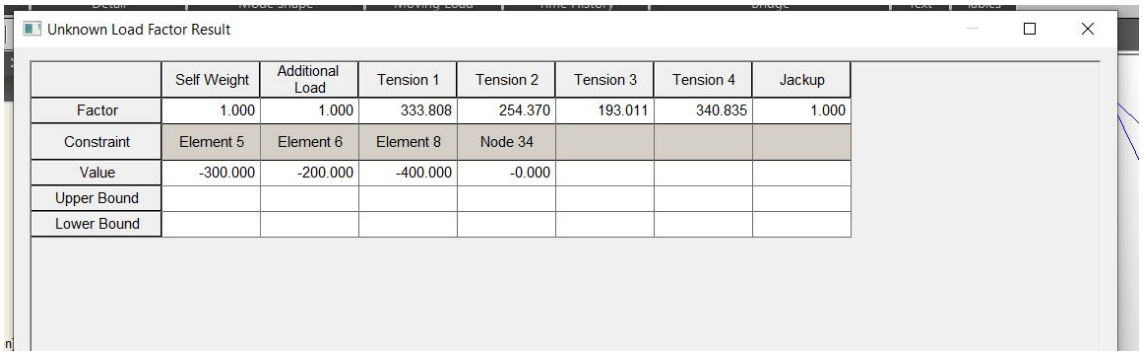
The next step becomes the application of the Unknown Load Factor function and, to calculate those unknown load factors that satisfy the boundary condition for LCB1, which was generated through load combinations, the constrains will be specified to limit the horizontal deflection (Dx) of the tower and the bending moment (My) of the girders.

To specify the load conditions, constraints and method of forming the object functions in Unknown Load Factor, first, the cable unit loading conditions as unknown loads must be defined.



23. UNKNOWN LOAD FACTOR DETAIL DIALOG BOX, IN MIDAS/CIVIL SOFTWARE

The constraints values given to Elements 5, 6 and 8 are, respectively, -300, -200 and -400 tonf/m; and a null value has been given to Node 34. With those values, the ULF function is ran.



Unknown Load Factor Result

	Self Weight	Additional Load	Tension 1	Tension 2	Tension 3	Tension 4	Jackup
Factor	1.000	1.000	333.808	254.370	193.011	340.835	1.000
Constraint	Element 5	Element 6	Element 8	Node 34			
Value	-300.000	-200.000	-400.000	-0.000			
Upper Bound							
Lower Bound							

24. RESULTS OF UNKNOWN LOAD FACTOR FUNCTION

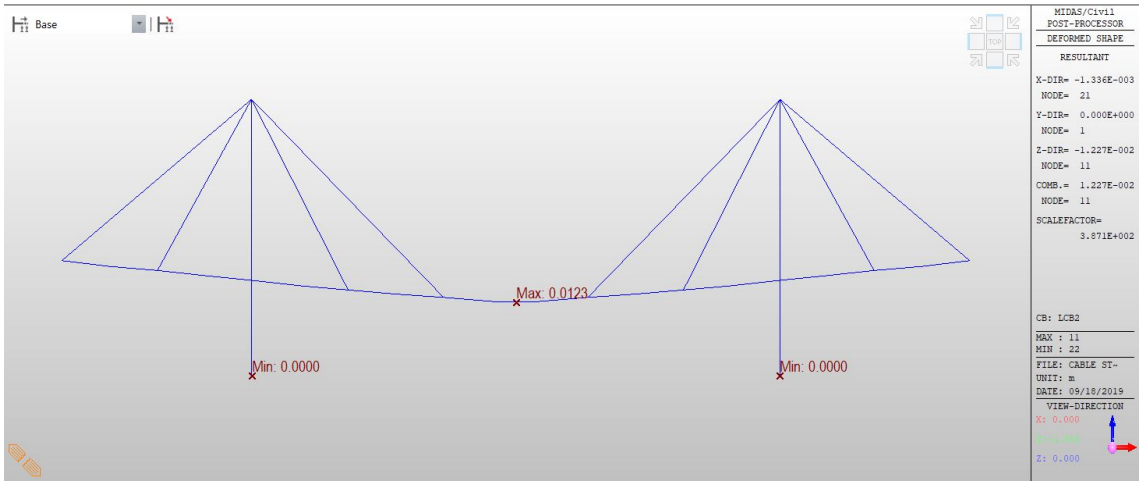
The program also gives the possibility of the extracting of the Influence Matrix, which is presented here below:

UnknownLoad Factor Result (Influence Matrix)					
File Name : MIDAS/Civil Tutorial			Date : 2019/8/15		
Number of Constraints : 4			Number of Load Cases : 7		
	Constraint	Element 5	Element 6	Element 8	Node 34
Factor	Upper Bound	2,12787E+93	2,12787E+93	2,12787E+93	2,12787E+93
	Lower Bound	2,12787E+93	2,12787E+93	2,12787E+93	2,12787E+93
	Value	-300,000026	-199,9998735	-399,9999865	-4,31086E-10
Self Weight	1	-3520,87207	-17,4434727	2047,740713	0,021406166
Additional Load	1	-1667,25251	-10,52741749	967,6863473	0,010115635
Tension 1	**333,8075867**	1,587423097	-0,440804703	-1,513357418	-0,000151982
Tension 2	**254,3696899**	3,088919832	0,727667189	-0,368708347	-7,86722E-05
Tension 3	**193,0109863**	6,42871332	-4,832804591	-1,912890857	4,74244E-05
Tension 4	**340,8348083**	6,533870693	1,942779922	-7,320172823	8,02664E-05
Jackup	1	104,7202361	60,63585197	47,71002036	0,002711636

25. INFLUENCE MATRIX AS A UNKNOWN LOAD FACTOR FUNCTION RESULT, IN MIDAS/CIVIL SOFTWARE

With those new constraints (highlighted in green) for Tension Load Cases, a new Load Combination (LCB2) can be created, and from here, the first deep analysis will be made.

The first step that the tutorial makes us follow is to check the Deformed Shape of the structure.



26. DEFORMED SHAPE OF LCB2 LOAD COMBINATION

Forward Construction Stage Analysis

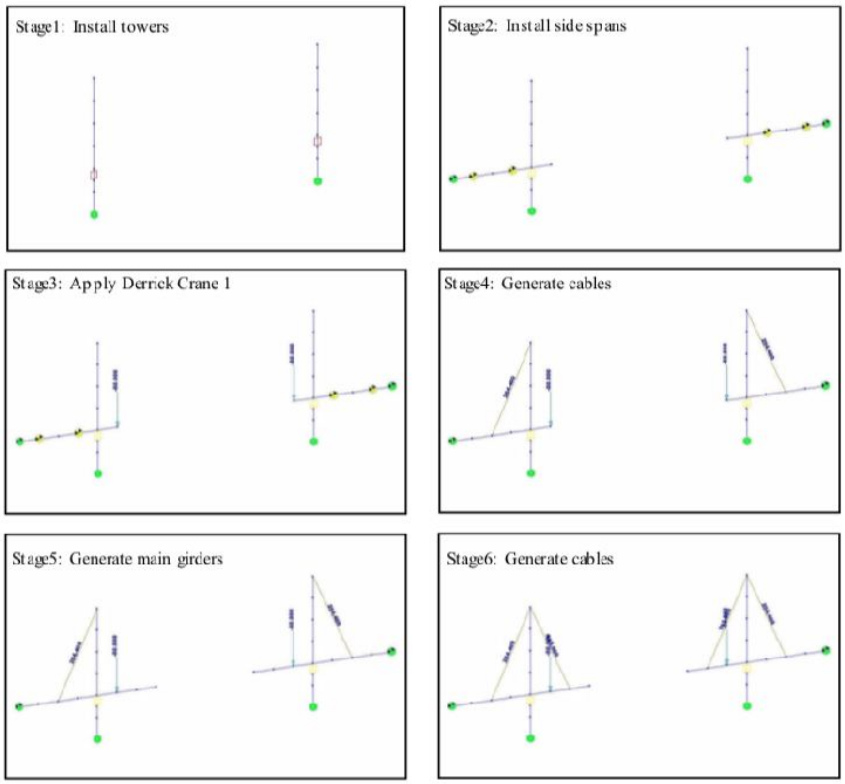
After the first approach, now the analysis of Forward Construction Stage will be presented. Following the tutorial's guide, a study of every construction stage of this cable-stayed bridge will be done, making focused observations on the resulting efforts during the constructive operations to evaluate how must the behaviour of the bridge be and obtain, like this, a work line to be based in on site.

When a cable-stayed bridge is designed, the structural configuration, cable sections and tension forces are generally calculated from the overall analysis of the completed state.

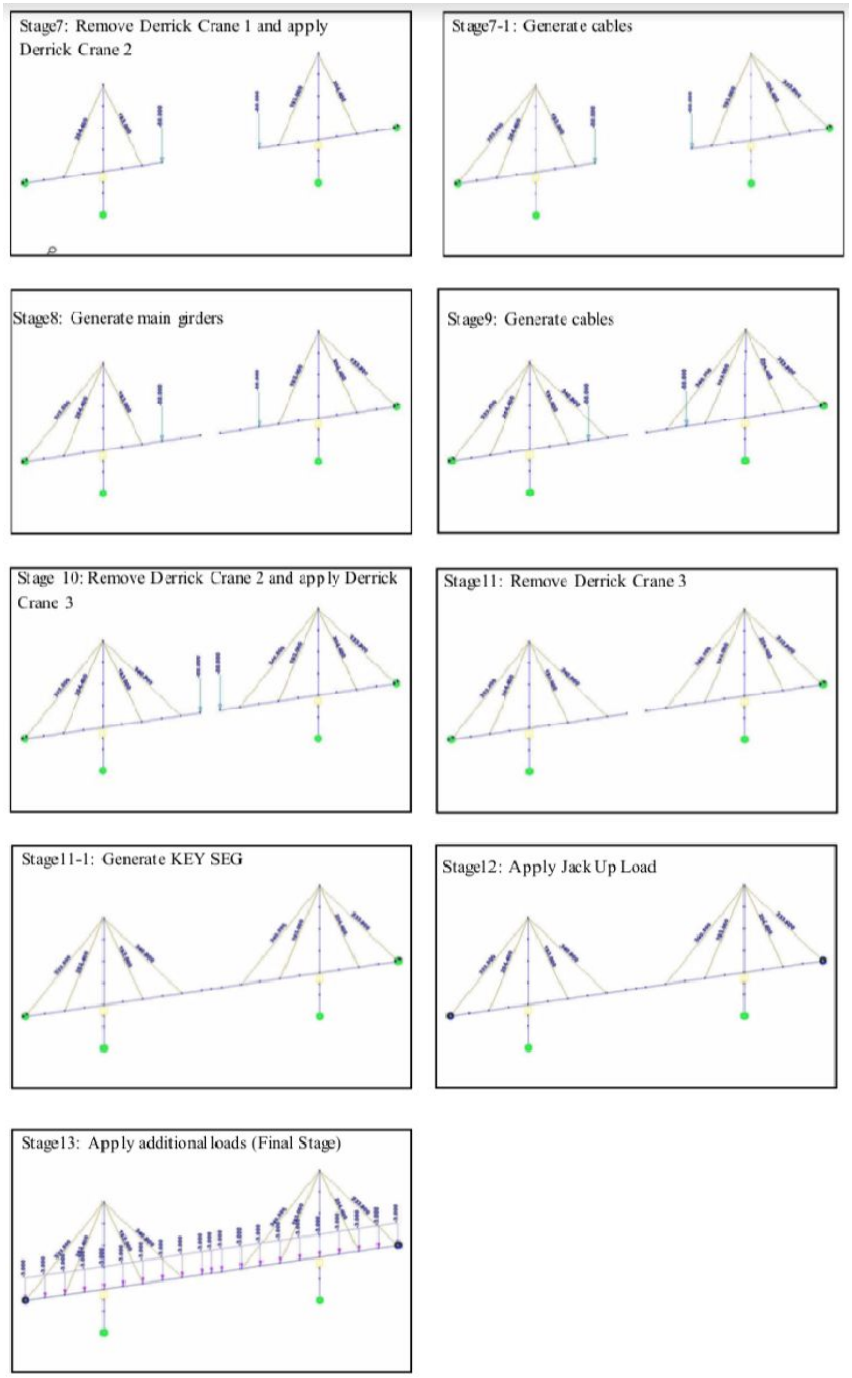
Apart from this analysis, it is also required to make another analysis of the construction stage to design the cable-stayed bridge. Depending on the temporary support method, the structural system of this kind of bridges changes drastically during construction, and it may become unsafe and/or unstable along the construction stage compared to the completed state. To make this construction stage analysis, Forward Analysis follows the natural construction sequence, which allows to check stresses, deflections, constructability…

But this analysis presents some difficulties, one of those is the calculation on tension forces at construction stages. MIDAS/Civil Software solved this problem creating the **lack of fit force** functionality, additional pretension loads, which are introduced during the installation of cables, and member forces to preload at Key Segment with equal efforts at the completion state. Using these pretension and member forces, forward stage analyses are performed.

To show how is the Forward Analysis running, construction stages must be defined to consider the effects of the activation a deactivation of main girders, cables, cable anchorage, boundary conditions, loads, etc. All of this is presented on the next table.



27. CONSTRUCTION SEQUENCE FOR A CABLE-STAYED BRIDGE (STAGE 1 TO 6)



28.CONSTRUCTION SEQUENCE FOR A CABLE-STAYED BRIDGE (STAGE 7 TO 13)

Lack of Fit Force

It is important to explain the concept of the "Lack of fit Force" function on MIDAS/Civil Software for the correct understanding of the calculation process.

This phenomenon not only affects the trusses, but also the beams. So to illustrate the difference on the calculation on those two elements, they are going to be explained separately.

In the case of trusses, as **[https://www.quora.com/What-is-the-significance-of-lack-of-fit-case-in-truss-structure]** explains, the lack of fit is an occurrence in which there is a difference between the length of a member and the distance between the nodes it is supposed to fit.

It is important to be noticed that that this phenomenon only occurs in indeterminate trusses, since in determinate trusses the variation in member length automatically adjusts the spacing between the nodes, hence, lack of fit cannot occur.

In cases of lack of fit, the member is either forcibly lengthened or forcibly shortened to fit the truss depending on whether it is too short or too long respectively. This causes the various members meeting at these joints to develop stresses of opposite nature, and these prestresses may be used advantageously in design of truss.

For the correct solving of this problem, the calculating follows this method:

1. First, displacements at each end of cables are calculated at a stage immediately before their installation.
2. Using these displacements, the additional cable pretension (ΔT) is calculated, which is the difference between the cable length (L) at the completed state and the cable length (L') during the construction.
3. This additional cable pretension is added to the initial pretension (T_0) determined from the initial configuration analysis.
4. That pretension value (T_f) is entered as Pretension during the construction to perform forward analysis.

In the case of beams, however, the calculation becomes more complicated due the fact that at the time the key segment closures the cable-stayed bridge, cantilevers of the centre span are deflected. If that key segment is closed in this state, no member forces takes place at it (only member forces due the self-weight take place) and there is a discontinuity between the cantilevers and the key segment.

What MIDAS/Civil Software achieves with the Lack of Fit Force function is to connect each cantilever member continuously calculating specified displacements required at each end of the key segment and converting them into member forces to apply these forces to the key segment.

Construction Stages

In the present tutorial, 13 construction stages are generated to simulate the changes of loading and boundary conditions. Those construction stages are outlined in the next table.

Stage	Content	Remark
Stage 1	Install towers, end supports in the side spans, temporary bents, and temporary connection between tower-girder	
Stage 2	Install side spans (Elements 1 to 5 & 16 to 20)	Lack-of-Fit Force
Stage 3	Apply Derrick Crane1 load	
Stage 4	Remove temporary bents and generate cables (Element 34, 39)	Lack-of-Fit Force
Stage 5	Generate main girders (Element 6, 7, 14, 15)	
Stage 6	Generate cables (Element 35, 38)	Lack-of-Fit Force
Stage 7	Remove Derrick Crane1 load and apply Derrick Crane2 load	
Stage 7-1	Generate cables (Element 33, 40)	Lack-of-Fit Force
Stage 8	Generate main girders (Element 8, 9, 12, 13)	
Stage 9	Generate cables (Element 36, 37)	Lack-of-Fit Force
Stage 10	Remove Derrick Crane2 load and apply Derrick Crane3 load	
Stage 11	Remove Derrick Crane3 load	
Stage 11-1	Generate KEY SEG (Element 10, 11)	Lack-of-Fit Force
Stage 12	Replace the connection between tower-girder and apply Jack Up load	Rigid link→ Elastic link
Stage 13	Apply additional dead loads (Final Stage)	

As it has been explained before, Forward Analysis reflects the real construction sequence, so it will allow to examine the structural behaviour of the analytical model and the changes of cable tensions, displacements and moments.

On first place, the LCB1 and LCB2 load combination will be erased, as will be the Tension 1-4 Static Load Cases. Those tensions won't be necessary because a new Static Load Case will be created: Pretension from Forward Analysis.

In construction stage analysis for a cable-stayed bridge, a geometrical nonlinear analysis for cable elements should be performed by means of the transformation of the truss elements, used in the final stage analysis, into the cable elements to consider the sag effect of them in cable-stayed bridge. The reason is that this element considers the stiffness due the tensioning.

Definition of Construction Stages

After the change of these element parameters, is time to define the constructions stages to run the forward construction stage analysis.

As shown before, 13 construction stages will be created, but it is important to mention that stages 7 and 11 will be doubled, this is to say that a 7-1 and 11-1 stages will be also created. The

reason of those extra-stages is that in stages 7 and 11 there is two operations to do. Each stage shows a single operation to make the evolution of the bridge construction more visual.

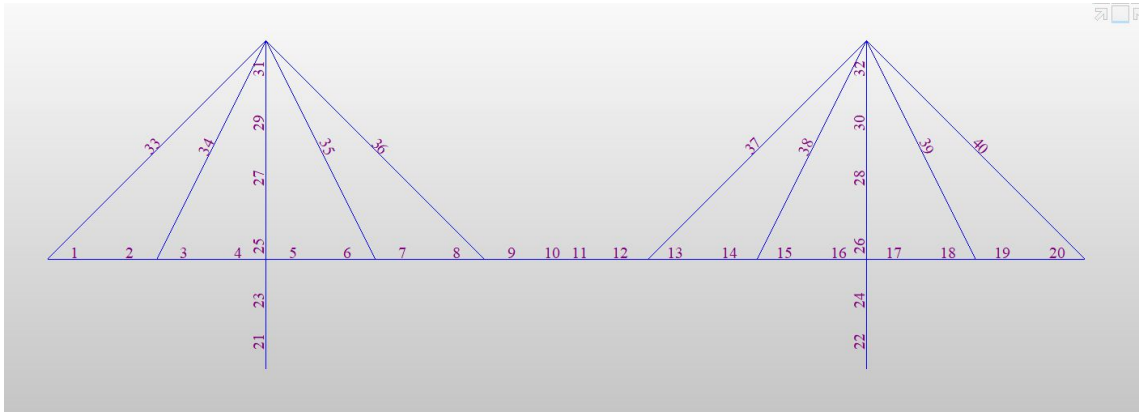
The extended explanation of every stage will be presented at the end of the tutorial, when the cable-stayed bridge is completely designed, and the evolution of the efforts could be analysed.

Structure Groups

To reach this final analysis, MIDAS/Civil Software relate structure groups with the construction stages created previously. This is not a direct relation, that means, not every single construction stage is linked with a structure group. What MIDAS/Civil Software defines as a structure group, is an operation where a element is added or deleted, for example, when a cable is placed, it is considered as a structure group directly related with the construction stage, but when the derrick crane is removed, it is only considered a constructions stage because it is not a part of the structure.

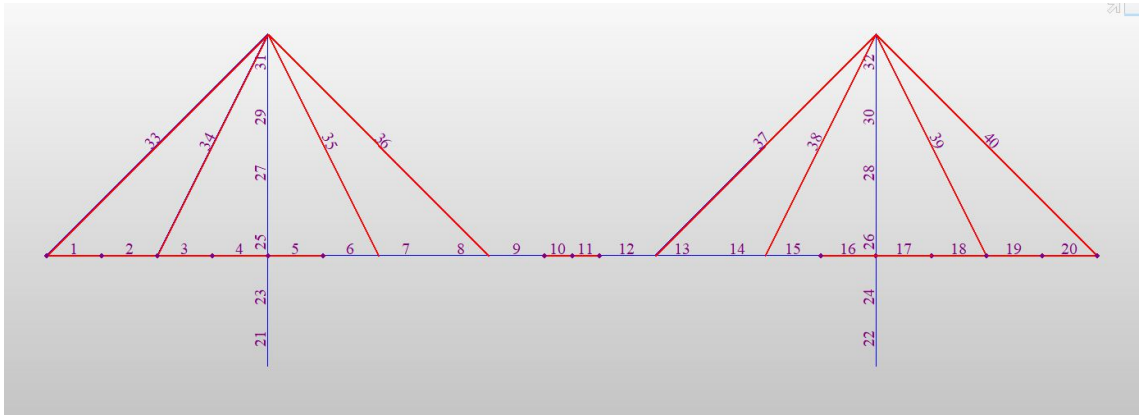
The designation of every structure group will be outlined here below:

- Structure Group 1: Tower erection stage
- Structure Group 2: Construction of side spans
- Structure Group 3: Generation of cables (elements 34 and 39)
- Structure Group 4: Generations of main girders (elements 6, 7, 14 and 15)
- Structure Group 5: Generation of cables (elements 35 and 38)
- Structure Group 6: Generation of cables (elements 33 and 40)
- Structure Group 7: Generation of main girders (elements 8, 9, 12 and 13)
- Structure Group 8: Generation of cables (elements 37 and 37)
- Structure Group 9: Generation of key segment (elements 10 and 11).



29. ELEMENT NUMBERS OF THE CABLE-STAYED BRIDGE

- Structure Group 10: Lack of Fit Force.



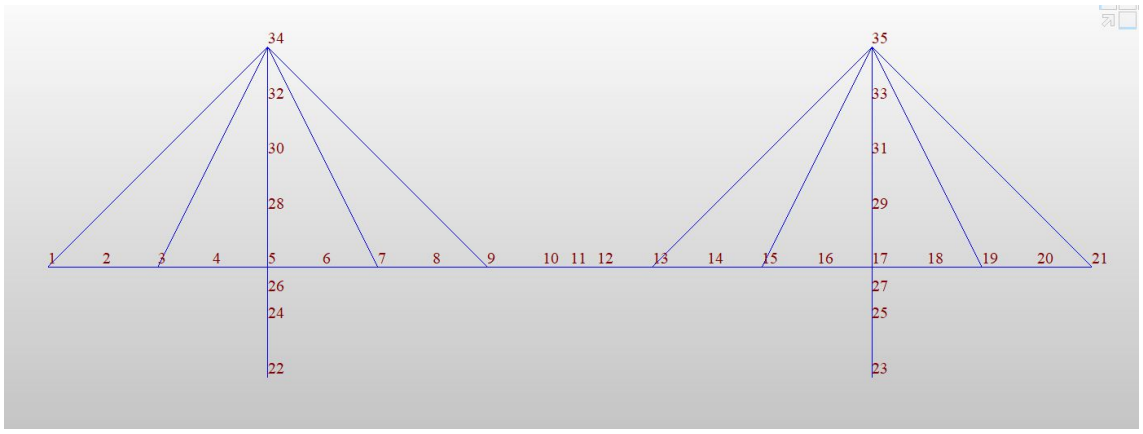
30. ELEMENTS WHERE LACK OF FIT FORCE IS APPLIED ON

Boundary Conditions

After the creation of all structure groups, the boundary conditions must be assigned.

The boundary groups are designed to join all the nodes with the same boundary conditions in a single group. The six boundary groups the tutorial designs are as follows:

- Fixed Support (Tower): nodes 22 and 23
- Hinge Support (Pier): nodes 1 and 21
- Elastic Link (Tower): between nodes 26 and 5, and between nodes 17 and 27. The stiffness values given for those segments are
    - SDx = 500.000 tonf/m
    - SDy = 10.000.000 tonf/m
    - SDz = 10.000.000 tonf/m
    - Being SRx, SRy and SRz null
- Temporary Support (Tower): between nodes 26 and 5, and between nodes 17 and 27. It is a reassigned boundary conditions which now turns into a rigid link.
- Temporary Bent: nodes 2, 4, 18 and 20, where the boundary condition is assigned as Point Spring Support and its stiffness takes values as
    - SDy = 10.000.000 tonf/m
    - SDz = 10.000.000 tonf/m
    - SRx = 10.000.000 tonf/m
    - SRz = 10.000.000 tonf/m.
    - Being SDx and SRy null



31. NODE NUMBERS ON THE CABLE-STAYED BRIDGE

Load Groups

Once the boundary groups are assigned to the nodes where they are applied on, the Load Groups are created and assigned to corresponding loading condition. The Load Group it is going to work with are outlined below:

- Self-Weight. This load condition, which is already assigned when the Final Stage analysis was performed, is now redefined with its own Load Group, since before it was defined with the default load condition.
- Additional Load. This condition will be designed as all the loads applied on the beams.
- Jack Up Load. For this load condition, MIDAS/Civil Software relate the specified displacements of the support to the whole structure.
- Derrick Crane Load. This new Static Load Case is generated to represent the load that the derrick crane creates on the nodes during the construction stages. As three movements per each half of the bridge will be done, this Load Group is divided in three subgroups: Derrick Crane 1, Derrick Crane 2 and Derrick Crane 3, and they will be applied on nodes 6 and 16, 8 and 14, and 10 and 12, respectively, with a 80 tonf/m value each one.
- Pretension Load. This Load Group will be divided in 4 subgroups since there is 4 pairs of cables where this pretension load will be applied on. The values of these pretension loads that are going to be used, are the values that have been obtained in the Final Stage Analysis, by means of Unknown Load Factor function. Those values are presented on the next table.

Load Group	Elements Nº	Pretension Loading	Load Group	Elements Nº	Pretension Loading
Pretension Load 1	**36, 37**	340,835	**Pretension Load 3**	**35, 38**	193,011
Pretension Load 2	**33, 40**	333,808	**Pretension Load 4**	**34, 39**	254,370

32. PRETENSION LOAD GROUPS

Once the Structure Groups, the Boundary Groups and the Load Groups have been created, they have to be assigned to each Construction Stage to reproduce as close as possible the real behaviour of the construction of the cable-stayed bridge.

To make it more visual, the next table represents the assignation of each condition to each construction stage:

	Structure		Boundary		Load Group	
	Activation	**Deactiva-tion**	**Activation**	**Deactiva-tion**	**Activation**	**Deactiva-tion**
Stage 1	Stage1		Fixed Sup-port (Tower) Temporary Support		Self-Weight	
Stage 2	Stage2		Hinge Sup-port (Pier) Temporary Bent			
Stage 3					Derrick Crane 1	
Stage 4	Stage4			Temporary Bent	Pretension Load 4	
Stage 5	Stage5					
Stage 6	Stage6				Pretension Load 3	
Stage 7					Derrick Crane 2	
Stage 7-1	Stage7				Pretension Load 2	
Stage 8	Stage8					
Stage 9	Stage9				Pretension Load 1	
Stage 10					Derrick Crane 3	
Stage 11						
Stage 11-1	Stage11					
Stage 12			Elastic Link (Tower)	Temporary Support (Tower)	Jack Up Load	
Stage 13					Additional Load	

Now, with all these conditions entered, and after adjusting some parameters on the program, like functions activation, number of iterations and tolerances, the final analysis can be performed.

As it was expounded previously, each construction stage will be analysed by means of efforts graphics, deformed shape and their explanation.

Chapter 4 – Construction Stage Analysis

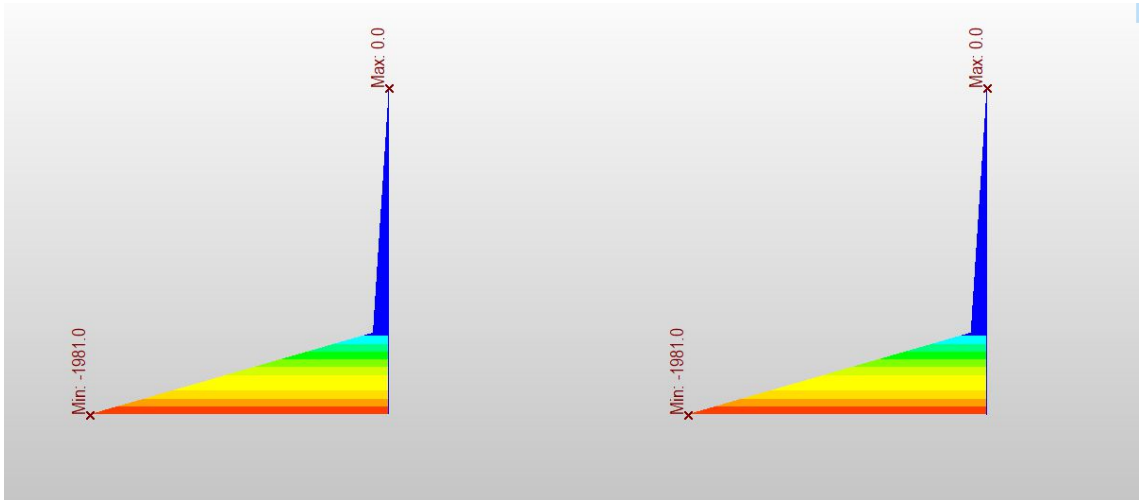
Stage 1

The construction stage 1 defines the construction of both towers.

33. STAGE 1 - INSTALL TOWERS

As they are vertical structures with no designed forces acting transversally, the only efforts that can be found are the ones due the self-weight, therefore only axial forces will be present on them.

It is important to notice that it is not a faithful representation of reality, since there are several factors acting against the towers once they are raised, like wind, temperature or even a reaction transmitted through the foundations due an irregular settlement. However, what this thesis wants to analyse is the efforts caused due the construction stage itself, obviating external factors.



34. AXIAL EFFORTS DIAGRAM ON STAGE 1

It can be appreciated that the most affected part of the structure the bottom of Lower Tower, since the self-weight of the pylon becomes maximum once the floor is reached.

It is also notable the difference between the bottom of Lower Tower and the bottom of Upper Tower. The inferior Modulus of Elasticity, Poisson's Ratio and Weight Density, given to the Lower Tower at the beginning of the tutorial when the material properties were defined, makes that the resistance against axial efforts turns into great reaction which must be transmitted to the foundations.

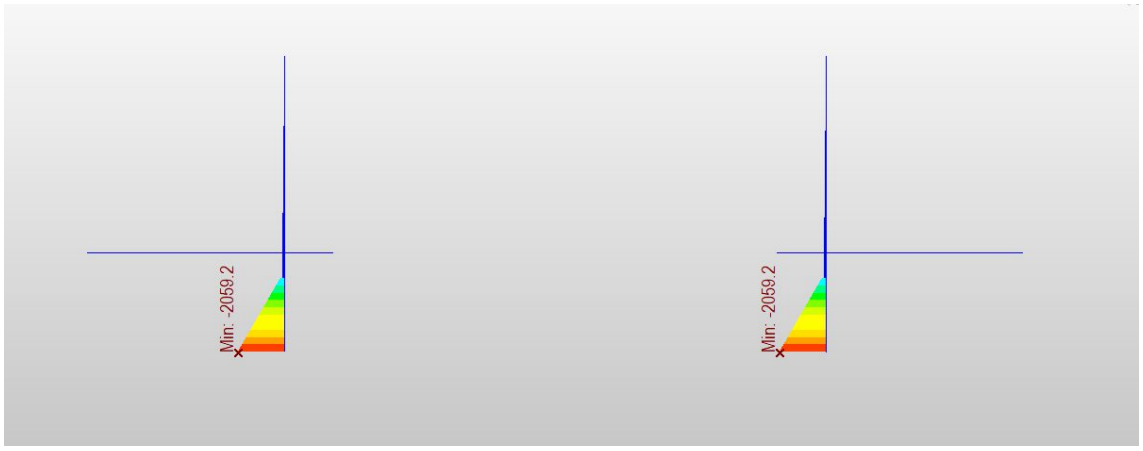
On the other hand, the rigid link between both parts of the tower makes that the whole axial force achieved on the bottom of Upper Tower gets transmitted to the Lower Tower, increasing the final negative effort.

Stage 2

The second construction stage includes the erection of the side spans and a cantilever segment of the main one. The incorporation of two parts linked transversally brings the appearance of bending moments, what is traduced in a bigger deformation of the structure.

35. STAGE 2 - INSTALL SIDE SPANS

First, the axial diagram will be reviewed, to observe the influence of the girder on the tower efforts.



36. AXIAL EFFORTS DIAGRAM ON STAGE 2

It can be noticed that the weight of the girder increases the axial effort resisted by the pylons, as predictable, and it creates new shears and bending moments along the modified structure as it can be checked on the next diagrams.

37. SHEAR EFFORTS DIAGRAM ON STAGE 2

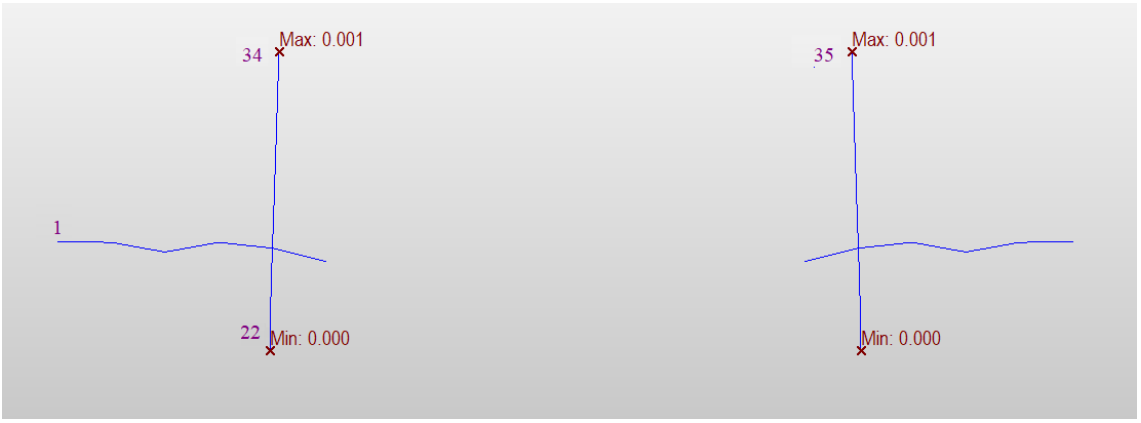
With Fz = ± 62,8 tonf as a maximum absolute value, the side span behaviour is the same of a fixed-hinged beam, since the Piers have been designed as a simple support and the connection between the girder and the tower, works as a rigid link. On the other hand, there is no shear efforts on the pylons because of that rigid link.

38. BENDING MOMENTS DIAGRAM ON STAGE 2

The static structural scheme of the bridge changed due the addition of the side spans. A negative bending moment of My = 314 tonf·m is created at the connection of the tower and the main girder because of the cantilever caused by element 6 (and symmetrically, element 16). This bending moment makes the necessity of a rigid link between the pylon and the deck becomes imperative to transmit those efforts to the foundations.

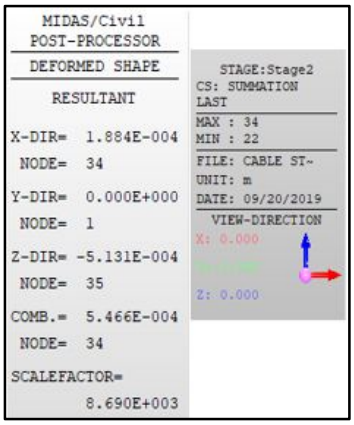
The bending moment suffered by the pylon, with a My = ± 299 tonf·m value, balance the efforts on the deck and makes the bridge become an hyperstatic structure that fronts of the mechanism it was on Stage 1.

The deformed shape of the bridge it also must be analysed. As it can be imagined, the displacements caused because of the self-weight follows the bending moments diagram and offers a deformed structure like next.



39. DEFORMED SHAPE DIAGRAM ON STAGE 2

MIDAS/Civil Software also gives the values of the displacements in some nodes on a legend. Those values are presented on the next table



40. LEGEND OF DEFORMED SHAPE ON STAGE 2

Stage 3

The variations between this stage and the second one are due the application of the Derrick Crane Load on nodes 6 and 16. This load increment only amplifies the efforts diagrams shown on the previous stage but it can be said that at this stage is when the cantilevered construction method begins.

41. STAGE 3 – APPLY DERRICK CRANE 1

The axial forces on Stage 3 are a reflection of a load increase due the placement of the Derrick Crane. The maximum absolute value still remains at the bottom of the tower with a value of Fx = 2145,3 tonf.

42. AXIAL EFFORTS DIAGRAM ON STAGE 3

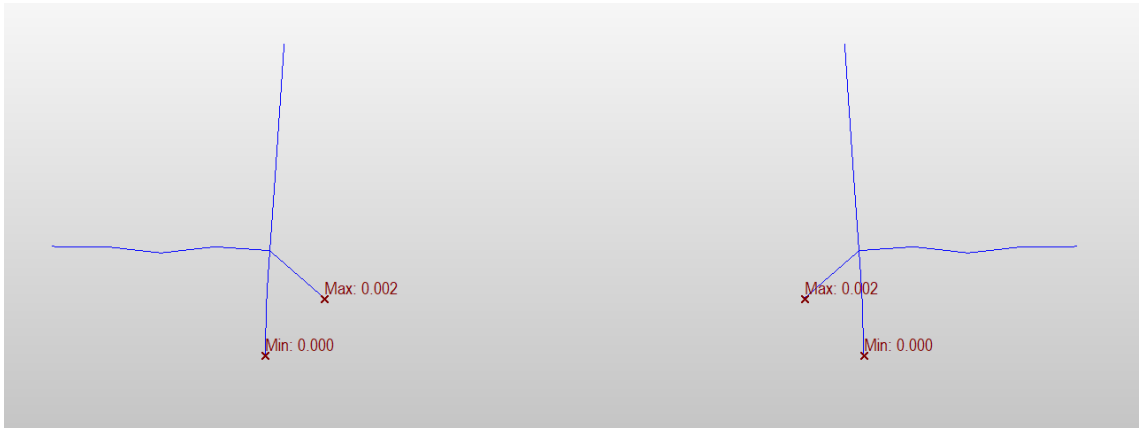
If shear diagram is analysed, it can be noticed how does the Derrick Crane load affects to the diagram as the summation of a distributed force (self-weight) and a vertical punctual force have as a result the draw of a parallelogram where a orthogonal side has the value of the punctual force itself.

43. SHEAR EFFORTS DIAGRAM ON STAGE 3

The bending moments on this stage have the same figure than in stage 2 because no new element is added to the structure. A vertical punctual force summed to a distributed force system, only increases the value of its diagram as it can be seen in the next diagram.

44. BENDING MOMENTS DIAGRAM ON STAGE 3

It is also remarkable the influence that the Derrick Crane has on the efforts in the side span, as it reduces the value of bending moments in the extremes of the beam and increases them in the central part of it. The transmission of the punctual force on the cantilever to the simply supported/fixed beam on the side span just affects to the efforts diagram in the way the connection point turns as a fixed point.



45. DEFORMED SHAPE DIAGRAM ON STAGE 3

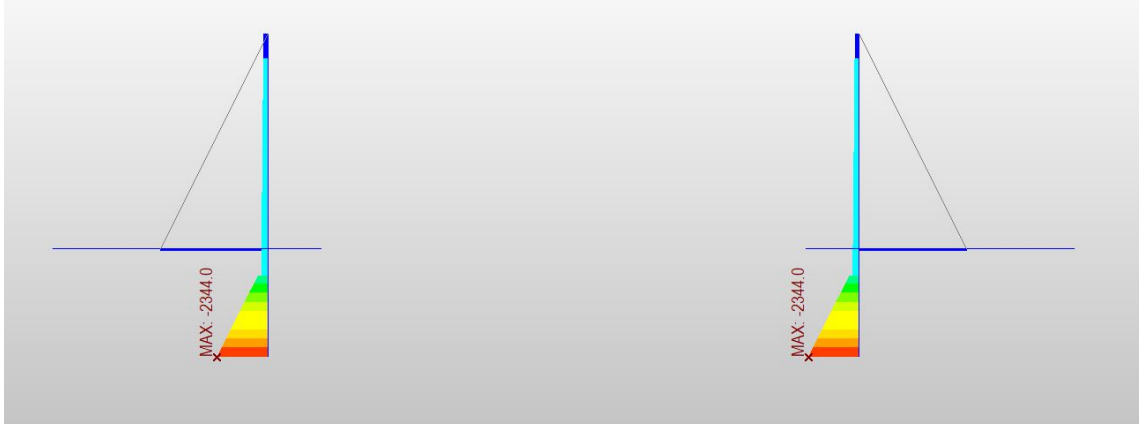
The deformed shape on stage 3 due the placement of the derrick crane increases the displacement values from stage 2 but keeps the same static scheme since there is no bridge's element added on the structure. However, the maximum displacement changes from the top of Upper Tower to the beam on cantilever.

Stage 4

On this stage the first cable is placed that begins transmitting forces in a non-orthogonal way so the efforts whose used to work only on one direction, they are now affected in a different way and makes the cable-stayed bridge become in a even more indeterminate structure.

46. STAGE 4 - GENERATE CABLES

The axial efforts diagram in that altered structure will be what is going to be analysed in first place.



47. AXIAL EFFORTS DIAGRAM ON STAGE 4

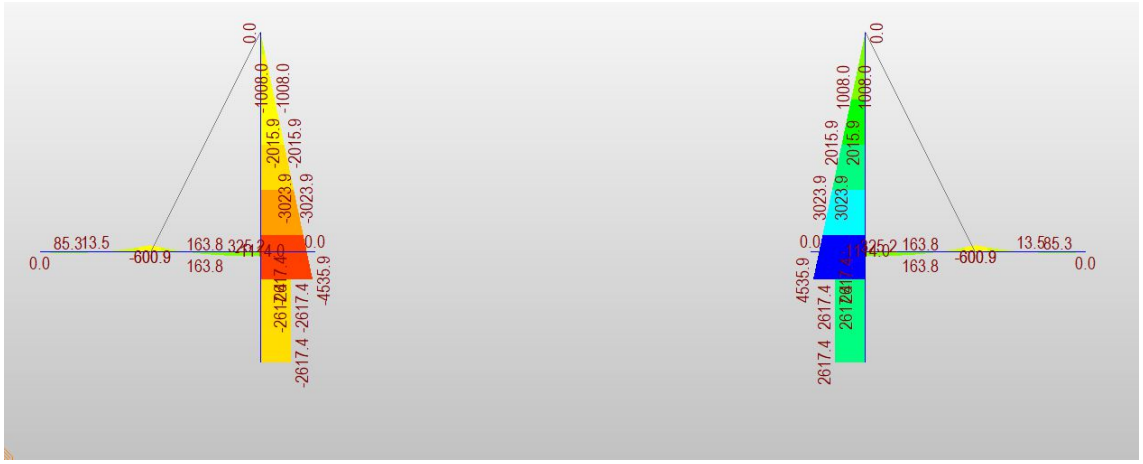
As it can be observed, the first axial forces appear on the deck because of the installation of the first cables, in this case, of elements 34 and 39. The tensions caused by the self-weight and the tension of the cable itself, are transmitted from the top of the Upper Tower to the side span girder changing again the static structural scheme and increasing compression forces in both parts of the bridge. This added axial force makes the bottom of the tower hold bigger efforts, in this case, efforts with a value of Fx = 2344 tonf.

48. SHEAR EFFORTS DIAGRAM ON STAGE 4

In shear diagram, it is important to notice how does the cable transfers its tension and converts it in a shear effort on the tower. The first pretension load applied is Pretension Load 4, with a value of 254,370 tonf. To calculate how does the Fz = 100,8 tonf appeared on the tower, the transmission of the horizontal component of the tension to the tower is found taking into account that the materials are extensible and deformable.

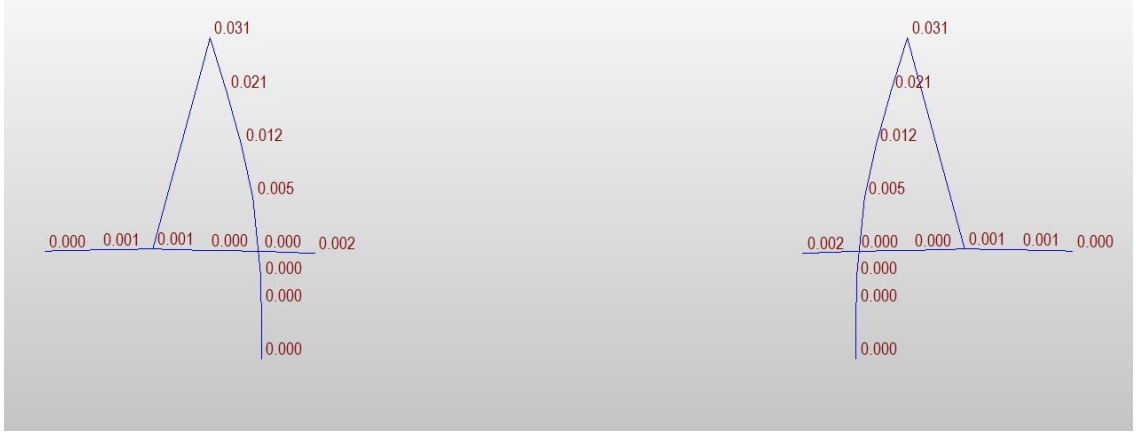
The cable's tensions also affect to the side span making the shear diagram take different form and increased values because the cable's tension is bigger than girders self-weight.

The change on the static scheme due the addition of the cable on the structure can be observed on the bending moments diagram. The pylon, which is used to have only axial forces, is now affected for shear forces and bending moments because of the transmission of a non-orthogonal tension. The girder, however, experiments a high increase of negative bending moments on nodes where the cable connects since the cable pulls the deck up creating an effort working on the opposite direction of gravity.



49. BENDING MOMENTS DIAGRAM ON STAGE 4

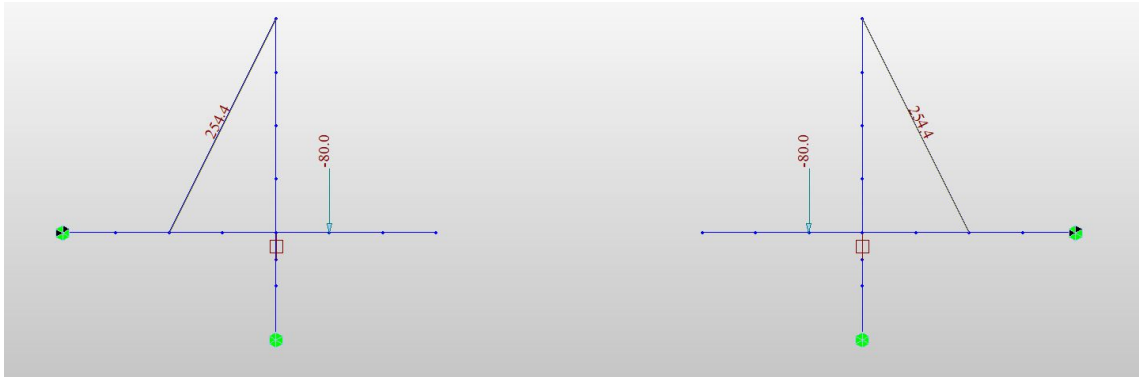
As it can be seen on bridge's deformed shape diagram, the displacements on the top of the Upper Tower reflect the efforts transmission due the placement of the cable and also the rise of the deck from its previous position. The displacement values are bigger in the Upper Tower due its stiffness is lower than Girder's.



50. DEFORMED SHAPE DIAGRAM ON STAGE 4

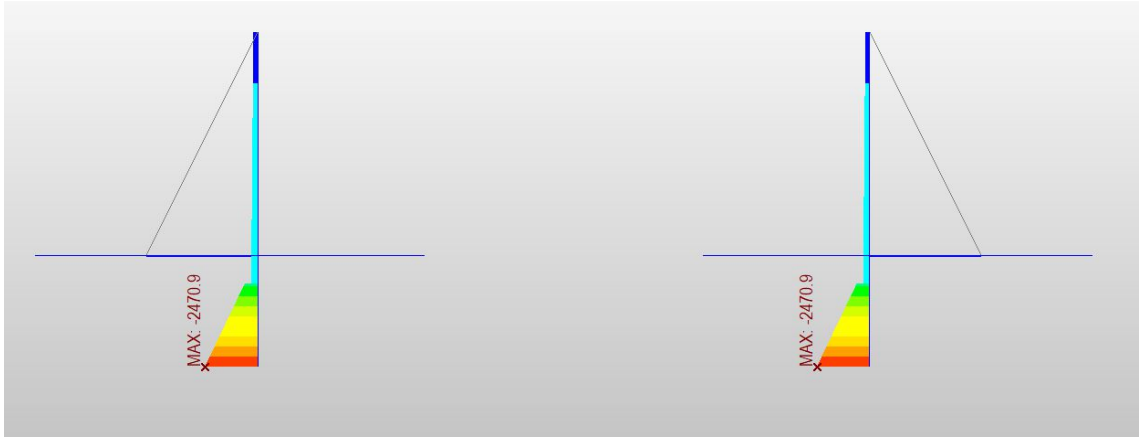
Stage 5

Once the first cable is placed, construction stage 5 consists in the continuation of the erection of the deck, creating two more elements with the cantilevered construction method.



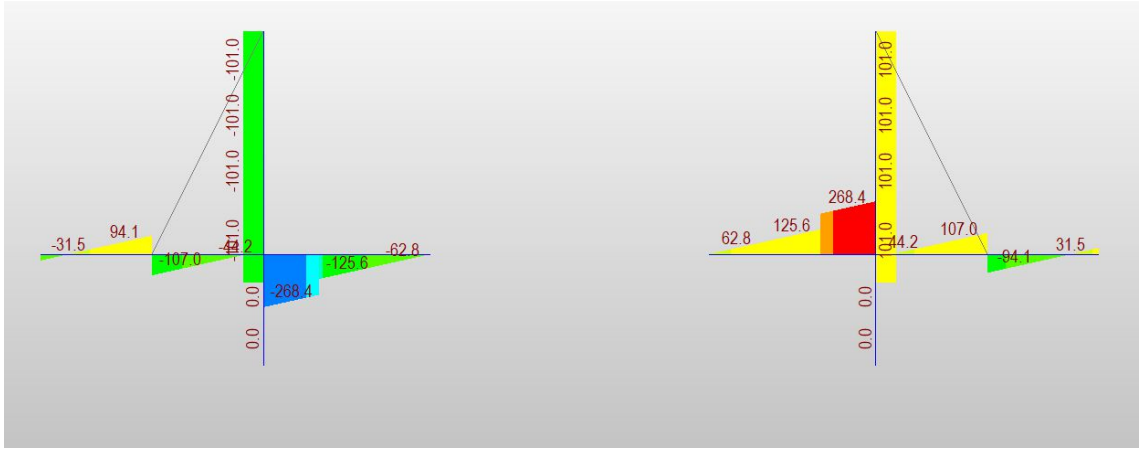
51. STAGE 5 - GENERATE MAIN GIRDERS

The axial efforts diagram is the less changed because of the advancing of the cable-stayed bridge construction. The more elements are added to the structure, the more weight will be transmitted to the foundations, so just an increasing value is obtained when axial forces are analysed.



52. AXIAL EFFORTS DIAGRAM ON STAGE 5

On the other hand, shear efforts are continuously changing. The addition of new cable elements brings a new diagram form because of the increasing of unknown reactions, but when a new part of the girder is erected, it only changes the value due the girder is built orthogonally to the direction of gravity.



53. SHEAR EFFORTS DIAGRAM ON STAGE 5

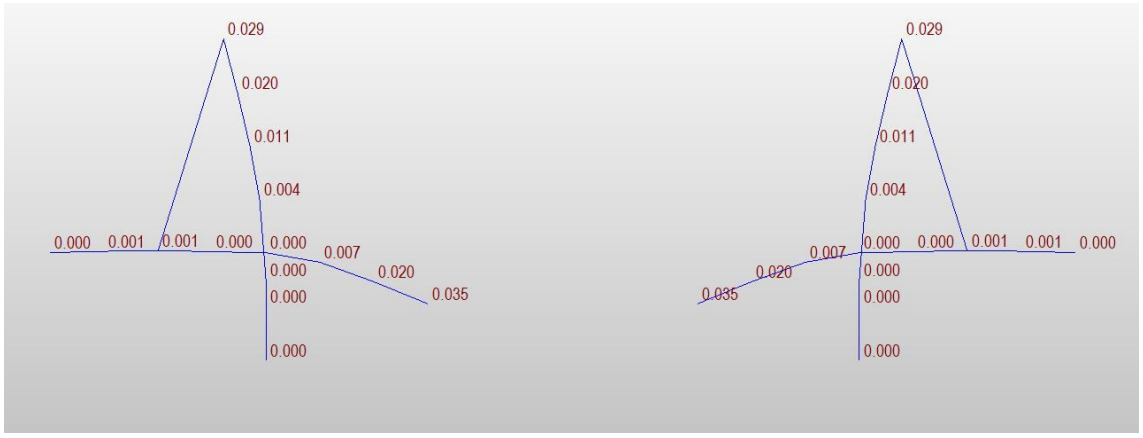
It is almost the same situation when the bending moments diagram is observed. From stage 2, the static scheme of the bridge is only changed when a cable is placed. The continuous-beam behaviour of the main girder working in cantilever doesn't make the number of unknown values increases, although the cable settlement does.

As it can be noticed, just 10 tonf·m are added to the girder on its maximum moment value due the new girder segment. On the other hand, the Lower Tower suffers a decreasing of almost

2500 tonf·m because of the relocation of that force on the main girder which has a 30 m cantilever beam with a 3.626 tonf·m bending moment.

54. BENDING MOMENTS DIAGRAM ON STAGE 5

The deformed shape attributed to stage 5 causes a 3,0 cm deflection on the extreme of the cantilever because of the self-weight and the derrick crane load. It also has a 2,9 cm displacement to the opposite side the top of the Upper Tower due the tension of the placed cable. Being conscient of these designed deflections, the construction process can be controlled.



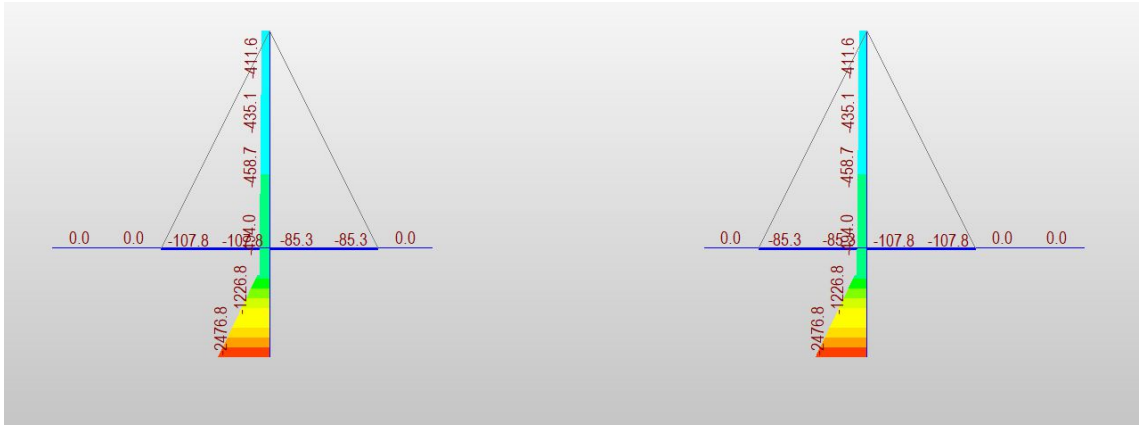
55. DEFORMED SHAPE DIAGRAM ON STAGE 5

Stage 6

On stage 6, a new cable is placed to reduce the bending moments caused due the long cantilever created on the previous stage. As it is been said before, the new cable element will change the static scheme of the cable-stayed bridge creating new diagrams whose need to be analysed.

56. STAGE 6 - GENERATE CABLES

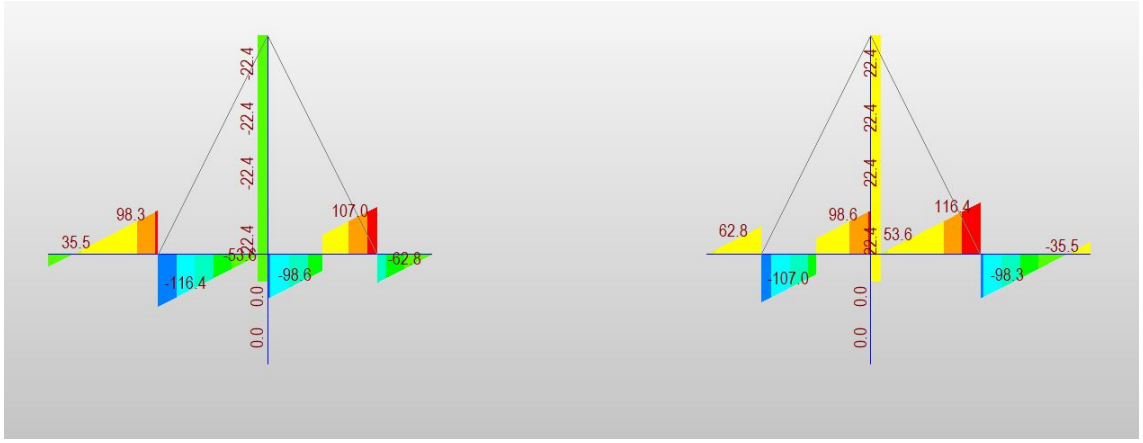
The different tension value for the two placed cables doesn't creates a symmetrical structure, even if we had the same length span on each side. That different tension is due that the correct OSS is pretended with those values so the construction state must be controlled other way. On Axial efforts diagram, it can be observed the apparition of a new compressed segment from the connection Tower/Girder (T/G) to the new-placed cable, with a 85.3 tonf value. Element by element, the axial effort suffered on the bottom of the tower becomes higher and with the placement of this new cable, it also does it the top of the tower almost duplicating its value.



57. AXIAL EFFORTS DIAGRAM ON STAGE 6

In the case of shear efforts, this stage transforms its shear diagram with the addition of the new cable. Where there was an efforts law for a cantilever with a distributed load, it can be found now an efforts law for two beams, one still in cantilever and the other acting like a bi-supported beam.

It can be evidently noticed the decreasing of shear value on the Tower/Girder connection since the cantilever is shorter than before, as it is the shear effort on the Upper Tower because of split of the cable tensions to both sides of the tower.



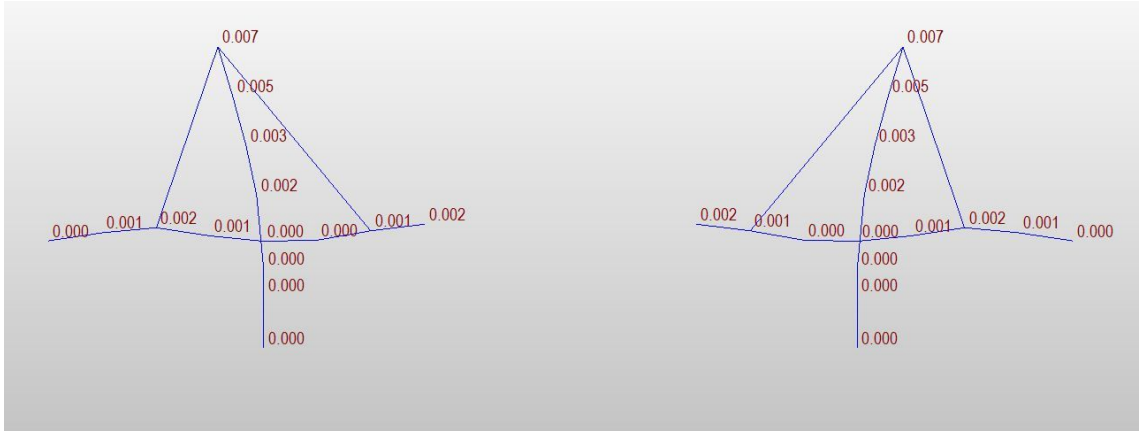
58. SHEAR EFFORTS DIAGRAM ON STAGE 6

The bending moments diagram shows that the placement of the new cable element releases the tensions supported by the tower decreasing the value on Tower/Girder connection. This happens because the main girder and the side span are now connected by somewhere else apart of the T/G connection so the tensions are splat to balance the structure.

59. BENDING MOMENTS DIAGRAM ON STAGE 6

The new placed cable obviously rises the main girder up by means of the given pretension load on this second cable. The -35 mm on axis z we had on the previous stage became a displacement que rises the cantilever until z=+2 mm, so the next stage, which is the advancing of the derrick crane to the extreme of the cantilever, can be achieved. It has to be tried the avoiding of an

excessive deflection that could unbalance the structure, so that's why a good construction process must be modelled.



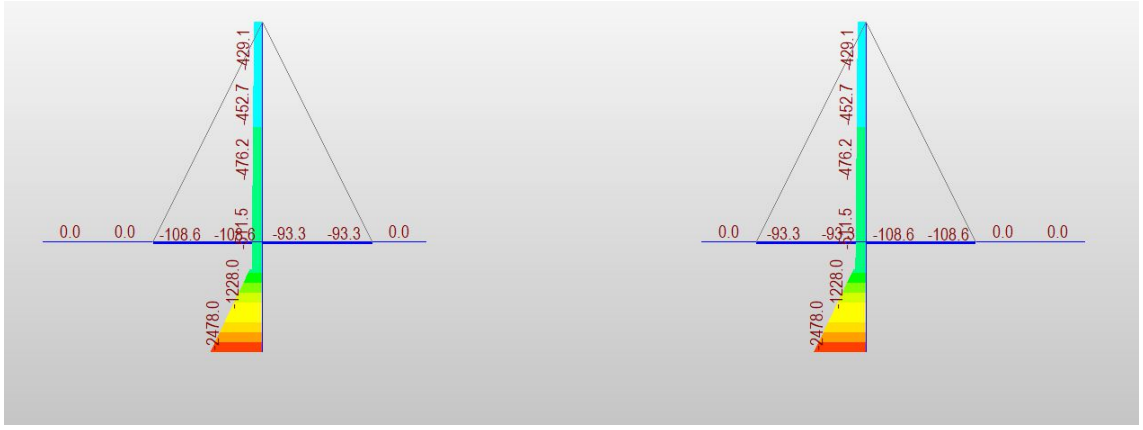
60. DEFORMED SHAPE DIAGRAM ON STAGE 6

Stage 7

As it has been said before, this stage represents the advancing of the derrick crane to the extreme of the cantilever. Once the second cable was placed, a better scenario to continue the erection of the main girder has been given.

61. STAGE 7 - ADVANCING OF THE DERRICK CRANE.

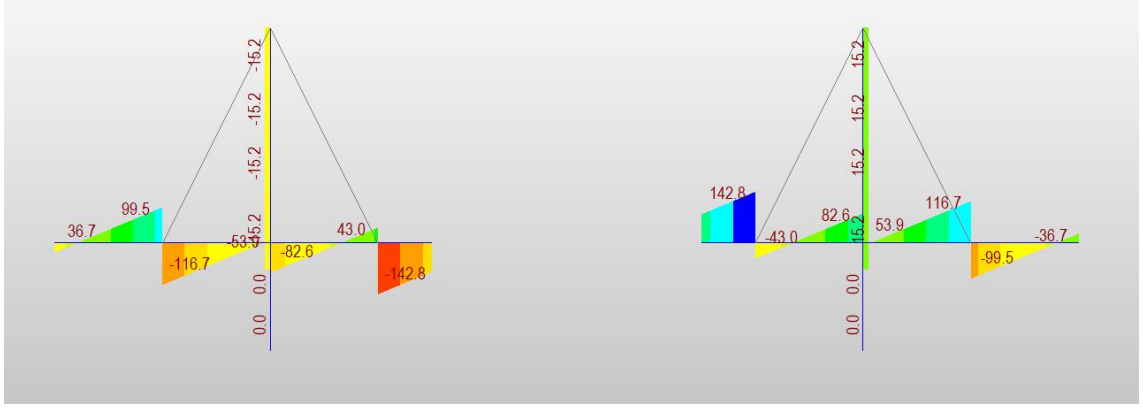
In the axial efforts diagram it can be noticed the small increasing there appears on the main girder as a result of the replacement of the derrick crane at the extreme of the cantilever. The 80 tonf of the crane barely modifies the axial efforts law but as a necessary step, it must be considered to avoid any unwanted stress.



62. AXIAL EFFORTS DIAGRAM ON STAGE 7

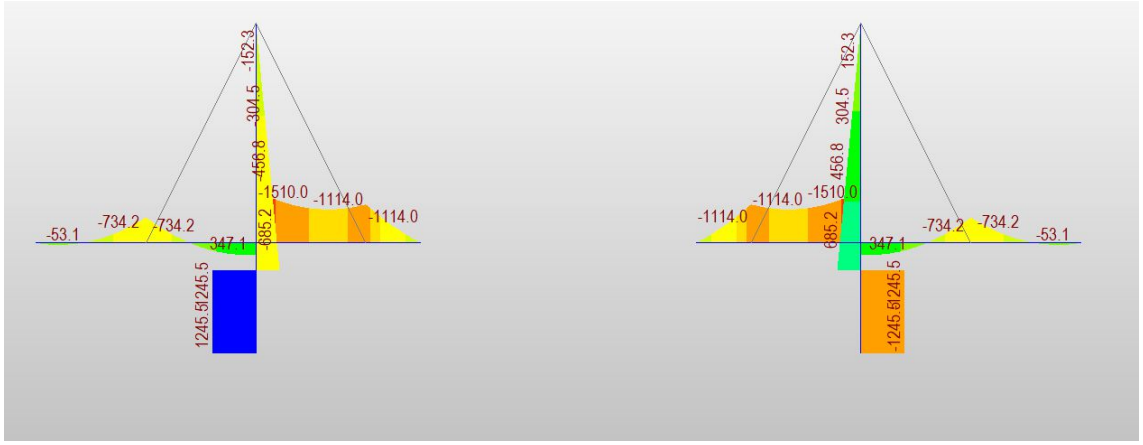
On Shear efforts diagram there is neither to big modifications, but it can be observed the influence that the derrick crane's punctual load has on the structure. Comparing with the previous stage, the maximum is displaced from the side span to the cantilever's extreme, having the first one almost the same value (from 116,4 tonf to 116,7 tonf).

It is also notable the reduction of the shear effort on the tower, which goes from 22,4 tonf to 15,2 tonf. This is due the remoteness of the nodal load, which it was "inside" the area created by the two cables and the girder and it is now "outside" of it.



63. SHEAR EFFORTS DIAGRAM ON STAGE 7

The bending moment diagram after the displacement of the derrick crane brings an effort law as follows.

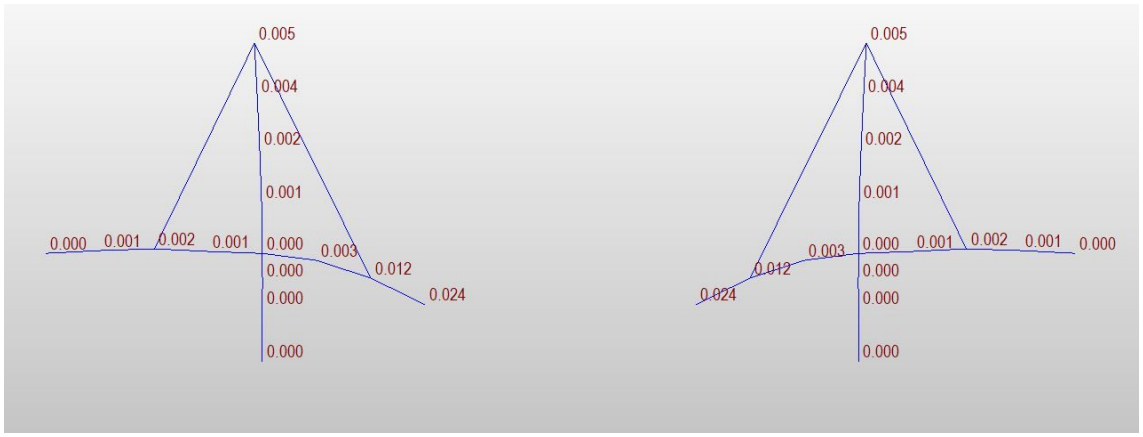


64. BENDING MOMENTS DIAGRAM ON STAGE 7

It is notable the difference between the previous stage and this stage since the moments on the Lower Tower have been multiplied by four, and the moments on the Tower/Girder connection, by three. The Upper Tower, on the other hand, has suffered a decreasing of its bending moments due the relocation of the first's mentioned.

The application of the punctual load on the extreme of the cantilever, with the fact that the main girder works as a bi-supported beam plus a cantilever, draws an efforts law that gives a maximum on the T/G connection.

As it was predictable, the Derrick Crane's load makes the structure descend z = -24 mm on the extreme of the cantilever beam from the horizontal. It is important to notice that the second placed cable experiment an increasing of tension, what is traduced in an increasing of length. It won't be an increasing of length to be worried about if the displacements of the nodes where the cable is connected to, are the same of the ones designed.

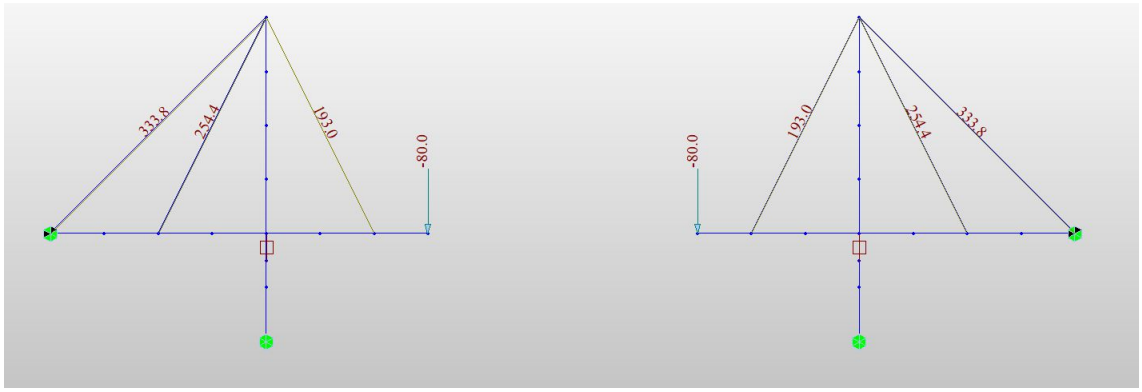


65. DEFORMED SHAPE DIAGRAM ON STAGE 7

Stage 7-1

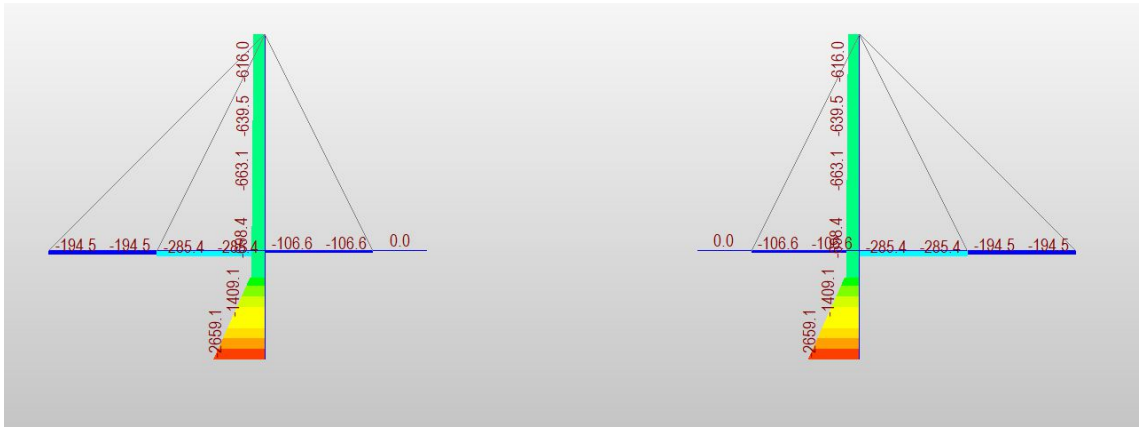
This is the second part of the stage 7, which is formed by two operations: one necessary for the construction process, the other, the adding of a new element bridge element.

A new cable is placed on this stage. It unites the pier with the top of the Upper Tower, so it can be said that the side spans are almost built, suffering only a few deflections because of the construction of the main span.



66. STAGE 7-1 - GENERATE CABLES

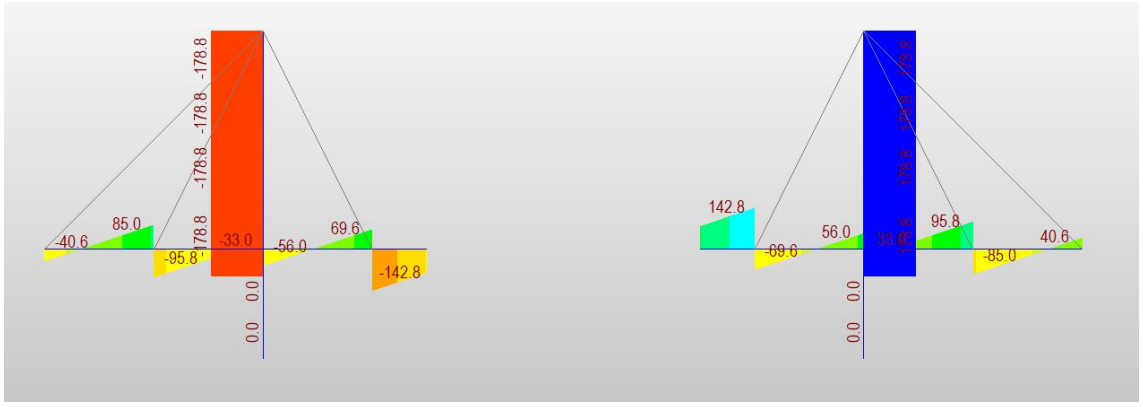
On the axial efforts diagram, as it has been seen until now, the only differences are the increasing of load values, and the apparition of a new compressed girder segment due the application of tension on the cable. The main form of the axial effort's law is basically following the same pattern during all the construction process.



67. AXIAL EFFORTS DIAGRAM ON STAGE 7-1

On shear efforts diagram, however, the placement of a new cable increases the value of the shear in the Upper Tower because of the unbalancing of tensions suffered for each side. This new-placed cable, with 333,8 tonf of tension, makes the rest of the shear values (except in the cantilever beam's extreme) decrease to compensate the efforts on the tower.

The cantilever beam has the same shear value because there is no new load or tension applied on it.



68. SHEAR EFFORTS ON STAGE 7-1

The bending moments, due the placement of this new bridge's element, are exponentially boosted. The tension given to the new-placed cable pulls the top of the Upper Tower creating an 8044,2 tonf·m bending moment at the bottom of it, therefore the bending moment transmitted to the Lower Tower is also increased to 5956 tonf·m.

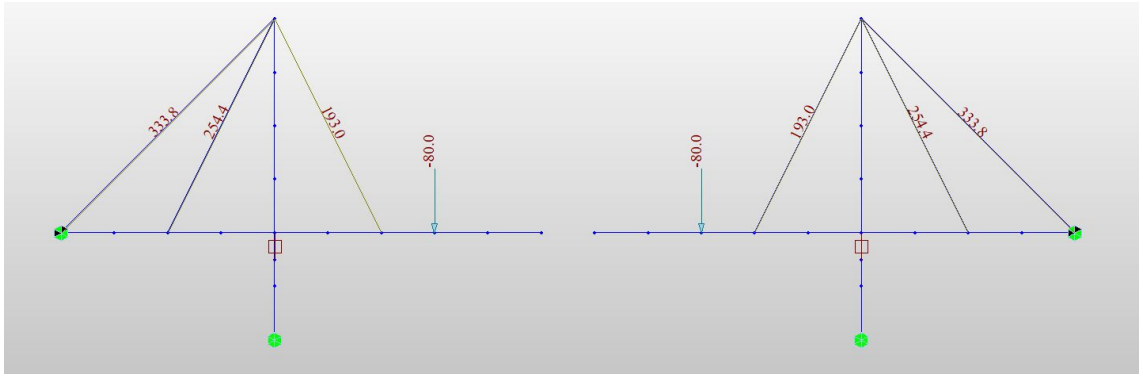
69. BENDING MOMENTS DIAGRAM ON STAGE 7-1

The deformed shape on this stage, as it can be seen, turns into a quite regular beam on the side spans due the placement of the two cables and the rigid link between the tower and the girder. The lower stiffness of the Upper Tower makes the side span girder remain flat while the pylon turns to equilibrate the imposed efforts. On the other side of the tower, the reduction of the descending of the girder is due the displacement of the top of the Upper Tower which pulls the deck up leaving a z = -15 mm descending in the cantilever.

70. DEFORMED SHAPE DIAGRAM ON STAGE 7-1

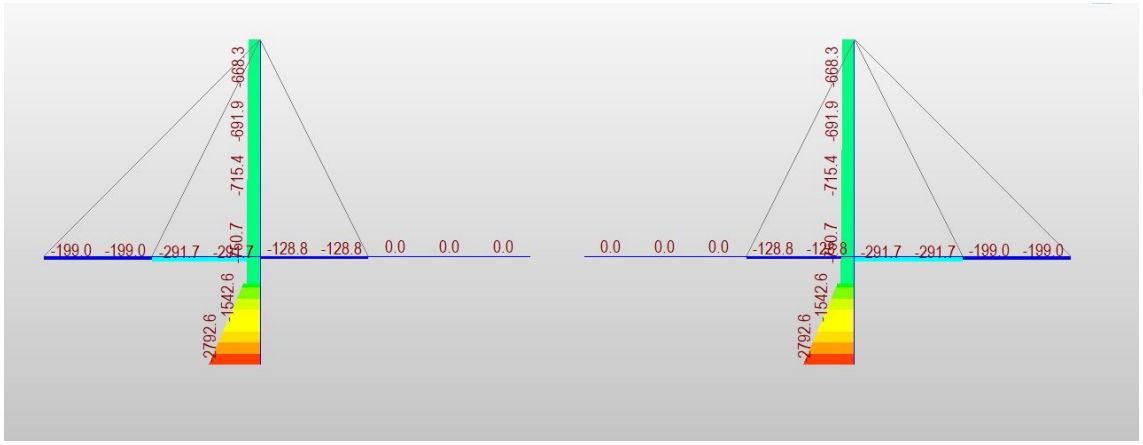
Stage 8

In this stage, a new girder segment is erected, the last one before the placement of the key segment. The cable-stayed bridge still advancing in its construction process with just un cable left to be placed, and the key segment to be built. Stage 8 could be seen as follows.



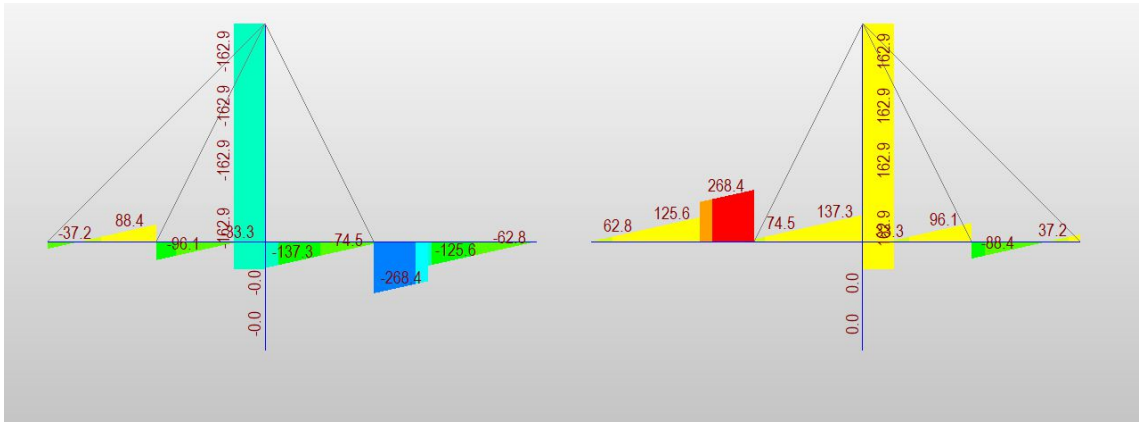
71. STAGE 8 - GENERATE MAIN GIRDER

The increasing of load due the new erected segments, increases the axial efforts in the whole structure. The placement of the cables makes, at the same time it holds the girder, the deck more compressed, which gives more resistance to the cable-stayed bridge.



72. AXIAL EFFORTS DIAGRAM ON STAGE 8

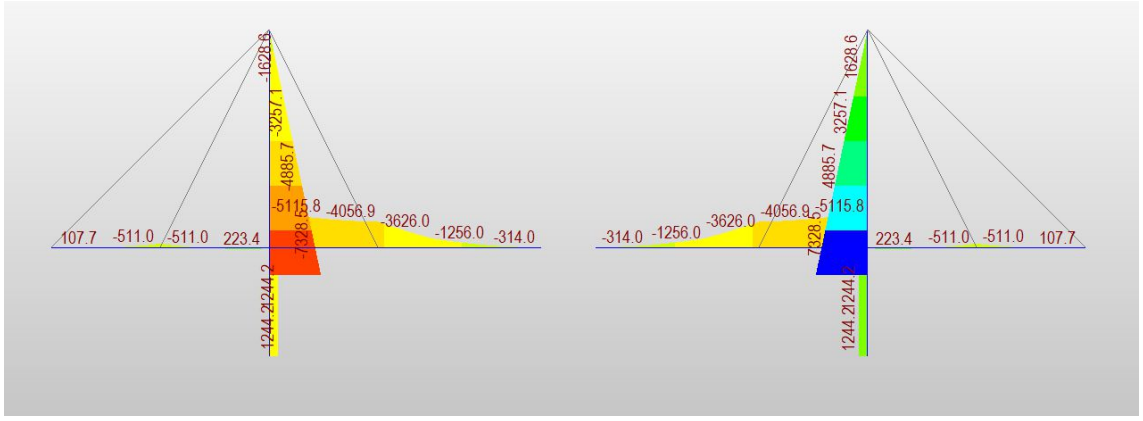
On this stage shears diagram's it can be observed the similarity with Stage 5, where just one cables was placed, because the cantilever's law has the same values that fifth stage but they are displaced 20 m to the centre of the main span. The cantilever beam has the same load applied on it but the fact the construction process is in a advanced state reflects a higher shear on the tower (101 tonf on stage 5 against 162,9 on this stage). The placed cables deform the structure in a way that the pylon's displacement creates higher efforts on it, but lower on the deck.



73. SHEAR EFFORTS DIAGRAM ON STAGE 8

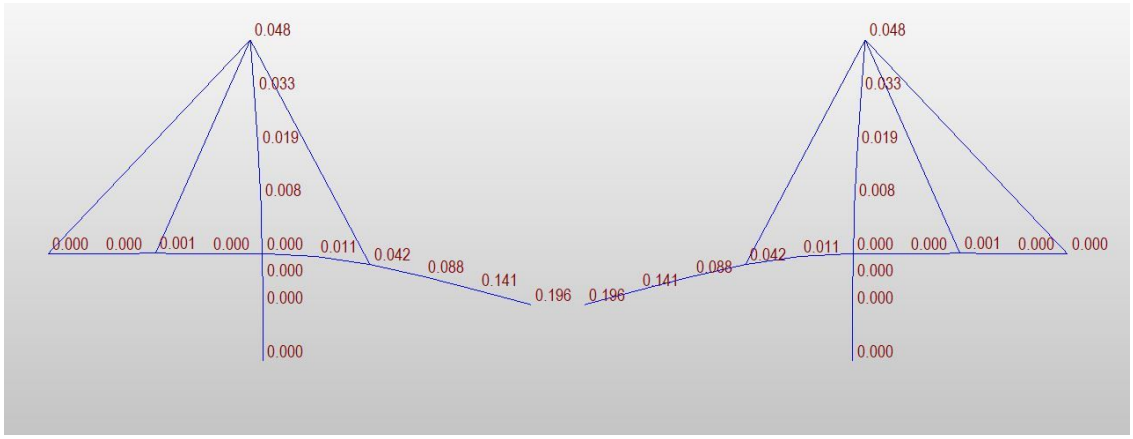
It happens something similar with bending moments diagram. While on 5th stage a 4545 tonf·m was hold by the bottom of Upper Tower, now It can be seen a 7328,5 tonf·m at the same place, and an increasing effort's law from the node 7 until the T/G connection, even if the bending moments are the same on cantilever beam.

Comparing with the previous stage, the bending moments has decreased on Upper and Lower Tower because of the raising of effort's law on the main span girder. The proximity to the finalization of the construction process creates a structure which suffers higher efforts and makes this stage becomes one of the most dangerous and delicate.



74. BENDING MOMENTS DIAGRAM ON STAGE 8

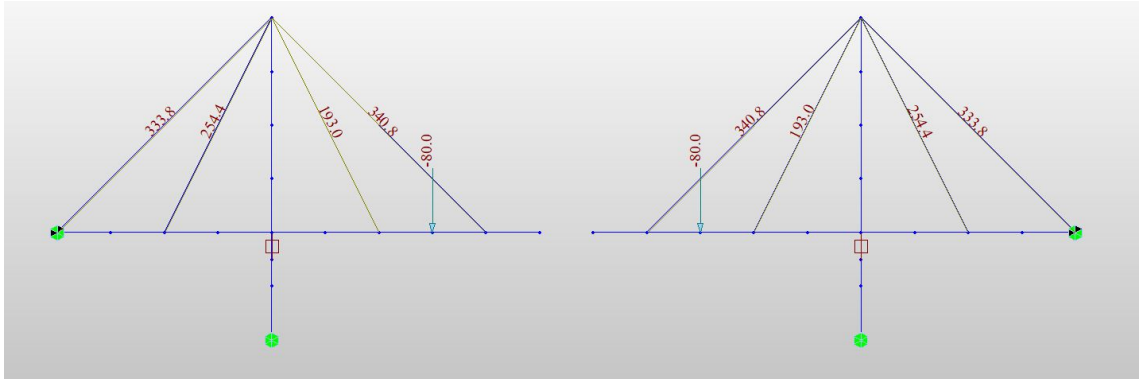
What it was said before is also reflected on Deformed Shape diagram. The deflections on the cantilever beam, just before the erection of Key Segment, are the highest the structure has to hold. Those 196 mm, as maximum displacement, have to be carefully controlled to avoid the collapse of the cable-stayed bridge.



75. DEFORMED SHAPE DIAGRAM ON STAGE 8

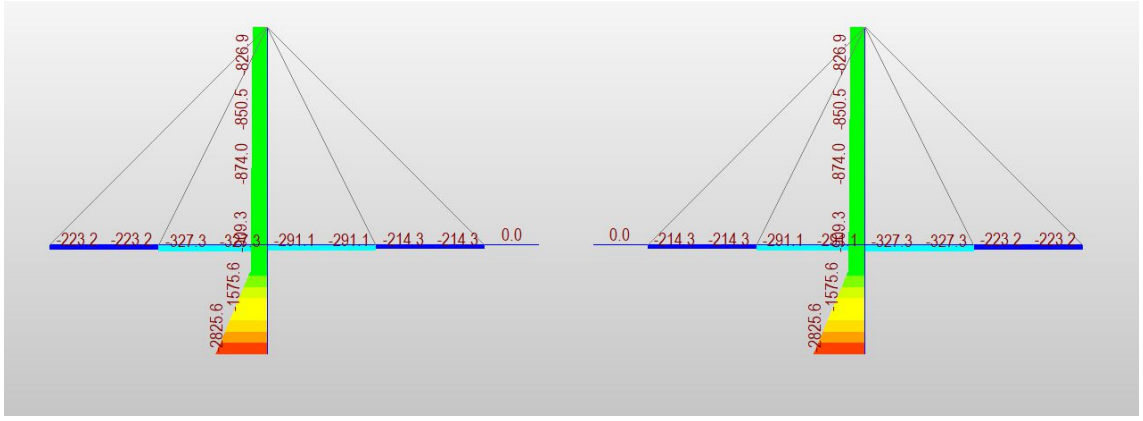
Stage 9

Before the erection of Key Segment, the last cable has to be placed. This is what happens in stage 9. The fourth cable is installed with a pretension force of 340,8 tonf/m.



76. STAGE 9 - GENERATE CABLES

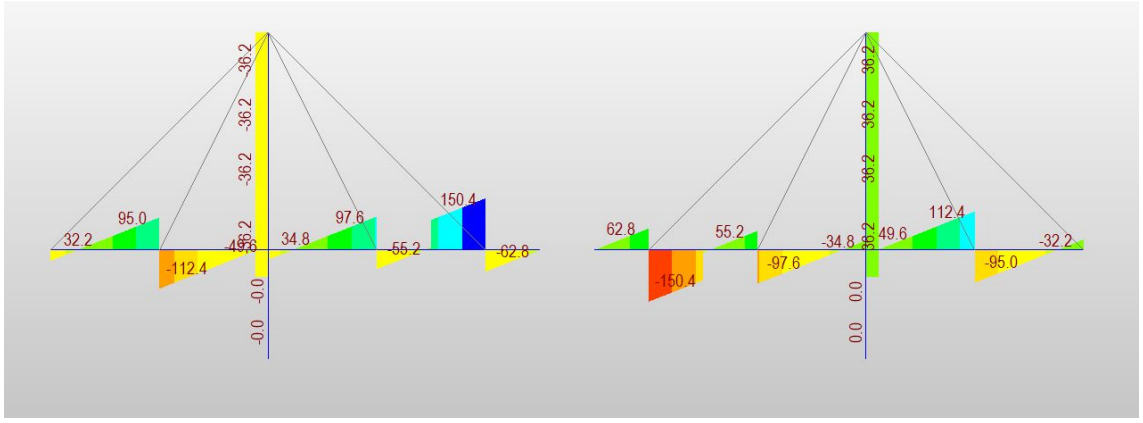
Axial efforts, as it has been observed until this construction stage, follows the same pattern rising effort's values and compressing girder segments whose are contained on those closed areas formed by cables and the girder.



77. AXIAL EFFORTS DIAGRAM ON STAGE 9

As it happens with stage 8, this stage can also be paired with a previous stage which has the same (but less expanded) efforts law. In this case, Stage 9 is very similar to Stage 6. Values on the towers and the deck are increased, but the pattern is the same.

It can be found a 36,2 tonf shear effort on the Upper Tower and a null effort on the Lower Tower, and the maximum shear is found just before the cantilever beam. It happens because of the transmission of cantilever's shear to bi-supported beam on its side.

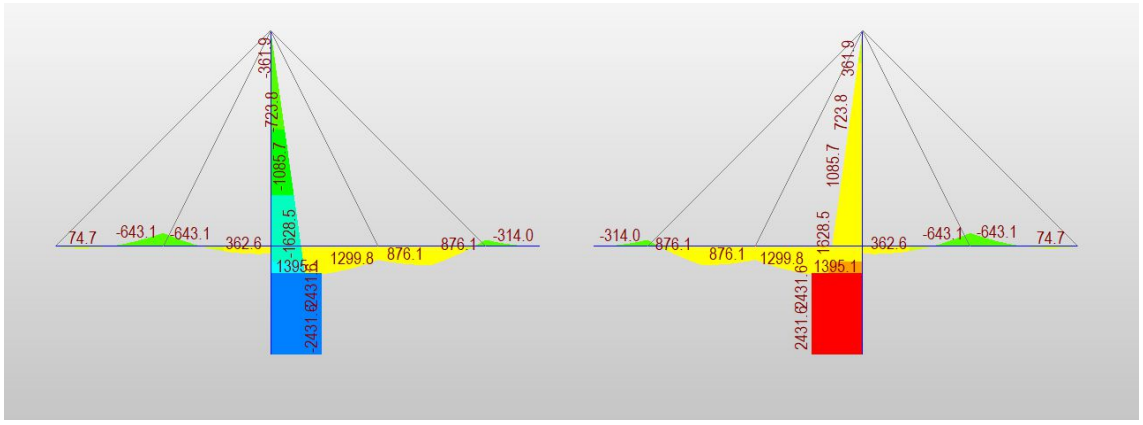


78. SHEAR EFFORTS DIAGRAM ON STAGE 9

For bending moments diagram, it can also be directly compared with Stage 6 since in stage 9 it can be found, like in shear efforts diagram, same form with higher values.

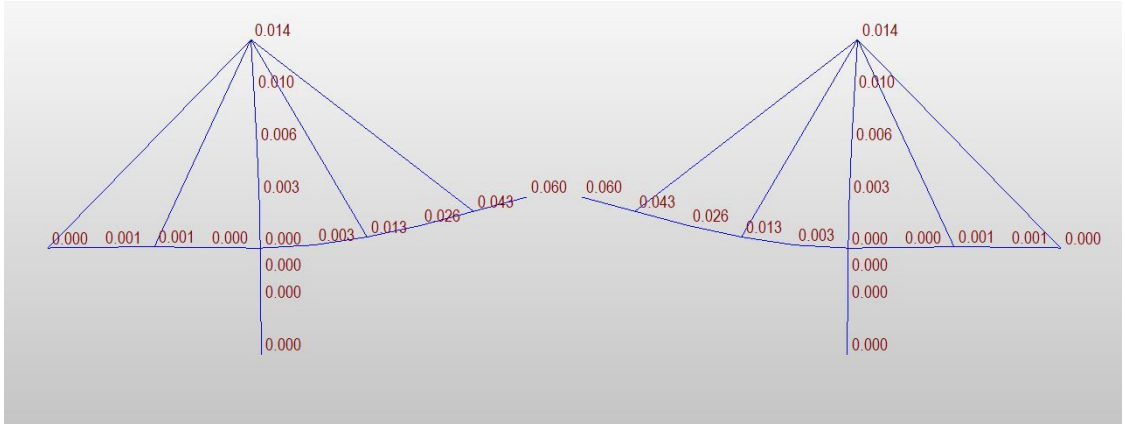
If we compare with previous stage, the main difference is observed on main girder since the bending moments increase until they go from negative moments in Stage 8 to positive moments in the present stage. The placement of the last cable brings some stability to the structure because it creates two (almost) symmetrical systems into a symmetrical structure, which is the whole bridge.

Even the values of the bending moment are considerably high, once the bridge is getting closer to the end, it becomes a closed system structure.



79. BENDING MOMENTS DIAGRAM ON STAGE 9

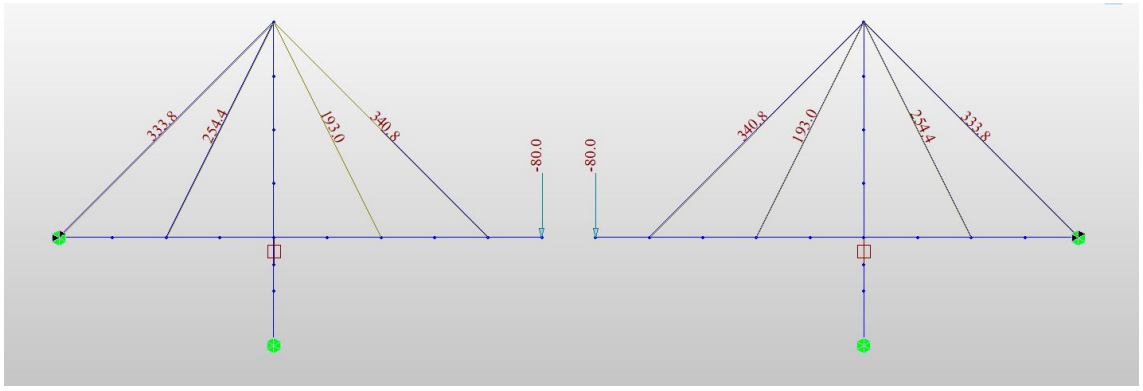
As it can be observed, the placement of the last cable raises the deck again but never achieving the negative displacement reached on previous stage. Just 60 mm from the horizontal before the preparation for the last step.



80. DEFORMED SHAPE DIAGRAM ON STAGE 9

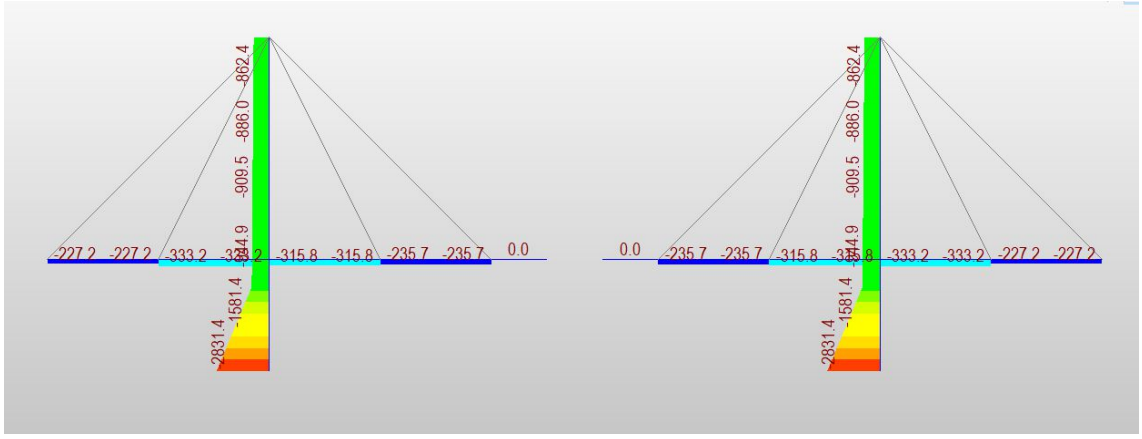
Stage 10

As the construction process is getting to the end, the last relocation of the Derrick Crane will be studied on this stage.



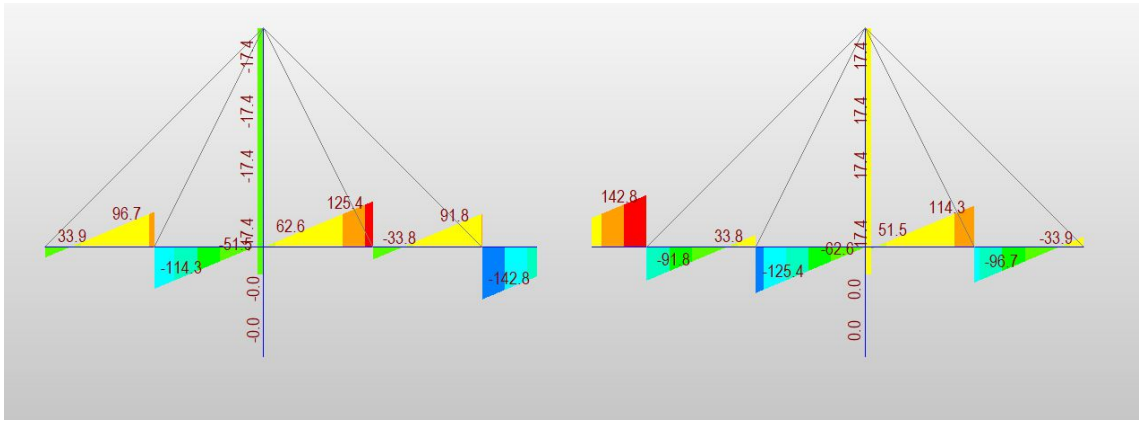
81. STAGE 10 – ADVANCING OF THE DERRICK CRANE

Axial efforts are scarcely modified on this stage. As the derrick crane's punctual load is getting far from the pylon, the influence on axial's value is increased. Just an increment of 6 tonf is noticed on the bottom of the pylon, and between 4 tonf and 24,7 tonf in the main girder.



82. AXIAL EFFORTS DIAGRAM ON STAGE 10

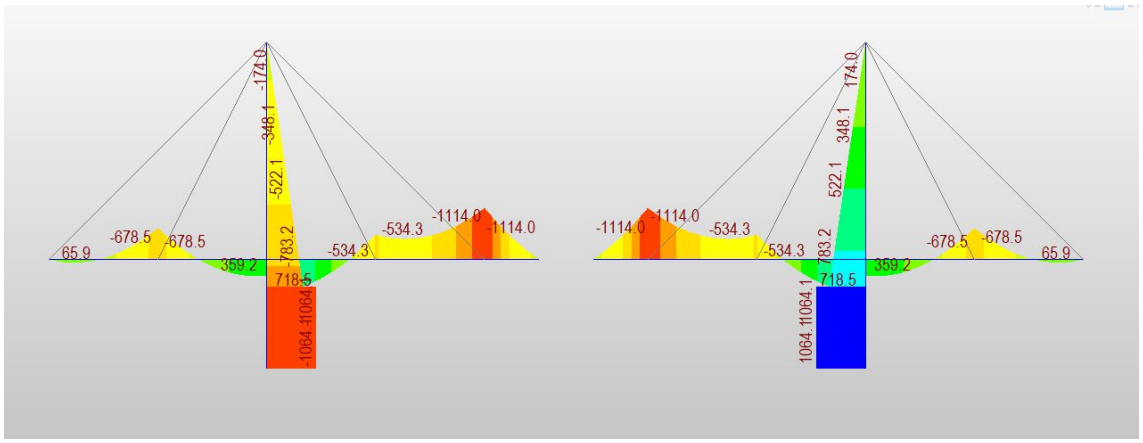
On shear efforts diagram, however, the relocation of a punctual load working orthogonally to the deck really changes shear values, and as it could be said that the construction cycle is repeated, a similarity with a previous stage is found again (Stage 7-1). In this case, it doesn't get so closer like it has been seen before. Shear efforts on the pylon are highly reduced because of the completed cables placement and as the structure is balanced. However, shear efforts on the girder turns higher since the structure has become bigger, and therefore, heavier.



83. SHEAR EFFORTS DIAGRAM ON STAGE 10

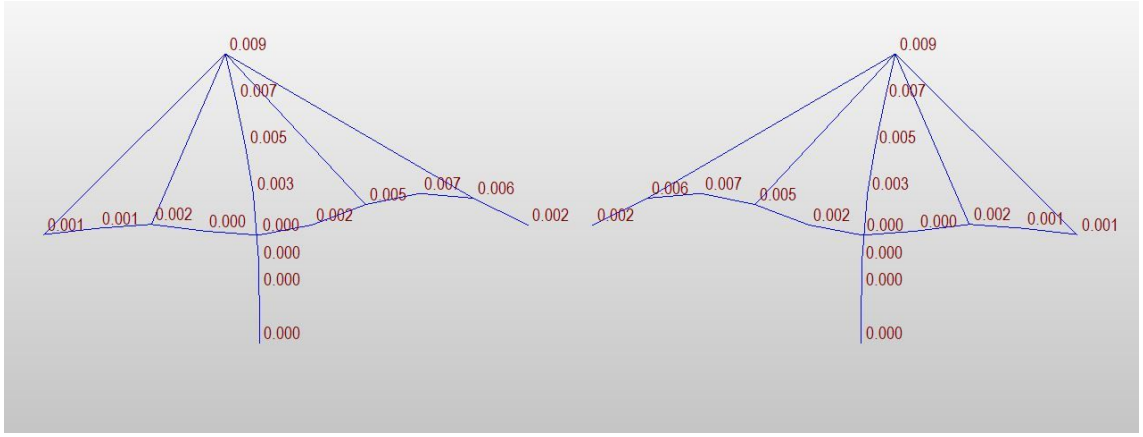
It could be said that the main difference with Stage 10's homonym stage is the fact that in this stage, the structure is more advanced than Stage 7-1. because the same bending moment on the cantilever is found, and the form of the diagram is very similar. However, there are other differences that also must be explained, for example, the high difference between the pylon's bending moment values or the increasing of the values on the side span girder.

The decreased value on the pylon is due the fact the cable-stayed bridge is getting stiffness as a structure and not only as a sum of its elements, as it splits the efforts in a way that the bridge balances itself.



84. BENDING MOMENTS DIAGRAM ON STAGE 10

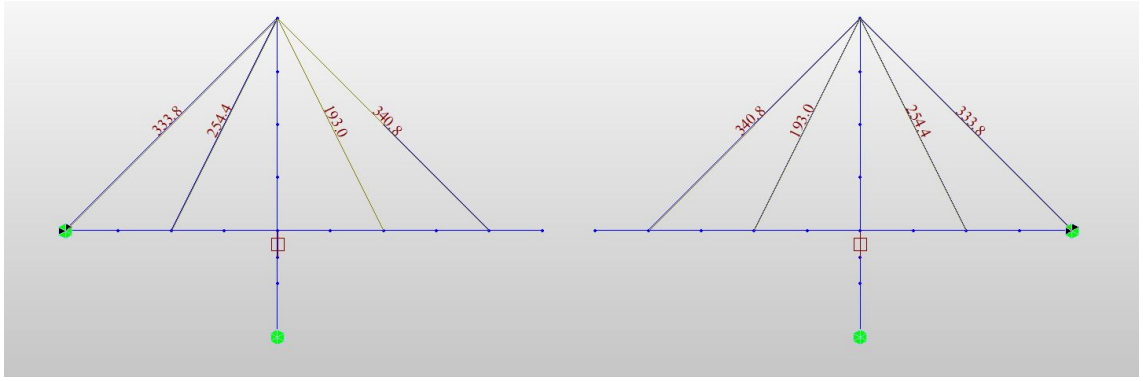
In the case of the Deformed Shape diagram, as it was predictable, the relocation of the derrick crane's load makes the extreme of the cantilever beam descend to z = +1,5 mm. Due this descending in the girder, the pylons experiment a displacement to the middle span leaving the deflection of the top's Upper Tower in Dx = -0,0057 m and Dz = -0,0066 m.



85. DEFORMED SHAPE DIAGRAM ON STAGE 10

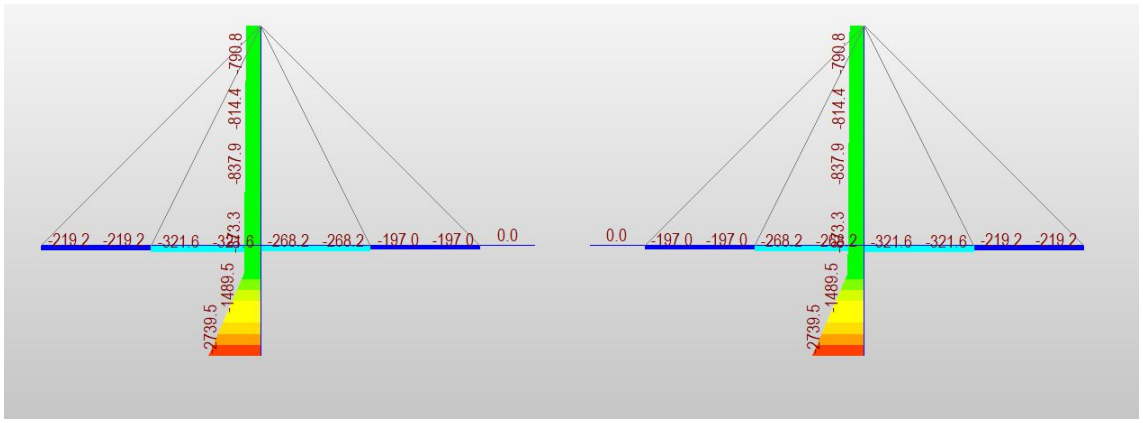
Stage 11

Once this stage is reached, the removal of the Derrick Crane leaves the structure near to be finally closed. As the tutorial says, this is a stage where 2 two steps are considered. First the removal of the Derrick Crane's load (Stage 11) and the construction of the Key Segment (Stage 11-1).



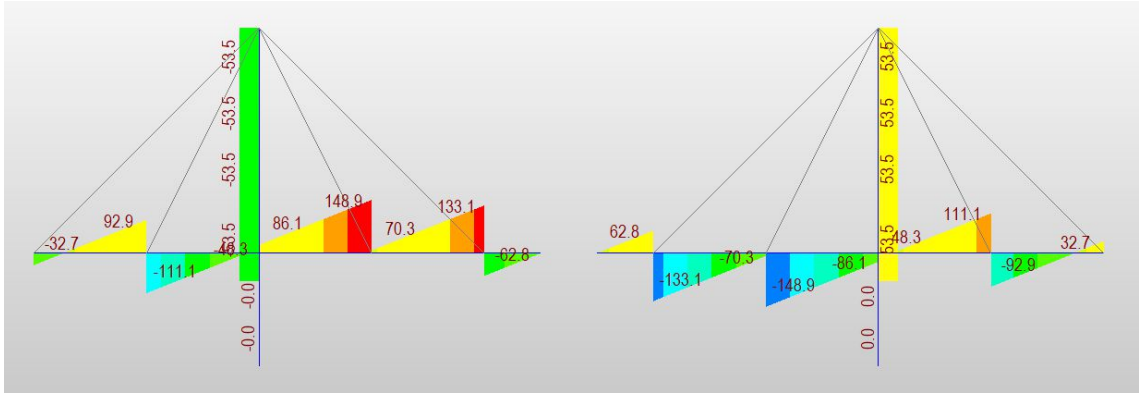
86. STAGE 11 - REMOVE DERRICK CRANE

In the case of axial efforts, it is clear to see what happens when a punctual load is eliminated. Even if only 80 tonf are removed, the difference between the previous stage and this one is of 91,9 tonf on the bottom of the pylon. That happens because of the bending moment created by the Derrick Crane's load on the extreme of the cantilever which makes the rest of forces being relocated to the tower's base. Once you remove it, the resultant created for that moment is redistributed on a different way and leaves the pylon less charged of the value of the punctual load.



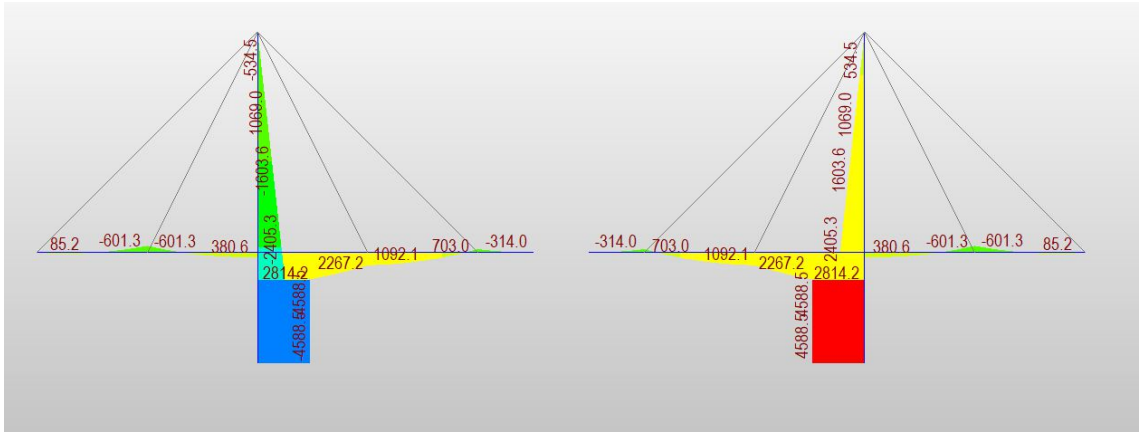
87. AXIAL EFFORTS DIAGRAM ON STAGE 11

In terms of shear effort, the direct relation between the punctual load and the sear diagram can be observed on the extreme of the cantilever, where the load reduction is effectively 80 tonf. The removal of the Derrick Crane's load affects also to the Upper Tower which takes higher values due the reduction of load on the main Girder. These are the two more noticed changes due this stage, since side span's loads are barely reduced.



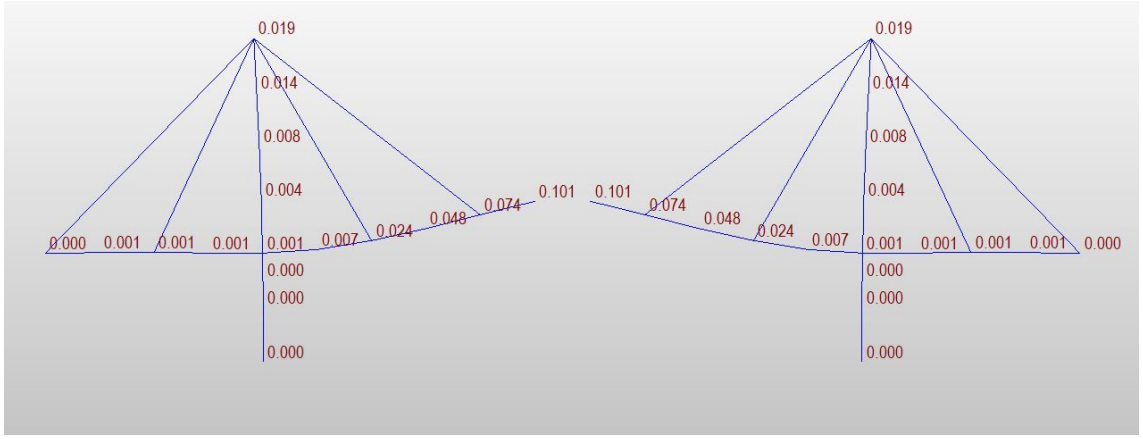
88. SHEAR EFFORTS DIAGRAM ON STAGE 11

On the other hand, for bending moments diagram, the change on efforts law are considerably high. The almost entire disappearance of negative bending moments on the main girder makes the girder takes positive efforts whose increase the bending moments on the bottom of the tower more than a 400%. So, close to finish girder's construction, the Bottom Tower takes one of the highest values suffered during construction process.



89. BENDING MOMENTS DIAGRAM ON STAGE 11

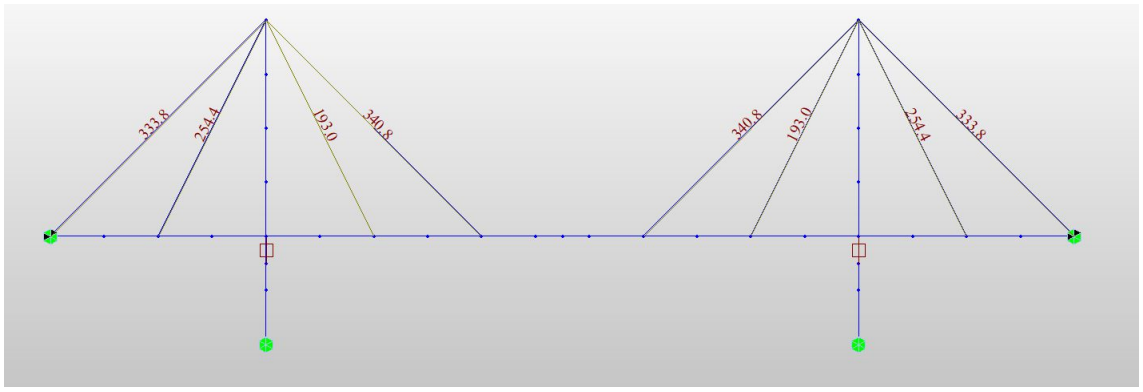
As intuitively happens, when a force against a displacement is removed, the displacement is increased, and that is what is seen on the deformed shape diagram. Cantilever's extreme raises its position until 101 mm above the horizontal. Those 101 mm prepare the bridge for the last step: erection of the Key segment.



90. DEFORMED SHAPE DIAGRAM ON STAGE 11

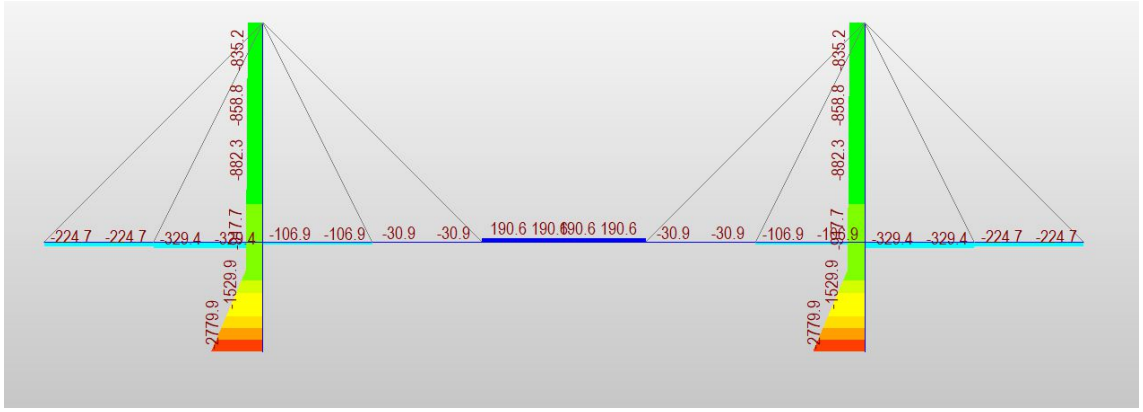
Stage 11-1

This is the final stage of the construction bridge itself. Once the Key Segment is generated, the bridge closes its static scheme and the efforts takes its final distribution.



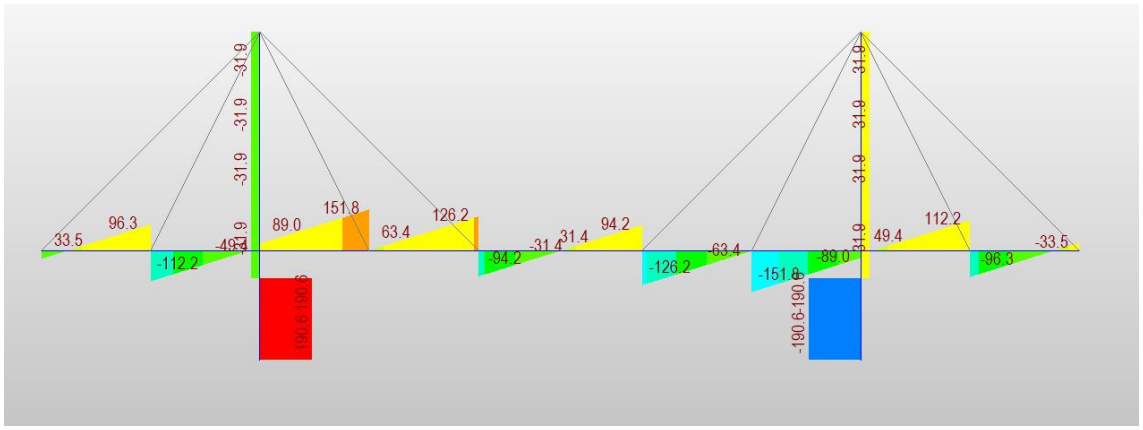
91. STAGE 11-1 - GENERATE KEY SEGMENT

On axial efforts diagram, for instance, it can be observed the first appearance of traction efforts on the girder. The cables pull the girder, which takes compression efforts in all its length, but the Key Segment, which has the function of unite the two parts of the bridge by means of traction forces. By the way, the traction forces generated on this part of the cable-stayed bridge are not excessively high and keeps the distribution of the efforts as it has to be for a good efficiency of the materials.



92. AXIAL EFFORTS DIAGRAM ON STAGE 11-1

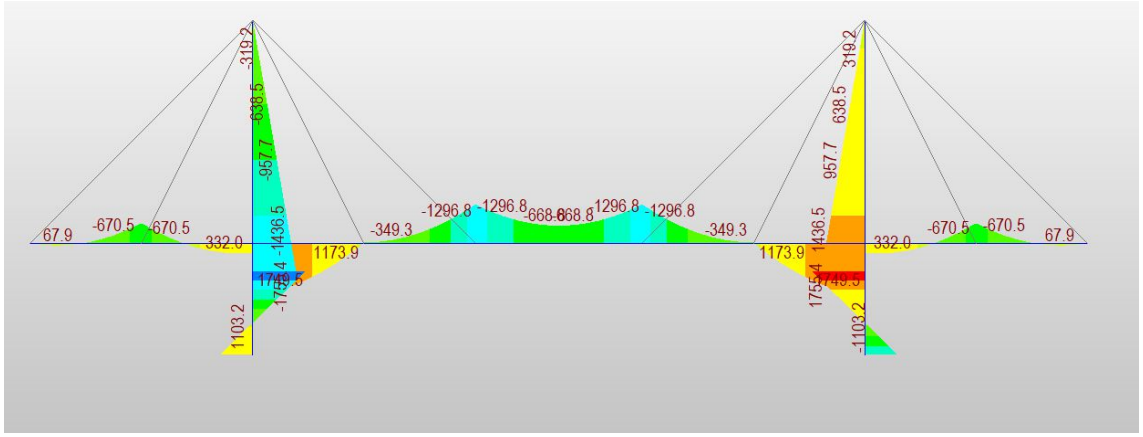
On shears diagram, its clearly seen the behaviour of a continuous beam along the girder, which has, obviously, a symmetric distribution and maximum values of My = 151,8 tonf·m on the girder, close to the pylon, and My = 190,6 tonf·m on the base of the Towers. As it can be noticed, this could be an acceptable distribution for an ended bridge, but it still needs to be prepared for the application of additional load, so jack loads will be needed to balance it.



93. SHEAR EFFORTS DIAGRAM ON STAGE 11-1

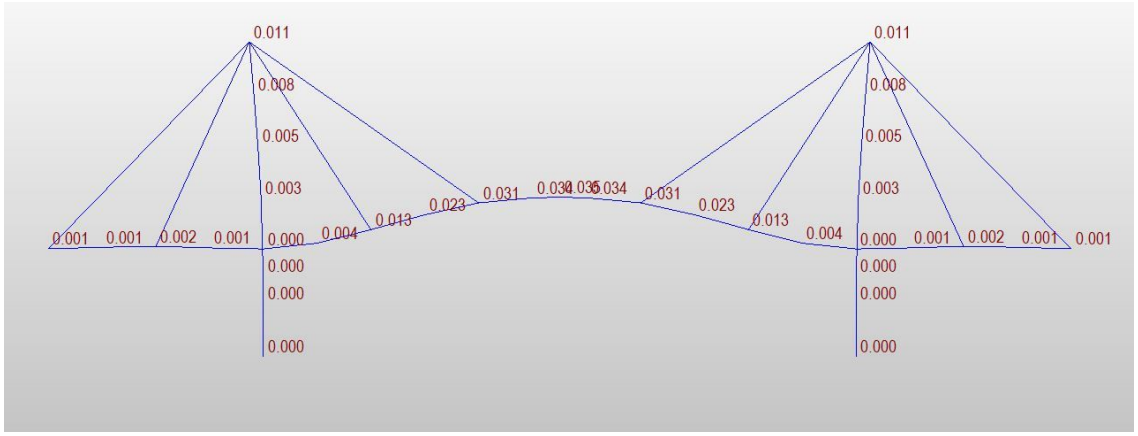
Once the bridge is closed, the redistribution of all efforts along the structure transforms completely the bending moments law. On one side, the appearance of positive and negative bending moments on the Lower Tower claims that the whole structure is finally erected because of the necessity to has a balanced behaviour on both axles.

In the case of the girder, high bending moments show up in the middle of the centre span. Due the fact the additional load is still not applied, the bending moments found on those elements are all negative because of the curvature the bridge creates due the tension of the cables. That curvature is also appreciated on the towers, which tend to turn to the exterior side.



94. BENDING MOMENTS DIAGRAM ON STAGE 11-1

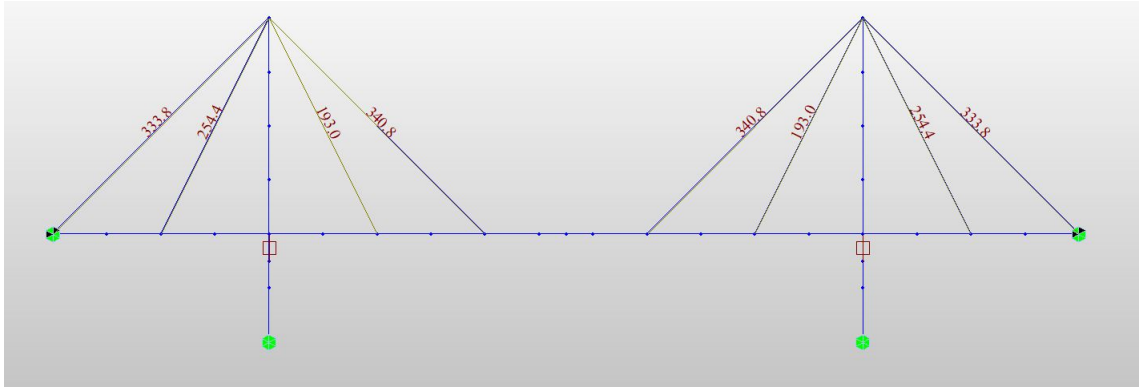
The curvature mentioned on bending moments' diagram is basically seen on Deformed Shape diagram. The deflection created in the middle of the bridge reaches z = +35 mm and Dxz = ±11 mm on the top of the towers.



95. DEFORMED SHAPE DIAGRAM ON STAGE 11-1

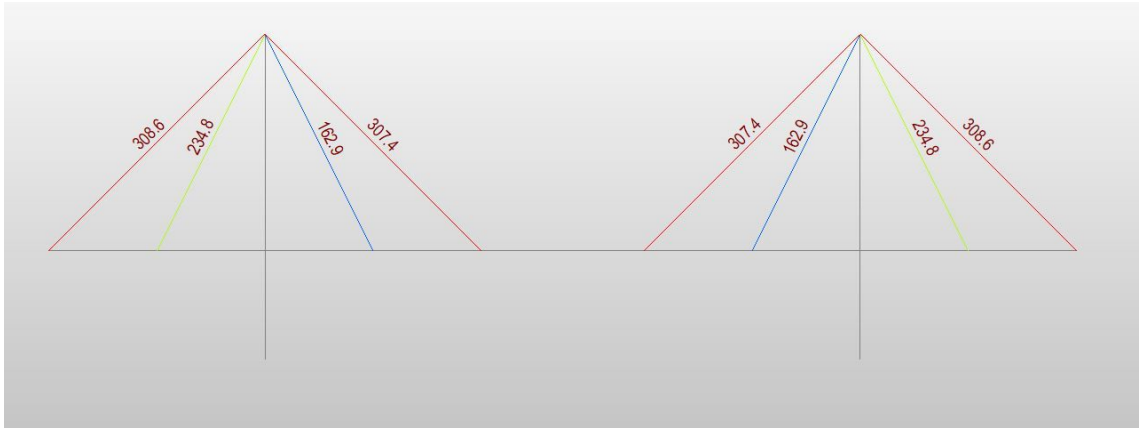
Stage 12

Once the bridge is entirely constructed, the application of the Jack Up Loads take place. Those tensions are found using the Forward Approach, which is thought to calculate the tensions that the bridge needs to balance the forces produced during the construction process and the forces to achieve its Objective Completion Stage.



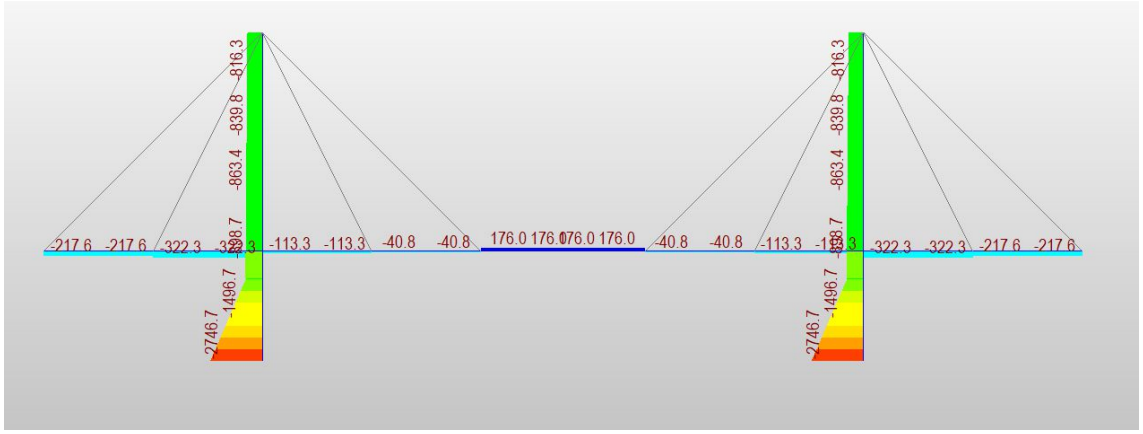
96. STAGE 12 - APPLY JACK UP LOADS

It can be checked on the next diagram, the tensions on the cables in this stage are lower than the ones that are going to be applied as a result of the calculation in the design stage. The difference between the actual state and the calculated tensions on the left side and the right side of the pylon creates a decreasing of axial forces in the key segment as it is going to be seen.



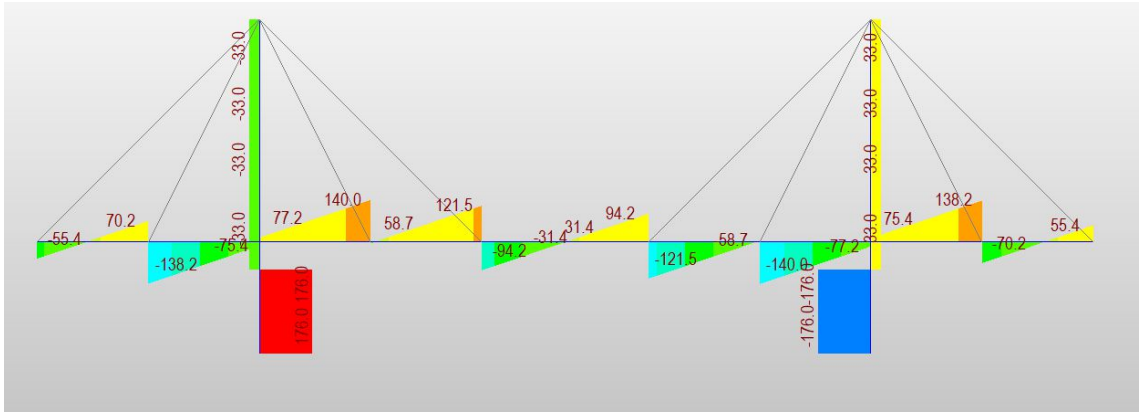
97. TRUSS FORCE DIAGRAM ON STAGE 12

Where there was an axial effort of Fx = 190,6 tonf, now it can be found a lower traction force due the Jack Up Loads applied on the cables. The compression forces on the segments of the main girder are increased pulling the top of the Upper Tower to the centre of the span, releasing the tensions on the Key Segment.



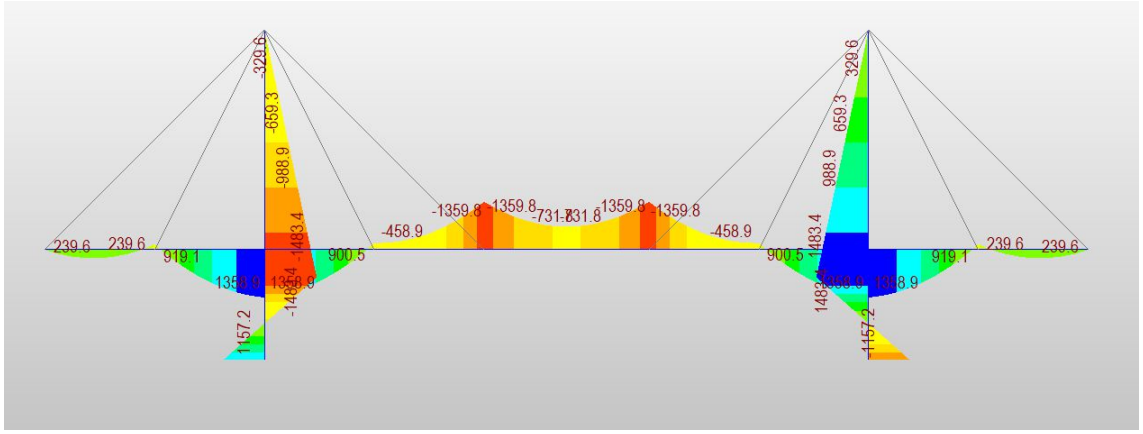
98. AXIAL EFFORTS DIAGRAM ON STAGE 12

The increasing of axial efforts becomes a decreasing of shear efforts, and vice versa, like it can be checked on the next image. While the shear efforts remain very similar on the Upper Tower, the Lower Tower suffers a decreasing of the same magnitude as the deck. The application of the Jack Up Loads rebalances the structure getting closer of its final state.



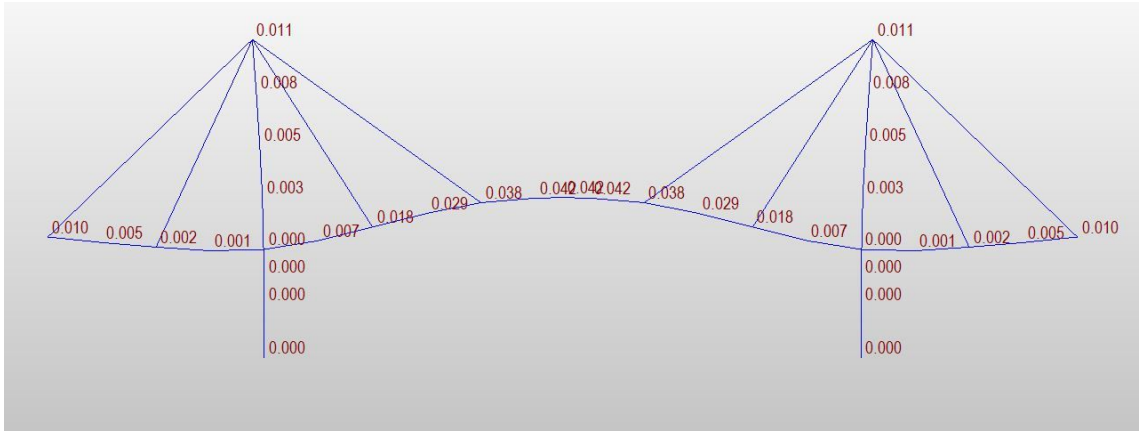
99. SHEAR EFFORTS DIAGRAM ON STAGE 12

All these applied loads also create a rebalance of the bending moments. The highest difference with the previous stage is found close to the pier, where the bending moments goes from My = -670,5 tonf·m, as a maximum value, to My = 919,1 tonf·m on the T/G connection. This stage suffers a great inequality of values all along the structure because is the previous stage before the Additional Loads application, which means that on this stage there is no load applied but the reactions to fight against it are already placed.



100. BENDING MOMENT DIAGRAM ON STAGE 12

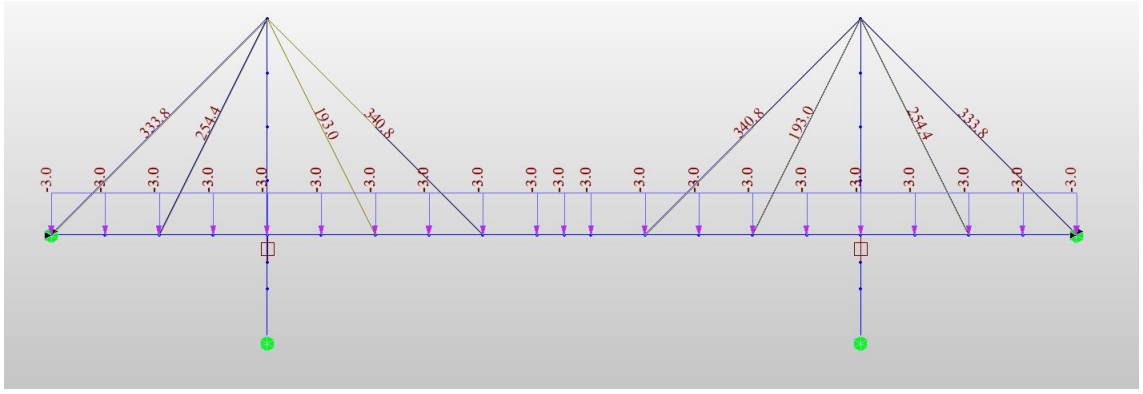
As it can be seen on the Deformed Shape diagram, the jack loads pull the deck up waiting for the application of the additional load whose will balance it all. Highest deflection is found in the middle of the main span with a z = +42 mm value.



101. DEFORMED SHAPE DIAGRAM ON STAGE 12

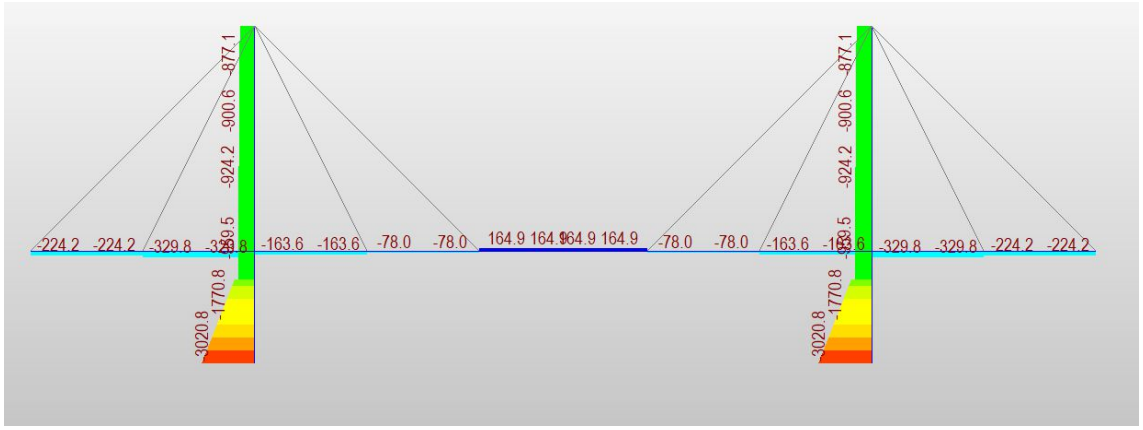
Stage 13

This is the final stage, where the additional loads are applied to balance the structure with the inside efforts created by means of imposed tensions.



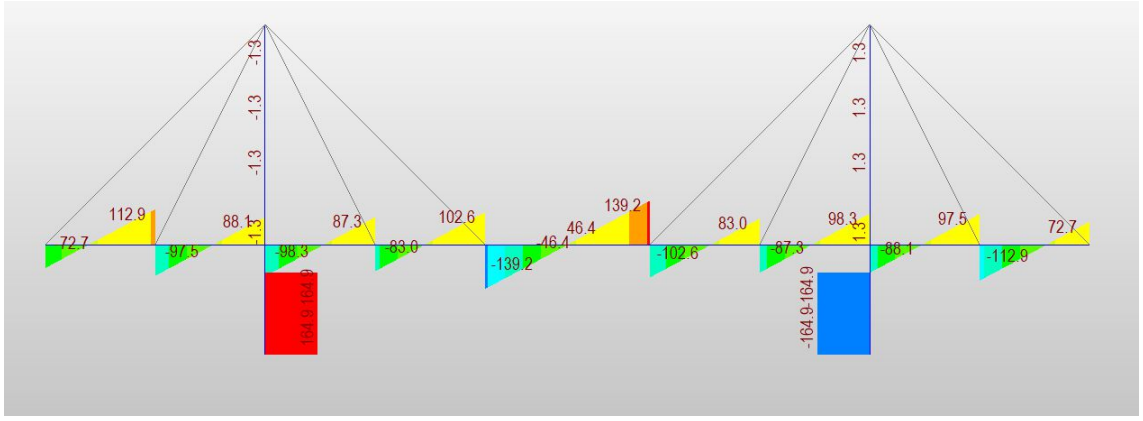
102. STAGE 13 - APPLY ADDITIONAL LOADS (FINAL STAGE)

What is found on axial efforts diagram of the last stage is a structure completely hold for its pylons and a compression efforts on the girder for a better work on deflection. This is also important to say the roll of the Lack Of Fit Force on the Key Segment. The traction suffered for the middle segments of the main span is traduced in a positive axial force (traction) due the tensions of the cables.



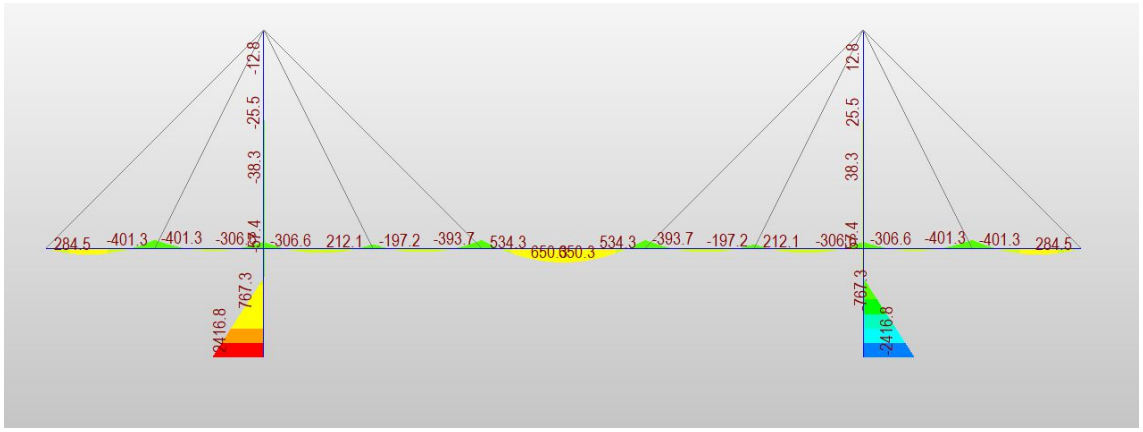
103. AXIAL EFFORTS DIAGRAM ON STAGE 13

The final connection by means of the key segment, makes the structure become in a continuous beam which draws the next shear efforts diagram. The Upper Tower is almost completely free of shear what allows it to work only with axial efforts.



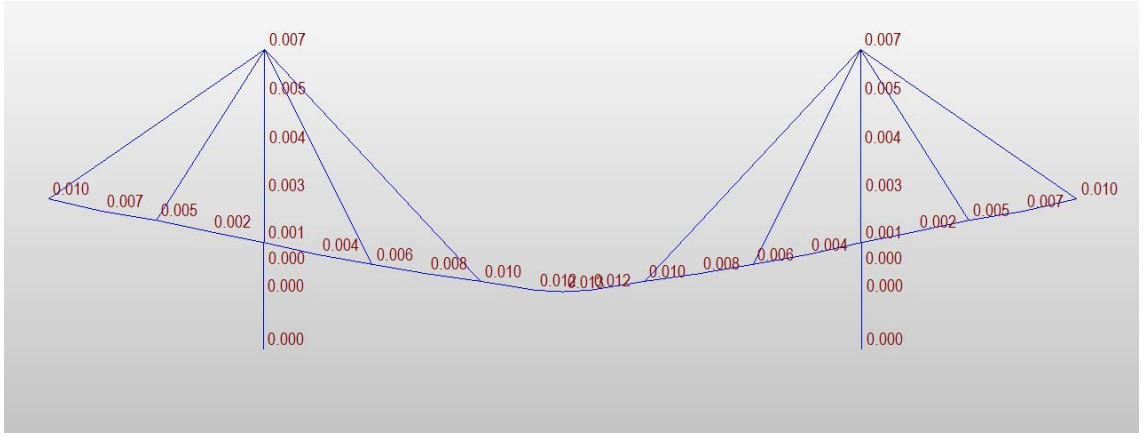
104. SHEAR EFFORTS DIAGRAM ON STAGE 13

In the referring of bending moments, the highest effort is found on the Lower Tower because of the traction of the Key segment. Once the structure is closed, the girder starts working as a continuous beam and the high bending moments are relocated to the foundations by means of the Lower Tower, which is the target of any structure.



105. BENDING MOMENTS DIAGRAM ON STAGE 13

As it can be observed, the final deflection caused by the application of the additional loads reaches the highest value in the middle of the main span, as it could be imagined. The maximum displacement is z = -12mm on node 11, but it is also important the displacement on the pier, which is z = +10 mm, caused by the transmission of displacements due the stiffness of the girder and the T/G connection working as a rigid link.



106. DEFORMED SHAPE ON STAGE 13

Chapter 5 – Results Analysis

As it is been illustrated on previous chapter, deck segments are added until the construction of Key Segment, the cables are placed to limit the girder's displacements and the structure reaches the stabilization on Stage 11, when the two half parts of the bridge become one.

The evolution of all the efforts suffered by the structure during the construction process reflects the high level of indeterminacy the bridge affronts. Like it was explained in the beginning of this thesis, cable-stayed bridges are very indeterminate structures that must be well-designed and controlled as much as possible.

The MIDAS/Civil Software has developed different algorithms for a good design of the cable-stayed bridges. Using tools as Lack of Fit Force function, the software compensates the deficiencies attributed to some construction methods bringing a program that can solve almost any type of bridge.

It has been seen how does the software works, calculating by means of iterations and extra functions that allows us to analyse deeply the behaviour of the cable-stayed bridge. Even if the majority of the possible functions on this program are not used, it can be noticed that MIDAS/Civil Software, as is well-known, is a powerful software for structures design.

The analysis of all these construction stages has brought some conclusions that must be commented. First, the analysis of the construction method itself will be presented, then, the evolution of all the parts, one by one, will be exposed.

Construction Method Analysis

As it could be seen during the diagrams' analysis, the static scheme of the structure has been changing a lot, since the bridge has been built element by element with cantilevering construction method.

This method is very efficient and useful when the difficulties of using a temporary supports method, like very deep ravines o the impossibility of settlement of the temporary structures, makes the construction of the cable-stayed bridge becomes very hard and expensive. It is important to say that this construction method becomes economic when a certain span length is projected, so the finding of a balance between span's length and construction expenses develops imperative.

As it has been checked on MIDAS/Civil Software tutorial and the results of the diagrams presented before, there is a difference that is has to be analysed in reference of the two calculation methods that has been used.

The difference between cable pretension and girder deflection obtained by Final Stage Analysis and Forward Stage Analysis are presented on the next tables

Element	Cable Pretension (Final Stage)		Cable Pretension (Forward Analysis)		Difference (%)	
	I	J	I	J	I	J
33	316,875	315,305	317,800	316,230	-0,29	-0,29
34	236,369	234,799	237,000	235,430	-0,27	-0,27
35	192,503	190,933	192,174	190,604	0,17	0,17
36	344,595	343,025	344,347	342,777	0,07	0,07
37	344,595	343,025	344,347	342,777	0,07	0,07
38	192,503	190,933	192,174	190,604	0,17	0,17
39	236,369	234,799	237,000	235,430	-0,27	-0,27
40	316,875	315,305	317,800	316,230	-0,29	-0,29

107. Comparison of Cable pretension for Final Stage Analysis and for Forward Stage Analysis

Nodes	Final Stage (mm)	Forward Stage (mm)	Difference (%)
1	10,000	10,000	0,000
2	6,902	6,946	-0,637
3	4,710	4,777	-1,423
4	1,948	2,000	-2,669
5	-0,622	-0,623	-0,161
6	-3,512	-3,594	-2,335
7	-5,752	-5,925	-3,008
8	-7,958	-8,211	-3,179
9	-9,752	-10,063	-3,189
10	-11,898	-12,238	-2,858
11	-12,270	-12,614	-2,804
12	-11,898	-12,238	-2,858
13	-9,752	-10,063	-3,189
14	-7,958	-8,211	-3,179
15	-5,752	-5,925	-3,008
16	-3,512	-3,594	-2,335
17	-0,622	-0,623	-0,161
18	1,948	2,000	-2,669
19	4,710	4,777	-1,423
20	6,902	6,946	-0,637
21	10,000	10,000	0,000

108. Comparison of girder Deflection for Final Stage Analysis and Forward Stage Analysis

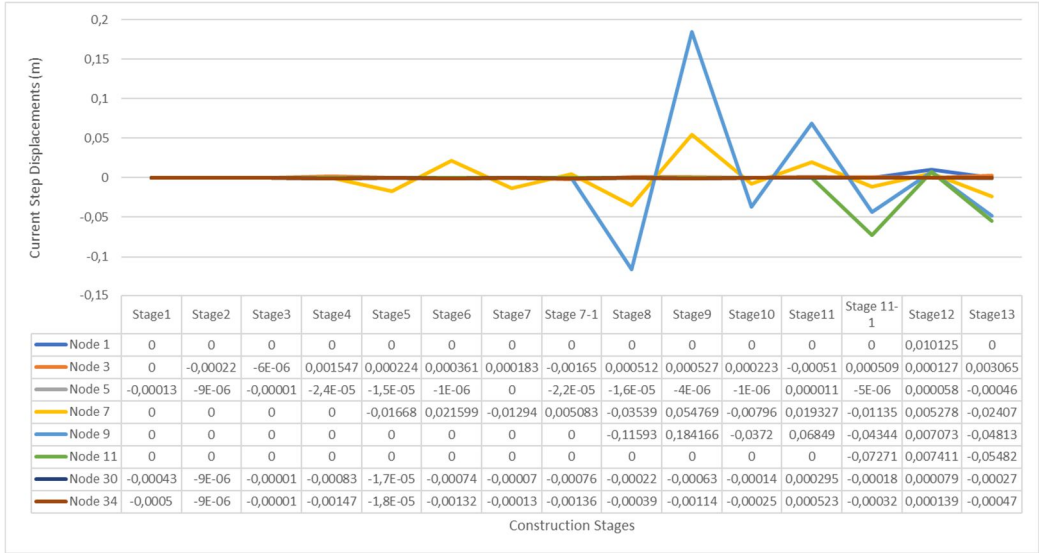
As it can be noticed, there are some differences that could be relevant, like the 3.189% on node 9 and 13 deflection between the two methods. The application of the Lack of Fit Force function can be determinant as it approaches the bridge more to reality.

As it has been observed in the previous analysis, the cables activation takes a decisive roll due they are placed in determined stages, when the vertical displacement of the girder takes excessive values that makes the structure suffer an unwanted unbalance.

Another important thing is been noticed on construction stage analysis. There is a notable oscillation on the Upper Tower and the Main Girder that must be controlled during the construction of the bridge due the unbalancing of the structure during the construction process. This oscillation makes those elements take values presented on the next tables.

	Stage1	Stage2	Stage3	Stage4	Stage5	Stage6	Stage7	Stage 7-1	Stage8	Stage9	Stage10	Stage11	Stage 11-1	Stage12	Stage13
Node 1	0	0,000045	0,000112	-0,00043	0,00037	-1,4E-05	0,000234	-0,00064	0,00072	-0,00011	0,000217	-0,00055	0,000698	0,000015	0,000131
Node 3	0	0,000045	0,000112	-0,00043	0,00037	-1,4E-05	0,000234	-0,00087	0,000714	-0,00014	0,000212	-0,00054	0,000692	0,000024	0,000123
Node 5	0	0,000045	0,000112	-0,00055	0,000369	-2,3E-05	0,000233	-0,00108	0,000707	-0,00018	0,000205	-0,00053	0,000682	0,000032	0,000114
Node 7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Node 9	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Node 11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Node 30	0	0,000117	0,000291	-0,01243	0,000935	0,008516	0,001393	-0,02066	0,003573	0,01336	0,002583	-0,00531	0,004055	-5,3E-05	0,003979
Node 34	0	0,000188	0,00047	-0,03147	0,001486	0,022635	0,003067	-0,05185	0,007569	0,035895	0,006296	-0,01265	0,008956	-0,0002	0,01001

109. DX DISPLACEMENTS ON DEAD LOAD COMBINATION

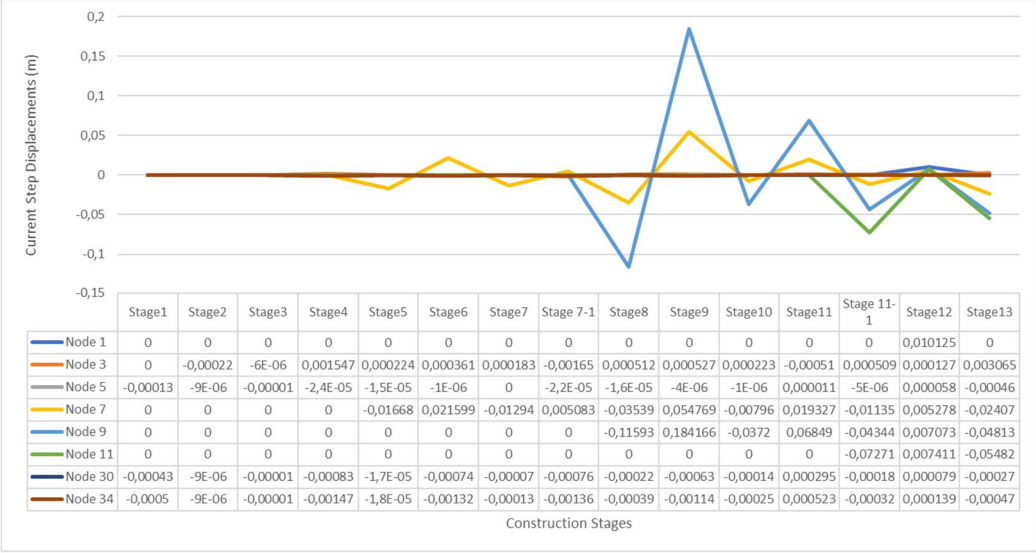


	Stage1	Stage2	Stage3	Stage4	Stage5	Stage6	Stage7	Stage 7-1	Stage8	Stage9	Stage10	Stage11	Stage 11-1	Stage12	Stage13
Node 1	0	0	0	0	0	0	0	0	0	0	0	0	0	0,010125	0
Node 3	0	-0,00022	-6E-06	0,001547	0,000224	0,000361	0,000183	-0,00165	0,000512	0,000527	0,000223	-0,00051	0,000509	0,000127	0,003065
Node 5	-0,00013	-9E-06	-0,00001	-2,4E-05	-1,5E-05	-1E-06	0	-2,2E-05	-1,6E-05	-4E-06	-1E-06	0,000011	-5E-06	0,000058	-0,00046
Node 7	0	0	0	0	-0,01668	0,021599	-0,01294	0,005083	-0,03539	0,054769	-0,00796	0,019327	-0,01135	0,005278	-0,02407
Node 9	0	0	0	0	0	0	0	0	-0,11593	0,184166	-0,0372	0,06849	-0,04344	0,007073	-0,04813
Node 11	0	0	0	0	0	0	0	0	0	0	0	0	-0,07271	0,007411	-0,05482
Node 30	-0,00043	-9E-06	-0,00001	-0,00083	-1,7E-05	-0,00074	-0,00007	-0,00076	-0,00022	-0,00063	-0,00014	0,000295	-0,00018	0,000079	-0,00027
Node 34	-0,0005	-9E-06	-0,00001	-0,00147	-1,8E-05	-0,00132	-0,00013	-0,00136	-0,00039	-0,00114	-0,00025	0,000523	-0,00032	0,000139	-0,00047

110. DZ DISPLACEMENTS ON DEAD LOAD COMBINATION

	Stage1	Stage2	Stage3	Stage4	Stage5	Stage6	Stage7	Stage 7-1	Stage8	Stage9	Stage10	Stage11	Stage 11-1	Stage12	Stage13
Node 1	0	0,000045	0,000112	-0,00043	0,00037	-1,4E-05	0,000234	-0,00064	0,00072	-0,00011	0,000217	-0,00055	0,000698	0,000015	0,000131
Node 3	0	0,000045	0,000112	-0,00043	0,00037	-1,4E-05	0,000234	-0,00087	0,000714	-0,00014	0,000212	-0,00054	0,000692	0,000024	0,000123
Node 5	0	0,000045	0,000112	-0,00055	0,000369	-2,3E-05	0,000233	-0,00108	0,000707	-0,00018	0,000205	-0,00053	0,000682	0,000032	0,000114
Node 7	0	0	0	0	0,000369	-0,00012	0,000223	-0,0011	0,000681	-0,00037	0,000176	-0,00047	0,000874	0,000025	0,000054
Node 9	0	0	0	0	0	0	0	0	0,000681	-0,00063	0,00015	-0,00043	0,001072	0,000013	0,00001
Node 11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Node 30	0	0,000117	0,000291	-0,01243	0,000935	0,008516	0,001393	-0,02066	0,003573	0,01336	0,002583	-0,00531	0,004055	-5,3E-05	0,003979
Node 34	0	0,000188	0,00047	-0,03147	0,001486	0,022635	0,003067	-0,05185	0,007569	0,035895	0,006296	-0,01265	0,008956	-0,0002	0,01001

111. Dx Displacements on Summation Load Combination



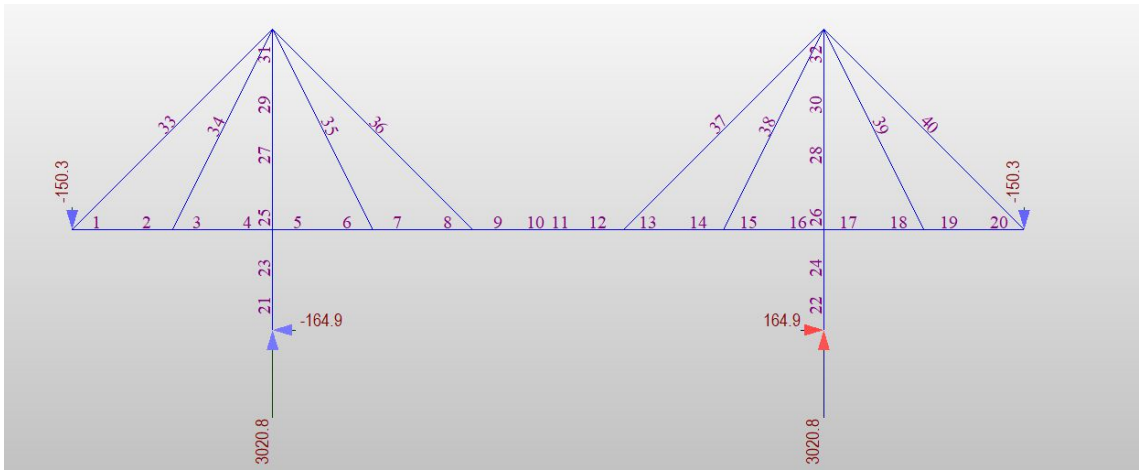
	Stage1	Stage2	Stage3	Stage4	Stage5	Stage6	Stage7	Stage 7-1	Stage8	Stage9	Stage10	Stage11	Stage 11-1	Stage12	Stage13
Node 1	0	0	0	0	0	0	0	0	0	0	0	0	0	0,010125	0
Node 3	0	-0,00022	-6E-06	0,001547	0,000224	0,000361	0,000183	-0,00165	0,000512	0,000527	0,000223	-0,00051	0,000509	0,000127	0,003065
Node 5	-0,00013	-9E-06	-0,00001	-2,4E-05	-1,5E-05	-1E-06	0	-2,2E-05	-1,6E-05	-4E-06	-1E-06	0,000011	-5E-06	0,000058	-0,00046
Node 7	0	0	0	0	-0,01668	0,021599	-0,01294	0,005083	-0,03539	0,054769	-0,00796	0,019327	-0,01135	0,005278	-0,02407
Node 9	0	0	0	0	0	0	0	0	-0,11593	0,184166	-0,0372	0,06849	-0,04344	0,007073	-0,04813
Node 11	0	0	0	0	0	0	0	0	0	0	0	0	-0,07271	0,007411	-0,05482
Node 30	-0,00043	-9E-06	-0,00001	-0,00083	-1,7E-05	-0,00074	-0,00007	-0,00076	-0,00022	-0,00063	-0,00014	0,000295	-0,00018	0,000079	-0,00027
Node 34	-0,0005	-9E-06	-0,00001	-0,00147	-1,8E-05	-0,00132	-0,00013	-0,00136	-0,00039	-0,00114	-0,00025	0,000523	-0,00032	0,000139	-0,00047

112. Dz Displacements on Summation Load Combination

A Current Step Displacement of Dead Load and Summation Load Combinations have been chosen because it is easier to realise the changing direction movement of those representative nodes. It is notable how those nodes oscillate due the construction process by means of the adding of new elements to the structure. The vertical displacement of node 9 is the highest of all nodes even when its placement starts at Stage 8, when a new segment of the bridge's Main Girder is erected. The advanced construction process creates bigger displacements due the bridge is close to be totally built.

The relocation of Derrick Crane's load is also a fact that must be controlled, since it is placed on the extreme of the cantilever, where the deflection is always bigger. There is a correspondence of the structural behaviour on the middle span, with the low deviation of the side span because of the application of loads on the whole structure. This correspondence shows the difficulty on the design if Forward Analysis is not used as the calculation procedure of the transient structure is carried on by starting from the beginning.

The final stage brings a structure hold by the reactions presented on the next figure.



113. REACTIONS ON STAGE 13

Those reactions can be easily explained due the fact that on the piers, a positive displacement of the nodes creates an opposite reaction to keep them in place; on the bottom of the towers, evidently a vertical reaction holding the structure and an horizontal reaction to balance the self-weight in the middle of the bridge are found.

The objective of obtaining these displacement values is to have a guide to follow during the actual construction of the cable-stayed bridge. At each stage of the construction process, the modelling of the bridge will give a reference value of displacement, bending moments or cable tensions to have a pattern with which to compare once the actual structure is erected.

Elements Evolution

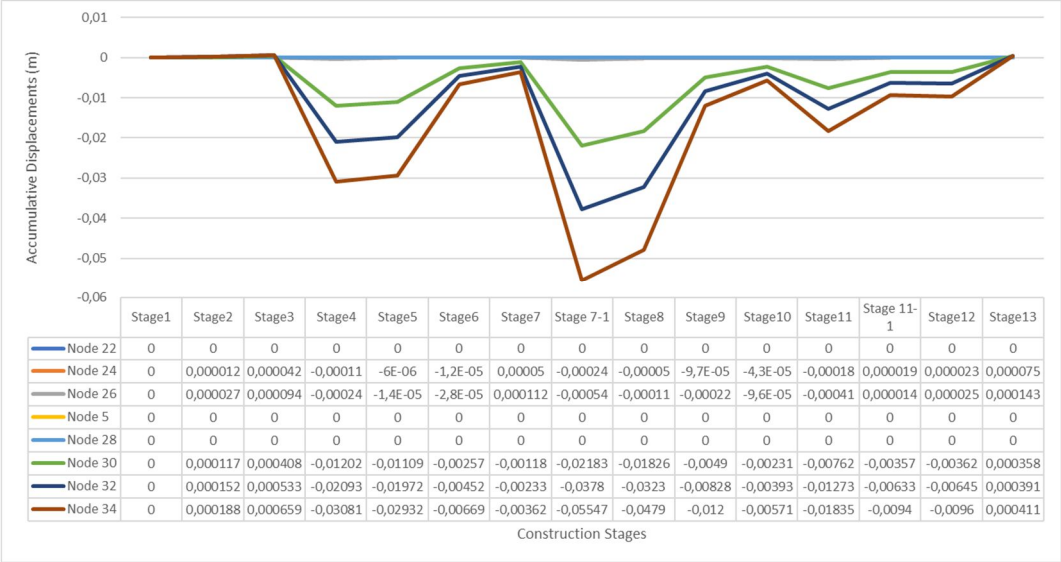
There is not only one analysis of the construction stage diagrams that could be done. Each element experiments an efforts evolution during the construction process that it is interesting to analyse.

Lower and Upper Tower

The pylons, for example, suffers several different efforts, starting with axial efforts due the self-weight of the structure, shear efforts because of the activation of the cables whose transmitted forces are in a non-orthogonal direction, and bending moments because of the same reason. The most affected part of the pylons is the connection between the Lower Tower and the Upper Tower reaching high contraries efforts on Stage 7 due the relocation of the Derrick Crane's punctual load. The maximum bending moment absolute value for both parts of the pylon is found on Stage 7-1, when the third cable is activated. Those Bending Moments' values are My = -5956,0 tonf·m for the Lower Tower, and My = -8044,2 tonf·m in the Upper Tower.

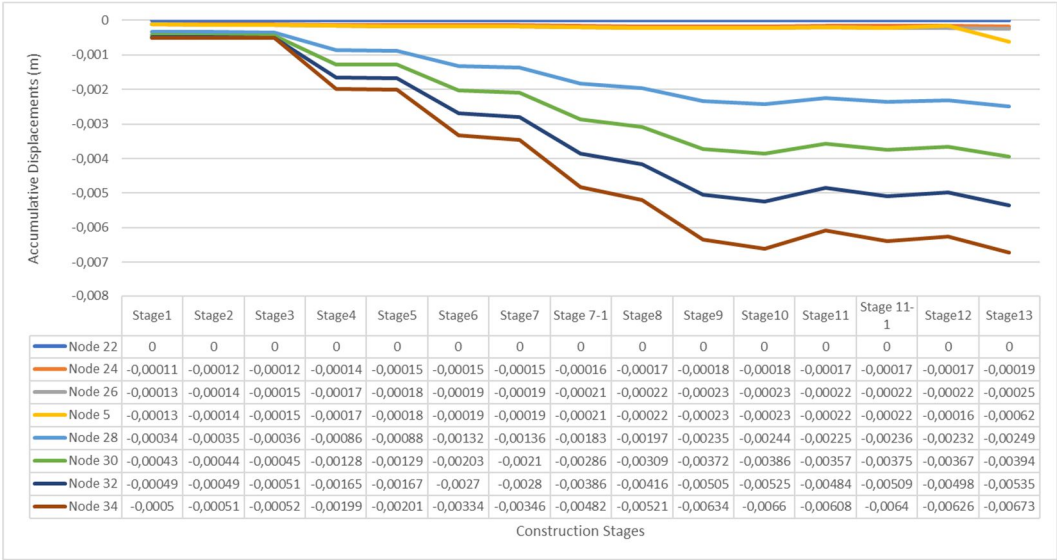
On the other hand, the displacements suffered by the top of the tower (node 34), whose are presented on tables 106 - 109, change in a way that an excessive movement could make the structure collapse.

While the tables 102 – 105 show the displacement of the nodes on its current stage, next table are presented with an accumulative displacement, which is going to illustrate the whole tower movements.



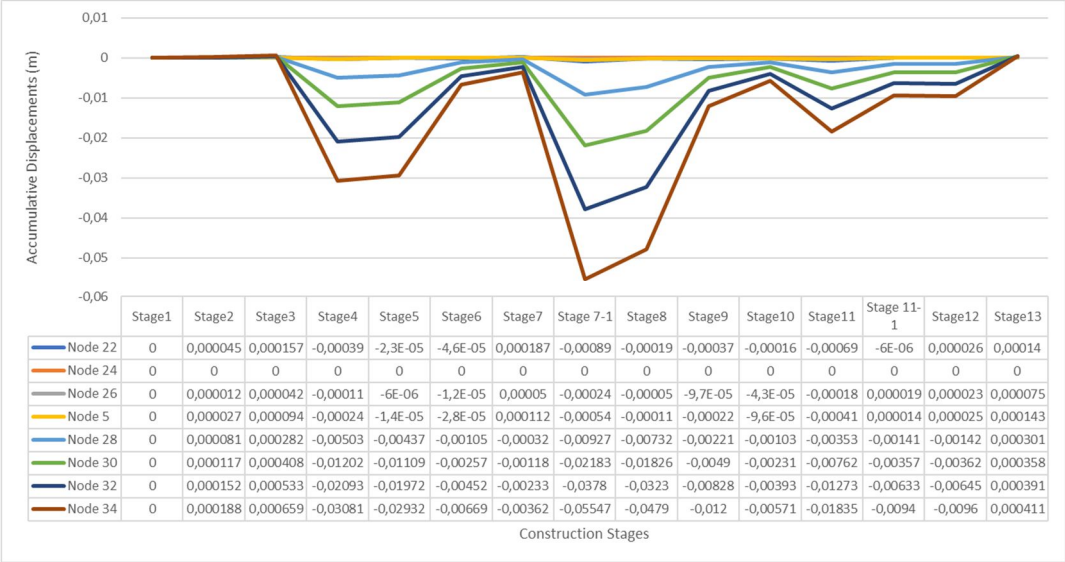
	Stage1	Stage2	Stage3	Stage4	Stage5	Stage6	Stage7	Stage 7-1	Stage8	Stage9	Stage10	Stage11	Stage 11-1	Stage12	Stage13
Node 22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Node 24	0	0,000012	0,000042	-0,00011	-6E-06	-1,2E-05	0,00005	-0,00024	-0,00005	-9,7E-05	-4,3E-05	-0,00018	0,000019	0,000023	0,000075
Node 26	0	0,000027	0,000094	-0,00024	-1,4E-05	-2,8E-05	0,000112	-0,00054	-0,00011	-0,00022	-9,6E-05	-0,00041	0,000014	0,000025	0,000143
Node 5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Node 28	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Node 30	0	0,000117	0,000408	-0,01202	-0,01109	-0,00257	-0,00118	-0,02183	-0,01826	-0,0049	-0,00231	-0,00762	-0,00357	-0,00362	0,000358
Node 32	0	0,000152	0,000533	-0,02093	-0,01972	-0,00452	-0,00233	-0,0378	-0,0323	-0,00828	-0,00393	-0,01273	-0,00633	-0,00645	0,000391
Node 34	0	0,000188	0,000659	-0,03081	-0,02932	-0,00669	-0,00362	-0,05547	-0,0479	-0,012	-0,00571	-0,01835	-0,0094	-0,0096	0,000411

114. DX DISPLACEMENTS ON DEAD LOAD COMBINATION



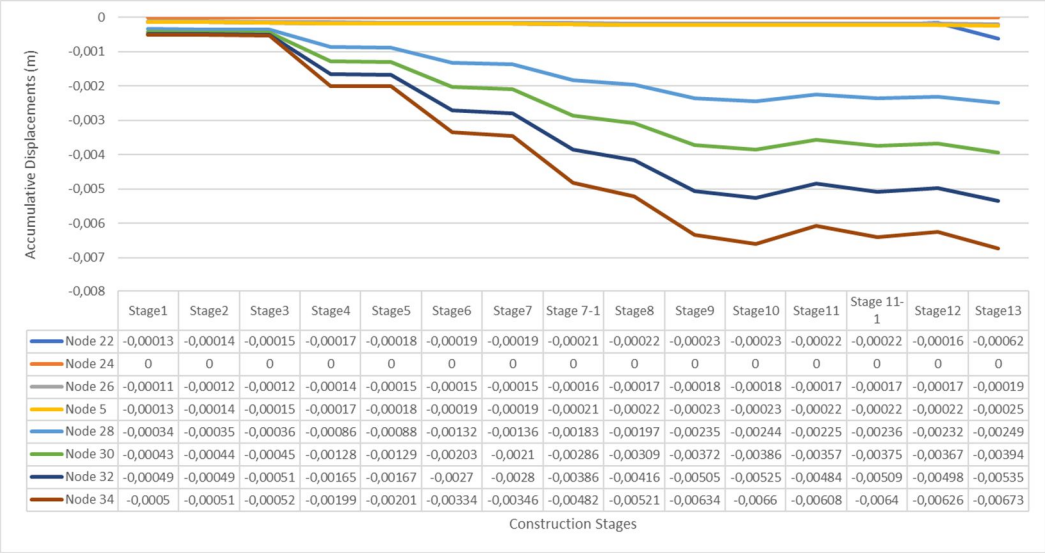
	Stage1	Stage2	Stage3	Stage4	Stage5	Stage6	Stage7	Stage 7-1	Stage8	Stage9	Stage10	Stage11	Stage 11-1	Stage12	Stage13
Node 22	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Node 24	-0,00011	-0,00012	-0,00012	-0,00014	-0,00015	-0,00015	-0,00015	-0,00016	-0,00017	-0,00018	-0,00018	-0,00017	-0,00017	-0,00017	-0,00019
Node 26	-0,00013	-0,00014	-0,00015	-0,00017	-0,00018	-0,00019	-0,00019	-0,00021	-0,00022	-0,00023	-0,00023	-0,00022	-0,00022	-0,00022	-0,00025
Node 5	-0,00013	-0,00014	-0,00015	-0,00017	-0,00018	-0,00019	-0,00019	-0,00021	-0,00022	-0,00023	-0,00023	-0,00022	-0,00022	-0,00016	-0,00062
Node 28	-0,00034	-0,00035	-0,00036	-0,00086	-0,00088	-0,00132	-0,00136	-0,00183	-0,00197	-0,00235	-0,00244	-0,00225	-0,00236	-0,00232	-0,00249
Node 30	-0,00043	-0,00044	-0,00045	-0,00128	-0,00129	-0,00203	-0,0021	-0,00286	-0,00309	-0,00372	-0,00386	-0,00357	-0,00375	-0,00367	-0,00394
Node 32	-0,00049	-0,00049	-0,00051	-0,00165	-0,00167	-0,0027	-0,0028	-0,00386	-0,00416	-0,00505	-0,00525	-0,00484	-0,00509	-0,00498	-0,00535
Node 34	-0,0005	-0,00051	-0,00052	-0,00199	-0,00201	-0,00334	-0,00346	-0,00482	-0,00521	-0,00634	-0,0066	-0,00608	-0,0064	-0,00626	-0,00673

115. DZ DISPLACEMENTS ON DEAD LOAD COMBINATION



	Stage1	Stage2	Stage3	Stage4	Stage5	Stage6	Stage7	Stage 7-1	Stage8	Stage9	Stage10	Stage11	Stage 11-1	Stage12	Stage13
Node 22	0	0,000045	0,000157	-0,00039	-2,3E-05	-4,6E-05	0,000187	-0,00089	-0,00019	-0,00037	-0,00016	-0,00069	-6E-06	0,000026	0,00014
Node 24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Node 26	0	0,000012	0,000042	-0,00011	-6E-06	-1,2E-05	0,00005	-0,00024	-0,00005	-9,7E-05	-4,3E-05	-0,00018	0,000019	0,000023	0,000075
Node 5	0	0,000027	0,000094	-0,00024	-1,4E-05	-2,8E-05	0,000112	-0,00054	-0,00011	-0,00022	-9,6E-05	-0,00041	0,000014	0,000025	0,000143
Node 28	0	0,000081	0,000282	-0,00503	-0,00437	-0,00105	-0,00032	-0,00927	-0,00732	-0,00221	-0,00103	-0,00353	-0,00141	-0,00142	0,000301
Node 30	0	0,000117	0,000408	-0,01202	-0,01109	-0,00257	-0,00118	-0,02183	-0,01826	-0,0049	-0,00231	-0,00762	-0,00357	-0,00362	0,000358
Node 32	0	0,000152	0,000533	-0,02093	-0,01972	-0,00452	-0,00233	-0,0378	-0,0323	-0,00828	-0,00393	-0,01273	-0,00633	-0,00645	0,000391
Node 34	0	0,000188	0,000659	-0,03081	-0,02932	-0,00669	-0,00362	-0,05547	-0,0479	-0,012	-0,00571	-0,01835	-0,0094	-0,0096	0,000411

116. DX DISPLACEMENTS ON SUMMATION LOAD COMBINATION



	Stage1	Stage2	Stage3	Stage4	Stage5	Stage6	Stage7	Stage 7-1	Stage8	Stage9	Stage10	Stage11	Stage 11-1	Stage12	Stage13
Node 22	-0,00013	-0,00014	-0,00015	-0,00017	-0,00018	-0,00019	-0,00019	-0,00021	-0,00022	-0,00023	-0,00023	-0,00022	-0,00022	-0,00016	-0,00062
Node 24	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Node 26	-0,00011	-0,00012	-0,00012	-0,00014	-0,00015	-0,00015	-0,00015	-0,00016	-0,00017	-0,00018	-0,00018	-0,00017	-0,00017	-0,00017	-0,00019
Node 5	-0,00013	-0,00014	-0,00015	-0,00017	-0,00018	-0,00019	-0,00019	-0,00021	-0,00022	-0,00023	-0,00023	-0,00022	-0,00022	-0,00022	-0,00025
Node 28	-0,00034	-0,00035	-0,00036	-0,00086	-0,00088	-0,00132	-0,00136	-0,00183	-0,00197	-0,00235	-0,00244	-0,00225	-0,00236	-0,00232	-0,00249
Node 30	-0,00043	-0,00044	-0,00045	-0,00128	-0,00129	-0,00203	-0,0021	-0,00286	-0,00309	-0,00372	-0,00386	-0,00357	-0,00375	-0,00367	-0,00394
Node 32	-0,00049	-0,00049	-0,00051	-0,00165	-0,00167	-0,0027	-0,0028	-0,00386	-0,00416	-0,00505	-0,00525	-0,00484	-0,00509	-0,00498	-0,00535
Node 34	-0,0005	-0,00051	-0,00052	-0,00199	-0,00201	-0,00334	-0,00346	-0,00482	-0,00521	-0,00634	-0,0066	-0,00608	-0,0064	-0,00626	-0,00673

117. DZ DISPLACEMENTS ON SUMMATION LOAD COMBINATION

As it can be seen, accumulative displacements show how the top of the tower experiments a considerable displacement. While, for both Load Combinations, the displacements on axle X of all nodes return at the end to its original position (or very close to it), on axle Z, the displacement of the node 34 (top of the pylon) becomes z = -6.73 mm. It can seem a very low displacement, but it is important to remember that this displacement is due the compression of the tower.

Cables

The use of the Lack of Fit Force function develops very important when the cables are placed and when the key segment is erected. As it can be seen on the next tables, the presence of Lack of Fit Force is only given on two types of stage: cables activation and key segment construction.

What is going to be analysed in this chapter is the influence of Lack of Fit Force function on the cables.

Elem	Node I	Node J	Pretension (tonf)	LOF Force (tonf)	SUM (tonf)	Local Vector		
						V-X	V-Y	V-Z
33	34	1	333,808	-7,135	326,673	-0,707	0	-0,707
34	34	3	254,37	0,12	254,49	-0,447	0	-0,894
35	34	7	193,011	51,664	244,675	0,447	0	-0,894
36	34	9	340,835	179,931	520,766	0,707	0	-0,707
37	35	13	340,835	179,931	520,766	-0,707	0	-0,707
38	35	15	193,011	51,664	244,675	-0,447	0	-0,894
39	35	19	254,37	0,12	254,49	0,447	0	-0,894
40	35	21	333,808	-7,135	326,673	0,707	0	-0,707

118. LACK OF FIT FORCE VALUES ON TRUSSES – 1

Elem	Node I	Node J	Angle ([deg])	I-Node Disp.			J-Node Disp.			Deform (m)
				DX (m)	DY (m)	DZ (m)	DX (m)	DY (m)	DZ (m)	
33	34	1	45	-0,004	0	-0,003	0	0	0	-0,005
34	34	3	63,435	0,001	0	-0,001	0	0	0	0
35	34	7	63,435	-0,029	0	-0,002	0	0	-0,02	0,029
36	34	9	45	-0,048	0	-0,005	0	0	-0,141	0,13
37	35	13	45	0,048	0	-0,005	0	0	-0,141	0,13
38	35	15	63,435	0,029	0	-0,002	0	0	-0,02	0,029
39	35	19	63,435	-0,001	0	-0,001	0	0	0	0
40	35	21	45	0,004	0	-0,003	0	0	0	-0,005

119. LACK OF FIT FORCE VALUES ON TRUSSES – 2

It can be observed that the pretension loads given to the cables is the one it was designed during the tutorial. However, Lack of Fit Force on the elements 36 and 37, which are the cables going

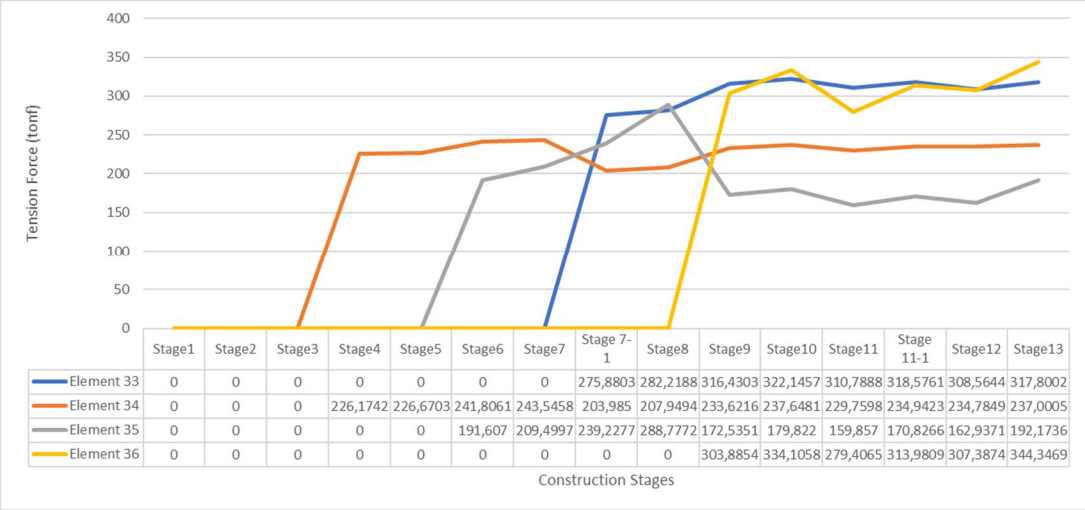
to the closest point of the centre of the span, are the ones whose experiment the highest deformation due the applications of 179,931 tonf from the Lack of Fit Force. With a final tension of 520,766 tonf, the cables are designed to support the major part of the Main Girder by means of those imposed tensions. Even if the two cables on the exterior side of the pylons had a higher designed pretension (the sum of them), the interior side of the tower has to affront bigger efforts, so that's why the balance of the structure becomes like this.

Due the pretension given to the cables, those elements are subjected to a variation of length between the cable's length measured in horizontal without forces applied on them, and the length they have when they are placed. That variation creates the tension and a deformation on them whose must be known to be hold.

The evolution of the tension of the cables during the construction process is presented in the next tables [112 and 113]. The first one is the tension calculated on the cable's origin point, the second table, presents the tension calculated on the cable's end point.

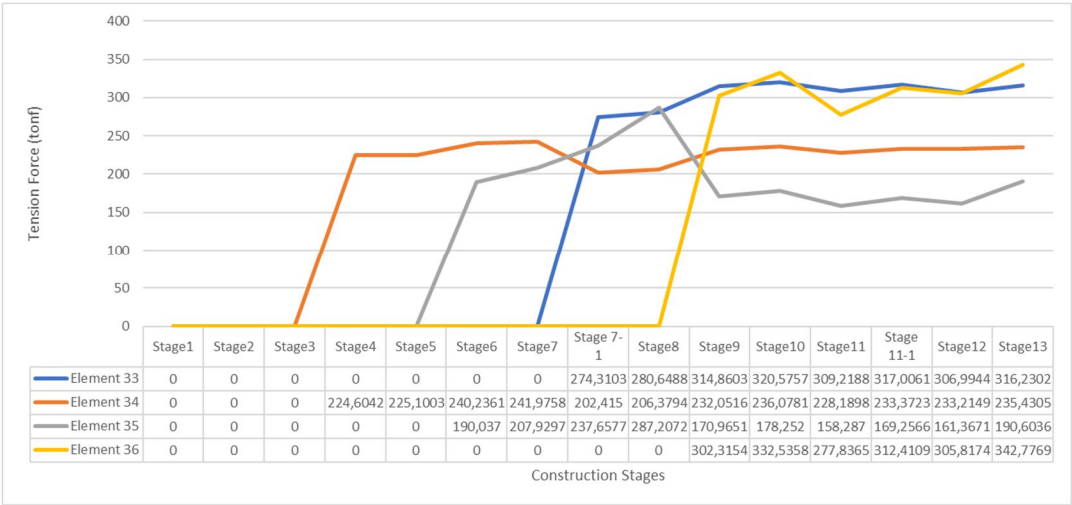
As it can be observed, the first cable is activated at Stage 4, where the cable takes tensions values as F_i = 226,1742 tonf and F_j = 224,6042 tonf. Those tensions still going up until the element 33 is placed and the tensions are splat. It happens the same when element 36 is placed. The element 35 gets its tension reduced because of the repartition of the axial efforts on the cables of the same side of the tower.

At the end, on Stage 13, the cables placed on the interior side of the tower have the maximum and the minimum tension values, but the sum of both tensions are higher than the sum of the tensions of the cables from the exterior side of the tower.



	Stage1	Stage2	Stage3	Stage4	Stage5	Stage6	Stage7	Stage 7-1	Stage8	Stage9	Stage10	Stage11	Stage 11-1	Stage12	Stage13
Element 33	0	0	0	0	0	0	0	275,8803	282,2188	316,4303	322,1457	310,7888	318,5761	308,5644	317,8002
Element 34	0	0	0	226,1742	226,6703	241,8061	243,5458	203,985	207,9494	233,6216	237,6481	229,7598	234,9423	234,7849	237,0005
Element 35	0	0	0	0	0	191,607	209,4997	239,2277	288,7772	172,5351	179,822	159,857	170,8266	162,9371	192,1736
Element 36	0	0	0	0	0	0	0	0	0	303,8854	334,1058	279,4065	313,9809	307,3874	344,3469

120. TRUSS FORCES ON CABLE'S ORIGIN POINT DURING CONSTRUCTION PROCESS



	Stage1	Stage2	Stage3	Stage4	Stage5	Stage6	Stage7	Stage 7-1	Stage8	Stage9	Stage10	Stage11	Stage 11-1	Stage12	Stage13
Element 33	0	0	0	0	0	0	0	274,3103	280,6488	314,8603	320,5757	309,2188	317,0061	306,9944	316,2302
Element 34	0	0	0	224,6042	225,1003	240,2361	241,9758	202,415	206,3794	232,0516	236,0781	228,1898	233,3723	233,2149	235,4305
Element 35	0	0	0	0	0	190,037	207,9297	237,6577	287,2072	170,9651	178,252	158,287	169,2566	161,3671	190,6036
Element 36	0	0	0	0	0	0	0	0	0	302,3154	332,5358	277,8365	312,4109	305,8174	342,7769

121. TRUSS FORCES ON CABLE'S END POINT DURING CONSTRUCTION PROCESS

Main Girder

In the case of the beams, next table illustrates its properties.

		LOF Force							Displacement						
Elem	Node	Axial (tonf)	Shear-y (tonf)	Shear-z (tonf)	Torsion (tonf*m)	Moment-y (tonf*m)	Moment-z (tonf*m)	Bi-Moment (tonf*m^2)	DX (m)	DY (m)	DZ (m)	RX ([rad])	RY ([rad])	RZ ([rad])	RW (rad/m)
1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	10	4173,795	0	-0,001	0	-11065,69	0	0	-0,001	0	0,101	0	-0,003	0	0
10	11	4173,795	0	-0,001	0	-11065,68	0	0	0	0	0,107	0	0	0	0
11	11	4173,795	0	0,001	0	-11065,68	0	0	0	0	0,107	0	0	0	0
11	12	4173,795	0	0,001	0	-11065,69	0	0	0,001	0	0,101	0	0,003	0	0
16	16	0	0	0	0	0	0	0	0	0	0	0	0	0	0
16	17	0	0	0	0	0	0	0	0	0	0	0	0	0	0
17	17	0	0	0	0	0	0	0	0	0	0	0	0	0	0
17	18	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18	18	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18	19	0	0	0	0	0	0	0	0	0	0	0	0	0	0
19	19	0	0	0	0	0	0	0	0	0	0	0	0	0	0
19	20	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20	20	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20	21	0	0	0	0	0	0	0	0	0	0	0	0	0	0

122. LACK OF FIT FORCE ON BEAMS

As it can be observed, the Lack of Fit Force only appears on the Key Segment once it is erected and all the forces are applied on the structure (Stage 13). That force is created due the tensions the cables give to the structure, pulling the contiguous elements to the pylons and generating traction efforts on elements 10 and 11. It appears again the Lack of Fit phenomena, creating differences between the length of the element placed and unplaced.

The evolution of the efforts on the Main Girder are basically linked to the construction process, and they are already analysed previously.

Chapter 6 – Conclusions

As it has been seen during this thesis, the importance of the successful construction of a cable-stayed bridge it is not only a matter of design, but also the way it is going to be built.

The necessity of patterns to control the changing static scheme of those structures becomes imperative in order to avoid excessive deflections that could bring the bridge to collapse.

With MIDAS/Civil Software, every stress, every deflection can be checked and analysed from every construction stage which expose a pattern to be followed during the real construction process of a cable-stayed bridge. Those programs become more and more important for the design of new structures because of the necessity of easier, faster and cheaper construction, and those factors can be decisive when the choosing of the construction method arrives.

The history of cable-stayed bridge shows a continuous evolution of the construction methods, calculation algorithms and design approaches. New technologies will bring new methodologies to revolute the way we construct, but meanwhile, it is important to know how does the actual methods work.

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