Mabey & Johnson Bridge Structural Behavior Assessment
Solutions to Increase Load Capacity and Span

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Abstract

Since the 1930s, military logistical bridges have been essential in numerous war theatres of operations to overcome natural obstacles in a short period of time. However, in the recent years they have played also a very important role in the assistance of civilian populations, whether being used in cases of environmental disasters or as a temporary replacement of bridges being repaired.

In order to increase the bridge length as well as the loading capacity of the most recent military logistic bridge in Portugal, Mabey Bridge, several numerical finite element models have been developed, duly benchmarked with other studies, which have determined the loading capacity of the bridge for a simple supported configuration.

Taking into account its limitations, both for military and civil loads, the study of several possible strengthening solutions was performed, from geometric change of the bridge to section changes and addition of new elements in order to find a solution that verifies the security for military loads according to NATO regulations and civil loads according to Eurocode 1 - Part 2.

Once a feasible solution was found, the construction stage of the bridge by incremental launching was studied considering the solution proposed by the supplier and a real launching of a bridge of the same type that was possible to follow at the Company of Bridges of the Engineering Regiment No. 1 of Tancos, which was used as a practical case for the study of the bridge launching.

Keywords: Military Logistic Bridge, Mabey Bridge, Structural Analysis, Incremental Launching

1. Introduction

The Mabey & Johnson Bridge is primarily used by the Portuguese Armed Forces in military operations and, when requested or in cases of emergency, in support of the population. The multiple possible configurations associated with its speed of assembly/disassembly have led to an increase in its use, either in scenarios of natural disasters, or as a temporary replacement of bridges that need to be repaired.

According to the manufacturer's technical manual, the use of the bridge is restricted to the settings of the various configurations, depending on the intended span and the load to be overcome. However, with the various uses of the bridge, there is interest in studying in more detail the operation of the bridge and its constraints, so that new configurations can be defined to achieve longer spans and higher load capacity.

For the study of the Mabey & Johnson Bridge, usually named as the "Mabey", numerical computer models were used, which were developed to simulate as much as possible the reality. The validation of the numerical model was done using experimental tests.

The main goals of this study are to assess the structural behavior of the Mabey bridge, both in service conditions and for the ultimate limit states, for different scenarios. For this purpose, a numerical model was developed, duly calibrated, based on the catalogued properties of the bridge components and based on previously performed models, in order to develop a consistent model for the various scenarios with which the bridge is to be operated.

Complementary, this work also aims to present a historical summary and the state of the art of the modular bridges and, more specifically, of the Mabey & Johnson military bridge, to understand how these modular bridges have developed and how they have been used since their initial design.

2. Mabey Bridge Description

2.1. Short Description

The Mabey bridge, widely operated by European Military Forces, can be used with spans up to 60.96 m and a maximum load capacity of 110 tons. Initially, the concept was to use always simply supported spans. However, the more recent models allow to have intermediate supports and a continuous bridge deck.

The possible configurations of trusses have been summarized by the Portuguese Military Forces which makes it easy to interpret the ratio of the number of panels, directly proportional to the length of the span, with their load capacity in relation to the type of truss and type of chords strengthening to be used.

OLHAL		MLC40 CIVIL	MLC60 NORMAL	MLC80 T NORMAL	MLC110 W NORMAL
VÃOS	METROS				
5	15,24	SSH	SSH +	DSH	DSH
6	18,29	SSH +	SSHRH +	DSH	DSH
7	21,34	SSHRH +	SSHRH ++	DSH	DSH
8	24,38	SSHRH ++	SSHRH ++	DSHR1H ++	DSHR1H ++
9	27,43	SSHRH ++	DSH	DSHR1H ++	DSHR1H ++
10	30,48	SSHRH +++	DSHR1H +	DSHR1H ++	DSHR2H ++
11	33,53	DSH	DSHR1H ++	DSHR1H +++	DSHR2H ++
12	36,58	DSHR1H ++	DSHR1H ++	DSHR2H +	DSHR2H +++
13	39,62	DSHR1H ++	DSHR1H +++	DSHR2H ++	DSHR2H +++
14	42,67	DSHR1H ++	DSHR2H +	DSHR2H ++	TSHR2H ++
15	45,72	DSHR2H +	DSHR2H +	DSHR2H ++	TSHR3H +
16	48,77	DSHR2H +	DSHR2H ++	TSHR2H ++	TSHR3H ++
17	51,82	DSHR2H ++	TSHR2H	TSHR2H ++	(TSHR3H ++ @ C)
18	54,86	TSHR2H +	TSHR2H +	TSHR3H +	X
19	57,91	TSHR3H	TSHR3H	TSHR3H ++	X
20	60,96	TSHR3H	TSHR3H +	(TSHR3H ++ @ C)	X

Figure 1 – Possible configurations of the Mabey bridge, adapted from Infrastructure Dep. Catalog (2014)

For example, the TSHR3H configuration has the following meaning:

- The letter "T" comes from the word "Triple" which represents the possible configurations of the panel trusses; they can be "S" for "Single", "D" for "Double" and T for "Triple".
- The letters "SH" are transverse to all panels of the Mabey Compact 200 bridge and come from "Super and Heavy".
- If it has the fourth letter "R", it has the meaning of "Reinforcement", that is, it represents a bridge configuration with chords strengthening. The number that follows, varies between 1 and 3, means the number of strengthening planes to be used.
- When chords strengthening are used, the fifth letter means their type; in this case the "H" stands for "Heavy".

If one or more "+" symbols appear after the name with letters, it means that the same number of "super high shear" panels, from 1 to 3, will be required to be placed at each end to strengthen the panels' shear resistance. These "super" panels are usually placed at the end of the bridge when the bridge is simply supported, since this is where the transverse stress is greatest.

The longest simply supported configuration of the Mabey Compact 200 bridge is the TSHR3H++ which has span of 60.96 m and a roadway load capacity of 80 tons.

2.2. Assembly process

The Mabey bridge has two main forms of assembling: i) using a crane that loads it after previously assembled and puts it in the desired position or ii) by incremental launching on a cantilever, from one of the margins.

The most used assembly process is with a crane, however, this requires the need to mobilize a crane, which requires a load capacity all the greater as the desired span, being very difficult to obtain a sufficiently robust crane in soft soils to carry the bridge span to the final position.

In 2006, in Scotland it was found that seven road bridges presented serious structural problems, and it was necessary to replace them. This would require interrupting the traffic for a long period. Thus, the Mabey Compact 200 bridge was used while the bridges were being repaired and, in a few days, the temporary bridges were assembled next to the existing bridges, with maximum spans of 18 meters, using the cranes that were being used in their intervention.

Figure 2 - Assembly using a crane, adapted from Mabey Bridge

In the incremental launching of the Bailey bridge, a "launching nose" is used which has as main functions to counteract the cantilever's deformation when reaching the opposite margin. The launching process is considered the most critical phase of the bridge erection because during this operation the equilibrium should be kept.

For the assembly process of the bridge prior to the launching, it is necessary to have some construction site space as the length corresponds to the length of the bridge itself plus the launching nose, that is at least 1.5 times the span of the bridge to be assembled.

Initially, the launching nose is mounted on the bridge, which is initially positioned with the deflection opposite the cantilever expected and does not pass through the launching rollers. Then, a hydraulic jack or a tracked tractor pushes the bridge that slides from the construction supports to the launching supports through the lower rope of the bridge, and through the launching supports the height and inclination of the launching of the bridge can be moderated according to the unevenness that exists in relation to the other side.

When the deck reaches the desired position, the launching nose is removed, and hydraulic jacks are used to lift the deck so that the launching rollers can be replaced with definitive supports. Since it would represent a great weight when assembling the bridge, once the final supports are assembled, the deck is only placed after the bridge launching.

Figure 3 - Launching nose as a cantilever, adapted from Companhia de Pontes (2011)

2.3. Intermediate support

There are two possibilities for executing this continuity of spans: a) the span junction equipment and b) the distribution equipment.

If a large degree of rotation in the support is required or too high transverse internal forces are expected, it may be necessary to use the span junction equipment, thus obtaining a better alternative than two separate spans simply supported (Figure 4). However, it is not fully continuous as it does not allow to have the full continuity. Even so, it reduces the rotation and the spans deformability, and there may be an increase in transverse effort which leads to a need for strengthening this support (Mabey Bridge, 2014).

The jointing equipment consists basically of a male/female system which, through steel bars and pins, guarantees the connection and rotation from one span to the other. This type of connection varies depending on the configuration used on the bridge.

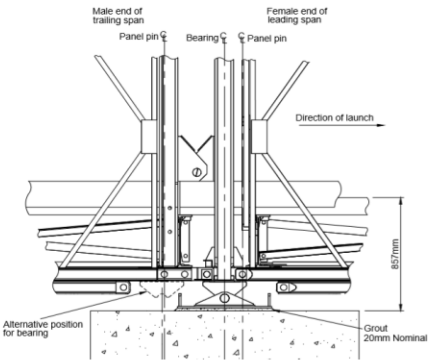

Figure 4 - Span junction equipment, adapted from Mabey Bridge (2014)

Although the span junction equipment is the most widely used method, sometimes a distribution beam is used below the bottom chords, specially when the piers are positioned at a distance from the end that makes it difficult to assemble the span junction equipment (Figure 5). In addition, the option of creating a continuous truss may also be the most economical solution since the maximum bending moment in the middle of the beam may be lower compared to the previous solution of separate spans. However, this can lead to higher forces at the chords of the support regions that may need some local strengthening of the steel structure.

One of the main drawbacks of this solution is when one very shorter lateral span exists, which may result in the lifting of the supports in a situation of a heavier vehicle crossing the bridge, since the supports are usually fixed only by the weight of the bridge.

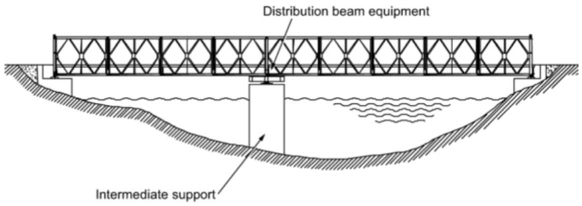

Figure 5 - Distribution Beam equipment at the intermediate support, adapted from Mabey Bridge (2014)

3. Design Criteria and Actions

3.1. Actions

Dead and live loads and the design states, service limit state and ultimate limit state, should be considered when designing a bridge deck. The live loads can have a civil or military nature.

3.1.1. Eurocode 1 live loads

Eurocode 1 – Part 2: Traffic Actions on Bridges is the standard that defines the civil design loads for road bridges in several countries of Europe.

Partial safety factors and combinations for Ultimate Limit State (ULS) and Service Limit State (SLS) are given for a temporary structure (applying a 0.9 coefficient to the ULS partial factors) by:

(1) **ULS:** 1.35 x 0.9 x (DL) + 1.35 x 0.9 x (LL)

(2) **SLS:** 1.0 x (DL) + 0.4 x (ULL) + 0.75 x (LV)

being

DL – Dead Load
LL – Live Load
ULL – Uniform Live Load
LV – Live Vehicle

3.1.2. Military live loads

According to NATO UNCLASSIFIED military vehicles are divided into 32 classes according to their gross weight in tons. Each class is always called the "Military Load Classification" (MLC) which is followed by its maximum gross weight in tons of the American system, rounded, where appropriate, to the next higher-class MLC. For example, a military vehicle weighing 45 tons is classified by MLC 50. This classification is of high importance, especially for bridges since the load capacity for high weight vehicles is generally low and thus makes it possible to quantify their maximum load capacity. As an example, a bridge allowing a maximum vehicle MLC 50 means that all military vehicles weighing less than or equal to 50 US tons are guaranteed safe passage (Figure 6).

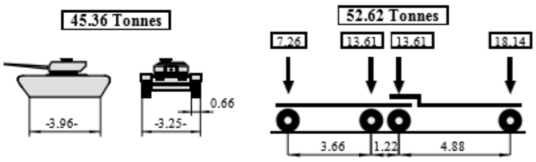

Figure 6 - On the left an example for tracked vehicle MLC 50 and on the right a wheeled armored fighting vehicle MLC50.

Since it was necessary to choose a class of vehicle to be used, the heaviest caterpillar armored vehicle of the different NATO armies is adopted. According to NATO UNCLASSIFIED (2017) the heaviest military vehicle is the "German LEOPARD 2 A5.

The weight of this vehicle is 59.23 tons (593 kN), approximately 66 American tons and so this vehicle is classified as MLC 70.

This vehicle has a width of 3.42 m and a length of caterpillars sitting on the ground of 4.93 m, however, the measures defined by the standard for MLC 70 have been used.

To ease the use of SAP model, especially to introduce moving vehicles, it was decided to simplify the distributed loads in concentrated loads equally spaced by the number of wheels of the bottom track in contact with the bridge deck. Thus, the 635 kN total load was divided by the 14 wheels, giving 45.45 kN per wheel and with equal spacings of 0.76 m between them (Figure 7).

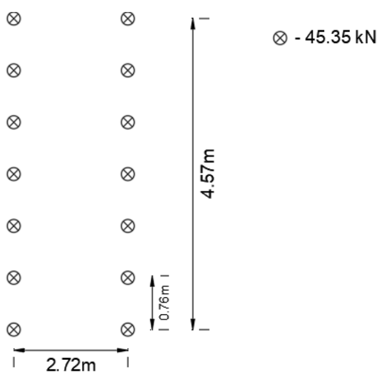

Figure 7 – Military vehicle loading model

For the military live load, the Ultimate Limit State combination is given by:

(3) **ULS:** 1.20 x (DL) + 1.22 x (LL)

4. Structural Analysis – Current Bridge

4.1. Analysis Model

Considering that the simply supported configuration is limiting, mainly in terms of span length, it was studied an alternative solution by creating the deck continuity in the central support.

Since the analysis model results were verified, with a small percentage of error, it was extended from 19 to 38 modules, and between the 19th and 20th modules it was introduced a continuity support, in order to have two spans with equal lengths of 57.91 m, for a total length of the bridge deck of 115.82 m (Figure 8).

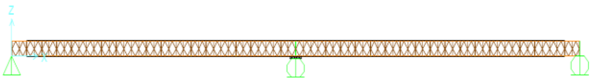

Figure 8 - Continuity supported model

Considering that there was no information about the distribution beam (currently not used by the Portuguese Military Forces), and the fact that it should be a much more robust beam compared to the chord profiles, a HEB 300 profile was adopted as the profile of the distribution beam. To simulate the link between with the bottom chords, rigid connections were adopted (Figure 9).

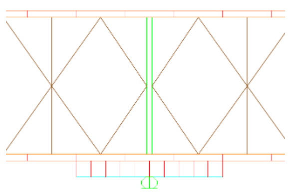

Figure 9 - Distribution beam on the SAP2000 model

4.2. Military ULS

The configuration used was a continuity support, with two spans of 57.91 m, for the bridge crossing of the Leopard 2 A5, with the right track at the edge of the deck to simulate the governing position of the vehicle.

The bridge crossing of two military vehicles simultaneously was not considered possible since the normal force / bending moment interaction ratios applied on the structure were approximately 30% above the available resistance. Thus, the bridge crossing of only one military vehicle was studied in more detail.

The interaction between the moments and the axial force proves that the bridge do not verify the ULS of resistance according to the Eurocodes:

- $\frac{473.49}{\frac{0,687 \times 1163.80}{1}} + 0.498 \times \frac{14.699}{\frac{0.82 \times 43.65}{1}} + 0.697 \times \frac{0.65}{\frac{66.70}{1}} = 0.70 < 1$ ✓
- $\frac{473.49}{\frac{0.687 \times 1163.80}{1}} + 1.162 \times \frac{14.60}{\frac{0.82 \times 43.65}{1}} + 0.829 \times \frac{0.65}{\frac{66.70}{1}} = 1.07 > 1$ X

However, as the bridge was originally designed according to British Standards, the same verification was done with the traffic loads and load combinations of the British Standard. Thus, according to Table 1 of "British Standard (2000) Part 2" the combination for the ELU is:

(4) ULS: 1.05 x (DL) + 1.25 x (LL)

With this load combination the same ULS verification have a ratio of 0.93. We can therefore conclude that the deck verifies the ULS security for the bridge crossing for the heaviest military vehicle in Portugal, class MLC70, which is in line with expectations, since the configuration adopted is one of the possible ones in the bridge manual, even if it is not clearly specified.

4.3. ULS with loads from EC and RSA

As seen above, since the bridge does not permit the passage of two 60-ton military vehicles, the use of a standard EC local vehicle of 60 tons was not considered, thus using only the uniformly distributed load, which corresponds to light vehicles crossing the bridge.

Three simulations were performed to test the load capacity of the bridge:

- For the first simulation, following EC1 - Part 2 (CEN, 2003), 2 lanes were used: one 3 m width using a live load of 9 kN/m^2 and the 1.2 m left with 2.5 kN/m^2.
- As expected, the bridge cannot withstand these live loads as there is failure of the chords over the intermediate support. To be able to resist, the chords should have double the tensile resistance and more than the double compressive resistance. And diagonals would have to be roughly 1.7 times more resistant than they are.
- For the second simulation, it was chosen to use the live loads defined by the "*Regulamento de Segurança e Acções para Estruturas de Edifícios e Pontes*". Thus, in accordance with Article 41.1) b) of the RSA (1983), a uniformly distributed load of 4 kN/m^2 and a "knife load" of 50 kN/ m should be adopted.
- For this live load, the results obtained indicate that the chords must have a tensile resistance 1.2 times higher and a compression resistance of 1.9 times higher. In the case of diagonals, they should be 1.4 times stronger.
- For the third simulation, having become clear in the two previous simulations that medium/ heavy vehicles applied along the hole deck should be a possible load scenario, it was chosen to use for this load case only light vehicles. According with the EC1 - Part 1, Article 6.3.3, the live load of category F, where the weight of the vehicle must have less than 3 tons, the live load of 1.5 to 2.5 kN/m^2 can be adopted. A distributed live load of 2.5 kN/m^2 was therefore used.
- The results obtained shown that the bridge can withstand the same live load in all sections. However, in the intermediate support area, the sections are almost at the maximum yield limit.

Thus, for the bridge with two spans with 57.91 m each – TSHR3H configuration, verifies security for the MLC70 military vehicle and allows the bridge crossing of several civil vehicles with less than 3 tons. If medium/heavy civil traffic is to be considered, this type of traffic should not cross the bridge simultaneously.

4.4. Deflection for service conditions

It is equally important to evaluate the behavior in service since excessive deformations can lead to problems in the circulation of vehicles, as well as difficulties with the correct drainage of rainwater.

Vertical deflection was predicted for two different situations: one with each load component isolated and the other for the ELS frequent combination.

For the first situation, the maximum deflection of the structure was evaluated separately and using a combination factor of 1.0. Likewise, the maximum deflection of the structure was evaluated only at the passage of the military vehicle. The results are presented in the following table:

	Dead load	**Leopard 2 A5**	**Total**
Deflection	51.1 mm	45.7 mm	96.8 mm

Table 1 - Deflection due to dead load and military vehicle

Since the Eurocode 0 does not define the maximum allowed deflection for SLS frequent combination, the British standard was adopted. Thus, according to BS5950- Part 1, Section 6, a deflection up to $L/360$ is allowed, which corresponded to a maximum allowable deflection of 160.8 mm for spans of $L = 57.91$ m.

Using the equation (2) a maximum elastic deflection was obtained for the civil SLS frequent combination of 110.2 mm, lower than the 160.8 mm limit and corresponding to approximately $L/500$. As it can be concluded the margin to the limit is high, mainly due to the continuing support configuration.

5. Structural Analysis – Modifications to the existing bridge

5.1. Increase the distance between chords

The first models of the Mabey bridge contemplated the possibility of panels with different heights in the areas where the bending moment was higher. This solution increased the resistant capacity of the bridge as it increased the distance between the chords. On the other hand, the increase in height led to an increase in the inclination of the diagonals, improving the resistance to shear.

For this purpose, the initial configuration of the Mabey bridge was used as the basis, in which the panels had a base height of 1.52 m and, if it were insufficient to resist the bending moment, a panel that made the transition from 1.52 m to 2.30m would be used. The support panels were 1.5 times higher than the "standard" panels.

Likewise, for the standard panel 2.13 m height, respecting the final panel's ratio being 1.5 times higher, a panel was created that made the transition from 2.13 m to 3.2 m (Figure 10).

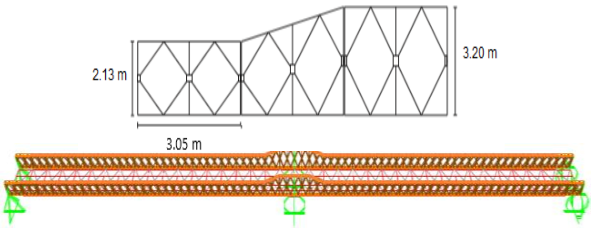

Figure 10 - From top to bottom: configuration adapted to the model under study and SAP Model

5.1.1. ULS checking – Military loads

The bridge was re-tested for the ULS with two Leopard vehicles with 30.5 m spacing between them. The results show that, given the changes made to the bridge, the bridge complies with the EC ULS safety checks. The results are presented in Table 2 for the governing member.

	Initial configuration	New configuration
Axial force - kN	-638.2	-400.9
Interaction Ratio	1.30	0.98

Table 2 - ULS Security checks for military loads

The results obtained prove that the distance between chords leads to a decrease in the chords internal forces.

5.1.2. ULS checking – UC loads

To carry out the security check with the EC loads, the same model of loading previously adopted was used. However, unlike the Military ULS, the configuration adopted was not sufficient to check the security of the bridge, with both diagonals and chords not verifying the ULS of resistance.

The most critical members, as expected, were the truss panels located on the side where the 9 kN/m^2 lane is applied, with diagonals having 2.6 times more load than their resistance and the chords 1.7 times more than allowed.

Contrary to the previous configurations, the increase of the distance between the chords has changed the governing sections from the chords to the diagonals since the increase of the distance, unlike the chords, worsens the resistance of the diagonals due to the increase of their buckling length.

Therefore, it can be deduced that without increasing the diagonal sections, it is unlikely that this configuration can be used with the EC live loads.

5.2. Panel overlapping

Like other types of military bridges, as the Bailey bridge, the Mabey bridge allows the overlapping of panels in height as in Figure 11. To do that, in the upper chord of one module instead of using a chord strengthening, another module is coupled through the lower chord of the latter, since the chord profiles and the chord strengthening are the same.

Since the main issue in ULS resistance is in the intermediate support, it was decided to place modules overlapping only over the intermediate support. However, this decision led to high local internal forces in the modules immediately after the overlapping modules. Thus, it was quickly found that overlaps should be made along the entire bridge, except for the end supports (Figure 12).

Figure 11 - Panel Overlapping on UK, adapted of Mabey Bridge 2020

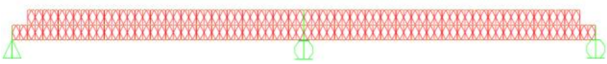

Figure 12 – SAP model adopting overlapping panels

5.2.1. ULS checking – Military loads

For the ELU security checks with the military loads, the same load model was used with two 30.5 m distant military vehicles and security was verified in all sections.

The results show that ULS security is verified with a relatively large margin, both on diagonals and chords. The result for the governing chord sections can be seen in the table below.

	Upper chord	Bottom chord	Diagonal
Axial Force - kN	195.0	-216.3	-73.4
Interation Ratio	0.19	0.29	0.44

Table 3 - ULS Security checks for military loads

5.2.2. ULS checking – UC loads

Regarding the ULS security verification for the EC loads, the same loading model as previously was used. The highest internal forces reported are shown in the table below:

	Upper Chord	Bottom Chord	Diagonal
Axial Force [kN]	738.3	-595.8	-342.1
Interaction Ratio	0.71	0.94	1.35

Table 4 - ULS security checks for EC loads

Unlike the ULS with Military loads, the change made to the bridge proved to be insufficient since, despite the resistant problem in the chords having been solved, due to the increased distance between the most extreme chord and the existence of an intermediate chord, the diagonals are still not sufficient resistant to withstand the ULS internal forces.

Nevertheless, the internal forces on these diagonals have decreased compared to the simple configuration, the latter being understood as the initial configuration without any change. This decrease in internal forces is due to the fact that there is a possibility of alternative shear path. However, for the intermediate support diagonals, since the forwarding of loads ends up having to be done to the support, these proved to be critical, even if less in relation to the initial configuration, going from 70% to 35% over the resistance according with the EC3 ULS verifications.

Thus, in addition to the overlapping of panels, it would be necessary to strengthen the diagonals using plates on the first four modules to both sides in relation to the intermediate support and to strengthen the first two modules at both ends. The cost of having almost twice as many panels would make this solution possible but very costly, and therefore not subject to further study.

5.3. Strengthening of the member sections

As seen above, changing the bridge geometry is a solution that will hardly allow checking the safety of the sections to the ULS for the loads from EC, since increasing the distance between chords decreases the internal forces on these, but increases diagonals buckling length.

A possible solution is therefore to change the sections, as this allows for an area increase and, consequently, a higher resistance to axial forces.

Since the bridge modules are defined a possible form of increasing the area, without having to modify these pre-defined modules is through the application of steel plates that can be both welded and bolted to the profiles.

Regarding the diagonals, it has been envisaging the solution of welding a 10 mm plate with a width equal to the height of the "C" profile, that is 76.2 mm, increasing the area by about 190% compared to the original section.

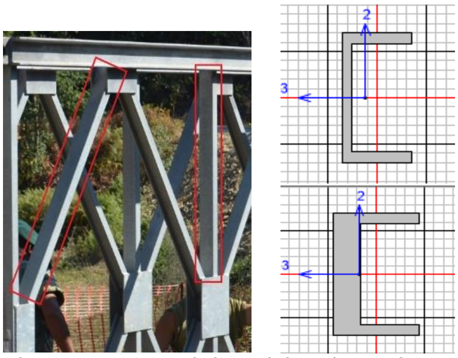

Figure 13 - From left to right: place of strengthening and section before and after the strengthening with the plate

Both the upper and lower chords have the chords-reinforcement and since the modules have no fixed orientation, it was decided to place the same plate solution on both chords. In the case of the upper chord, it was placed above the chords-reinforcement and in the case of the lower chord, it was placed below the chords-reinforcement.

The added plates have a thickness of 20 mm and a width capable of covering the "C" profile flanges and the space between them, that is 50+80+50 = 180 mm, leading to an area increase of 240% compared to the existent section.


Figure 14 – Location of the chords strengthening (in red)

Regarding the security verification of the ELU using the EC loads, and adopting same load model as previously, ULS of resistance was verified in all sections, as shown in the following table.

	Upper Chord	Bottom Chord	Diagonal
Axial Force - kN	1172.0	-1017.0	-459.1
Iteration Ratio	0.98	0.89	0.97

Table 5 - ULS security checks for EC loads

Being a solution that checks safety, it is also important to verify that the connections safety.

Connections between chord and the chord strengthening have been found to be the most critical with a maximum transverse shear of 149 kN. This connection is made through bolts M24 cl 8.8, with an area of 353 mm⁴ and a f_{ub} = 800 MPa, having a shear resistance given by:

$$F_{v,Rd} = \frac{\alpha\, fub\, As}{\gamma m2} = \frac{0.6 \, x \, 800 \, x \, 10^3 x \; 353 \, x \, 10^{-6}}{1.25} = 135.6 \text{ kN} < F_{v,Ed} = 149 \text{ kN} \quad \text{X}$$

Regarding the possibility of increasing the class to 10.9 could be used, resulting in an increase of the ultimate tension from 800 MPa to 1000 MPa and resulting in a shear resistance:

$$F_{v,Rd} = \frac{0.5\ x\ 1000\ x\ 10^3 x\ 353\ x\ 10^{-6}}{1.25} = 141.2\ \text{kN} < 149\ \text{kN} \quad \text{X}$$

The fact that for Portuguese bridge models do not allow for a connection with bolds diameters bigger than M24, makes the implementation of this strengthening solution very difficult since all the connection holes would have to be increased. by the manufacturer to assure control and quality.

5.4. Increasing of the strengthening of chord

Finally, the possibility of creating an extra reinforcement at the chords has been considered, being the coupling of one more element under the existent reinforcement, in the case of the lower chord and, above the existing reinforcement, in the case of the upper chord.

Thus, similarly to the option studied before, it was chosen to keep the strengthening on the diagonals of the five modules on both sides of the intermediate support and on the module of the end support on both sides, as the resistant problem is easily solved with the addition of a 10 mm plate in the compressed diagonals.

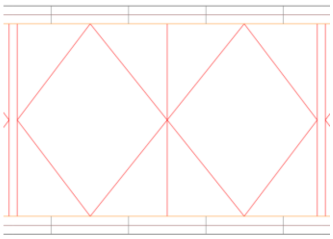

Figure 15 - SAP 2D model of a module with double layer strengthening (black added element).

5.4.1. ULS checking – EC loads

The checks were carried out successfully, however, a problem appears in the area where the distribution beam ends, generating a slight increase in axial force, but mainly an increase in the local bending moment. Thus, being the distribution beam a HEB 300 commercial profile, an alternative was used to reduce the impact of the sudden decrease in rigidity, a variable inertia profile.

In this way, the web will have a progressive reduction of height, reaching a final solution which consists of a distribution beam with 2.30 m of constant inertia and at both ends a portion with variable inertia with a transition length of 0.90 m, varying from a HEB 300 profile to a profile with the same geometric properties of the flanges, the same thickness of the web, however, a gradual variation in the height of the profile from 30 cm to 10 cm, to assure the web welding can still be carried out.

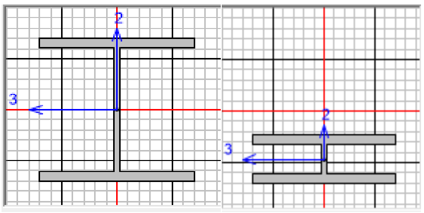

Figure 16 - From left to right: initial and final variable inertia distributed beam cross-section

The application of the beam with variable inertia at the ends made it possible to verify the safety of the most external strengthening, immediately after the end of the distribution beam, and as the internal forces and ratios are shown in the following table.

	N_{Ed} (kN)	M_{yy} (kNm)	Interaction Ratio
Uniform Inertia	-1045.00	-12.64	1.14
Variable Inertia	-1039.00	-5.14	0.99

Table 6 – Internal forces with a variable inertia beam

5.4.2. ULS checking – Military loads

For the final solution of strengthening the ULS security checks using military loads, the model used was two military vehicles, 30.5 m distant. and security was checked in all sections. The results can be seen in the following table.

	N_{Ed} (kN)	M_{yy} (kNm)	Interaction Ratio
Chord	-304.40	-1.36	0.26
Diagonal	-116.03	-0.30	0.35

Table 7 - ULS security checks for Military loads

5.4.3. ULS checking – Joints

Using the maximum internal forces for both the military and EC loads, ULS security checks of the connections were carried out bellow.

- Simple Shear resistance with M24 cl.10.9 between chord and chord reinforcements:

$$F_{v,Rd} = \frac{0.5\ x\ 1000\ x\ 10^3 x\ 353\ x\ 10^{-6}}{1.25} = 141.2\ \text{kN} > 139\ \text{kN} \quad \checkmark$$

- **Bearing Resistance:**

$$F_{b,Rd} = \frac{1.5x\ 800\ x\ 10^3\ x\ 24\ x\ 10^{-3}\ 9\ x\ 10^{-3}}{1.25} = 207.36\ \text{kN} > 201\ \text{kN} \quad \checkmark$$

- **Block Rupture**

$$V_{eff,2,Rd} = \frac{0.5\ 800\ x\ 10^3\ Ant}{1.25} + \frac{460\ x\ 10^3\ Anv}{1.25} =$$

$$= \frac{0.5\ x\ 800\ x\ 10^3\ x\ (5.5\ x\ 8) x\ 10^{-4}}{1.25} + \frac{460\ x\ 10^3\ x\ (5.5\ x\ 2.4) x\ 10^{-4}}{1.25}$$

$$= 1045.6 >> 201\ \text{kN} \quad \checkmark$$

6. Incremental launching of the bridge

6.1. Verification during launching

As the adopted solution has a considerable cantilever length during launching, it is important also to make the safety checks during this stage. Moreover, even though the launching process for these lengths will be proposed by the manufacturer, the rules used to carry out the safety checks are different and may be less conservative as it is a temporary stage.

In this way, the launching model of the most critical phase has been created, that is, immediately before the counterweight is removed and when the launching cantilever is 62.5 m long (Figure 17).

Figure 17 - Model for the launching for the adopted solution

The most stresses element is the lower chord just above the first support. The interaction between the internal forces led to an (N,M) ratio of 1.50 = 0.27(N) + 1.23(M), as the local bending moment is excessively high.

However, it should be noted that for this launching procedure, as shown in the catalogue, only one launching roller was used on the inner chord of each triple truss plane and, in practice, it is possible to use two launching rollers: internally, as used previously, and externally (Figure 18).

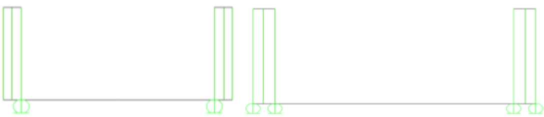

Figure 18 - From left to right: solution proposed by catalogue and suggested alternative arrangement

The duplication of the launching rollers allowed a reduction of approximately 43% of the vertical reaction applied in one roller compared to the layout adopted by the catalogue. In addition, the interaction between the internal forces led to an (N,M) ratio of 0.91 = 0.27(N) + 0.64(M) verifying the safety according to the EC3.

6.2. Identifying the effective supports

One of the main concerns in the incremental launching assembly process is the existence of tension in some supports. In this case the bridge is supported only due to gravity loads and can decollate from some supports. The vertical reactions of the bridge to the various phases of its launching have been verified and there have been tension supports. The phase where there were more tension forces was in the phase immediately before the bridge reaches the intermediate piers where the launching nose cantilever is longer (Figure 19).

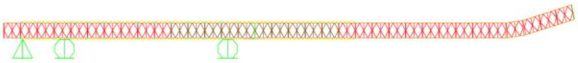

Figure 19 - Support working during the launching.

Having studied the reactions in the supports, it is possible to conclude that in the most critical phase of the launching, only three of the six supports are working as supports. Although the other supports are not removed as they are used for the construction of the bridge at the site, in practice, the actual configuration of this launching phase is based on only three.

6.2. Maximum deflection during launching

The solution under study consists of one module with a slope of one launching link followed by three modules with a slope of two launching links:

Elevation: 363 + (119.10 + 115.49) x 3 x 3.048 = 2508 mm

As already observed, to measure the maximum expected deflections, in addition to considering a rigid support, a study was made of the possibility of the support having flexible stiffness, which in large cantilever lengths can result in an appreciable difference at the end of the cantilever. To carry out the simulations, 2 models were used: the initial model proposed in the catalogue, and the model with all the compression supports, that is, with three supports (Table 8).

Stiffness in Support (kN/m)	Settlement in Support (cm)	Deflection (cm)	Settlement in Support (cm)	Deflection (cm)
	With all Supports		With 3 Supports	
K = ∞	0	45.29	0	43.78
K = 50000	1.02	50.00	0.68	45.89
K = 40000	1.20	50.80	0.87	46.41
K = 30000	1.46	52.04	1.16	47.27
K = 25000	1.64	52.86	1.39	48.00

Table 8 – Settlements on the extreme support and maximum deflection at the tip of the cantilever

7. Summary and conclusions

The main conclusions of this study can be drawn:

- The bridge without any strengthening verifies ULS safety for military loads, with the partial factors of the British Standard. However, with the partial load factors proposed by the NATO standards, ULS is not verified.

- For SLS, the deflection obtained was approximately half of the values reported by Pereira (2014) for a configuration simply supported, proving to be a configuration with a considerably smaller deflection.

- The duplication of the chord reinforcement, for both the upper and lower chords, and the strengthening of the diagonals on the end modules, as well as on five modules for both sides of the intermediate support, make it possible to carry out the safety checks successfully, both at section level and at connection level, with the aid of a variable inertia distribution beam allowing a gradual reduction in stiffness. The connections must be made with bolds of class M24 cl 10.9, to check the connection between the chord and the chord strengthening.

- The launching of the proposed bridge according with in catalogue and using just one roller bellow each group of three trusses, especially in the stage in which the cantilever has the maximum length, since the sections that are immediately above the first launching roller have a very high local bending moment. However, this problem is easily overcome by using four launching rollers, two for each triple truss plane (on the outer and inner chord).

- The launching process, as shown in the catalogue, has throughout the launching process “tension supports”, that is, supports that effectively are not performing any function since the supports work exclusively due to the structural weight, there is no mechanism that makes the anchorage with the ground, thus not being able to work with tension forces; Nevertheless, these supports are used for the construction of the deck, but not for launching.

- The maximum expected deflection from the launching nose is well below the elevation given before the launching, which allows for vertical deflections at the end support and yet the elevation is higher than the deflection at the end of the launching nose.

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