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DESIGN OF NETWORK ARCH BRIDGES

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In memory of my father

Inspiration exists, but it has to find you working.

Pablo Picasso

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ABSTRACT

In this thesis, several hanger arrangements for tied arch bridges are developed and presented. Starting from a basis configuration with vertical hangers, different settings, namely network and Nielsen type, are explored in accordance with different variables, such as number of hangers, cross angle of the hangers, hanger node position, etc. Moreover, the force evolution for the ultimate limit states is analyzed and the weight obtained for each structure is compared.

Finally, a set of recommendations are prescribed in order to render the design and optimization process more efficient and swift.

KEYWORDS: bridges, network arch, road bridges, hanger arrangement, composite structures.

RESUMO

Nesta tese são apresentadas diversas configurações de pendurais para pontes em arco com tabuleiro suspenso. Partindo de uma configuração base com pendurais verticais outras configurações, do tipo network ou nielsen são exploradas, segundo diversas variáveis -número de pendurais, ângulo dos pendurais, posição dos nós dos pendurais, entre outros- analisando a evolução dos esforços nos estados limites últimos e comparando o peso obtido para cada estrutura.

Por último, um conjunto de recomendações são prescritas de modo a tornar o processo de concepção e optimização de pontes do tipo network mais eficiente e expedito.

PALAVRAS-CHAVE: pontes, arco network, pontes rodoviárias, arranjo de pendurais, estruturas mistas.

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INTRODUCTION

1.1. PREAMBLE

The arch is the strongest embodiment of a bridge. Its shape expresses directly its ability to carry loads across a river, a valley or a gorge [1]. After two centuries of being the most widely used, standard solution for middle and long spans nowadays, arch bridges are considered an expensive solution to be only applied when aesthetics and integration into the Environment are keys issues [2].

However, a convincing case can be made that this solution is indeed competitive in middle spans, which is what this study sets out to do.

1.2. SCOPE AND OBJECTIVES

The purpose of this work is to investigate the design performance of a tied arch bridge, focusing on the hanger arrangement. The main objectives are to develop a state of the art review on tied arch bridges, with a focus on the network type and to examine, understand and compare how the hanger arrangement can lead to a structurally more efficient bridge, using the actions prescribed in Eurocode 1 on a road network arch bridge with convergent arches.

1.3. THESIS OUTLINE

This thesis consists of six chapters and it can be divided into two parts. The first part, which comprises Chapters 2 and 3, provides a general literature review on the state of the art of tied arch bridges and network arch bridges. The second part, which consists of Chapter 4 and 5 deals with modelling and design aspects related to arch bridges. In Chapter 4 a preliminary design of an arch bridge is provided along with a description of the numerical modelling of this type of structure. This is then followed by a study on the influence of hanger arrangement, which is presented in Chapter 5. In the last chapter, conclusions are drawn and an outlook on future developments is presented.

2
TIED ARCH BRIDGES
STATE OF THE ART

2.1. CONCEPT OF TIED ARCH BRIDGE

A tied arch bridge is an arch bridge in which the compression of the arch is balanced with a tensile action developing in the bottom chord, a tie or deck, resulting in no horizontal forces at the abutments. The downward forces applied to the deck of tied arch bridges are transmitted by the hangers towards the curved top chord. This results in a flattened bridge where the tips are moved outward into the abutments, similar to other arch bridges. The main difference to the other types of arch bridges is that, here, the downward thrusts are restrained by the bottom chord and not by lateral reactions of the abutments [3].

Due to the similarity of this mechanism with the design of the string of a bow, which also results in the strings being flattened, tied arch bridges are often referred to as "bowstring arch" bridges. By eliminating the horizontal forces at the abutments, these bridges have the advantage of requiring less robust foundations, which is beneficial in terms of location - it is thus possible to build tied-arch bridges on top of elevated piers as well as in areas with unstable soil. Another advantage concerns the possibility of prefabricating tied arch bridges off the construction side. Since their integrity does not depend on horizontal compression forces, they can be build elsewhere and then floated, hauled or lifted into place.

The definition of tied arch bridge presented above is the most common one. Nonetheless, some authors define a tied arch bridge as a bridge with a compressed arch and a suspended deck. Figure 2.1, shows other types of bridges often referred to in literature as suspended deck arch bridges.

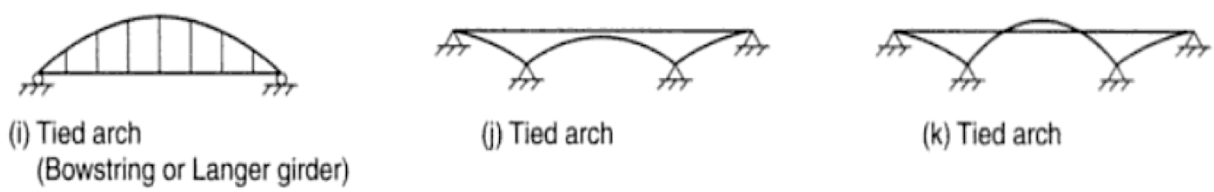

Fig. 2.1 – Tied arch bridge – alternative definition [4]

For the purpose of this thesis, tied arch bridges refer to the most accepted definition, in which the deck behaves as a tie. However, and in order to better perceive the different configurations that can be adopted for an arch bridge with a suspended deck, the broad definition will be applied in the examples listed under section 2.5.

2.2 Classification

Taking into account the existing arch bridges, a classification can be defined based on the shape, number and relative position of the elements that compose a tied arch bridge.

2.2.1. Number of arches

A tied arch bridge can have a single, double or even multiple arches. Single arches are often used in pedestrian bridges, while multiple arches are usually adopted in highway bridges. The double arch is a standard for regular road and rail bridges.

2.2.2. Shape of the arch

The most common functions adopted to define an arch are the parabola, the catenary and part of a circle. The catenary follows accurately the anti-funicular of the self-weight of the arch, while the parabolic function is the anti-funicular for uniform permanent loads. This means that for arches where the self-weight is dominant, such as concrete arches, the anti-funicular that best suits is the catenary, while for bridges in which the self-weight is less conditioning than the permanent loads, as in steel arches, a parabolic function is adopted. Part of a circle is the anti-funicular for radial loads, a function that is often used in steel arch bridges with inclined hangers. A polygonal function can also be adopted when concentrated loads are dominant compared to the self-weight of the arch.

2.2.3. Relative position of the arches

When employing two arches, these can be defined as parallel, convergent or divergent. Parallel arches are usually referred to as “classic” whereas any other configuration out of the plane is referred to as “spatial” arch bridges. Convergent arches have the advantage of requiring less wind bracing, though leading to a wider deck in order to prevent that higher vehicles might clash with the arch or the hangers. A single arch can be also defined as either centred or inclined eccentric.

2.2.4. Hanger arrangement

The simplest hanger (or cable) arrangement is the one in which all the hangers are vertical. In those cases where the hangers are positioned in an inclined manner, they can be classified as:

- “Nielsen”, where the hangers only intersect once;
- “Wheel”, where hangers are placed in the radial direction;
- “Network” where the hangers intersect at least twice [4].

The latter are at the focus of this thesis.

2.2.5. Other characteristics

The characteristics listed above are the most commonly used to classify tied arch bridges. Nonetheless, there are several other characteristics that should be taken into consideration when defining spatial arch bridges, such as:

- The position of the deck, which can be straight or curved;
- The relative position between the deck and the arch(es), which can be centred or eccentric;
- The horizontal projection of the arch, which can be straight or curved, etc.

2.3 Bridge Aesthetics

Over the years, there have been a number of attempts to set out criteria against which the success or failure of bridge design in aesthetics terms can be judged. However, due to the subjective nature- probably aesthetics is the single most subjective field when it comes to engineering- such qualifications remain controversial. Some distinguished engineers refuse to define rules, such as Eduardo Torroya. Nevertheless, in his texts he readily refers to concepts of harmony, proportion, rhythm and function, among others [6]. On the other hand, Fritz Leonhardt offers in his widely acclaimed “Brücken” a ten-point framework for the evaluation of bridge aesthetics, namely: fulfilment of purpose/function, proportion, order, refinement of form, integration into the environment, surface texture, character, complexity and incorporating nature [1].

Rules alone will not guarantee the achievement of a beautiful design, since intuition and imagination cannot be apprehended as if they were a formula. Nevertheless, these rules provide a good starting point that can help defining a critical appraise, thereby making the designer aware of aesthetic design errors.

Tied arch bridges demand a flat ground. According to Leonhardt, the best appearance is obtained when the arches are designed to carry all the loads, the deck thus being as shallow as possible in order to emphasize the character of the deck suspension, as shown in Figure 2.2.

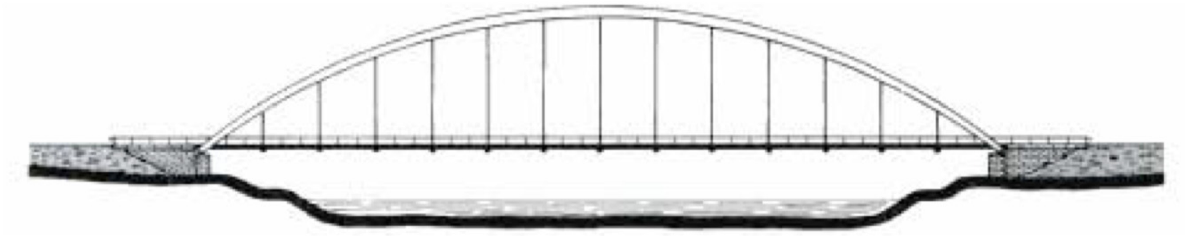

Fig. 2.2 – Stiffer arch and thin suspended deck [1]

Arch bridges allow distributing the bending moments between the deck and the arch, which can lead to stiffer beams and slender arches, or to more balanced beams and arches. However, from an aesthetic viewpoint, this kind of arch bridges tend to look bulkier, as shown in Figures 2.3 and 2.4.

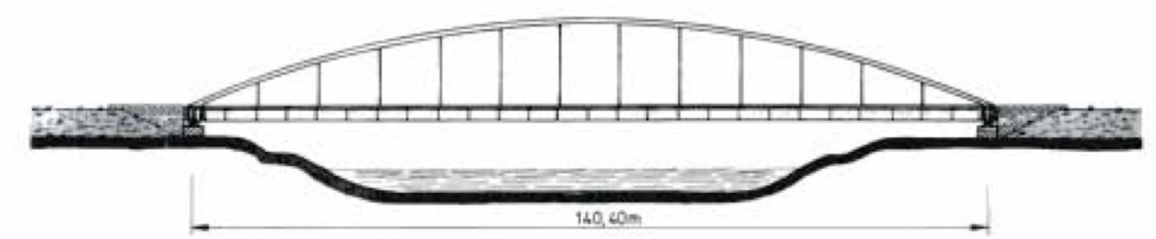

Fig. 2.3 – Thin arch and stiffer deck [1]

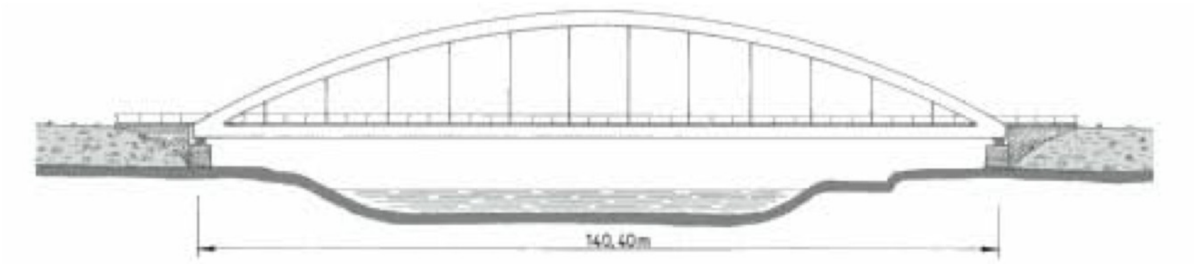

Fig. 2.4 – Balanced stiff in the arch and the deck [1]

The simplest way to get both the arch and the deck slender is to define inclined hanger arrangements, such as the Nielsen or the network approach (Figure 2.5). Such configuration is one of the main foci of this thesis.

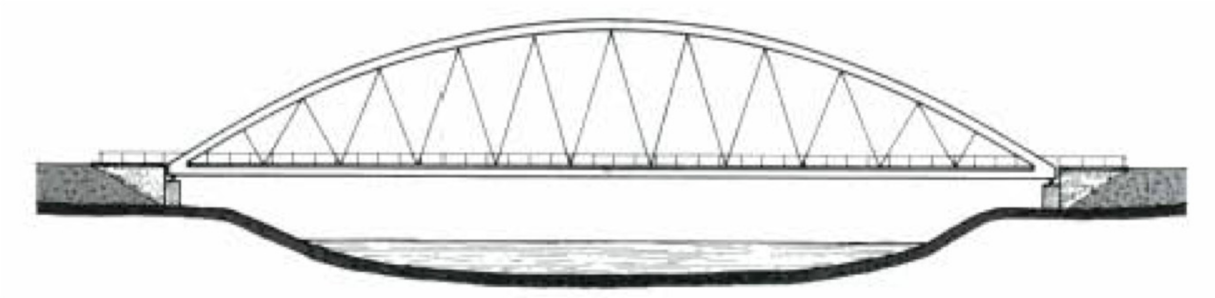

Fig. 2.5 – Slender arch and deck, due to use of inclined hangers [1]

2.4 The Art of Bridge Design

The most common definition when it comes to bridge design is that a bridge should be economic, efficient and elegant. Understandably, this has been a point of reflection for many bridge designers and is widely discussed in the literature.

According to Menn, "*A truly well designed bridge balances economy and aesthetics while responding to the functional requirements and technical and environmental boundary conditions."* [7].

Menn considers the "functional requirements" to comprise the traffic, alignment and state-of-the art construction technologies, while the "technical and environmental boundary conditions" include topography, geology, clearances, available programme, social and environmental impact, etc.

But Menn develops: "*On the basis of the above considerations, the real art of bridge design is to elaborate a suitable technically appropriate structural system that aims at achieving an optimal balance of economy and appearance. (…) This pragmatic, simple and purely functional approach not only leads to technically proper structures but also to aesthetically convincing ones.*" [7].

Leonhardt adds an ethical mandate to aesthetics refusing the "*tendency toward the spectacular, the sensational and the gigantic*" [1] and Menn states on a similar note that any significant increase in cost "*should be abandoned*".

These definitions and prescriptions should be read bearing in mind the time when they were produced. At that time, the identification of an optimal form allowed materials to be minimized and the least cost arose from the least materials; ultimately, the technically most efficient design would often coincide with the most slender and elegant solution. However, the conditions of production have changed, and nowadays the lowest-cost solution means in most of the cases minimizing site labour while maximizing the use of off-site fabrication and assembly. Also, the cost of a structure cannot only be evaluated by the initial cost but it rather has to include the maintenance during the whole life of the structure.

An important issue that engineers are usually reluctant to take into account is the public opinion. It is far from clear whether a good design is shared by the public that benefit from a bridge and even from those who have funded it. Widely respected architects like Santiago Calatrava are often praised by the public due to their aesthetic lavish- even where his avant-garde designs leave the field of structural efficiency and move towards the sculpturing side of designing an –albeit artistic- object. Public opinion has proven to be indifferent to the structural efficiency of a bridge but it does care if whether it is ugly or more expensive than another alternative. Virlogeux criticises this position, stating that "*Economy has been too much the unique goal of narrow-minded engineers, resulting in some poor, ugly and repetitive structures which discredited the profession. (...) Engineers used to live in their narrow professional world, sure of legitimacy based on rationality and competence. They have not been able to feel the evolution of our Society and the growing power of politicians and media, and of the lobbies which are able to influence them."* [8].

The dilemmas engineers face at the crossroads of efficiency, costs, aesthetics, etc. cannot be solved by ignoring the legitimate wishes of the public, who ultimately finances the projects. The challenge, at times frustrating, of using engineering expertise in a way that different social stakeholders might relate to needs to be realized and incorporated into designs. In order to find balanced solutions, engineers have to carefully weight different factors and alternatives: what is appropriate in a given context, how can a design challenge notions of a space without disrespecting its character?

2.5 EXAMPLES OF TIED ARCH BRIDGES

2.5.1. SVINESUND BRIDGE

Designed by Bilfinger Berger and Meyer & Schubart it has an overall length of 704 m. The main span between the abutments is 247 m and it is supported by a single vertical reinforced concrete arch that suspends two decks that consist of two steel orthotropic boxes, on each side of the arch. Transverse beams join the two bridge decks, each 25.5 m being these connected by vertical hangers to the arch.

Fig. 2.6 – Svinesund Bridge – Norway/Sweden border [9]

2.5.2. HOGE BRUG FOOTBRIDGE

Designed by René Greisch and also known as the Passarelle Céramique it has a total length of 261 m. Entirely made of steel, the main span is 164 m long and is supported by a single vertical arch with variable geometry. The deck is a 7.20 m wide box girder shaped as a sector of circle and the 14 full locked inclined cables, which cross each other only once, possessing a 50 mm in diameter each.

Fig. 2.7 – Hoge Brug footbridge – Maastricht [10]

2.4.3. YORK MILLENNIUM BRIDGE

Designed by Whitby Bird & Partners Engineers it is a good example of an eccentric single arch. It has an overall length of 150 m, with the main span being 4 m wide and 80 m long. The deck is a trapezoidal steel box girder and is suspended using 19mm diameter cables from the arch, which is inclined at 50 degrees from the horizontal. Both the deck and the arch are in stainless steel. The cables are inclined, being each one perpendicular to the circular arch towards the centre of the circle that the arch would form.

Fig. 2.8 – York Millennium footbridge – York [11]

2.4.4. JUSCELINO KUBITSCHEK BRIDGE

Designed by the architect Alexandre Chan and the engineer Mario Vila Verde it has a total length of 1200m. The main span supported by the three arches has 720 m with 240 m each, which are in reinforced concrete since the abutments until the deck level being the subsequent part in steel. The composite deck has a width of 24 m, with three lanes in each direction and lateral sideways with 1.5 m on each side. The arches are vertical and follow a parabolic shape, while cables are disposed in an apparent asymmetrical way in the deck. The main feature of this bridge is that the asymmetrical arches crisscross the deck diagonally, without touching it.

Fig. 2.9 – Juscelino Kubitschek bridge – Brasilia [12]

2.4.5. Straubing bridge

It has a total length of 615 m. The main span of 200 m is supported by a circular steel arch, with a cross-section of 1.25x0.80 m. The rise of the arch is 31.2 m and the deck is defined with two longitudinal beams, with a width of 15 m, being the overall height 1.7 m. The deck being stiffer than the arch is however slender, which leads to an agreeable configuration of the bridge.

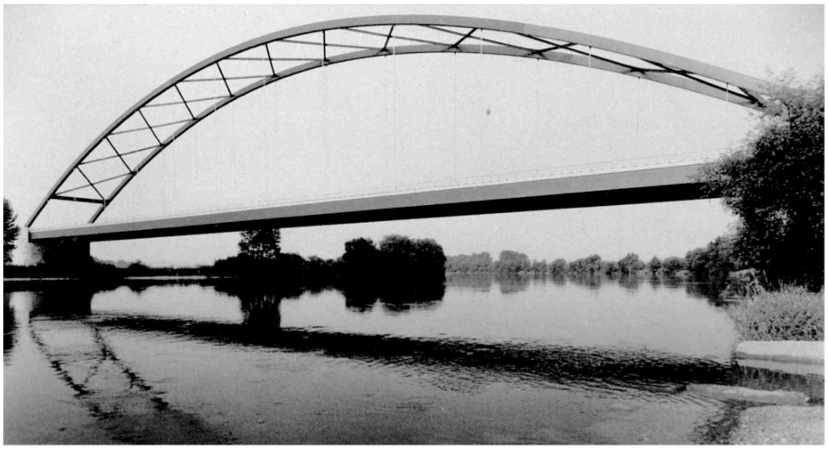

Fig. 2.10 – Straubing Bridge – Bavaria [13]

2.4.6. Bayonne bridge

Designed by Othmar Ammann, it is a steel trussed arch bridge, which was the standard for arch bridge design in the beginning of the 20th century. It has a total length of 1760 m, being the main span 510 m. The trussed arch cuts a clear parabolic shape with a pleasing pattern of regular triangles through the articulation of the arch into 40 segments. The deck has a width of 26 m and is suspended from vertical steel cables at approximately 12.5 m centres.

Fig. 2.11 – Bayonne Bridge – New Jersey [14]

2.4.7. CASTELMORON BRIDGE

This bridge was designed by Aage Nielsen, the first engineer to patent arch bridges with inclined hangers. It was built in 1933 and consists of two parallel reinforced concrete arches of 143 m span, which support the concrete slab deck of 10 m width with inclined hangers.

Fig. 2.12 – Castelmoron Bridge - Castelmoron-sur-Lot [15]

2.4.8. TRI-COUNTRIES BRIDGE

Designed by Feichtinger Architects and Leonhardt, Andrä und Partner it has a total length of 248 m. The main span is 229 m and the piers were avoided to reduce the risk of a ship impact. One of the main characteristics of this footbridge consists in the asymmetrical cross-section of the arches. The northern arch is made of two hexagonal steel boxes while the southern arch is an hollow steel circular inclined of 18º, leaning the arches towards each other. The bridge deck is an orthotropic slab with a width of 5 m and the hangers intersect only once. Besides comprising the largest span for a footbridge it is also one of the most slender bridges ever built.

Fig. 2.13 – Tri-Countries Bridge - Weil am Rhein – France/Germany [16]

2.4.9. James Joyce bridge

Designed by Santiago Calatrava has a span of 40 m and it is an example of an arch bridge with divergent arches. The rise of the arch is 6 m and due to the curved directrix of the road the arch itself should be considered as a spatial arch. The bridge deck consists of box sections of constant depth but varying width, which is greatest at mid span. Transverse girders span between, with varying width from 13 m to 18 m, and cantilever outside of the longitudinal girders to give the pedestrian walkway, which have a varying width between 3 m and 6 m. The cables that suspend the bridge deck have 40 mm and are arranged in pairs at 2.353 m centres, on each side of the deck.

Fig. 2.14 – James Joyce Bridge – Dublin [17]

2.4.10. Third Millennium bridge

Designed by Juan Arenas with an overall length of 270 m, being the main span of 216 m. The reinforced concrete single arch suspends the reinforced concrete deck, in a Nielsen hanger arrangement, with a set of cables to each side of the deck, and with the pedestrian walkway as a cantilever. The arch has a rise of 44 m and it spreads into a triangle near at the abutments, following the natural flow of the forces.

Fig. 2.15 – Third Millennium Bridge – Zaragoza [18]

2.4.11. Sheikh Rashid bin Saeed bridge

Project by FxFowle Architects, it will create two separate arch bridges that converge on an artificial island. Also known as the Sixth Crossing, the overall length of the bridge will be approximately 1600 m, with the east span being 380 m while the west span will stretch 610 m, making it the world's longest spanning arch bridge.

Fig. 2.16 – Sheikh Rashid bin Saeed bridge – Dubai [19]

2.6 Concluding Remarks

In this chapter, the definitions of tied arch bridges were introduced and different examples of tied arch bridges were presented. Since a more elaborate critical review of the bridges is beyond the scope of this thesis, it might be said that generally most of the bridges comply with the presented criterions of a good design, also bearing in mind the time they were designed. One exception becomes more than obvious: the Sixth Crossing will be the most expensive arch bridge ever built. It might be interpreted as a post-modernist extravagance, whereas more discrete and cheaper alternatives could have been adopted.

In the following chapter, the concept of a network arch bridge and the different hanger arrangements are presented in detail and illustrated with the aid of examples.

3
NETWORK ARCH BRIDGE

3.1 CONCEPT OF A NETWORK ARCH BRIDGE

A network arch bridge is a tied arch bridge with inclined hangers intersecting at least twice. Compared with regular tied arch bridges, i.e. those with vertical hangers, the network arch bridge exhibits low moments in both of the chords, which typically leads to important material savings. In Figures 3.1 and 3.2 can be seen that the network arch tends to behave like a simple beam, due to its higher stiffness, leading to small deflections. As shown by the Figures, partial loading on half of the span will lead to deflections on the upper and lower chord in the arch with vertical hangers while the arch with inclined hangers only observes deflections on the lower chord.

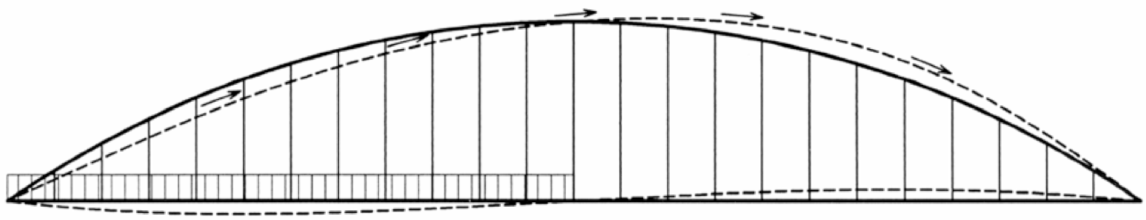

Fig. 3.1 – Tied arch with vertical hangers submitted to partial loading [4]

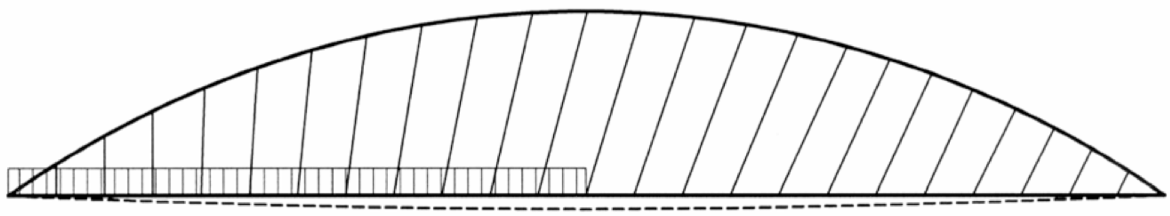

Fig. 3.2 – Tied arch with one set of inclined hangers submitted to partial loading [4]

As a consequence, in the arch with vertical hangers, bending is a decisive factor when it comes to the choice of the cross-section of the chords. In the network arch, bending will only occur due to local loading, and therefore the arch and the tie are only subjected to axial forces. Figure 3.3 compares the influence lines for bending moments in the chords of an arch with vertical hangers and of a network arch.

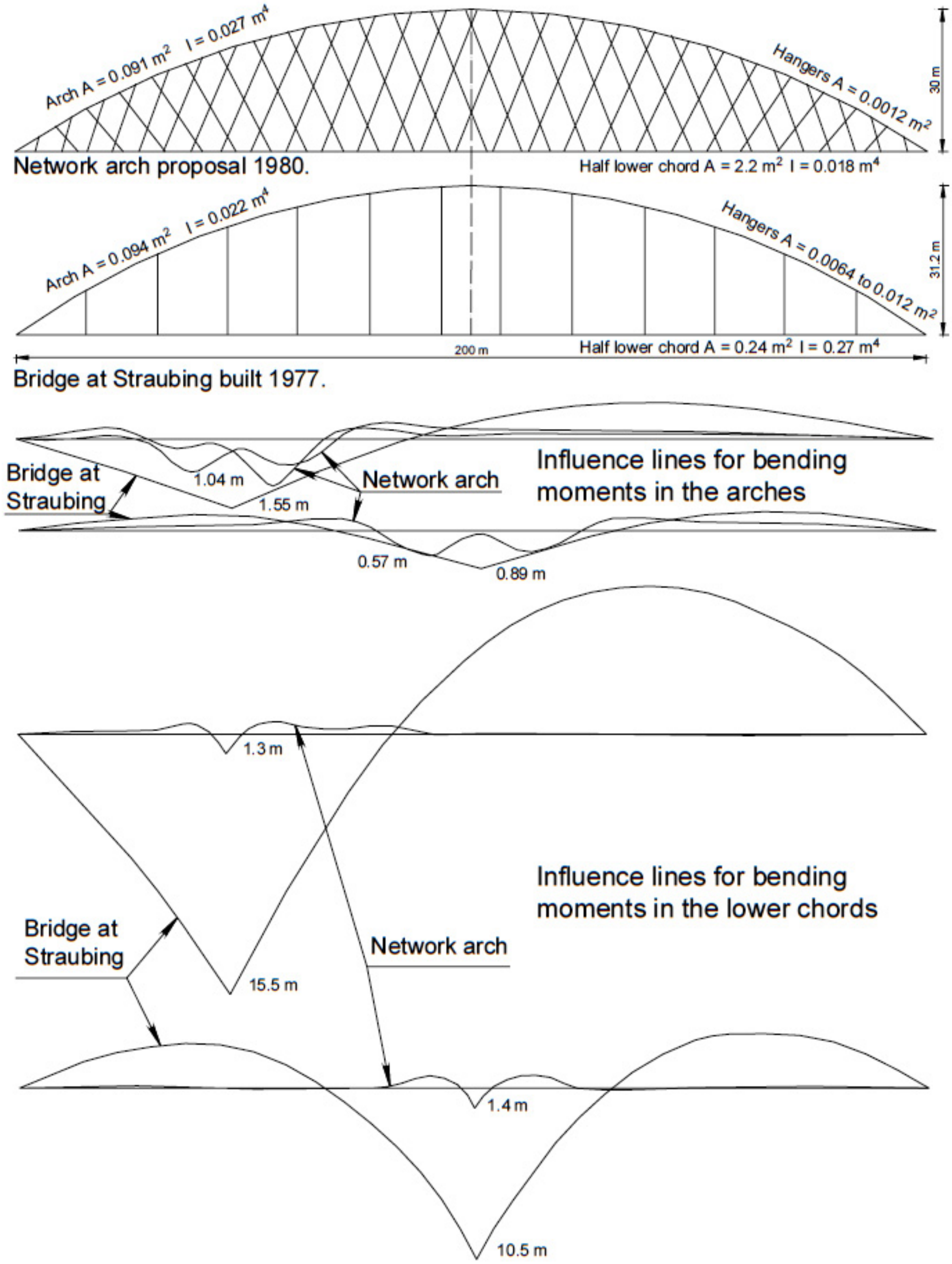

Fig. 3.3 – Influence lines for bending moments [4]

It is the stiffness of the hanger web that leads to such small bending moments in the lower chord of the network arch, indeed about ten times smaller in comparison with the conventional arch bridge.

As a result, longitudinal bending does not govern the network arch design. As shown later, this leads to a more efficient use of material, to lower steel weight and to more slender arch cross-sections. Transversal bending moments are usually greater than longitudinal bending moments, causing transversal loads to determine the design of the concrete or of the composite steel-concrete tie.

3.2 HANGER ARRANGEMENT

As a simplification, hangers are often placed with equal spacing along the upper chord, in order to turn the constructive process easier while also leading to more uniform bending moments and smaller buckling lengths along the arch. As a consequence, the location of the nodes in the bottom chord is the only variable in the hanger arrangement.

An alternative consists in placing the hangers with equal distance along the deck. In this case, the location of the nodes in the upper chord is the only variable. If the deck is a composite concrete-steel solution, where the anchorages of the hangers lie near the connection between the longitudinal and transverse beams, an almost null bending moment in the longitudinal beams is expected.

3.2.1. CONSTANT SLOPE CONFIGURATION

It is the most common and ancient method of defining a network hanger arrangement. In Japan almost every arch network bridge, also known as Nielsen-Lodghe, was defined like this. By fixing the position of the node in the upper chord, a constant slope is set for the hanger, thereby defining the position of the node in the lower chord where it intersects with the tie, as shown in Figure 3.4.

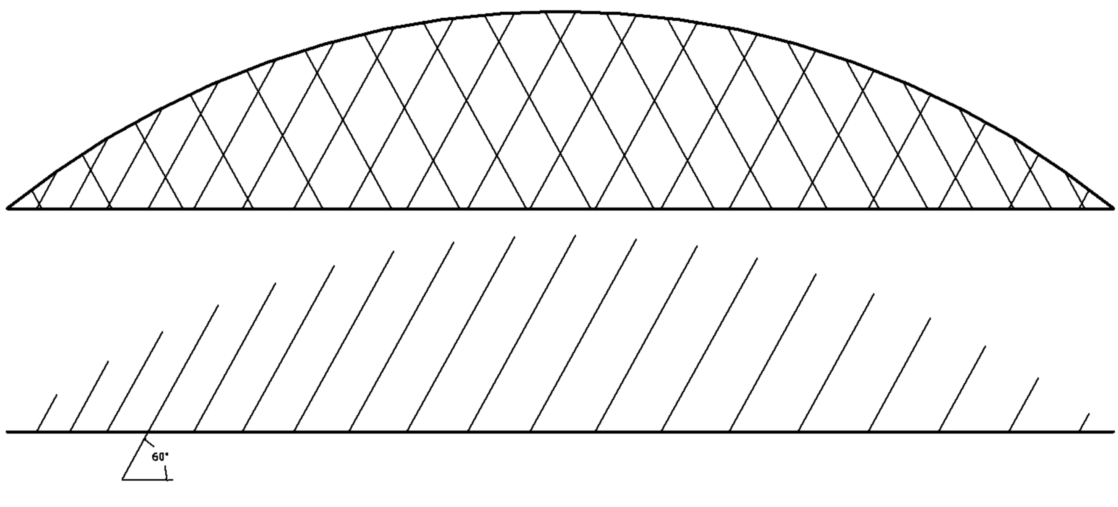

Fig. 3.4 – Definition of slope

3.2.2 Variable slope configuration

This method follows the same concept as the previous one. In this case, however, the slope of each hanger varies following, for instance, a linear function like **$\Delta\varphi=\alpha.x+\beta$**, where **x** is the number of the hanger, **α** and **β** are the parameters that make the hanger arrangement vary along the length of the arch. A general case is illustrated in Figure 3.5. Assigning a constant slope is a particular case of this configuration, when **α** is taken as 0.

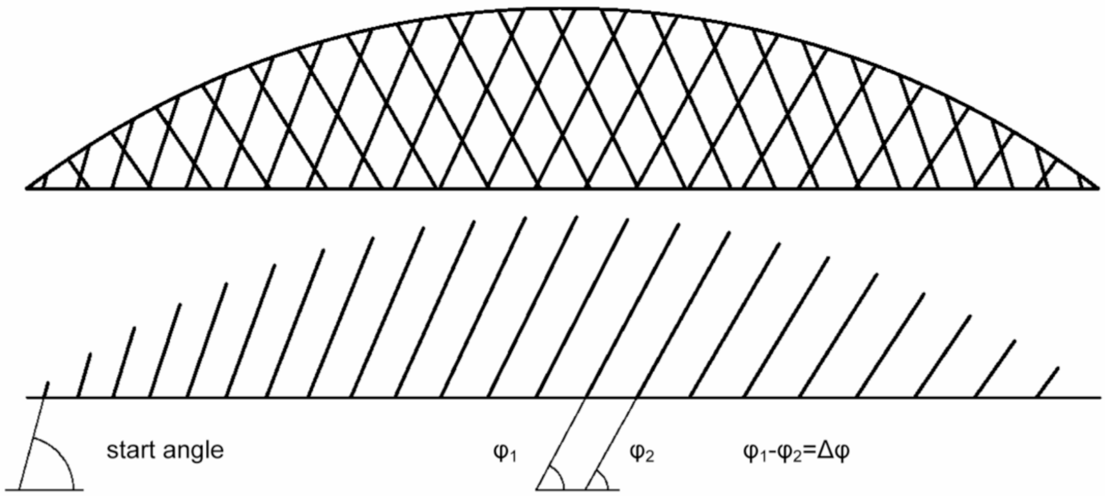

Fig. 3.5 – Definition of start angle and angle variation

3.2.3 Advanced hanger configuration

When subjected to uniformly distributed loads, the forces in the hangers are minimum in the radial direction, like in a "spoked wheel", as shown in Figure 3.6. Thus, bending in the arch is minimized when the line of thrust deviates very little from centreline of the arch, being more evident when the arch is defined as a part of a circle, the anti-funicular for radial loads. However, this assumption only applies if the forces in the hangers are equal, which does not happen when live loads are applied to the bridge. Nevertheless, this arrangement is quite efficient to sustain dead loads.

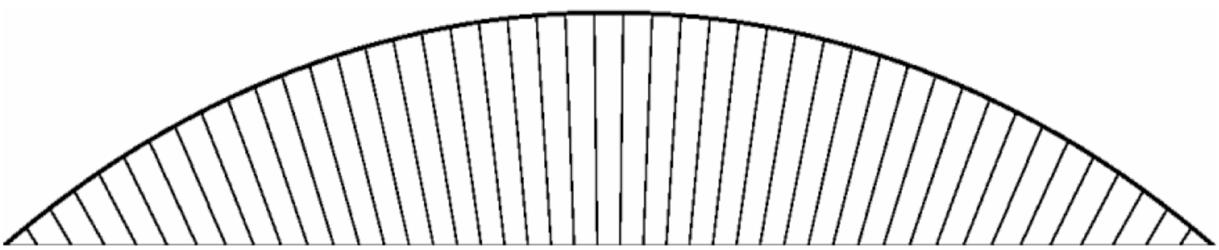

Fig. 3.6 – Spoked Wheel

In Figure 3.7 an extrapolation of the "spoked wheel" model for a Network configuration is drawn. As mentioned before, this model shows that if the forces in each hanger were approximately equal, the "resulting force" would lie on the radii of the arch circle.

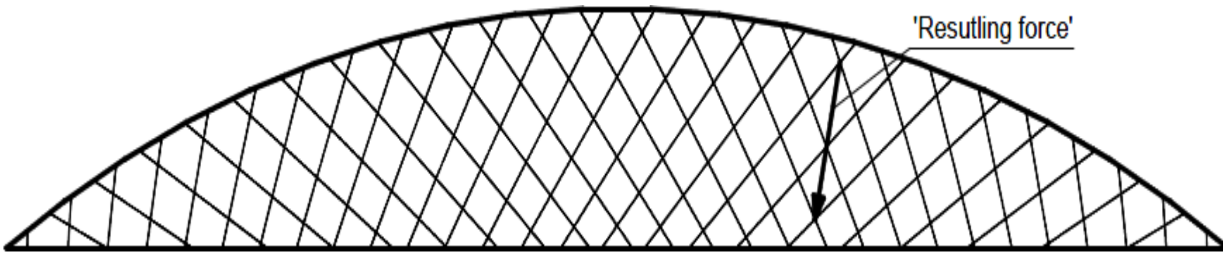

Fig. 3.7 – Concept of the advanced hanger configuration [20]

Hence, and in order to simulate a similar structural behaviour in the network arch configuration, Brunn and Schanack have defined that the first intersection between hangers below the arch should aim the radii of the arch circle [20]. This way, the only variable involved is the angle between hangers when they cross each other, as illustrated in Figure 3.8. Here, the hangers are placed with equal space along the upper chord. Throughout this investigation, the angle marked in grey will be the key variable.

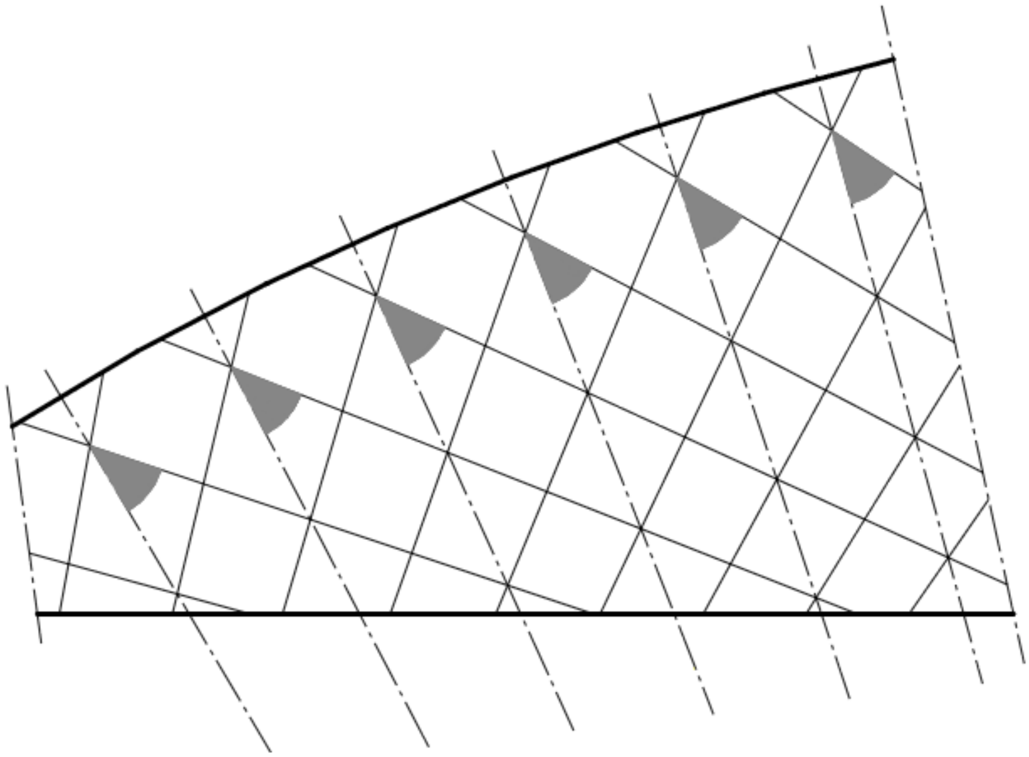

Fig. 3.8 – The hangers cross symmetrically the radii with same angle [20]

3.2.4. Alternative constant slope configuration

This configuration is similar to the constant slope configuration with the main difference that the starting nodal points are fixed not in the upper chord but in the lower chord. Each pair of hangers is merged in one nodal point and spaced with equal distance along the tie, as illustrated in Figure 3.9.

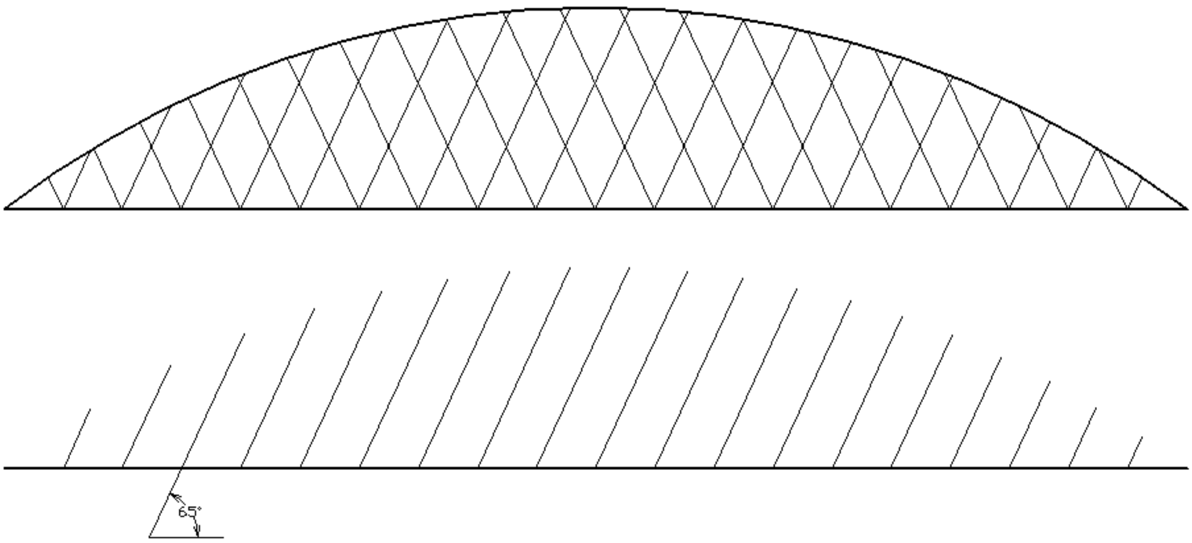

Fig. 3.9 – Definition of slope

3.2.5. Other configurations

Defining a network arch bridge only requires that two hangers (or cables) cross at least twice. Departing from this simple and broad definition, any web arrangement that meets this criterion can be classified as such.

One of the most straightforward hanger arrangements consists of assuming a constant slope as discussed above while ensuring that in the neighbourhood of the middle half of the span, the distance between lower hanger nodes would be the same. To achieve this, either the distance in the upper chord would change or the angle of each hanger would vary. This model is not further considered due to the fact that it is mostly empirical, thereby making it hard to extrapolate generalized conclusions, and also because such partial change of the web arrangement, in the opinion of the author, will lead to a rather disagreeable configuration.

Another hanger arrangement, based solely on the position of the lower hanger node distances was defined by Brunn and Schanack [20]. In this case, the nodal distance between the hangers of one set increases till the end of the arch, as shown in Figure 3.10.

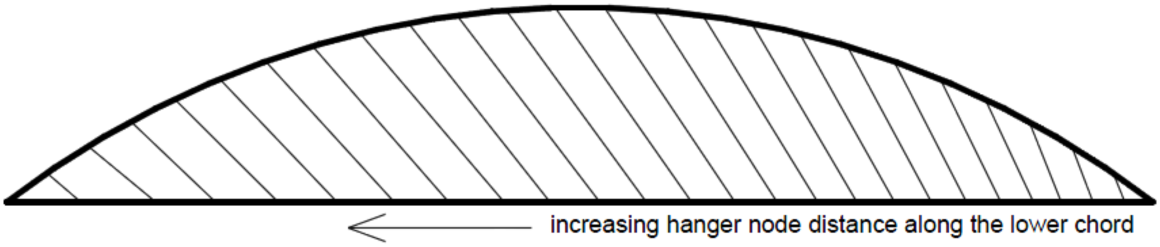

Fig. 3.10 – The hangers cross symmetrically the radii with same angle [20]

In order to adopt the considerations of the previous model, which suggests that it would be beneficial for the hanger nodes along the middle of the span to be positioned at equal distances, an elliptical function is adopted, as suggested in Figure 3.11.

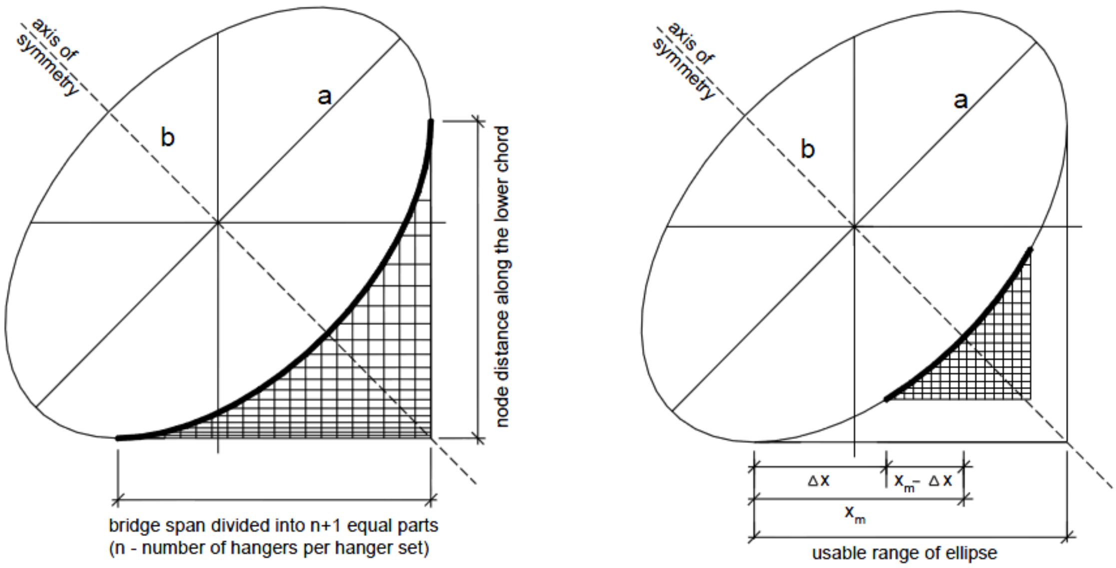

Fig. 3.11 – Node distance defined with elliptical curves [20]

The ratio between the semi-minor axis b and semi-major axis b can define a variable, so as the ratio between the unused range and the usable range.

Despite displaying an interesting mathematical configuration, this model is not going to be used in the following investigation, due to the fact that it is mostly a mathematical model which is detached from a physical model showing little constructive advantages when compared to the other configurations.

3.3 Examples of Arch Network Bridges

3.3.1. Steinkjer bridge

Situated in Norway, this is most likely the first network arch bridge ever built. Designed by Per Tveit in 1963, it spans 80 m with two parallel arches and the deck has a total width of 8 m. The arch is a steel box with triangular shape while the deck is made of concrete with longitudinal pre-stressing. Each hanger does not cross more than twice any other hanger.

Fig. 3.12 – Steinkjer Bridge – Norway [21]

3.3.2. Fehmarnsund bridge

Designed by the Gutehoffnungshuette Sterkrade AG, this bridge presents an overall length of 963 m. The main span has 240 m and is defined by two convergent arches made of steel. The deck is 21 m wide and has two road lanes and one rail line, also made of steel. It is probably the first network configuration where the hangers cross each other more than twice.

Fig. 3.13 – Fehmarsund Bridge – Germany [22]

3.3.3. PALMA DEL RIO BRIDGE

Designed by Ideam to cross the Gualdiquivir River, it spans out 130 m. The two arches are inclined and convergent, with a tubular cross section of 900 mm of diameter and 50 mm of maximum wall thickness. The deck has a total width of 20.4 m and is defined with two longitudinal beams of 900 mm of diameter and 40 mm of maximum thickness. Transverse beams are positioned every 5 m to sustain the platform. In order to avoid the effect of local loads on the tie, the anchorages of the hangers are located right where the transverse beams meet the longitudinal ones.

Fig. 3.14 – Palma del Rio Bridge – Spain [23]

3.3.4. PROVIDENCE RIVER BRIDGE

Designed by the Rhode Island Department of Transportation it encompasses a main span of 121 m. Serving as a highway, the bridge entails three parallel steel arches. The composite deck, consisting of transversal steel beams between the ties, has a total width of 47 m.

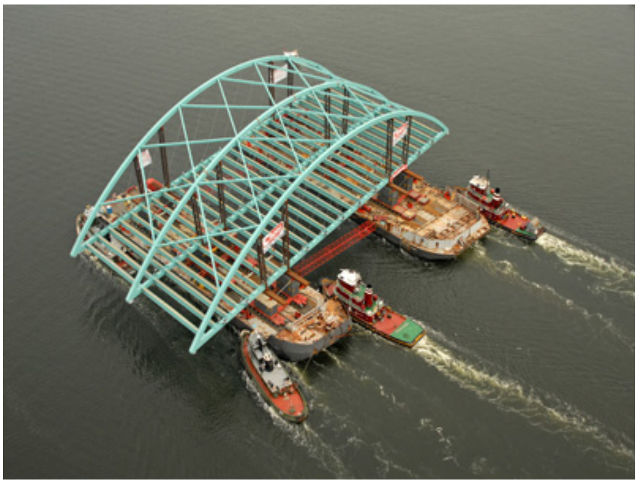

Fig. 3.15 – Providence River Bridge – USA [24]

3.4 Design considerations

Per Tveit designed two network arch bridges where the cross-section of the arches was a steel triangle box girder. Later, Tveit stated that the use of American Wide Flange beams leads to a less costly solution [4]. Nonetheless, the author considers that such cross-section in an arch bridge can only be used when surroundings and location lent themselves to it, as could be the case in an industrial zone. Nowadays, arch bridges are mainly used in contexts where the environmental visual impact must be reduced, thereby bringing up aesthetic concerns as a main issue. Positive and creative solutions, such as the Palma del Rio Bridge, are defined with closed cross-sections, which can be tubular, triangle, squares, etc.

Placing the hangers with an equal distance along the upper or the lower chord leads to different results. When the hanger nodes are placed with equal distance along the arch, bending moments are minimized in the arch. In a similar way, when the hanger nodes are placed with equal distance along the tie, bending moments are minimized in the tie.

3.5 Concluding Remarks

In this chapter the definition of network arch bridge was presented and different types of hanger arrangements were provided. Within the concept of the network web, any configuration where hangers cross at least twice can be defined as such, which means that the examples presented do not necessarily have to follow the prescriptions above defined.

In the next chapter, the model for the deck bridge will be defined according to the actions prescribed in the latest version of the Eurocode 1 [25]. A preliminary design of the arch and hangers will also be presented.

4
PRELIMINARY DESIGN AND MODELLING OF A NETWORK ARCH BRIDGE

4.1. SCOPE

In this chapter, the preliminary design of the elements that constitute the bridge model are presented. Per Tveit stated that a network arch bridge is most economic when the tie is a concrete slab, either pre-stressed or not [4]. Nevertheless, several designers have adopted different configurations for the deck, especially in Japan and Spain, where composite steel concrete decks are a standard. Using a composite deck is also one of the aims of this thesis.

The loads and combination hereafter used are according to the most recent version of the Eurocode 1. Where the Portuguese National Annexes are yet to be defined, the British ones will be utilized.

For the sake of simplicity, the preliminary design is not extensive, and hence actions such as temperature, wind or seismic are not considered. Traffic actions are considered in simplified manner, while assuming a conservative approach. Finally, the solutions adopted are compared with projects that hold similar characteristics.

4.2. GEOMETRY

The bridge has a span of 100 m and the rise of the arch is 17 m, as illustrated in Figure 4.1. There is no particular reason to select this kind of span except their established commonness in tied arch bridges. The rise of the arch of 17 m is a value close to the optimal [4], allowing to minimize the cost of the material. As is well known, higher rises lead to minimal moments but also imply the use of more material.

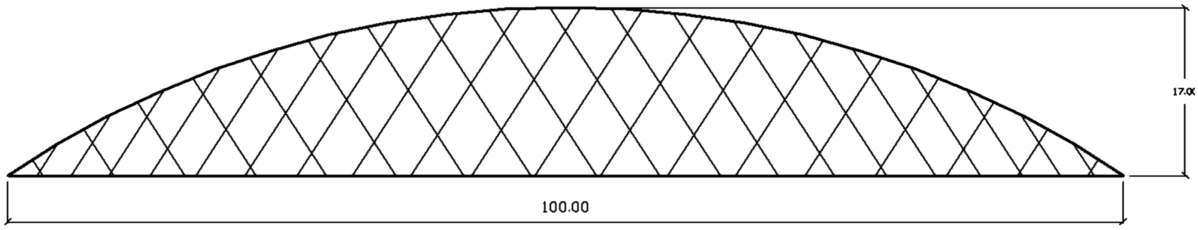

Fig. 4.1 – Arch bridge considered in the study

This bridge is an interesting deviation from the rule: whereas normally network arch bridges are endowed with parallel arches, here, the two arches are convergent at mid span. That has an impact on the deck construction, which not only will have to accommodate the track lanes but also provide additional space so that higher vehicles, like trucks, can circulate without damaging the hangers or the arch itself.

The carriageway has two notional lanes with a width of 3.5 m each and a hard strip of 1 meter wide on each side. The distance between the two arches amounts to 13 m.

The composite steel concrete deck follows a geometry similar to that found in Spanish bridges of the same kind. The carriageway is designed as a concrete slab supported by transverse steel beams spaced at 5 meters, of variable height, which are supported by the longitudinal beams, as illustrated in Figure 4.2. In the following parametric study the longitudinal beams are referred as tie.

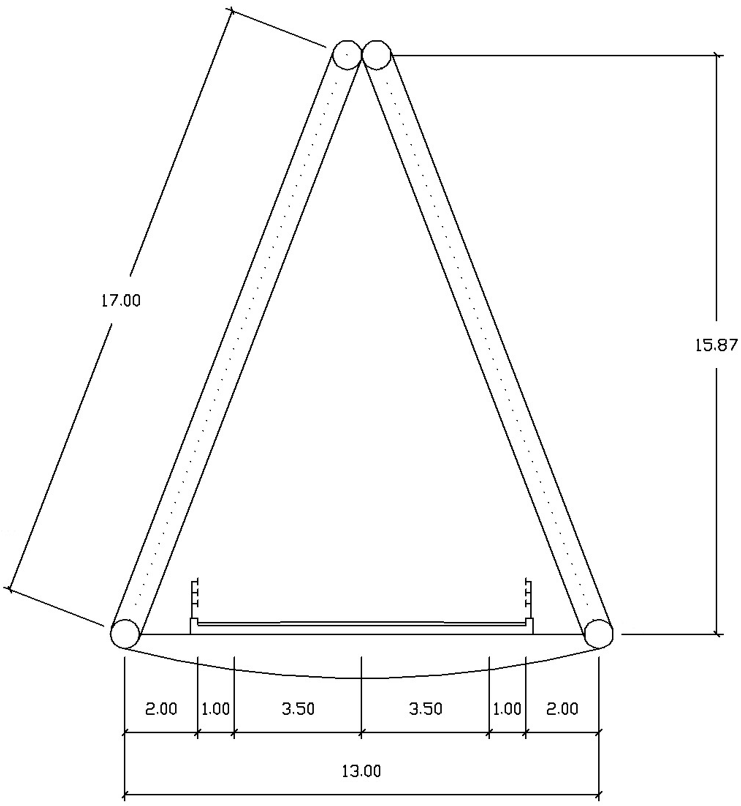

Fig. 4.2 – Composite steel concrete deck

4.3. MATERIALS

In what concerns to the materials that the bridge is made of, the choice has been for the most common in structures of this kind. The concrete class for the carriageway slab is C50/60, the steel reinforcement used is S500.

All the other members, such as hangers, transversal beams, longitudinal beams and arch profiles are made of steel grade S355.

4.4. Loads

4.4.1. Dead Loads

For the purpose of simplification, the dead loads only include the self-weight of the concrete slab and the pavement of the carriageway.

Table 4.1 – Dead Loads in concrete deck

	gk [kN/m3]
Road Pavement	20
Concrete Slab	25

For the composite steel concrete deck, it is assumed that the average height of the tarmack is 0.10 m. For the carriageway slab an average height of 0.25 m is assumed, taking built bridges of the same type as an example. In order to simulate the transversal beam self-weight, a dead load of 5 kN/m is considered.

4.4.2. Live Loads

The live loads on this bridge are the road traffic actions and also other actions specifically defined for road bridges. For bridges with spans of less than 200 m, the Eurocode 1 is applicable and therefore used in the context of this thesis [25].

In order to represent the road traffic actions, the Eurocode 1 defines models of road traffic loads according to the bridge class. Each carriageway is divided into notional lanes, being numbered from the most to the least unfavourable effect, as can be seen in Figure 4.3. The parameter w stands for the carriageway width while w_1 is the notional lane width.

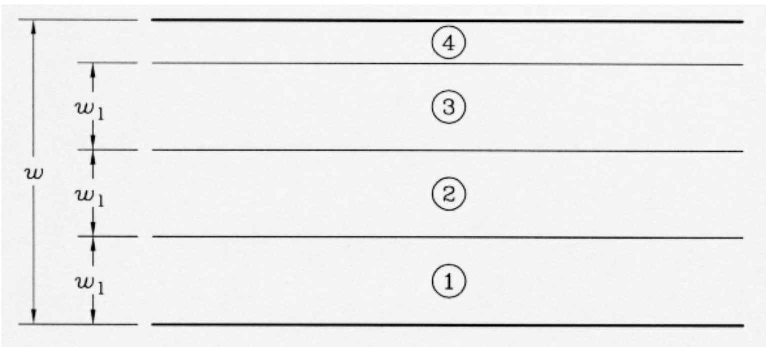

Fig. 4.3 – Lane Numbering in general case

A notable difference between the Eurocode and other codes is that the application of load models could potentially diverge from the notional lanes defined in the project. As a matter of fact, the number of lanes to be considered as loaded, their location on the carriageway and their number should be chosen in such a way that the effects on the load models leads to the most adverse action effects. The methodology to define each notional lane number and width is presented in Figure 4.4.

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

Fig. 4.4 – Number and width of notional lanes

The bridge for the current model has a carriageway of 9 m and two lanes of 3.5 m each, as well as 1 m of hard strip on each side. Thus, to comply with the Eurocode 1, the carriageway has 3 notional lanes, each one with 3 m of which the most unfavourable one is located in the mid span of the deck.

The load dodel serves to calculate vertical loads, representing concentrated and uniformly distributed loads, which cover most of the effects of the traffic of lorries and cars. It consists of two partial systems, a tandem system (TS) and a uniformly distributed system (UDL system).

The tandem system consists of two double-axle concentrated loads, with each axle being represented by two wheels and having the following weight: $\alpha_Q Q_k$, being α_Q an adjustment factor, taken as 1.0, as stated in the British National Annex (NA). In this system, no more than one tandem system should be taken into account per notional lane and each axle should be taken into account with two identical wheels, being the load per wheel equal to $0.5\alpha_Q Q_k$.

The UDL system is only applied to unfavourable parts of the carriageway, having the following per square meter of notional lane: $\alpha_q q_k$, where α_q is an adjustment factor, that accounts for different traffic classes and is taken as 1.0, as stated in the NA. The characteristic values for the TS and UDL system are shown in Table 4.2.

Table 4.2 – Load Model 1 characteristic values

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

4.5. Combinations

The preliminary design is undertaken for the ultimate limit state (ULS), for both persistent and transient design situations. The design values of actions for ULS in persistent and transient design situations are the ones defined on expressions 6.9b to 6.10b of the Eurocode 0, which are in agreement with the Tables A2.4(A) to (C), of the Eurocode [26].

The partial factors adopted are similar to those considered in the design of regular structures.

4.6. Preliminary design of the bridge

4.6.1. Preliminary design of the deck

The static system and characteristic loading per composite beam is presented in Figure 4.5. Taking into account that each beam has an influence width of 5 m of concrete slab on the carriageway, a distribution of the tandem system loads has to be defined. According to a longitudinal analysis it can be expected that each axle of the TS contributes with 80% of the total loads when applied to the beam centred between the two wheels of each axle.

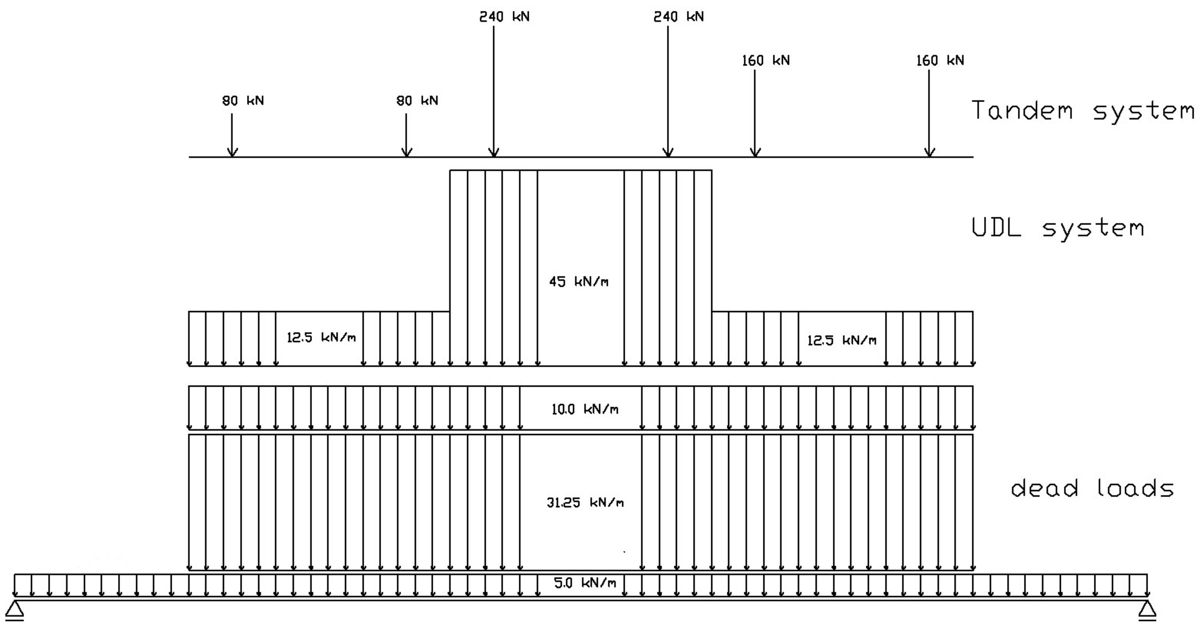

Fig. 4.5 – Characteristic loads applied to the composite steel concrete deck

Bending moments and shear forces in ULS, in a persistent design situation, are depicted in Figures 4.6 and 4.7.

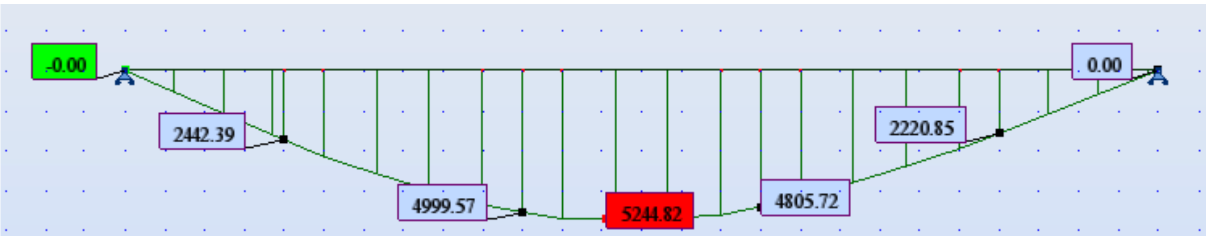

Fig. 4.6 – Bending moments in ULS [kN.m]

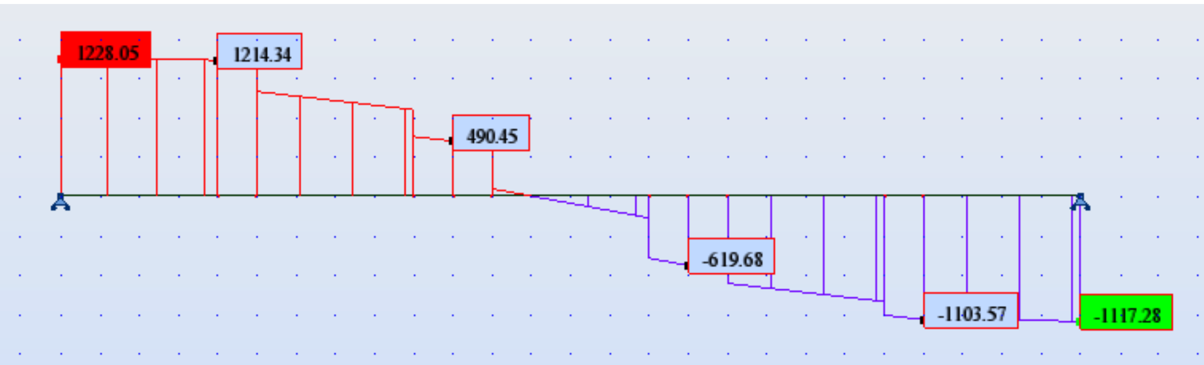

Fig. 4.7 – Shear forces in ULS [kN]

Starting from a conservative approach, an effective width of 2 m is employed for the concrete slab. An HEB900 commercial profile is used, and it is found that it remains elastic under the actions considered. The self-weight of the steel profile is around 3 kN/m, which is in line with the value adopted in the preliminary design. The cross section of the composite beam is illustrated in Figure 4.8.

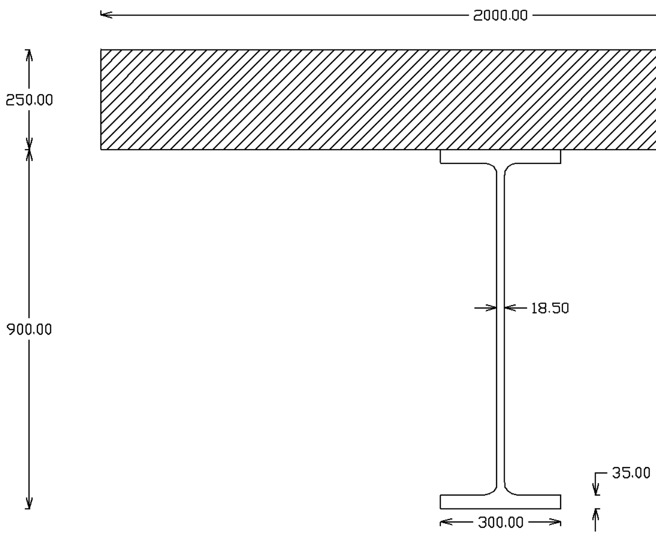

Fig. 4.8 – Composite beam

4.6.2. Preliminary Design of the Arch and Tie

Tveit proposed an expression to estimate an approximate axial force on a vertical arch. This expression assumes that the bending moment on the arch is so little that it can be neglected.

in which w is the uniformly distributed load per unit length of lane, l is bridge span, h is the rise of the arch, v_h is the average angle of the hangers crossing a vertical line at a distance x from the support, which for this case is taken as 60º.

The compression on the arch provided by this expression is due to a uniformly distributed load, which is not the case when the tandem system is applied. For the purpose of this thesis, a simplified method is adopted replacing the concentrated loads of the tandem system by equivalent uniformly loads with a amplification factor of 1.30, in order to prevent the local effects that are not taken into account. The dead loads corresponding to the tarmac, concrete or steel, in the case of the composite deck, are scaled by 10% in order to simulate the loads of the other equipments in the bridge. For the estimation of the tensile force in the tie, a similar expression is used, where the second term is assumed as negative, meaning that in absolute values, the axial force in the arch is always greater than in the tie.

The loads listed in Table 4.3 concern half of the deck, and exhibit the expected axial forces for the arch and for the tie

Table 4.3 – Compression in the arch

	Dead loads [kN/m]	Live Loads [kN/m]	w=1.35gk+1.5qk [kN/m]	K_u [kN]
Arch	48.1	29.06	108.5	8484.3
Tie	48.1	29.06	108.5	7475.30

Though bending moments are known to be small in network arches, they are nevertheless important to be considered, as they can significantly reduce the resistance of a cross-section. As mentioned before, in the model bridge only box cross sections are used for aesthetic reasons. Therefore, no use is made of bending optimized cross sections, such as the American Wide Flanges. As a result, bending will have to be considered when defining the cross-section. A rather easy sum of ratios, between the design values and the design resistances, is taken as a simplified criterion for checking the cross-sections:

Only circular sections are adopted, which means that the bending moment is the resultant of the bending moments along the y-y and the z-z axils.

For the tie, a bending moment of 1000 kN.m is assumed, while for the arch a bending moment of 2000 kN.m is considered. Compared to the regular values obtained in most of the network bridges studied, the actions considered are indeed conservative.

The cross-section adopted for the arches is a Circular Hollow Section (CHS) 800x45, with a diameter of 800 mm and a wall thickness of 45 mm, while for the longitudinal beams a Circular Hollow Section (CHS) 800x25, with a diameter 800 mm and a wall thickness of 25 mm is adopted.

4.6.3. Preliminary Design of the Hangers

When it comes to designing the hangers of an arch bridge, it is crucial to be aware of the influence lines to similar configurations in order to establish a similar value of design tension. Due to the fact that those influence lines were not available to the author, it was decided to adopt smooth bars of S355 with a diameter of 60 mm, which is a solution often seen in network arch bridges with similar spans. Thus, a design value of approximately 1000 kN is expected for the tensile resistance of the hanger.

The standard number of hangers considered is 19 per set and 38 per arch, which in a vertical web arrangement would result in a maximum tension lower than 500 kN, in this manner providing a wide margin which enables not to exceed the resistance of the hangers on others web configurations.

4.7. MODELLING

4.7.1. DESCRIPTION OF THE MODEL

The transverse beams support the concrete carriageway and transmit the forces to the longitudinal beams spaced by 5 m each. The longitudinal beams are suspended by the hangers to the arch.

Though the bridge is a composite structure, the concrete slab is not modelled, which means that all the loads of the carriageway are defined as linear or local loads along the transversal beams. This process not only simplifies the massive bridge design, but also allows for a major simplification in what concerns data treatment. Since the loads of the slab are in fact transmitted to the structure via the transverse beams, this method leads to results close to the results exhibited by the real structure.

The bridge is modelled in Robot Structural Analysis as a spatial structure (3D), where all the elements are defined as beam elements. A wind bracing is not designed due to the fact that wind loads, horizontal loads, or second order effects are not considered in this work. Some preliminary results also showed that for the asymmetric configuration of vertical live loads, the results variation, with or without wind bracing, was less than 3%. In fact, the structure could have been modelled as a 2D model, since a comparative study showed that the results variation between the 2D and 3D models was within 5%. Nevertheless, in the course of future studies, the structures hereby modelled might be used in order to fulfil the assessment of the different variables. On the other hand it also resulted in a more intuitive way of proceeding design.

An example of a model with 44 hangers, in the advanced configuration, is presented in the following Figure.

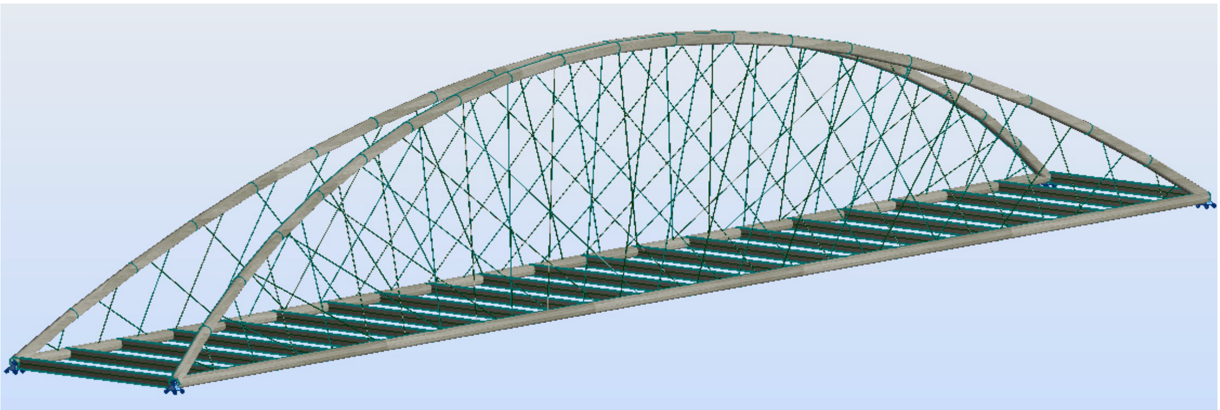

Fig. 4.9 – Model of bridge in Robot Structural Analysis

As mentioned before, no slabs are modelled, which leads to an equivalent cross section being chosen, in which a steel HEM900, having an equivalent inertia, is adopted. This cross section, stemming from the same family of HEB900, has thicker flanges and web. The hangers were initially modelled as cable elements, but the existence of nonlinear convergence issues lead to a modelling in which the hangers are considered as bars. This amount in fact to a limitation of the FEM program which does not allow designing the hangers as simple bars that can only be tensioned and not compressed. This way, as will be shown in the next chapter, when hangers are found to be under compression, they are treated as relaxed, i.e. removed from the model and the structure then recalculated. In all the configurations tested the circular arch is divided into 20 beam elements.

4.7.2. Loads and Combinations

Without the consideration of the carriageway slab, simplified measures had to be adopted. The first attempt to model the vertical live loads, such as the tandem system, was to use moving loads along the longitudinal beams. However, this approach is not accurate due to the fact that all the loads of the carriageway only transmit to the ties through the transverse beams. This way, four standard transverse beams are considered: one that corresponds to the most unfavourable position of the tandem system, resisting 80% of its loads; the adjacent beams, each resisting 10% of the total loads from the concentrated loads; a beam which only has the distributed loads from the tandem system and the last one, which only comprises the self-weight loads.

Two loading models for the live loads were considered. The first was the symmetrical Load Model 1 (LM1), in which the UDL system is placed throughout the entire bridge deck, in the 21 transverse beams, with the tandem system changing its position throughout all transversal beams that constitute the deck. The second is the asymmetrical Load Model 1, where the UDL system is only considered in half of the deck and with the tandem system being applied to the first 11 transverse beams.

In *Robot Structural Analysis*, 11 load cases were created for each of the two leading actions, giving way to 22 combinations in total. Nevertheless, and more so when it comes to data treatment, the envelope defined by the 11 combinations represents the envelope of a leading action combination.

4.8. Concluding Remarks

In this chapter the preliminary design of the main elements that define the arch bridge was described. A simplified interpretation of Eurocode 1 was adopted while bearing in mind the advantages of a conservative approach. Furthermore, the modelling of the structure was conducted with the help of a simplified hypothesis that served both when defining the structure as well as in the actions simulation.

In the next chapter, a parametric study on different hanger arrangements is performed, and the main results are compared and discussed.

5
HANGER ARRANGEMENT

5.1. SCOPE

In this chapter a parametric study on the hanger arrangement is carried out. Several hanger configurations, ranging from the vertical to network arrangements are studied and based on a set of criteria.

For this purpose, over one hundred bridges were analysed. The simplified design approach allows to obtain a clear overview of the benefits of specific arrangements. However, one should not overlook that this approach can only serve as a first approximation. It is incomplete, since it does not take into account important structural issues, such as buckling, fatigue or the response to dynamic loading scenarios.

5.2. PARAMETERS

Defining parameters in order identify the most efficient solution is a complex process that involves several variables intending to fulfil certain goals, such as aesthetics, economy, efficiency, etc. In the following approach, aesthetics evaluation is taken in account after discussing other goals such as:

- As far as the hangers are concerned, the attributes to minimise are the maximum axial forces and the number of relaxed hangers. The maximum tensile forces in the hangers not only influence the ultimate limit states but also play a role in fatigue assessment, together with other variables such as the stress variation. Relaxed hangers lead to redistribution of forces along the bridge, increasing the average axial force per hanger, and inducing severe fatigue issues along the hangers;
- In the arches, the main attributes to minimise are the maximum bending moments and the maximum compressive forces. The interaction between these two variables is well known, with the axial resistance being reduced significantly in the presence of bending moments.
- On the tie, the main attributes to minimise are the maximum bending moment and the maximum tension. Like in the arches, a criterion of the interaction between bending and axial force is defined, but buckling does not occur in ties, due to their tensile behaviour;
- A general criterion based on the minimum global weight of the bridge, deck and arch, is developed in order to determine the best configuration of each type of hanger arrangement. As mentioned before, only circular hollow cross-sections are used for the arch and tie, being

limited to a diameter of 1,40 m. The overall weight considered is due to the arches, longitudinal beams and hangers.

5.3 Vertical Configuration

5.3.1. Model Description

In order to evaluate several models, this study departs from the vertical configuration as the control case. A total of 10 bridges were designed, with the number of hangers ranging from 5 to 50. The cross-sections of the hangers, arches and tie are those defined in the preliminary design. Later in this chapter, the cross-sections are redesigned in order to fulfil the resistance criteria, in the ultimate limit states.

Two examples of different configurations, for 15 and 40 hangers, are presented in Figures 5.1 and 5.2.

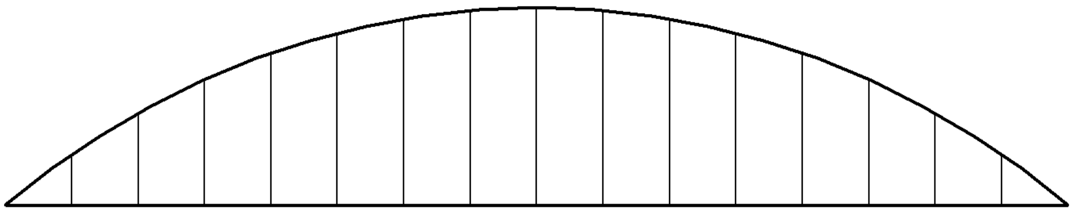

Fig. 5.1 – Arch rise of 17 m, span of 100 m and 15 hangers

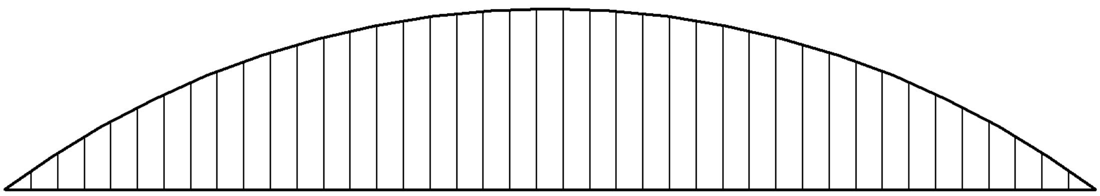

Fig. 5.2 – Arch rise of 17 m, span of 100 m and 40 hangers

5.3.2 Discussion of the Results

5.3.2.1. Hangers

In this configuration no relaxed hanger are found and, as expected, increasing the number of hangers leads to smaller axial forces in the hangers and to lower average tension per hanger. The maximum tensile forces in the hangers are illustrated in Figure 5.3, as well as the average tensile in the hanger arrangement.

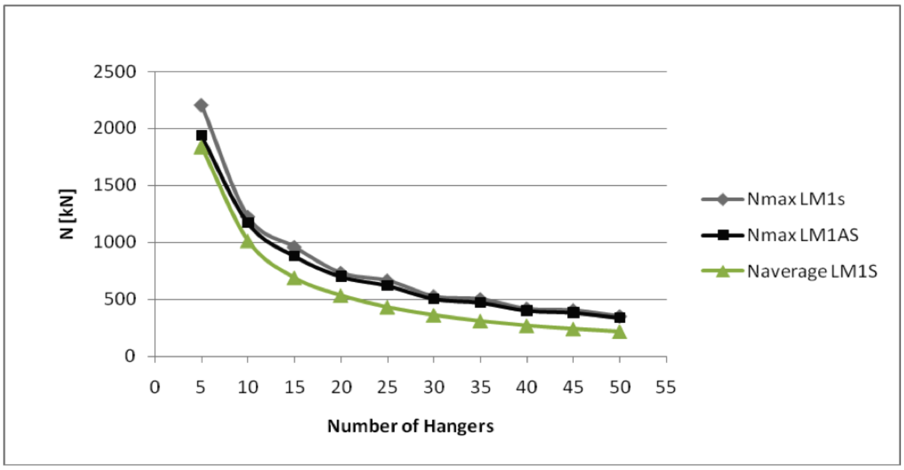

Fig. 5.3 – Axial forces in the hangers

5.3.2.2. Arch

In Figure 5.4 it is observed that the compression force in the arch increases with the number of hangers, while the shear forces decrease. With more hangers the anti-funicular becomes smoother, while for fewer hangers, and in order to achieve an arch behaviour, the anti-funicular should be a polygonal function. Due to the fact that the arch is defined as a part of a circle, for fewer hangers the shear forces increase. Nevertheless, the difference between each arrangement is not very expressive, which confirms that the number of hangers do not influence dramatically the axial forces in the arch.

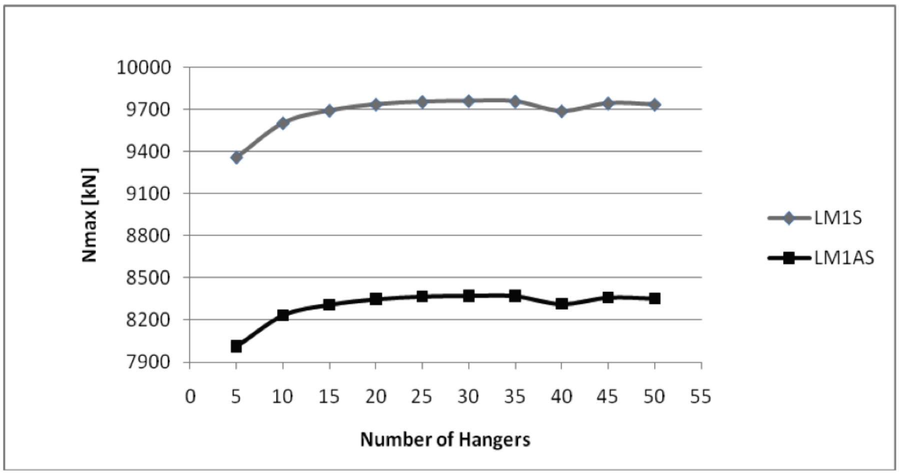

Fig. 5.4 – Compression in the arches

When it comes to bending moments, it is clear, from Figure 5.5, that increasing the number of hangers leads to smaller bending moments. This difference is indeed noticeable for configurations with fewer hangers, such as five hanger bridges, and tends to stabilize when the number of hangers is increased.

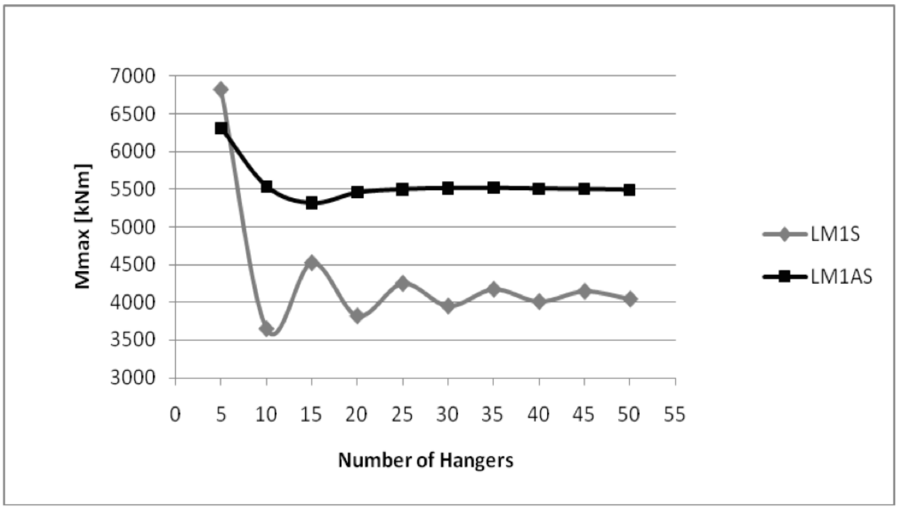

Fig. 5.5 – Bending moments in the arches

5.3.2.3. Tie

The number of hangers has no influence at all when it comes to the maximum tension in the tie, as shown in Figure 5.6. For vertical hangers, the tension along the tie is the horizontal component of the compression of the arch at the abutments, which dependent on the vertical loads, leading to values of tension in the tie that are the same for every configuration.

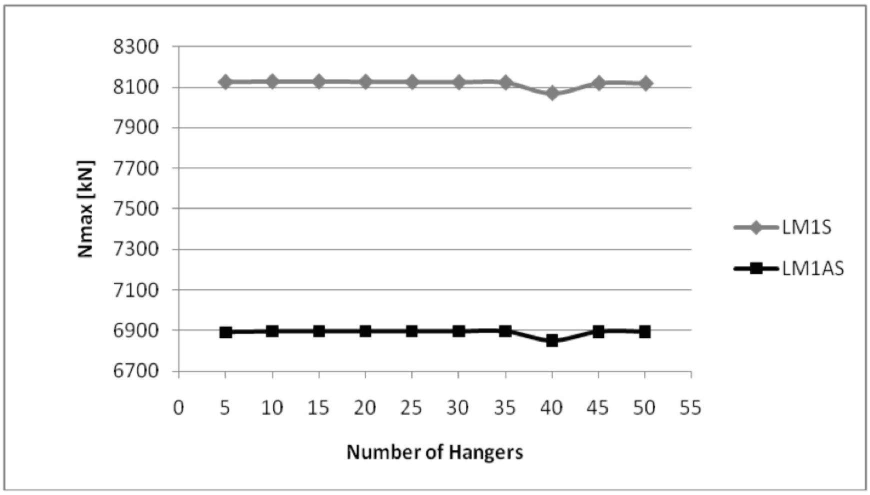

Fig. 5.6 – Axial forces in the tie

In Figure 5.7, the bending moments decrease dramatically compared to arrangements which have fewer hangers, stabilizing on configurations with 15 hangers or more. It becomes thus evident that symmetric live load results in smaller bending moments in the tie in comparison with the asymmetrical live load.

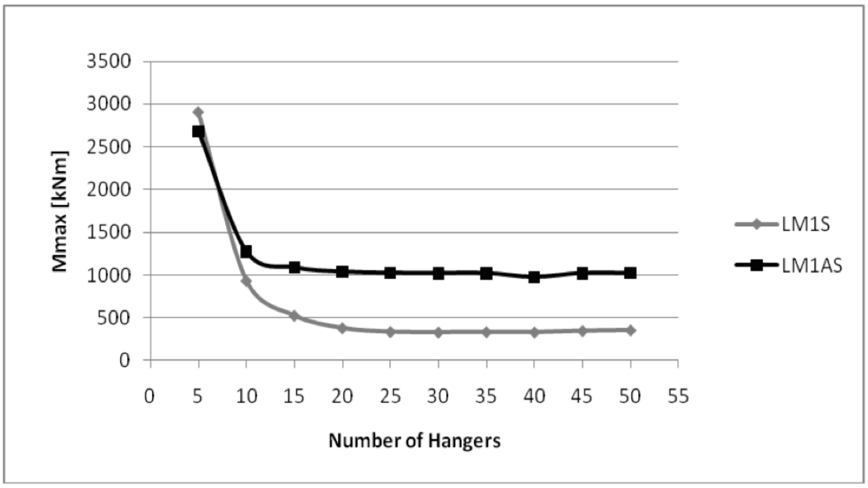

Fig. 5.7 – Bending moments in the tie

5.3.2.4. Bridge Weight

Taking as design criteria the interaction between the axial force and the bending moments, a minimum weight for the arch and tie are calculated and presented in Tables 5.1 and 5.2.

Table 5.1 – Minimum weight of the arch

Hangers	5	10	15	20	25	30	35	40	45	50
Min Weight (ton)	71,81	62,95	62,95	62,95	62,95	62,95	62,95	62,95	62,95	62,95
Cross-section	1400x40	1400x35	1400x35	1400x35	1400x35	1400x35	1400x35	1400x35	1400x35	1400x35

It should be acknowledged that the catalogue limits the cross-sections. It restricts the cross-section to a diameter of 1.40 m, which was indeed the least diameter required to fulfil the criteria of minimum weight. If wider cross-sections were available, bulkier diameters would be adopted to fulfil the criteria of minimum weight in the arch.

Table 5.2 – Minimum weight of the tie

Hangers	5	10	15	20	25	30	35	40	45	50
Min Weight (ton)	41,87	33,56	28,86	28,86	28,86	28,86	28,86	28,86	28,86	28,86
Cross-section	1300x25	1300x20	900x25	900x25	900x25	900x25	900x25	900x25	900x25	900x25

The total weight of the arches, ties and hangers is presented in Table 5.3

Table 5.3 – Total weight

Hangers	5	10	15	20	25	30	35	40	45	50
Min Weight Arches (ton)	143,62	125,89	125,89	125,89	125,89	125,89	125,89	125,89	125,89	125,89
Min Weight Tie (ton)	83,74	67,13	57,73	57,73	57,73	57,73	57,73	57,73	57,73	57,73
Hangers Weight (ton)	10,05	9,96	11,13	10,76	13,33	16,00	18,47	21,03	23,60	26,17
Total Weight (ton)	237,41	202,97	194,75	194,38	196,95	199,62	202,09	204,65	207,22	209,79

5.3.3 Final Remarks

The values of bending moments, axial forces and weights serve as a comparative base for the following hanger arrangements. The arch cross-section is governed by the resistance to the design bending moment value. The most efficient solution is the configuration with 20 hangers, though it must be stated that when using more than ten hangers, the total weight variation is smaller than 10%.

5.4. Constant Slope Configuration

5.4.1. Model Description

In this model the only variable is the slope of each hanger. The hangers are spaced at an equal distance along the arch, being the angle of the hanger defined with the horizontal. Thus, steeper hangers are obtained when larger angles are considered.

The tested models have angles that vary from 40º to 85º and two examples are illustrated in Figures 5.8 and 5.9.

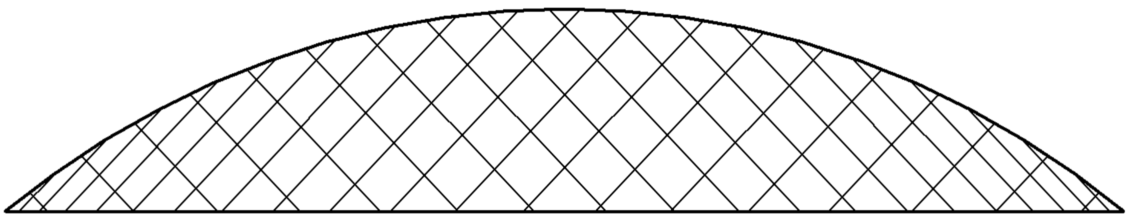

Fig. 5.8 – Arch rise of 17 m, span of 100 m, 38 hangers with 45º of slope

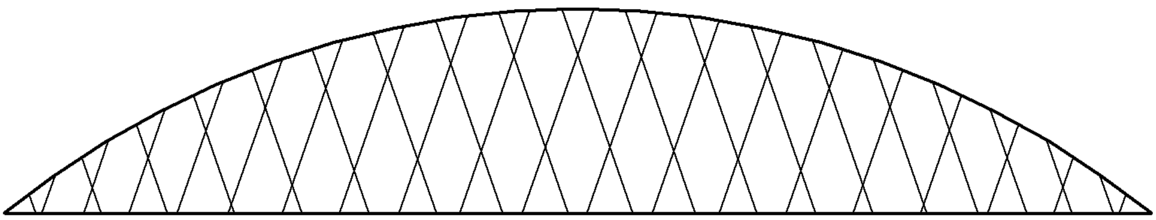

Fig. 5.9 – Arch rise of 17 m, span of 100 m, 38 hangers with 70º of slope

5.4.2 Discussion of the Results

5.4.2.1. Hangers

For this configuration the number of relaxed hangers is rather large, as can be seen in Figure 5.10. With 38 hangers per arch, 76 in total, the minimum number of relaxed hangers is 8, with a maximum of 32 relaxed hangers, when a 75º angle is considered.

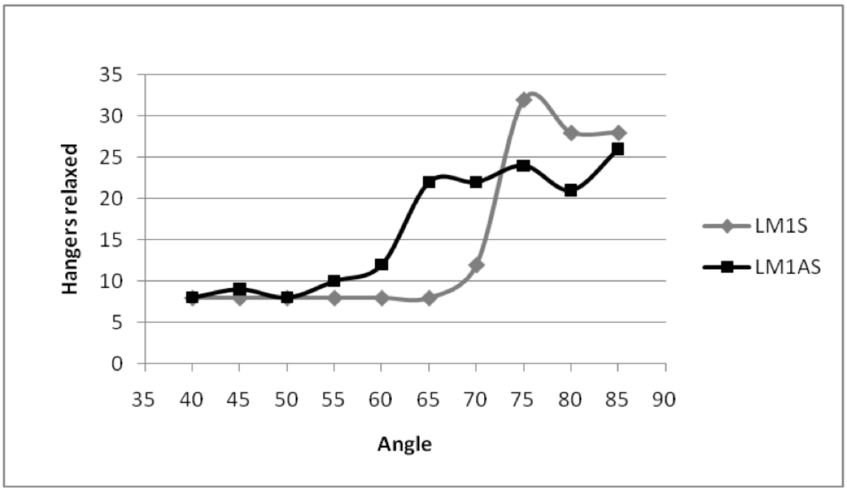

Fig. 5.10 – Number of hangers relaxed

It has been mentioned before that the hangers are modelled as bar members, which enables them to potentially work in compression. In a regular case, the bars are extracted from the model and the structure is reanalysed. However, with so many relaxed hangers, this procedure is not always recommendable since it might represent the structural behaviour of the bridge in an inaccurate way. For instance, removing 3 or 4 hangers results in a configuration that can present more or less hangers relaxed than the total sum of the original relaxed ones.

However, it must be underlined that in every configuration the first two hangers of each set are relaxed, reaching a total of 8 relaxed hangers when considering the two arches. This phenomenon is explained by the arch clamping effect towards the tie, if these hangers are allocated in a range of the arch where this occurs. In order to prevent this effect, the position of these hangers should be manually shifted.

The results without the relaxed hangers are presented in the following diagrams. It can be concluded that the maximum axial force in the hangers is compatible with the graphic that describes the number of relaxed hangers.

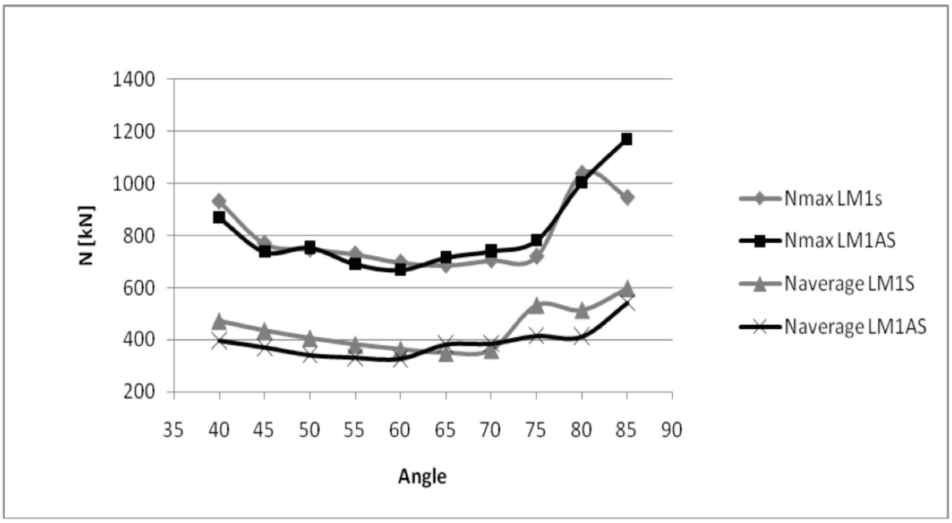

Fig. 5.11 – Axial forces in the hangers

In Figure 5.11, a trough is shown for an angle range between 50º and 70º, which could potentially represent a good design range. Nevertheless one must bear in mind that these results, as mentioned before, are not totally accurate due to the hanger relaxation. In fact, taking into account the relaxed hangers, an angle higher than 60º is not be recommended.

5.4.2.2. Arch

Figure 5.12 confirms that the maximum compression force tends to decrease with steeper hangers, which can be explained by the steeper hangers being less tensioned, due to a small horizontal component. The most efficient results are obtained for angles between 60º and 80º.

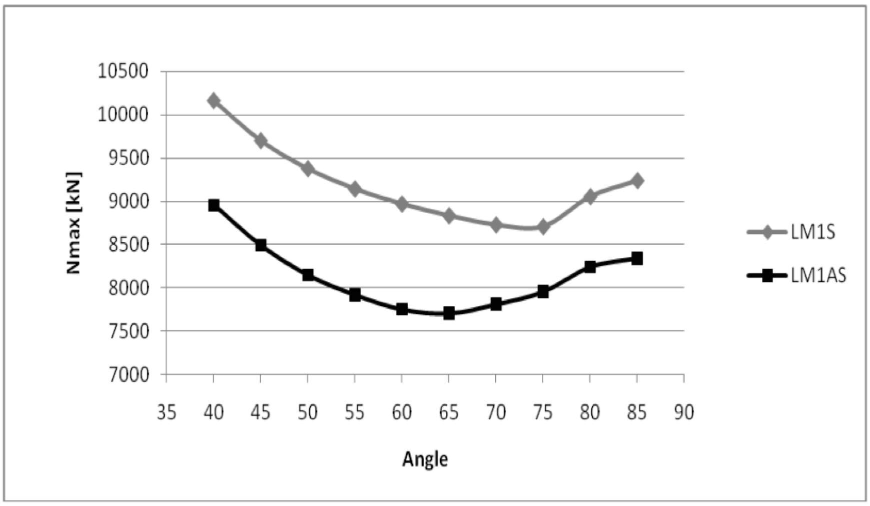

Fig. 5.12 – Axial Forces in the arches

The bending moments exhibited in Figure 5.13 already include the removal of relaxed hangers. Removing a large number of hangers can result in high bending moments, which occurs for angles between 80º and 85º. In the 80º configuration a large amount of relaxed hangers in a part of the bridge leads to a great bending moment. In the 85º configuration, the relaxed hangers are more "dispersed" than in the 80º configuration, resulting in smaller maximum bending moments.

It must be stated that the hanger arrangement for this configurations is no longer a network one, due to the fact that there is not more than one intersection point between the hangers. It is interesting to note that when looking at the maximum bending moment on the base load combinations, one finds that both symmetric and asymmetric bending moments are almost equal. There are differences between the force distributions along the bridge for each combination, but the maximum bending moment in the arch seems to be insensitive to them.

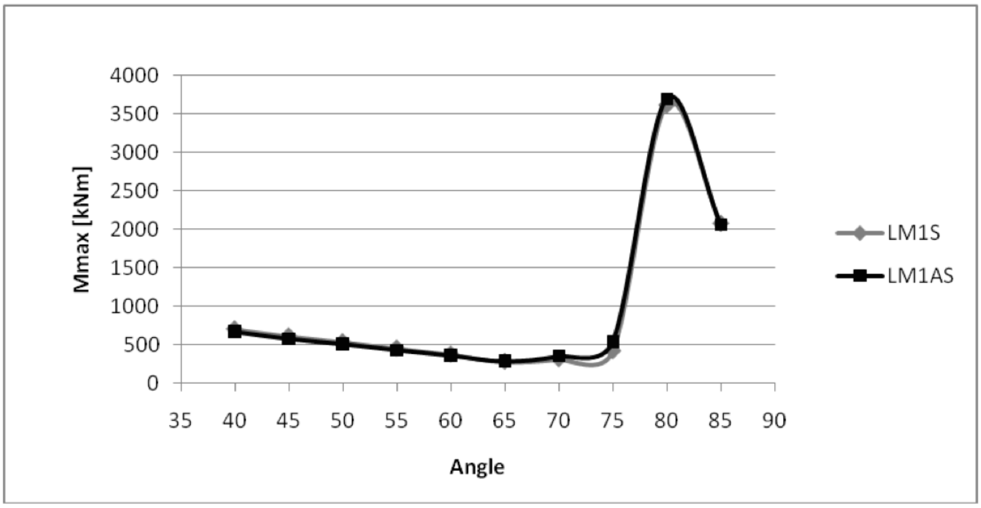

Fig. 5.13 – Bending moments in the arches

5.4.2.3. Tie

The maximum axial forces in the tie are presented in Figure 5.14. As a general rule, the maximum axial force tends to increase when steep hangers are considered. When the angle approaches 90º, the vertical configuration is no longer a network configuration, over 80º, the maximum axial forces tend to get closer to those obtained for the vertical hanger configuration.

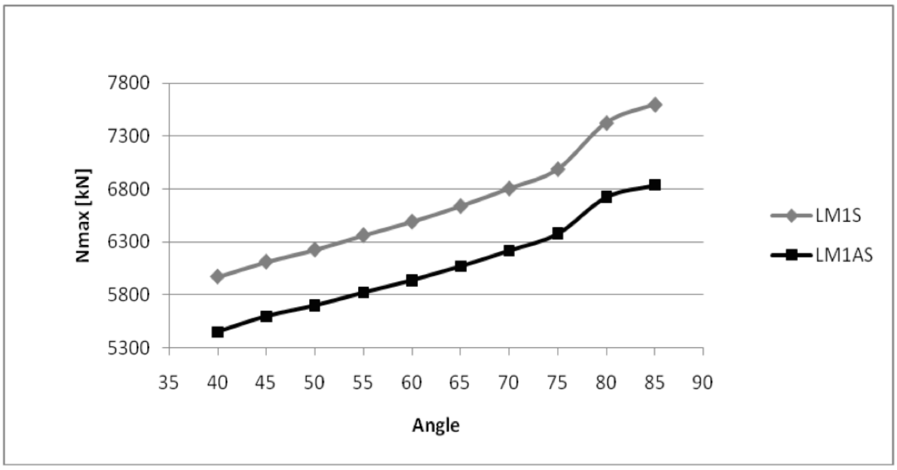

Fig. 5.14 – Axial forces in the tie

The maximum bending moments in the tie seem to be influenced only marginally by the angle of the hanger configuration till angle of 70º, as Figure 5.15 illustrates. When the number of relaxed hangers increases, there is a tendency for an increase of bending moment, as expected.

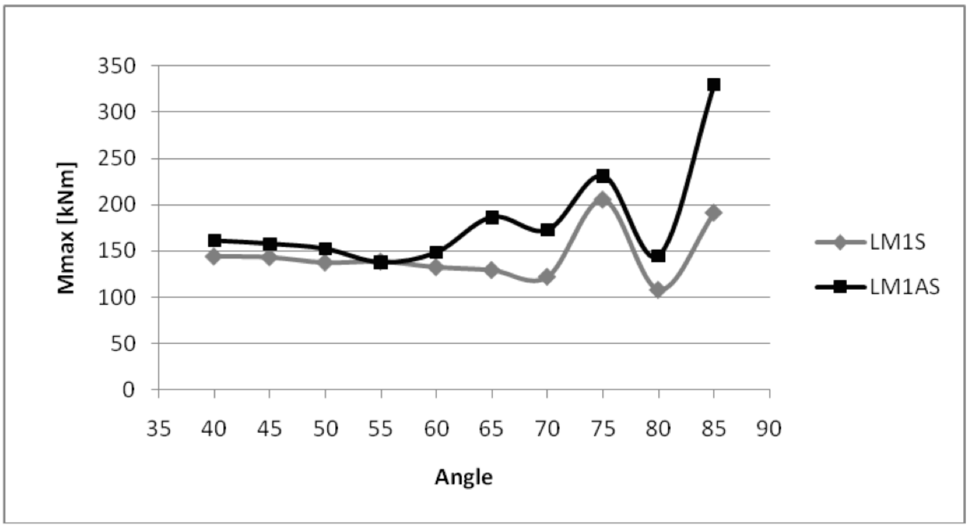

Fig. 5.15 – Bending moments in the tie

When employing hangers with 80º or more, thus a non-network configuration, as in the arch, a disturbance range can be observed, where bending moments can increase significantly.

5.4.2.4. Bridge Weight

The minimum weight for the arch and tie are presented in Tables 5.4 and 5.5.

Table 5.4 – Minimum weight of the arch

Angle	40	45	50	55	60	65	70	75	80	85
Min Weight (ton)	34,54	30,64	28,86	28,86	25,61	25,61	25,61	25,61	62,43	45,79
Cross-section	900x30	800x30	900x25	900x25	800x25	800x25	800x25	800x25	900x55	900x40

Table 5.5 – Minimum weight of the tie

Angle	40	45	50	55	60	65	70	75	80	85
Min Weight (ton)	17,42	17,42	17,42	17,42	17,42	17,42	17,42	17,95	18,93	20,55
Cross-section	900x15	900x15	900x15	900x15	900x15	900x15	900x15	700x20	500x30	800x20

The most efficient solution when it comes to total weight is obtained for a slope of 70º, as can be seen in Table 5.6. Nonetheless, it was stated before that configurations with an angle higher than 60º present severe problems on hanger relaxation. Thus, only solutions with less than 60º are considered, where the actual 60º configuration stands out as the most efficient one.

Table 5.6 – Total weight

Angle	40	45	50	55	60	65	70	75	80	85
Min Weight Arch (ton)	69,08	61,27	57,73	57,73	51,22	51,22	51,22	51,22	124,85	91,58
Min Weight Tie (ton)	34,83	34,83	34,83	34,83	34,83	34,83	34,83	35,90	37,86	41,11
Hangers Weight (ton)	30,21	27,46	25,35	23,71	22,42	21,43	20,66	20,10	19,72	19,49
Total Weight (ton)	134,12	123,56	117,91	116,27	108,47	107,48	106,71	107,23	182,43	152,18

5.4.3 Final Remarks

The results presented for the “classic” Tveit configuration serve to illustrate what was already known about arch network bridges already: the lighter a bridge is, the steeper hangers it requires and the more of those hangers are found to be relaxed.

The most efficient solution, solely taking into account the minimum weight criterion, cannot be adopted. To reach an efficient solution, more sophisticated models should be used, such as cable elements instead of bar members to model the hangers. Also, a fatigue assessment should be performed in order to identify the most efficient configuration is in the hanger arrangement.

Nevertheless, as shown in Table 5.7, the constant slope configuration results in an almost 44% lighter solution in comparison with the vertical configuration.

Table 5.7 – Weight comparison

Most Efficient Solution	Weight (ton)	Improvement
Vertical	194,38	--
Constant Slope	108,47	44,2%

5.5. Variable Slope Configuration

5.5.1. Model Description

The constant slope configuration is a particular case of the variable slope case. In the latter configuration, each set of hangers has a starting angle, which evolves into a positive or negative angle variation towards the other extreme of the span.

The starting angles considered range from 55º to 70º, the angle variation being 0.5º, 0.8º, 1.0º, positive and negative. Two examples of this network web are provided in Figures 5.16 and 5.17.

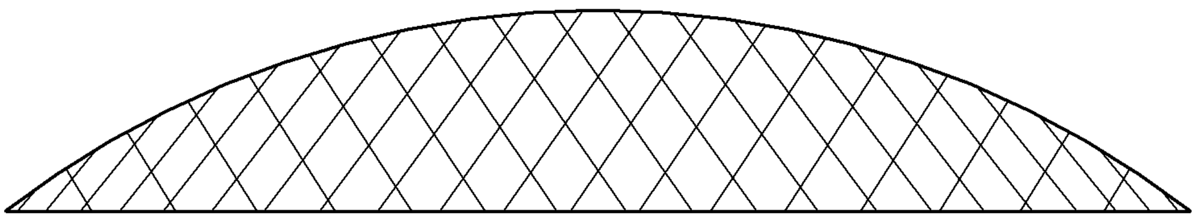

Fig. 5.16 – Starting angle of 55º, with a positive variation of 0.5º per hanger

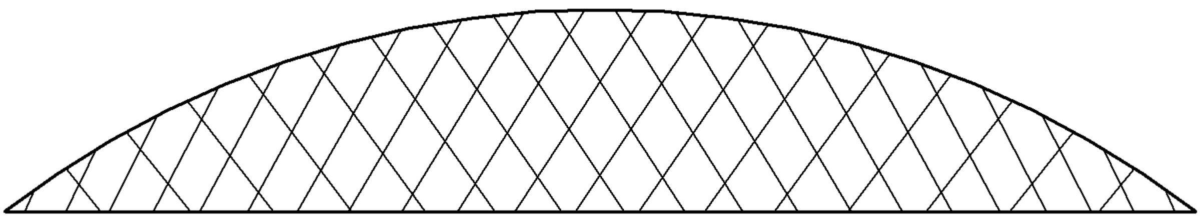

Fig. 5.17 – Starting angle of 65º, with a negative variation of 0.8º per hanger

In the discussion of results, the values of the constant slope configuration are added as null angle variation configurations.

5.5.2 Discussion of the Results

5.5.2.1. Hangers

As shown in Figures 5.18 and 5.19, there is not a single configuration for which relaxed hangers are found.

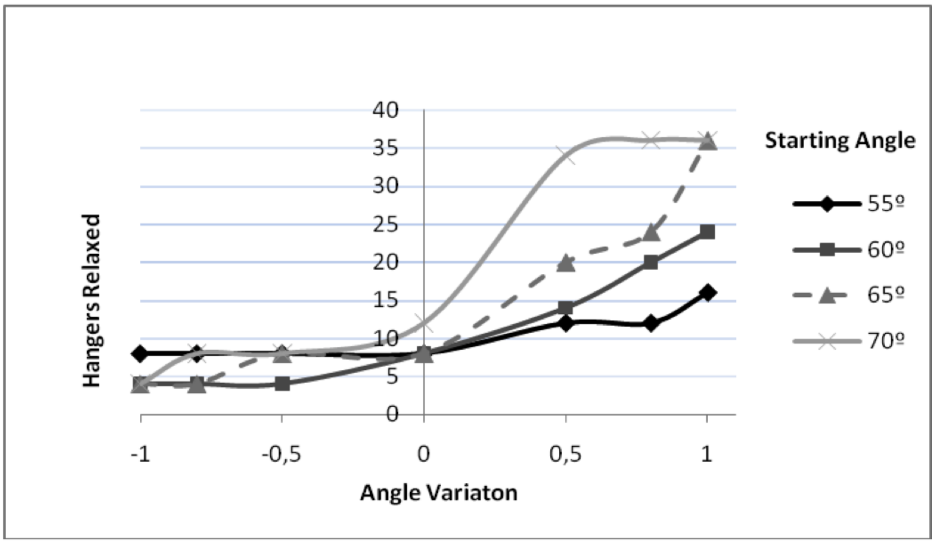

Fig. 5.18 – Relaxed hangers – LM1S leading

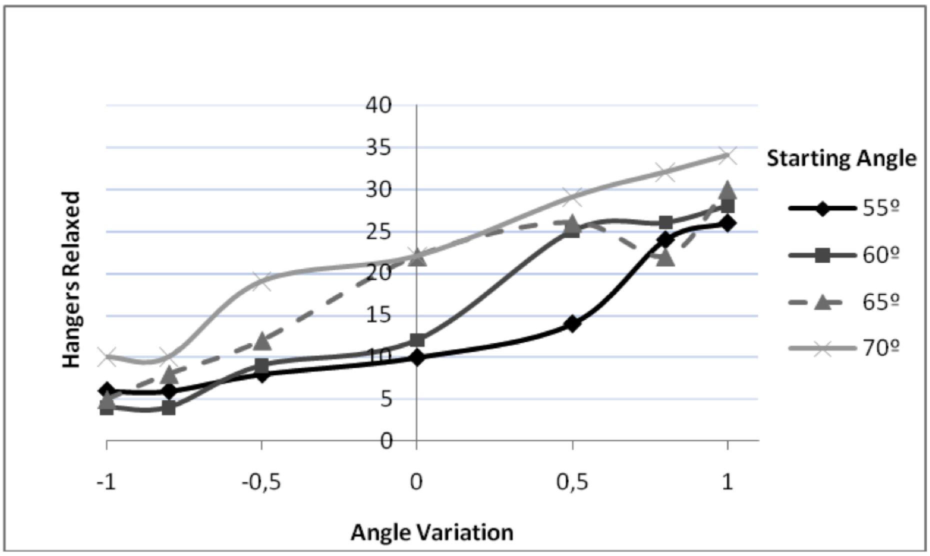

Fig. 5.19 – Relaxed hangers – LM1AS leading

A negative angle variation, where hangers are allocated less and less steep, results in a smaller number of relaxed hangers. This amounts to being the best result for a configuration with a starting angle of 60º. Generally, a positive angle variation leads to more relaxed hangers.

In some of the configurations, such as the ones with a starting angle of 70º, removing the relaxed hangers would result in even more relaxed hangers. This phenomenon is quite remarkable for the 0.8º and 1.0º positive angle variations, where the hanger arrangement is no longer Network but Nielsen.

Regarding the maximum axial forces, in the symmetric loading model, the values show that a variation of the starting angle leads to worse results than the ones obtained with no angle variation.

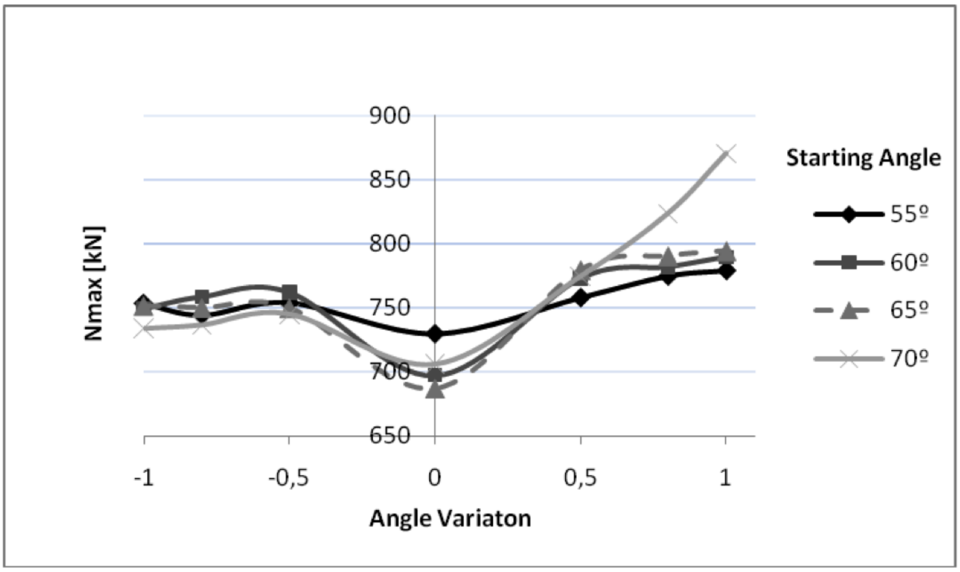

Fig. 5.20 – Axial forces in the arches – LM1S leading

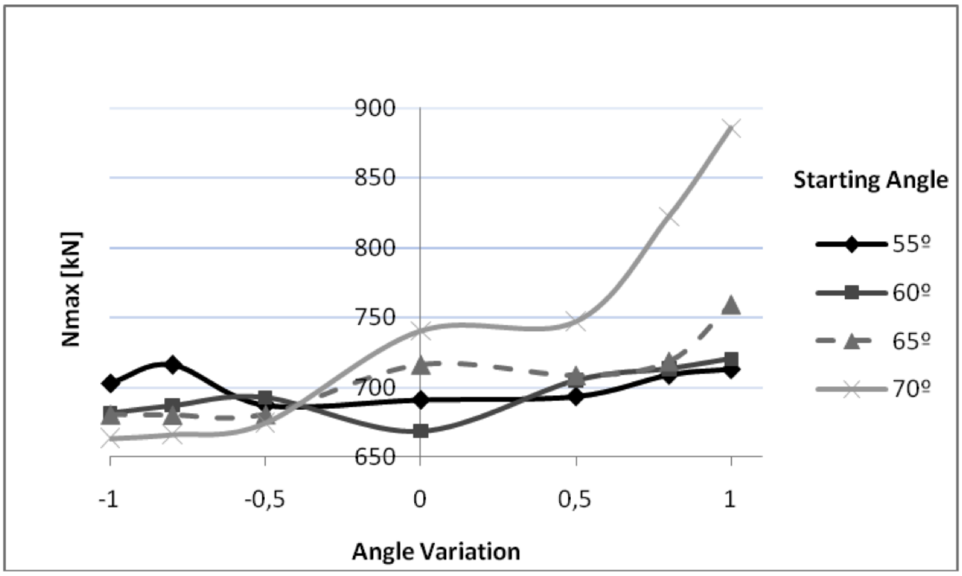

Fig. 5.21 – Axial forces in the arches – LM1AS leading

5.5.2.2. Arch

Figures 5.22 indicate that the maximum axial force in the arches tends to be lower when the hangers become steeper. Nevertheless, the force variation is not greater than 10%, between the best and the worst configuration.

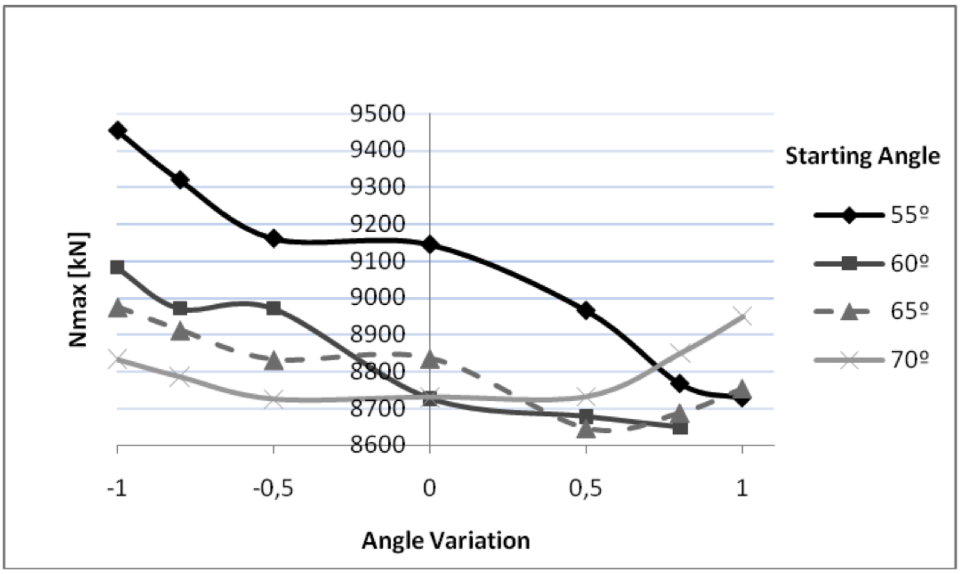

Fig. 5.22 – Axial forces in the arches – LM1S

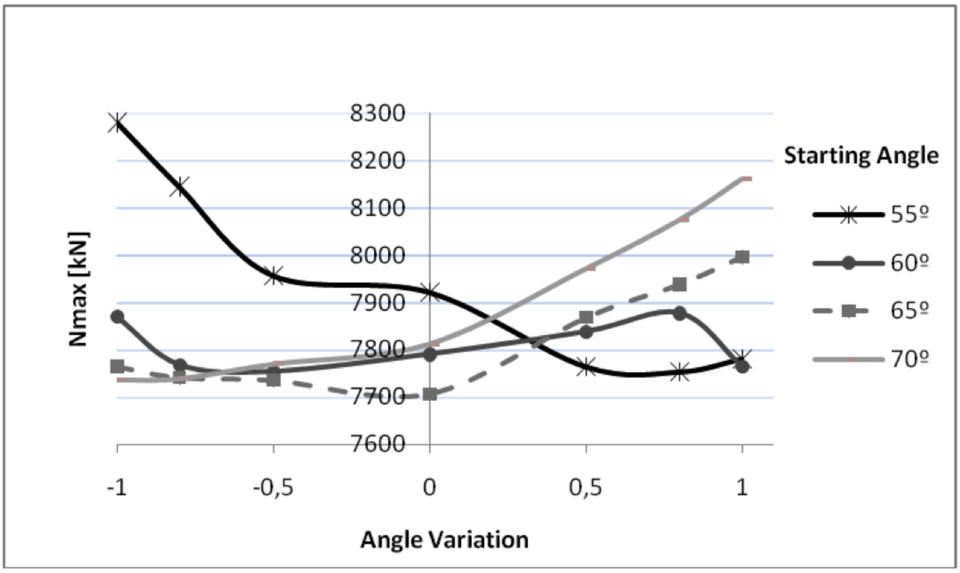

Fig. 5.23 – Axial forces in the arches – LM1AS

The maximum bending moments in the arches are plotted in Figures 5.24 and 5.25. The 0º variation - constant slope configuration- exhibits the lower bending moments for each starting angle, and as a general rule, steeper starting angles lead to smaller bending moments.

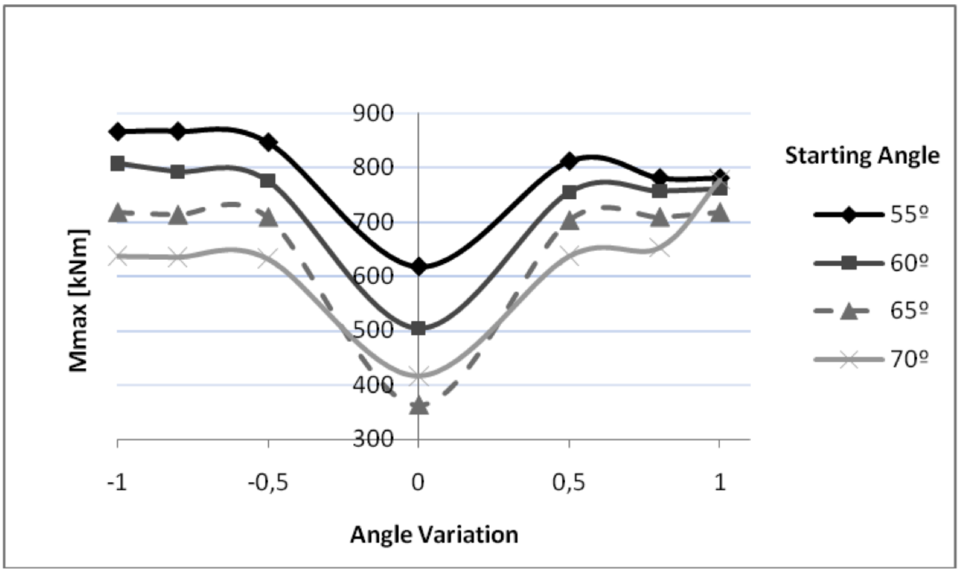

Fig. 5.24 – Bending moments in the arches – LM1S

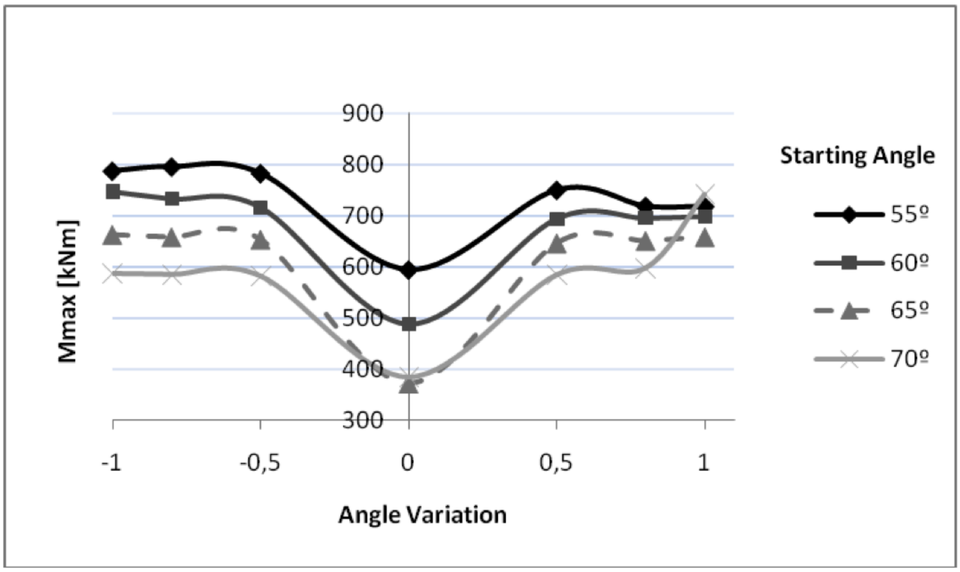

Fig. 5.25 – Bending moments in the arches – LM1AS

5.5.2.3. Tie

In Figures 5.26 and 5.27 can be seen that smaller starting angles generally lead to smaller hanger forces. The variation of the angle of the hangers does not seem to produce better results than a non variation.

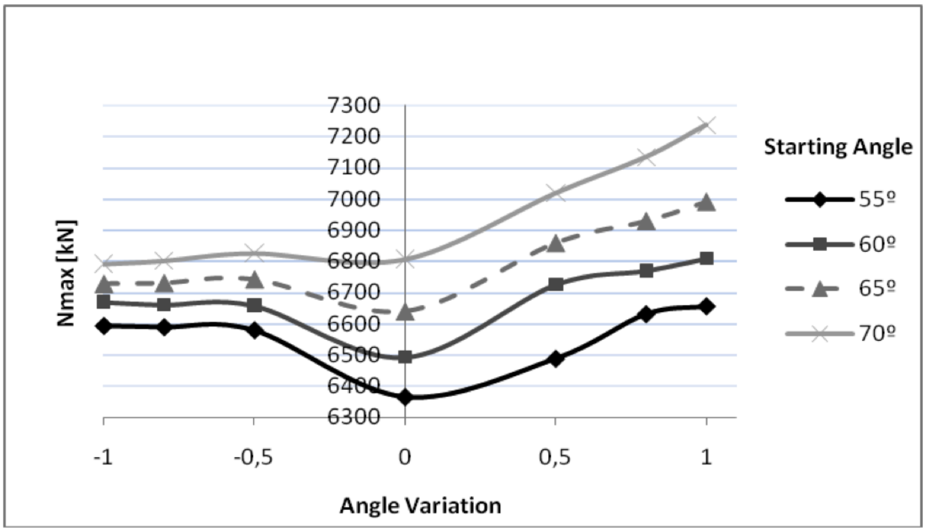

Fig. 5.26 – Axial forces in the tie – LM1S

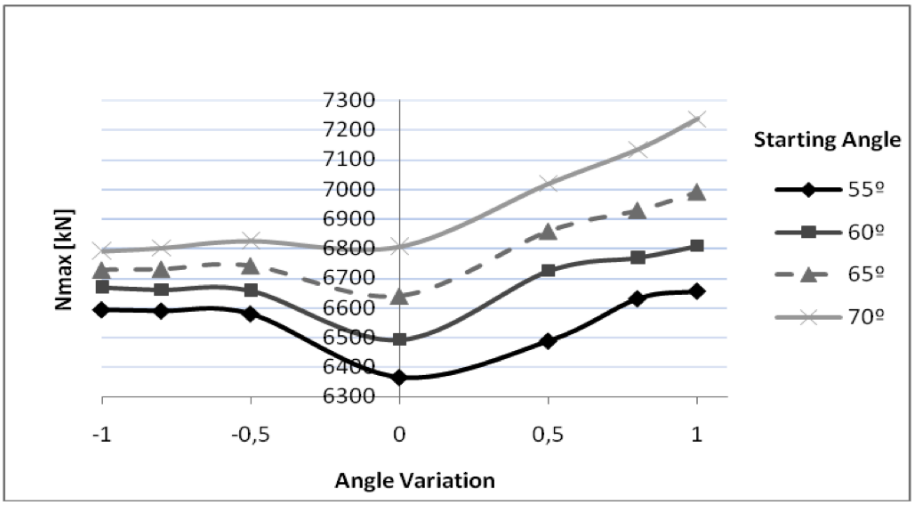

Fig. 5.27 – Axial forces in the tie – LM1AS

The maximum bending moments along the tie exhibit values close to the ones present in the arch. Moreover, it can be noted that the steeper the starting angle is, the smaller the tendency for higher bending to occur becomes.

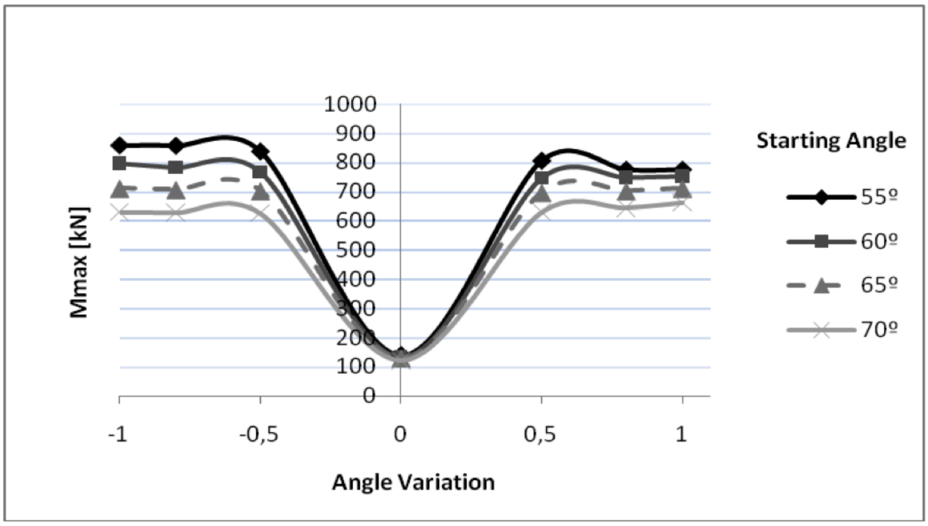

Fig. 5.28 – Bending moments in the tie – LM1S

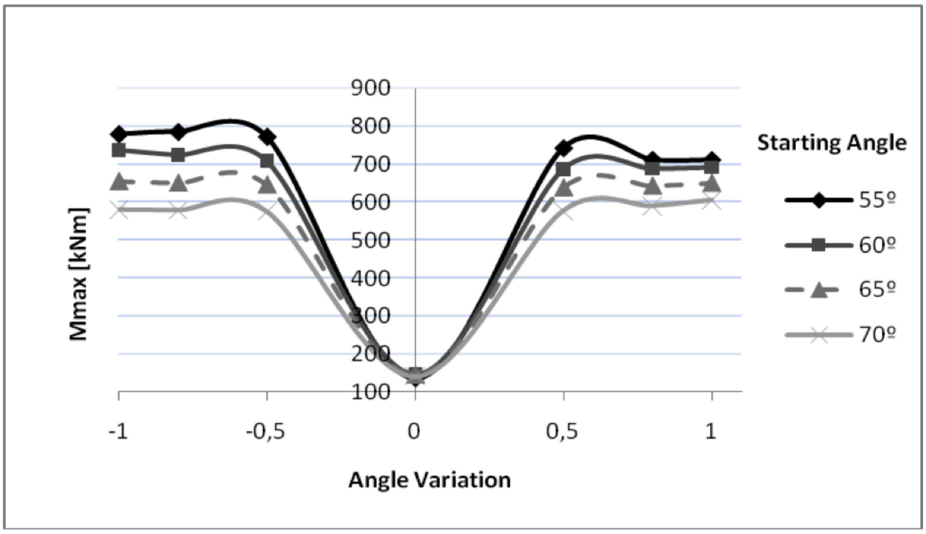

Fig. 5.29 – Bending moments in the tie – LM1AS

Similarly to the bending moments for the arch, the constant slope configuration appears to show better results than a model where a variation of angle for each hanger is adopted.

5.5.2.4. Bridge weight

The minimum weight for each configuration starting angle, for both arch and tie, is presented in Tables 5.8 and 5.9. The constant slope configuration is omitted and the results only comply to configurations with an angle variation.

Table 5.8 – Minimum weight of the arch

Angle	55°	60°	65°	70°
Min Weight (ton)	28,86	28,86	28,86	28,86
Cross-section	900x25	900x25	900x25	900x25

Table 5.9 – Minimum weight of the tie

Angle	55°	60°	65°	70°
Min Weight (ton)	23,16	23,16	23,16	23,16
Cross-section	900x20	900x20	900x20	900x20

It can be seen that the starting point has no influence in the total weight of the bridge; similar observations have been made for almost every configuration studied. Thus, the hanger arrangement is truly the decisive factor when it comes to determining which configuration is most efficient.

Table 5.10 – Total Weight

Angle	55º-0.5	60º-0.5	65º-0.8	70º-0.8
Total Weight (ton)	130,05	128,38	127,81	125,03

5.5.3. Final Remarks

Regardless of the details of a given configuration, positive variable angles cannot be accepted since they lead to excessive hanger relaxation. Compared with the constant slope configuration, which is a particular case of this one, the results are worse and thus not justifying the complexity that this model requires.

Generally, positive angle variations on this model cannot be accepted because they lead to a significant number of relaxed hangers, even when manually changing the position of the first hangers. Considering configurations with a starting angle of 65º or 70º, only negative variables angles of 1.0 or 0.8 can be accepted, due to an excessive number of relaxed hangers found in the other configurations. Nevertheless, this configuration represents an improvement of 34% on the bridge total weight.

Table 5.11 – Weight comparison

Most Efficient Solution	Weight (ton)	Improvement
Vertical	194,38	--
Constant Slope	128,38	34,0%

5.6. ADVANCED HANGER CONFIGURATION

5.6.1. MODEL DESCRIPTION

In this model, the first intersection of two hangers lies on the radii of the arch circle. The cross angle is half of the angle defined by the two hangers, or between a hanger and fictitious line towards the radii of the arch circle.

The range of cross angles varies from 0º, the “spoked wheel” model, and 50º. Three examples of this configuration are presented in the following figures.

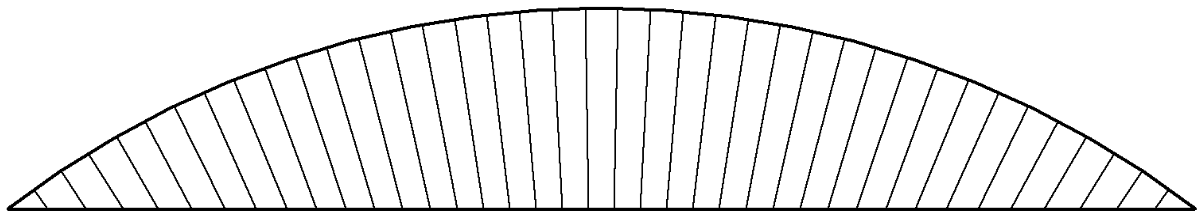

Fig. 5.30 – Cross angle of 0º, the “spoked” wheel model

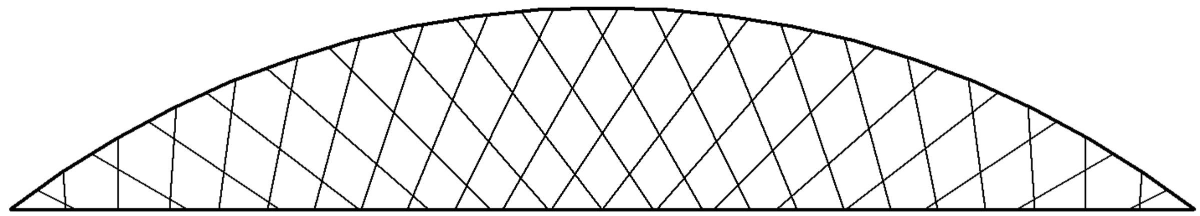

Fig. 5.31 – Cross angle of 30º

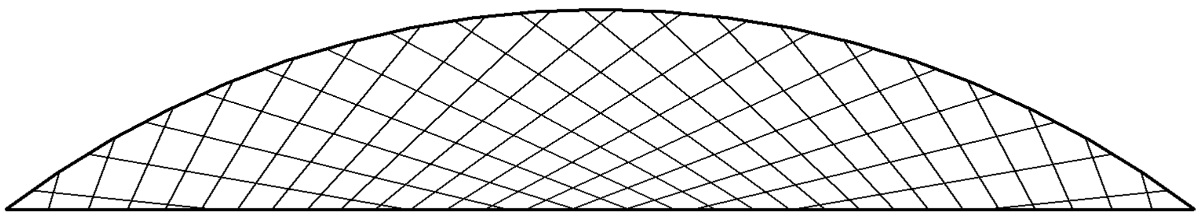

Fig. 5.32 – Cross angle of 50º

5.6.2 DISCUSSION OF THE RESULTS

5.6.2.1. Hangers

In this model, the hangers are steeper for lower angles. For the maximum axial forces, two valleys of minima can be identified, between 15º and 25º, and between 35º and 45º.

No relaxed hangers are detected when the symmetric Load Model 1 is leading, while plenty of hangers can be found relaxed, when the asymmetric Load Model 1 is leading, between 5º and 15º. In a way, the relaxed hangers are responsible for a distribution of forces that leads to higher maximum axial forces, like in the configuration with 5º, which has a maximum of 21 relaxed hangers. The configurations between 5º and 10º are not of the network type, while in the configuration of 15º only some hangers, at mid span, cross each other more than once.

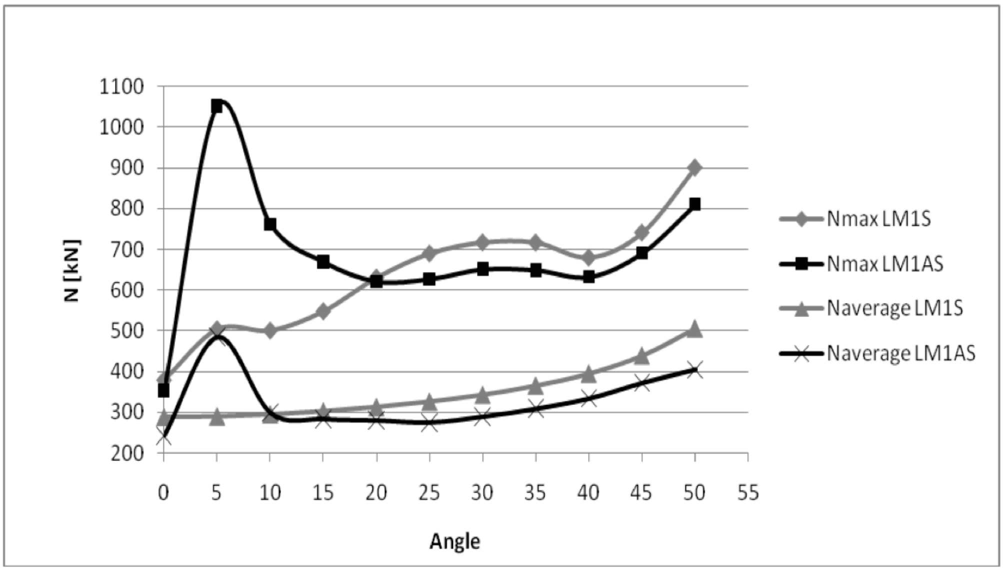

Fig. 5.33 – Maximum axial forces in the hangers

It can therefore be concluded that the most preferable configuration -when only considering the minimum tension in the hangers- is the "spoked wheel" model (0º angle).

5.6.2.2. Arch

The maximum compression in the arch tends to increase when the hangers become less and less steep (Figure 5.34), which occurs for bigger cross angles. The smallest maximum compression, in every combination, is obtained for the "spoked wheel" model.

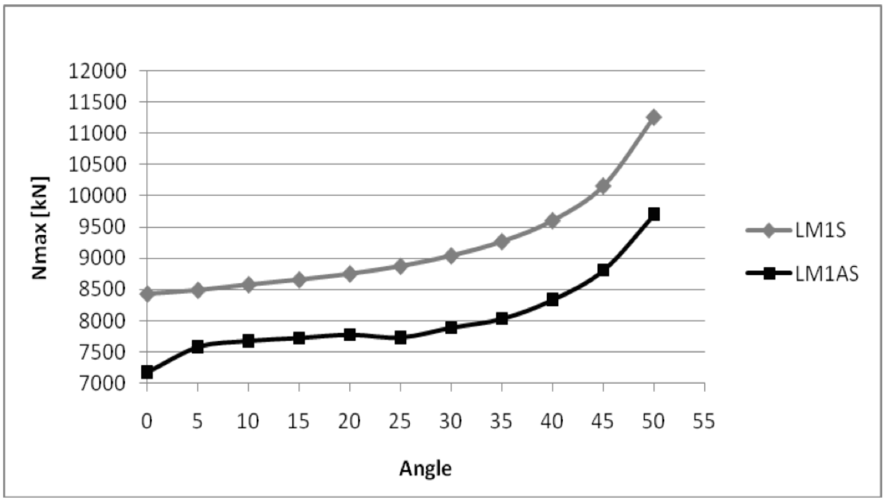

Fig. 5.34 – Axial forces in the arches

The situation is inverted for maximum bending moments. The worst results are obtained for the "spoked wheel", with bending moments up to ten times higher than the ones registered for angles between 15º and 30º. As in the vertical configurations, the maximum bending moments occur when the asymmetric Load Model 1 is leading, which contributes to demonstrate the inadequate capacity of this kind of hanger arrangement to deal with live loads.

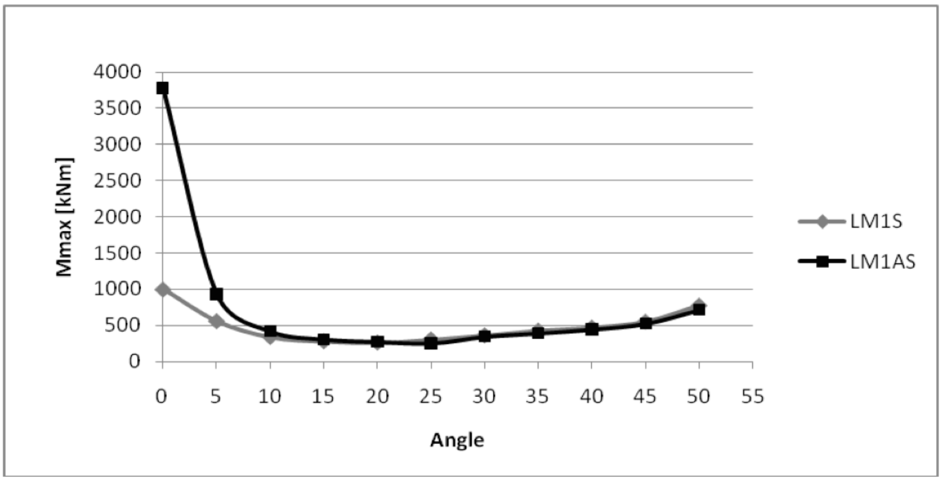

Fig. 5.35 – Bending Moments in the arches

The best results are obtained for angles between 10º and 35º, though bending moments for configurations with angles wider than 35º up to 50º, have little bending moment when compared with configurations in which the hangers are not of the network type.

5.6.2.3. Tie

Figure 5.36 shows that the best results for maximum axial forces in the tie are obtained for cross angles between 15º and 45º.

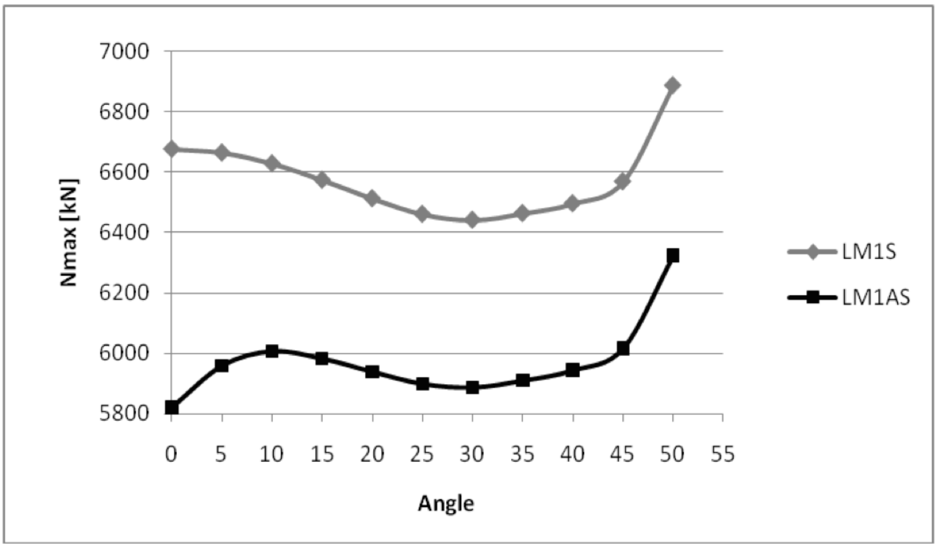

Fig. 5.36 – Axial forces in the tie

For the maximum bending moments, a similar situation to the one of the arch is observed. The worst results are obtained for the spoked wheel case and the maximum bending moments tend to decrease while the hangers become less steep.

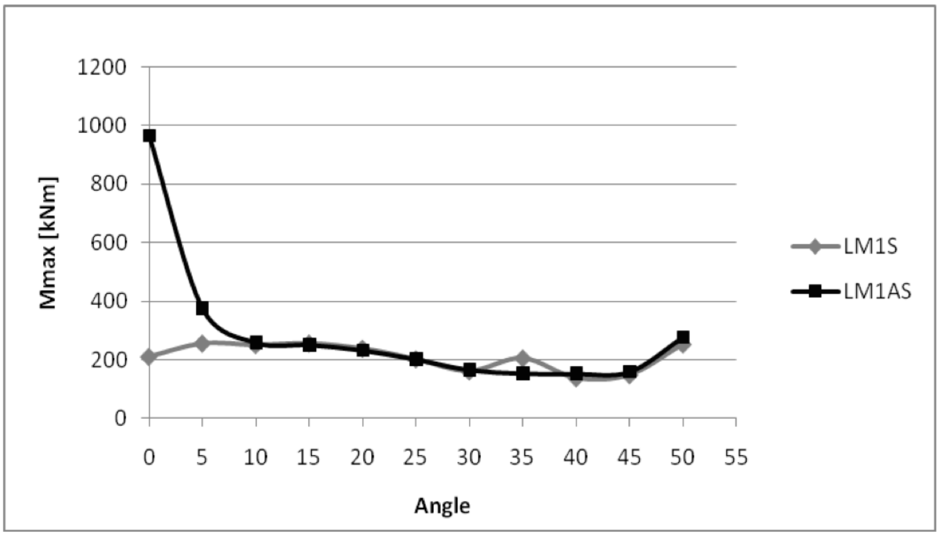

Fig. 5.37 – Bending moments in the tie

Like in the arch, the leading combination, after 10º, seems to have little influence on the maximum bending moments in the tie, with the two diagrams almost overlapping.

5.6.2.4. Bridge Weight

The weight of the arch and the tie, and cross-sections required, for each configuration are presented in Tables 5.12 and 5.13.

Table 5.12 – Minimum weight on the arch

Angle	0	5	10	15	20	25	30	35	40	45	50
Min Weight (ton)	62,43	28,86	25,61	23,16	23,16	25,61	25,61	28,86	28,86	30,64	39,66
Cross-section	900x55	900x25	800x25	900x20	900x20	800x25	800x25	900x25	900x25	800x30	700x45

Table 5.13 – Minimum weight on the tie

Angle	0	5	10	15	20	25	30	35	40	45	50
Min Weight (ton)	25,61	20,55	20,55	20,55	19,11	17,42	17,42	17,42	17,42	17,42	20,55
Cross-section	800x25	800x20	800x20	800x20	600x25	900x15	900x15	900x15	900x15	900x15	800x20

When it comes to total weight, the most efficient solution can be found for a configuration of 20º. However, the webs with angles ranging 5º to 45º also exhibit competitive values.

Table 5.14 – Total weight

Angle	0	5	10	15	20	25	30	35	40	45	50
Min Weight Arches (ton)	124,85	57,73	51,22	46,31	46,31	51,22	51,22	57,73	57,73	61,27	79,32
Min Weight Ties (ton)	51,22	41,11	41,11	41,11	38,21	34,83	34,83	34,83	34,83	34,83	41,11
Weight Hangers (ton)	20,32	20,33	20,58	21,04	21,72	22,67	23,96	25,7	28,12	31,6	37,14
Total Weight (ton)	196,39	119,16	112,91	108,46	106,24	108,72	110,01	118,26	120,68	127,70	157,57

5.6.3 Final Remarks

The advanced network arrangement, developed by Brunn and Schanack is the most efficient configuration so far tested. Road bridges are lighter than rail bridges, and so steeper hangers are required. Relaxed hangers can be found, only for smaller angles, which renders this configuration rather reliable and robust. Comparing to the vertical arrangement, it represents an improvement of over 45%, as shown in the following table.

Table 5.15 – Weight comparison

Most Efficient Solution	Weight (ton)	Improvement
Vertical	194,38	--
Advanced Method	106,24	45,3%

5.7. Alternative Constant Slope Configuration

5.7.1. Model Description

The alternative constant slope configuration is a web arrangement often seen in bridges, like in the Fehmarnsund bridge. The bridge model is relatively similar to the one defined on this thesis, where the deck is defined with transverse steel beams. The main difference is that the hangers are merged next to the intersection between the transverse beams and the tie. Thus, a constant slope is obtained, wherein the angle is defined by the hanger and the horizontal. As a result, the upper node of each hanger, in the arch, is not equidistant as in the models defined before. The angles considered range from 50º to 70º. Two examples are plotted in the following figures.

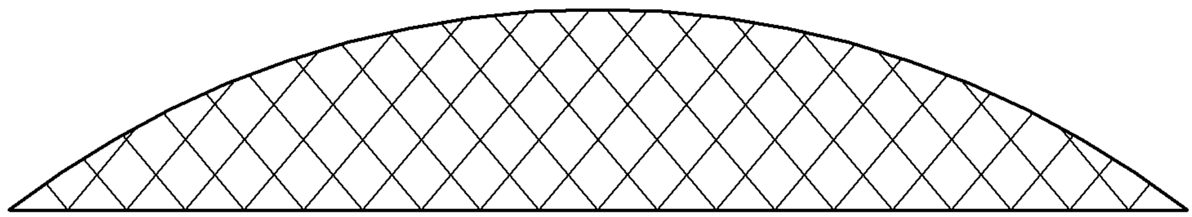

Fig. 5.38 – Arch rise of 17 m, span of 100 m, 38 hangers and 50º of slope

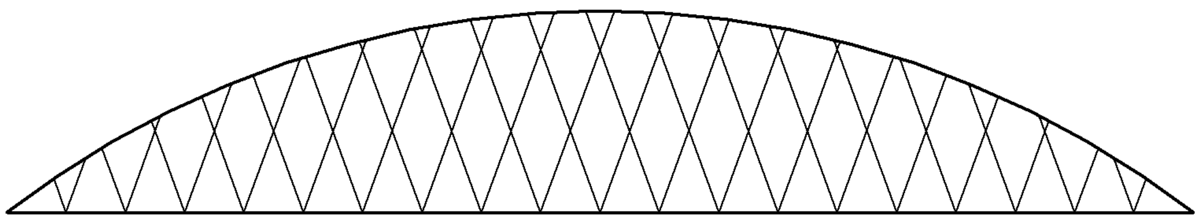

Fig. 5.39 – Arch rise of 17 m, span of 100 m, 38 hangers and 70º of slope

5.7.2. Discussion of the Results

5.7.2.1. Hangers

In this configuration two tendencies for the axial forces in the hangers can be found, as seen in Figure 5.40. In the first, steeper hangers lead to smaller maximum forces, due to the fact that a lower horizontal component is required. In the second, the number of relaxed hangers increases, and thus resulting in bigger tensile forces in the hangers close to the ones relaxed.

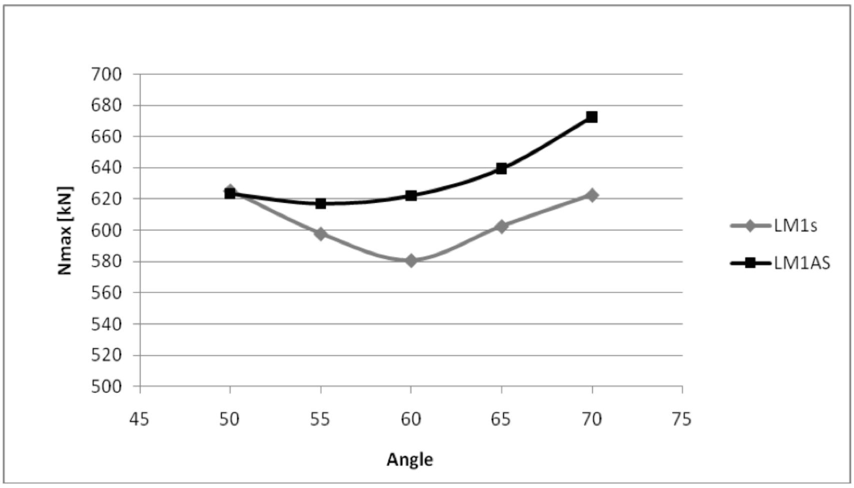

Fig. 5.40 – Maximum axial forces in the hangers

Concerning relaxed hangers, it is observed that every configuration has at least four relaxed hangers. Those are the first hanger on the birth of each arch, being relaxed in every configuration tested, due to the clamping of the arch towards the tie. When the number of relaxed hangers starts to grow, the hangers likely to relax are those ones parallel to the previously relaxed hangers.

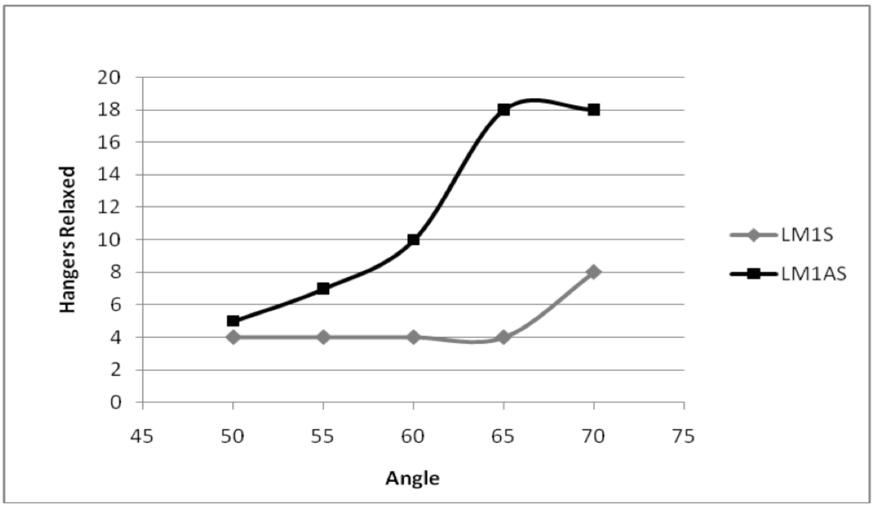

Fig. 5.41 – Number of relaxed hangers

5.7.2.2. Arch

The results for the maximum compression force in the arch are presented in Figure 5.42. Compared to the same range in the constant slope configuration – Figure 5.11 - it can be seen that it has the same tendency for lower values with steeper hangers. The extreme values stabilize at a lower value than those found in the constant slope configuration.

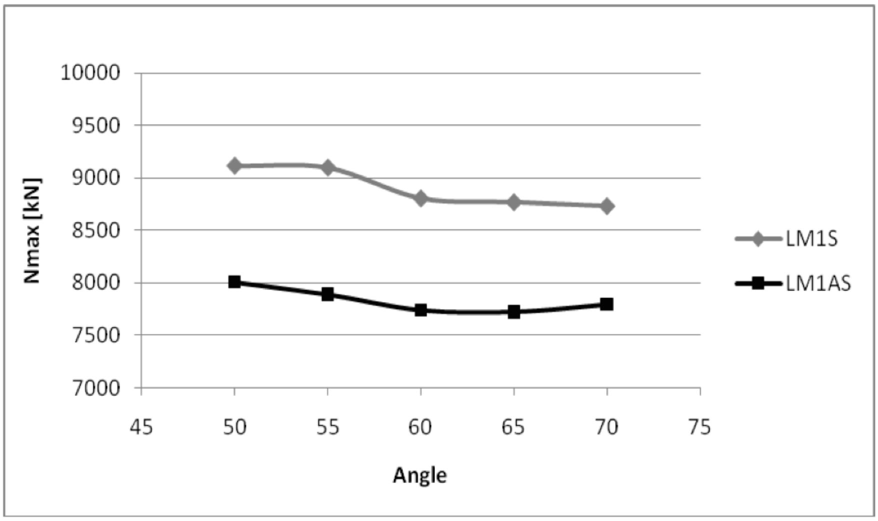

Fig. 5.42 – Axial forces in the arches

Figure 5.12, shows that the maximum values for bending moments are almost constant for a range between 50º and 70º. In this configuration, as illustrated in Figure 5.43, the variation for each configuration is at times high, thus related to the number of relaxed hangers.

It must be stated that no relaxed hangers were removed in the range between 65º and 70º. Removing relaxed hangers on those configurations would lead into more relaxed hangers. This procedure was not convergent and therefore produces non-reliable results.

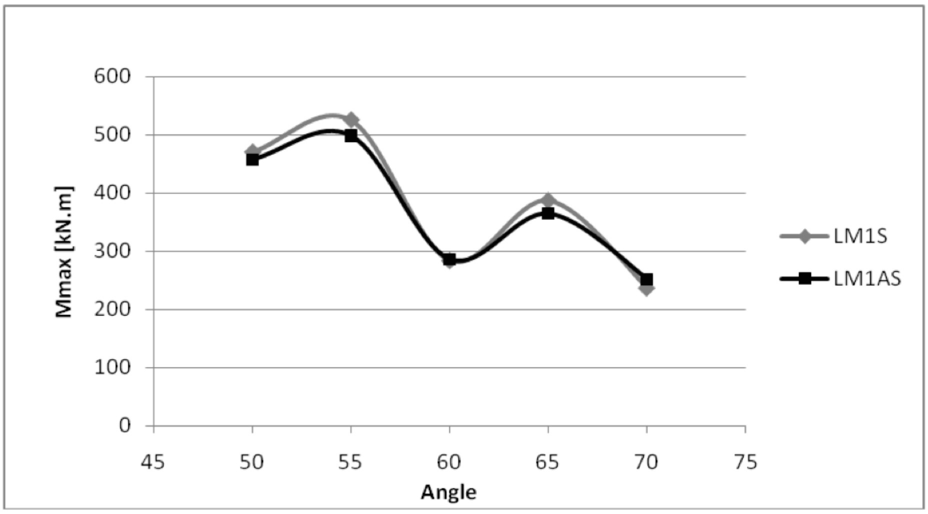

Fig. 5.43 – Bending moments in the arches

5.7.2.3. Tie

Comparing the results for the same range, plotted in Figure 5.44, with the ones exhibited in Figure 5.13, it can be stated that the maximum axial forces are smaller for the alternative constant slope configuration. After changing the position of the upper node in the first hangers, the maximum tensile force in the tie would occur next to the first hanger.

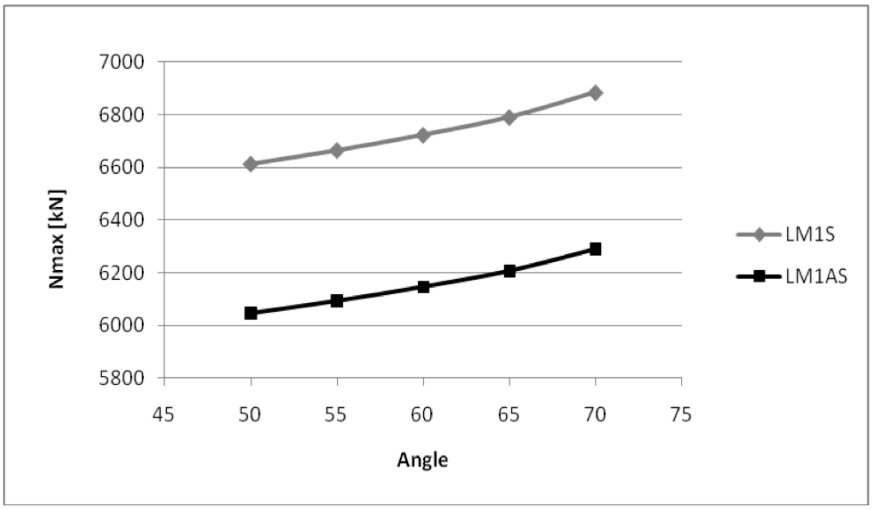

Fig. 5.44 – Axial forces in the tie

The bending moments in the tie for the alternative configuration are as small as the bending moments obtained for the constant slope configuration, as can be seen when comparing Figures 5.14 and 5.45. It becomes explicit that the bending moment has no overall influence when designing the tie for the alternative configuration.

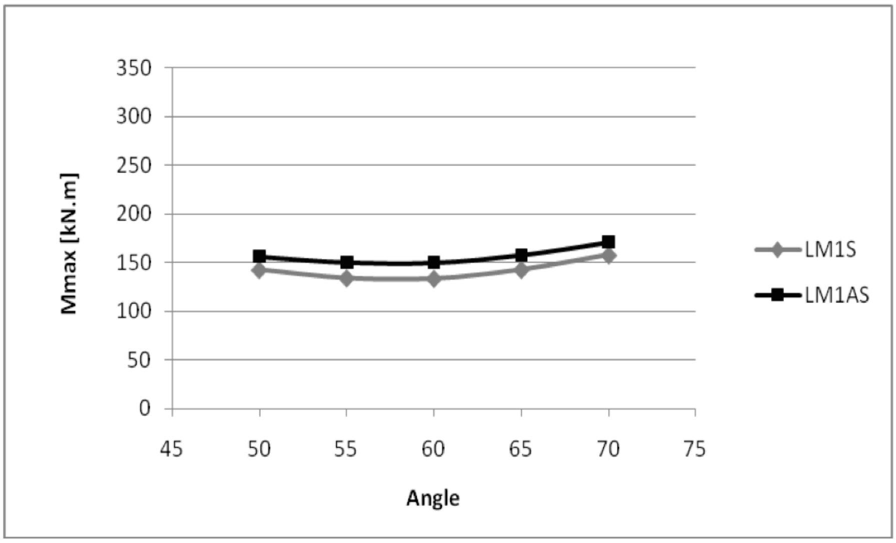

Fig. 5.45 – Bending moments in the tie

5.7.2.4. Bridge Weight

The minimum weight and respective required cross-sections for the arches and ties are presented in Tables 5.16 and 5.17.

Table 5.16 – Minimum weight for each arch

Angle	50	55	60	65	70
Min Weight (ton)	28,86	28,86	25,61	25,61	23,16
Cross-section	900x25	900x25	800x25	800x25	900x20

Table 5.17 – Minimum weight for each tie

Angle	50	55	60	65	70
Min Weight (ton)	17,42	17,42	17,42	17,42	19,11
Cross-section	900x15	900x15	900x15	900x15	600x25

The configurations with the lightest cross-section for the arches are the 60º and the 65º configurations. For the ties, the cross-section required is the same, except for the last configuration, due to the influence of a bigger tension. When adding the weight of the hangers, the best arrangement is the one obtained for 70º, being closely followed by the 60º and the 65º slope configurations.

Table 5.18 – Total weight

Angle	50	55	60	65	70
Min Weight Arches (ton)	57,73	57,73	51,22	51,22	46,31
Min Weight Tie (ton)	34,83	34,83	34,83	34,83	38,21
Hangers Weight (ton)	26,68	24,97	23,64	22,58	21,79
Total Weight (ton)	119,24	117,53	109,69	108,63	106,31

5.7.3 Final Remarks

Arguably, when using this model, it is clear that it is not possible to find a configuration without relaxed hangers. The results found in the alternative constant slope configuration are quite similar to the results obtained in the constant slope configuration, even though in the constant slope configuration more relaxed hangers are found.

The first hangers that are found relaxed in each configuration should have a different position in the upper node, manually defined for each case, or possibly even removed. With this process, better results are obtained, in spite of the fact that so far it was not possible to extrapolate a methodology applicable to all configurations of this kind, when dealing with relaxed hangers.

Considering the number of relaxed hangers present in the 70º configuration this is clearly not the most efficient configuration, less so when one takes into consideration that no convergence was found when removing the relaxed hangers. Thus, the 55º configuration, where the position of the first hangers is changed in order to not obtain relaxed hangers, is taken as the most efficient solution. The improvement of this configuration results in about 40%.

Table 5.19– Weight comparison

Most Efficient Solution	Weight (ton)	Improvement
Vertical	196,74	--
Constant Slope	117,53	40,3%

5.8 VARIATION OF THE NUMBER OF HANGERS

5.8.1. MODEL DESCRIPTION

For the bridge model adopted, with an arch rise of 17 m, span of 100 m and 38 hangers, the advanced hanger configuration has produced the best results.

This model explores the influence of the number of hangers in the advanced configuration, with a cross angle of 30º. While a configuration of 30º does not produce the lightest results, it does guarantee that no hangers are found relaxed, thereby leading to a very reliable set-up. In following Figures, two hanger arrangements, with 20 and with 50 hangers respectively, are presented as examples.

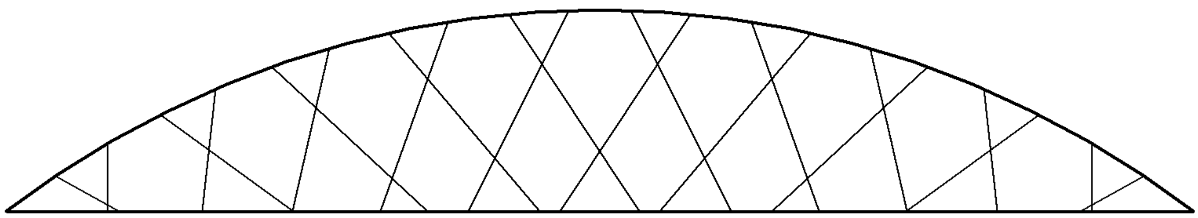

Fig. 5.46 – Cross angle of 30º with 20 hangers

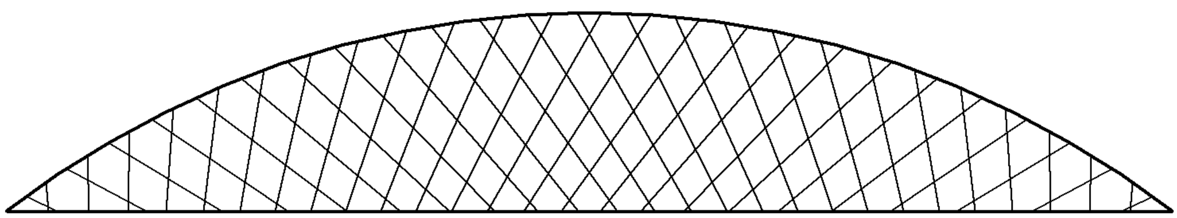

Fig. 5.47 – Cross angle of 30º with 50 hangers

5.8.2 Discussion of the Results

5.8.2.1. Hangers

For all calculations up to the 50 hangers configuration, no hangers are found in relaxed state. The number of relaxed hangers in that model is two. As it is illustrated in Figure 5.48, the maximum tension does not vary intensively when using more than 28 hangers per set. The average tension decreases when increasing the number of hangers, as expected.

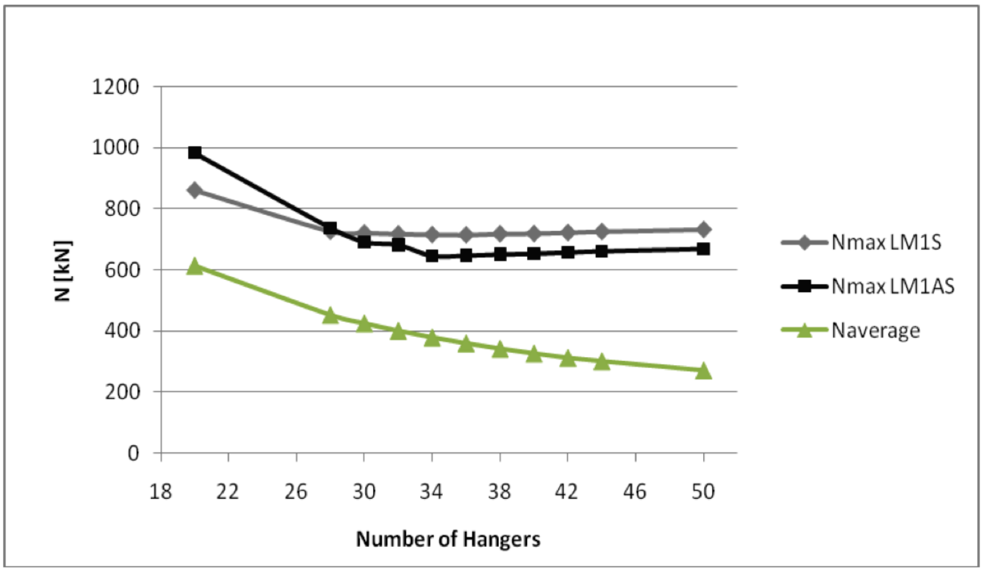

Fig. 5.48 – Maximum axial forces in the hangers

5.8.2.2. Arch

The maximum value of compression for the arch occurs when the symmetric LM1 is leading, as presented in Figure 5.49. The variation of the maximum compression in all the configurations is not significant.

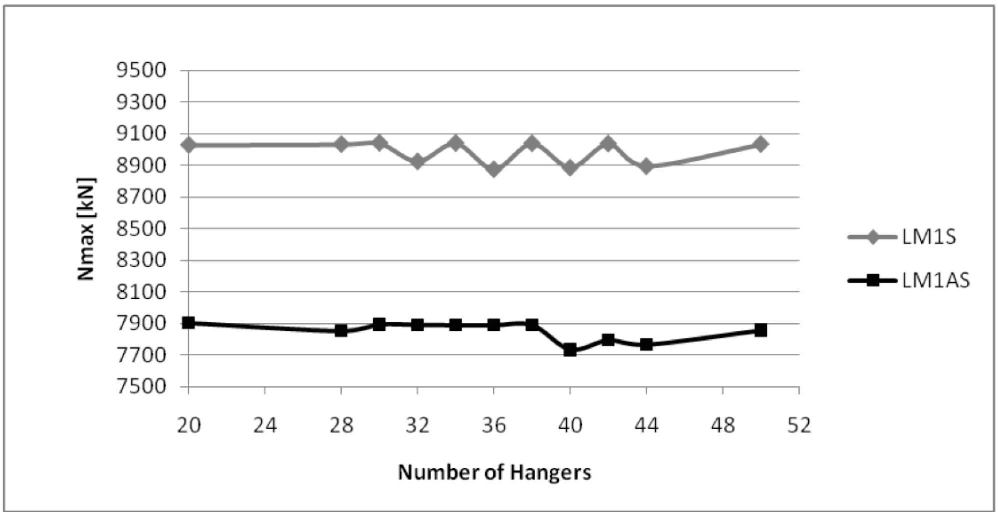

Fig. 5.49 – Axial forces in the arches

Figure 5.50 shows that the maximum bending moments have a decreasing tendency. However, it is also observable that local minima are achieved between 28 and 44 hangers, when half of the number of the hangers set is an odd number, for instance 15, 17, 19, 21.

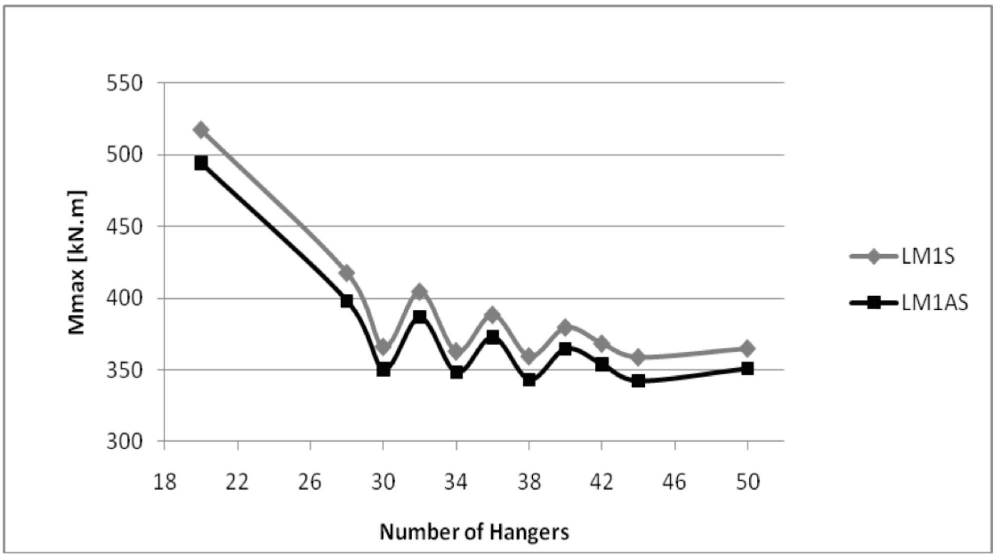

Fig. 5.50 – Bending moments in the arches

5.8.2.3. Tie

Confirming the expectations, increasing the number of hangers results in smaller maximum tensions along the tie, as shown in Figure 5.51.

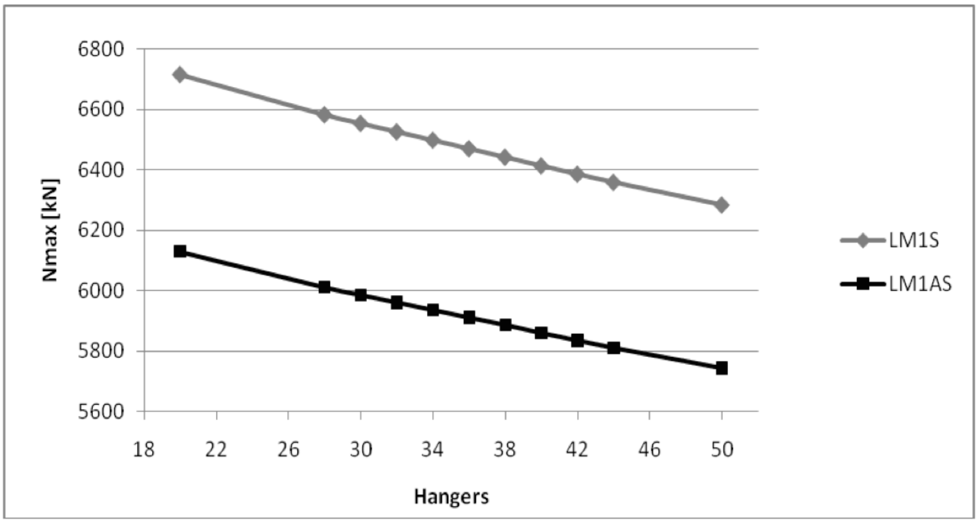

Fig. 5.51 – Axial forces in the tie

In Figure 5.52 a minimum for maximum bending moments can be identified for an arrangement with 30 hangers. However, the values at stake, when using more than 28 hangers, are all smaller than 200 kN.m, which is indeed a small value when compared to the values registered for any vertical solution. Thus, only a small percentage of the bending resistance of the cross-section is required to fulfil the resistance in tension, in ULS.

Figure 5.52 shows that the response of the tie in terms of bending moments to the asymmetric LM1 and to the symmetric LM1 is basically the same. This confirms the expectations that this type of hanger configuration does not allow substantial bending moments to develop along the tie.

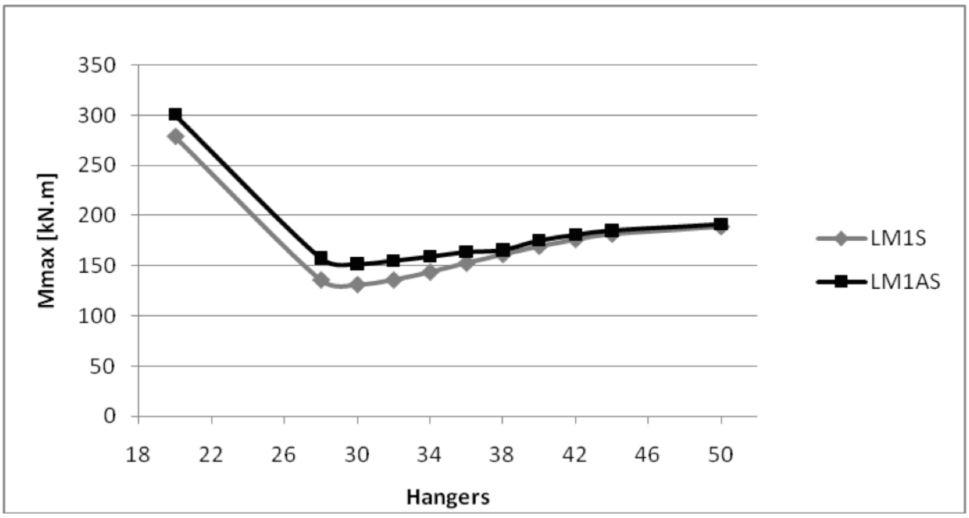

Fig. 5.52 – Bending moments in the tie

5.8.2.4. Bridge Weight

The weights and cross-sections required to fulfil the resistance in ultimate limit states are presented in Tables 5.20 and 5.21.

Table 5.20 – Minimum weight for each arch

Number of Hangers	20	28	30	32	34	36	38	40	42	44	50
Min Weight (ton)	28,86	28,86	25,61	26,73	25,61	25,61	25,61	25,61	25,61	25,61	25,61
Cross-section	900x25	900x25	800x25	700x30	800x25	800x25	800x25	800x25	800x25	800x25	800x25

Table 5.21 – Minimum weight for each tie

Number of Hangers	20	28	30	32	34	36	38	40	42	44	50
Min Weight (ton)	20,55	17,42	17,42	17,42	17,42	17,42	17,42	17,42	17,42	17,42	17,42
Cross-section	800x20	900x15	900x15	900x15	900x15	900x15	900x15	900x15	900x15	900x15	900x15

In the tie, any hanger arrangement with more than 28 hangers leads to cross-sections that are only conditioned by the tension in the element. In the arches, any configuration with more than 30 hangers –except the one with 32- will lead to a cross-section that weights 25,61 ton per arch.

Table 5.22 – Total weight

Number of Hangers	20	28	30	32	34	36	38	40	42	44	50
Min Weight Arches (ton)	57,73	57,73	51,22	53,47	51,22	51,22	51,22	51,22	51,22	51,22	51,22
Min Weight Ties (ton)	41,11	34,83	34,83	34,83	34,83	34,83	34,83	34,83	34,83	34,83	34,83
Weight Hangers (ton)	12,74	17,73	18,97	20,22	21,47	22,71	23,96	25,21	26,46	27,71	31,45
Total Weight (ton)	111,57	110,29	105,02	108,52	107,52	108,76	110,01	111,26	112,51	113,76	117,50

Considering the weight of the hangers, which increases with the number of hangers adopted, the most efficient solution is obtained for a hanger arrangement with 30 hangers.

5.8.3 Final Remarks

At this point of the research, the advanced hanger configuration proved to be the most efficient hanger configuration. Almost no relaxed hangers were found, except for the arrangement with 50 hangers, confirming the robustness of this configuration.

Increasing the number of hangers leads to smaller tension in the tie due to the increase of stiffness provided by the web.

Though the most efficient solution was obtained with 30 hangers, it should be underlined that using fewer hangers leads to smaller construction costs, which could make the 20 hangers solution the most favourable, at least in economical terms. Nevertheless, a preliminary fatigue assessment conducted for some of the bridges presented in this work indicated that in some cases there may be a potential need to redesign the hangers.

When compared with a vertical arrangement, an improvement of over 46% can be found, as expressed in Table 5.23.

Table 5.23 – Weight comparison

Most Efficient Solution	Weight (ton)	Improvement
Vertical	196,74	--
Advanced Method	105,02	46,6%

5.9. SUMMARY AND DESIGN CONSIDERATIONS

The parametric studies discussed in the previous sections allowed an assessment of a number of variables related to bridge design. Some design considerations are summarized:

- Smaller hanger forces are achieved in the “spoked wheel” model. However, this configuration presents similar issues as have been observed for the vertical arrangement with large bending moments in the arch and in the tie;
- Generally, more relaxed hangers are found when the hangers become steeper;
- Both the advanced hanger configuration -in the network range- and the alternative constant slope configuration, present the most efficient solutions when it comes to minimum compression and bending moments in the arch, followed closely by the constant slope hanger arrangement;
- The constant slope configuration provided the most reasonable results in the tie, with the smallest bending moments and tension.

Concerning the different network arrangements studied, the following conclusions can be drawn:

- The constant slope configuration, being the simplest to define, provides good results, proving that a simple cross of hangers at a constant angle can lead to better results in comparison with the vertical hanger configuration.
- The variable slope configuration is clearly the least efficient arrangement. The minimum bending moment, in both tie and arch, is almost twice compared to other configurations. The presence of a large amount of relaxed hangers leads to difficulties in the structural analysis of the bridge. In summary, and as an overall conclusion, the benefits of using the variable slope configuration do not justify a more complex design and construction process.
- The advanced hanger configuration produces the most effective results for equidistant upper nodes in the hangers. Departing from an intuitive structure (the “spoked wheel”) and with almost no relaxed hangers, it is a robust configuration, and clearly the best tested in this parametric study.
- The alternative constant slope configuration, in which the hangers are merged close to the transverse beams in the tie and that involves variable positions for the upper nodes, provided good results. However, the number of relaxed hangers remained a problem. A slight change in the position of the first hangers has to be undertaken in order to minimize this effect.
- Similarly to other configurations, increasing the number of hangers in the advanced hanger configuration generally tends to produce smaller internal forces in all the elements. However, the improvement obtained in the advanced method by varying the number of hangers seems

too insignificant to overrule aesthetic considerations as pivotal decision point in the bridge design process.

Other considerations should be taken into account when designing arch network bridges:

- Near the supports of the arch a disturbance range can be found, where the tension in the hanger is above the average due to a clamping effect. In order to eliminate these localised problems, the position of lower and upper nodes can be shifted manually in an iterative process;
- When smaller hanger forces are less significant than smaller stress variations, less inclined hangers should be adopted. In the case of the constant slope configuration that would mean the consideration of smaller angles, while in the advanced hanger configuration it would consist of larger cross angles.

5.10. Concluding Remarks

In this chapter, several hanger arrangements were investigated and results compared, according the different criterions. Conclusions and design considerations were identified which may be considered when designing tied arch bridges.

6
CLOSURE

5.1. Summary and Conclusions

In this thesis, the performance of several hanger arrangements was investigated. The previous studies undertaken on arch bridges showed that the network web solution can be competitive for arch bridges with a concrete tie, achieving better results with heavier decks, such as rail ones. This study shows that the network solution can also be considered as a competitive alternative for smaller weights, such as the road loads, with a composite deck.

A composite steel concrete arch bridge, with a span of 100 m, an arch rise of 17 m, a deck with a width of 13 m and convergent arches was studied in this work. The results clearly show that a network configuration leads to better structural efficiency when compared with any vertical hanger arrangement.

Concerning the hanger arrangements examined, it was proven that a simple cross of hangers, at constant angle, leads to good results. The most efficient solutions are obtained by making use of the advanced hanger configuration. The variable slope configuration, being more sophisticated than other configurations, clearly did not produce better results in comparison with other arrangements.

Finally, it must be acknowledged that a bridge with convergent arches is necessarily heavier than a bridge with parallel arches. Nevertheless, convergent arches structurally allow for a more slender wind bracing system that is likely to lead to a higher aesthetical appeal by looking more interesting and harmonious.

5.2. Recommendations for Future Developments

In the course of this investigation several topics remained unanswered. Some were not studied or not presented due to the constraints of time and others should be the focus of further investigation, such as:

- Consideration of all the actions prescribed in the Eurocodes, such as traffic loads, wind and temperature actions;
- A detailed fatigue assessment for the hangers;
- Due to the fact that the buckling length varies according to the position and number of hangers, a buckling assessment must be carried out;

- The concrete slab deck should be studied in more detail and different solutions need to be compared. Being subjected to tension, variables as creep or shrinkage may take an essential role in the serviceability limit states, as well as in the detailing of the shear stud connectors;
- A more flexible FE analysis should be employed in order to ease the conduction of extensive parametric studies.

The author believes that the reason why network web configurations did not become massively used globally is not only to be found in the extra construction process it implies, but also the in the fact that there is not a clear and straightforward methodology to design this type of structure. Creating such a methodology –which would have to comprise simple topics and optimum ranges over how to design a network arch bridge- would therefore be of outmost benefit to the bridge engineering community, since it would free time resources that could then be dedicated to other variables that are no less important, such as durability, aesthetics and environmental integration.

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