

# **Influence of the construction stages process and the concreting sequence in the tension state of a composite steel-concrete bridge deck**

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## **Abstract**

This work has the objective of studying the influence of the constructive phases in the tension state of a steel-concrete composite bridge deck. This constructive stage is composed initially by the assembly of all the steel structure, followed by the installation of prefabricated slab panels over the entire length of the bridge deck and, finally, the “in situ” pouring of all the deck slab, fulfilling a process of sequential concreting by sections. At the end of this constructive phase of the steel-concrete bridge deck, a different stress distribution is introduced than if one is not considered.

Firstly, it is presented a synthesis of the design of bridges constructed with steel-concrete composite decks, identifying the main characteristics associated to the various structural solutions, accompanied by some examples of application with images illustrative of them. A brief description of the construction processes with a larger range of application for this type of decks is given below, with reference to the assembly processes of the steel structure, as well as the construction and solidification of the deck slab. The actions and design criteria to be considered during the construction of the deck are defined. The characteristics of the elements that make up the composite steel-concrete deck under study and the different phases of assembly and release of the steel part of the deck are described, as well as the concrete sequence of the slab to be used.

The proposed model for the composite steel-concrete deck is presented, with the SAP2000 software, using finite bar elements to simulate the steel structure and shell elements to simulate the concrete slab. The modelling used allows the consideration of the specific behaviours associated to each material and its joints action, as well as the simulation of the constructive phase, with and removal of elements of the structure at different moments of the analysis.

The structural analysis is performed during the various stages of construction, evaluating the stresses, tensions and deformations during these phases. Safety checks are carried out for the various parts of the structure during construction. The results obtained with the constructive phases are compared with the ones obtained in case it is not considered the constructive phases.

**Keywords** Steel-concrete composite bridge deck; Construction phase; Assembly of the steel structure; Concreting sequence; Construction safety checks.

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## **1 Introduction**

The construction of bridge decks in reinforced concrete and prestressed reinforced concrete, located at small heights and without any impediment to occupying the space below it, is currently carried out by using a shoring supporting the total weight of the deck structure during the entire period of the concrete cure. In this case the proper weight of the structure is only exerted when, after deactivation of the shoring, the structure has already acquired the characteristics of strength and stiffness that will have along the life of the bridge. Accordingly, the evaluation of the effects (stresses, displacements) of the weight of the structure is carried out using the same calculation models and following the same analysis routines used to evaluate the effects of the remaining actions.

Such simplification is not, however, feasible in the case of bridge decks consisting of prefabricated elements which are assembled on the bridge site without recourse to any temporary shoring. To the

evolution of the shape of the structure, as elements are added to it, successive additions of the loads are applied to the elements that compose the deck. It becomes necessary to use calculation models which evolve in accordance with the sequence of the assembly of the deck and the progressive application of successive plots of the weight of the structure, according to the same sequence.

Two large sets of decks may be distinguished, to which the considerations set out in the last paragraph apply: decks made of prefabricated elements of reinforced concrete and/or prestressed reinforced concrete and composite steel-concrete decks. The case evaluated in the present work is that of a composite steel-concrete deck, continuous with three spans.

In addition to the evolution of the calculation model of the deck, accompanying, step by step, the respective assembly process, it is also intended to compare the state of tension resulting at the end of the construction with that which would result from the application of the entire weight of the structure on a complete representative model.

## 2 Common configurations of composite steel-concrete beam decks

The mixed beam decks transmit bending loads to the abutments. Thus, the cross-sectional dimensions of the steel beams along the longitudinal span acquire a key role in ensuring the resistant capacity of the deck due to the bending moments and transverse stresses acting on the deck.

The cross sections of the main beams with the greatest area of application in this type of decks, as they are the ones that offer the best dimensional characteristics in the guarantee of the resistant capacity to the flexural and cutting efforts are the slender plate girders with the I-shape, the box girders and triangular beams (trusses).

In relation to the concrete slab to be executed on the steel structure, a distinction will be drawn between the possibility of using an “in-situ” concreting process or prefabricated slabs.

### 2.1 Steel Structure - Slender plate girders with the I-shape

The slender plate girders are structural members composed of three steel plates welded together to create an I-shape with the necessary dimensions to withstand the stresses exerted. These beams present a wide range of options, since it is possible to create cross sections with the necessary dimensions to give flexural and cut stiffness to the structure, in order to comply with the regulatory safety criteria [3]. Typically, these beams have a variable height along the longitudinal span, and can reach heights of 4-6 meters in the zones of the intermediate supports, which increases their moment of inertia and, consequently, the vertical stiffness of the structure, making its application feasible in spans of much larger dimensions, reaching these ones 200 meters in length [4], [5].

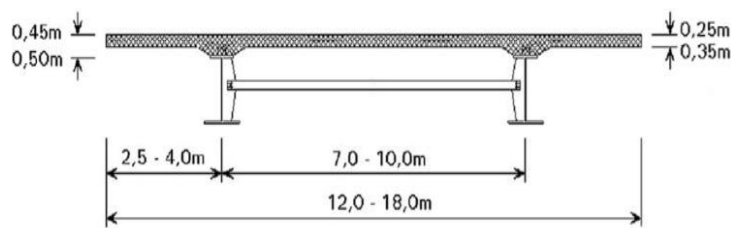


Figure 2.1 - Cross section of a composite steel-concrete deck with slender plate girders [W1]

The supply of resistance to bending stress is achieved by removing as far as possible the entire mass of the section of its axis of greatest inertia. Thus, for I-shaped cross-sections, the resistant capacity to flexural strength is mostly conferred by the flanges [4]. However, in the case of mixed decks, the contribution of the concrete slab, which acts as a reinforced upper flange, is taken advantage of, so it is common to observe asymmetries in relation to the axis of greater inertia in the dimensions of these elements. In this way, the quantities of steel used are reduced without violating the resistant capacity of the structure, which represents an increase in section efficiency and resource savings. With respect to the lower flange, it is dimensioned to withstand the acting moment, which justifies that they are wide and thick pieces, since they need dimensions that guarantee the resistance required [6].

The supply of shear resistant capacity is guaranteed by the web of the sections. Thus, the main objective in the design of these elements is to enable them to have a sufficient web thickness - in the face of cutting requests - to ensure compliance with regulatory safety criteria. Considering the possible buckling modes of these steel elements in general, webs up to 1,50 m in height only require vertical reinforcements in the internal supports, to resist the effect of the concentrated forces of the reactions and to stabilize the web to the effects of local buckling under concentrated loads [2]. Webs between 1.50

m and 2.50 m in height require vertical reinforcements and possibly longitudinal reinforcements near the inner supports, since they are zones of negative moments in which the lower part of the web is strongly requested to compressed forces. For profiles with heights above 2.50 m, it is convenient to require both types of reinforcement, vertical and longitudinal [3].

In Figure 2.2 are represented some of the main components of a composite steel-concrete deck, composed of a solution in slender plate girders with the I-shape.

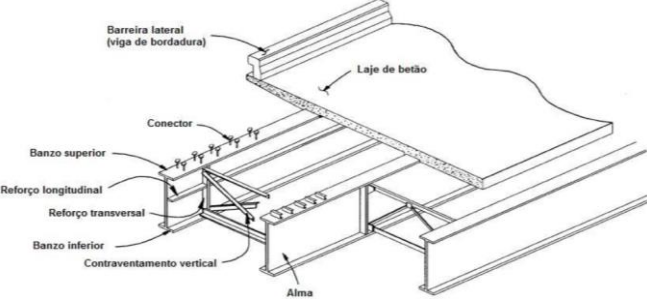


Figure 2.2 - Components of a solution in slender plate girders with the I-shape [7]

In Portugal, one of the most relevant structures of this type is the rail board used in the Maia Norte and Sul Viaducts (completed in 2006, Figure 2.2.3 and Figure 2.2.4). These viaducts have lengths of 524 m (Maia Norte Viaduct) and 497 m (Maia Sul Viaduct). The width of the decks in the current zone is 12.7 m - Maia Norte, and 10.1 m, Maia Sul [8], [9]. Two I-shape beams with a constant height of 2.25 m were adopted along the entire length of the deck, with 30 to 50 mm flanges. Due to the planar curvature of the deck, tubular bracings in plan and profile were adopted with CHS tubes [10].



Figure 2.3 - Maia Norte Viaduct [8], [9]

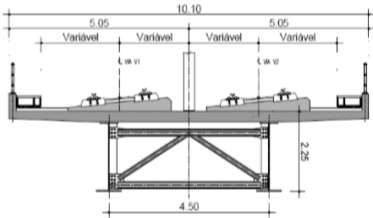


Figure 2.4 - Maia Sul Viaduct [8], [9]

**2.2 Deck slab – “In-situ” concreting process**

The "in-situ" concreting, to be applied after the assembly of the steel deck has been completed, requires a working platform where the workers can circulate whilst the concrete is processed. At the same time, the platform can be used as formwork for slab concrete. However, it is not appropriate for this formwork to be applied manually, since it becomes an expensive solution, with a high degree of complexity and a long delay. It is justified, therefore, the use of pre-slabs, these functioning as lost formwork. These pre-slabs are placed on the beams by mobile cranes or cranes on barges, if conditions are favourable [3].



Figure 2.3 - Concreting of the deck of the Penacova Bridge over the Mondego River – use of pre-slabs [11]

After the laying of the slabs, the in-situ concreting takes place, thus ensuring the joining of the slab panels between themselves - through transverse ribs - and to the beams - by means of negatives left for the purpose - at which point additional armour may also be applied.

### 3 Construction of mixed decks - Possibility of changing the initial tension state of the deck

Perhaps in none of the works of the field of structural engineering does the constructive process condition conception as much as in the case of bridges[2]. As the constructive process of a bridge develops, its structure goes through several configurations and types of loads, which generate stresses and deformations that evolve during the process. Such stresses cannot be calculated as if the permanent load of the structure were simultaneously acting in all its elements and on the final structural system as is currently recognized in the design of buildings [2]. It is therefore imperative to consider the evolution of the static system throughout the process and the effects of the permanent load acting during the various phases of the work and to make the corresponding stability checks of the structure, especially in situations that may be understood to be particularly conditioning factors. In calculating the final stresses to which the deck is subjected due to the permanent actions, it is necessary to take into account the effects of redistribution of the stresses caused by the deferred deformations of the concrete as well as the effects of shrinkage, which are influenced by the concrete sequence adopted[2].

### 4 Presentation and characterization of the deck

The analysed deck is mixed, steel-concrete and continuous in three spans of 63.00m + 105.00m + 63.00m, making a total of 231.00m between the extreme supports of the main beams (supports with meetings).

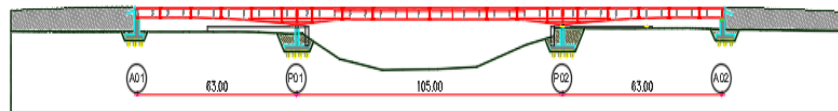


Figure 4.1 - Longitudinal configuration of the mixed deck (dimensions in m). Adapted from [16]

The deck is straight over a length of approximately 211m, but the slab is slightly curved at the end near the abutment A02. The curved shape and the width variations do not have any influence on the horizontal arrangement of the steel structure. The three main girders are straight and parallel. Only the slab reflects the curved deck shape and width variations.

The deck width is 11,30m in the straight extension and is variable between 11,30m made 11,75 m at the curved extremity.

The main girders are built-up welded sections and have variable depth between 2,5 meters and 4,50m. The two distances between the three beams are equal: 3,750 meters. Due to the pavement transversal inclination between the internal pier P02 to the abutment A02, the external beams have a small longitudinal slope on that part of the bridge. The slab thickness is constant, and the beams move up and down to follow the transversal inclination. On the central part the deck is entirely symmetric about a longitudinal central axis and the central beam is in a higher position than the lateral ones.

As the three main girders are straight and parallel the slab lateral cantilevers spans varies from a minimum of 1,10 meters to a maximum of 3,20 meters. At the central part of the bridge the cantilevers span is 1,90 meters.

The total slab thickness is 0,30 meters which results from the addition of the precast plank thickness – 0,05 m – to the in-situ concrete thickness – 0,25 m. At the central straight length of the deck the slab is symmetrical and has two equal transversal inclinations of 2,5%. Near the abutment A02 the slab transversal slope is 5,5%.

Transversely to the main beams several cross girders connect them. The distance between the cross girders varies from 6,0 to 9,0 meters and they are built-up welded sections, similar but smaller than the main beams.

### 5 Structural Modelling

The model of the board was developed using CSi SAP2000 v.19 commercial structural analysis software [17]. The model of calculation, formed by linear finite elements - representing the main beams and the bracing systems - and by finite elements of shell - simulating the slab of reinforced concrete - reproduces the real behaviour of the deck. The actions carried out on the deck in the various phases of its construction were applied in the model.

Regarding the modelling of concrete, the simulation of its correct structural response throughout the construction process makes it imperative to consider its deferred behaviour, which is translated by the effects of creep and shrinkage, intrinsic characteristics of the same and that have already been discussed.

A fundamental aspect to take into consideration when trying to know the structural response of the deck is its constructive phasing. It results from the need to create a model that illustrates the evolutionary process in the state of tension of the deck as one advances in the constructive process of the same. To analyse the successive constructive steps, it was used the nonlinear staged construction tool that CSI SAP2000 v.19 [17] commercial software provides and allows non-linear static analysis. This tool allows to define a sequence of constructive phases, being possible to add or remove zones of the structure, to apply loads selectively to certain zones of the structure, as well as to consider the time dependent properties of the materials that constitute them, as for example in concrete, the effect of creep and shrinkage.

The modelling of the structure is carried out in the reference frame X, Y, Z, with the X direction coinciding with the longitudinal axis of the bridge, the Y direction transverse to the deck and the Z direction coincident with the vertical axis. The structure is simulated using a three-dimensional model that comprises the steel structure and the concrete slab, Figure 6.2 identifies the full version of the model used.



Figure 5.1 - Finite element model of the deck (image, CSI SAP2000)

## 6 Safety check during deck construction

### 6.1 Security check in the launch phase

During the assembly process of the steel structure, the most unfavourable stresses that can condition the dimensioning of the structural elements are: locally the effect of the concentrated load - which can result in the local instability of the web "Patch Loading effect" - and globally, the effect of the bending moment on the cantilevers, resulting from the weight of the steel structure and the counterweights placed on the outer free ends of the steel sections. CSI SAP2000 v.19 commercial analysis software removes the following values for the stresses and vertical support reactions related to the fundamental combination of actions:

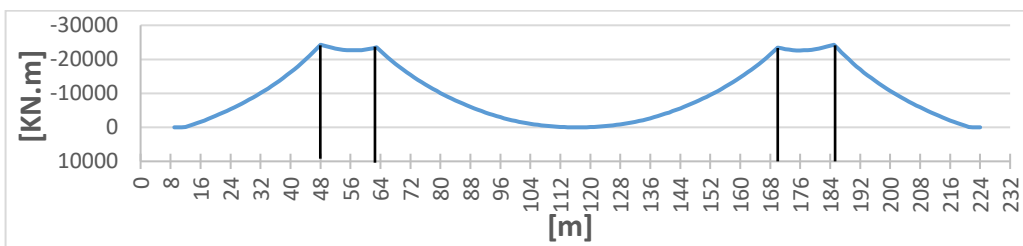


Figure 6.1 - Bending moments on the main beams of the deck - Fundamental Combination of Actions

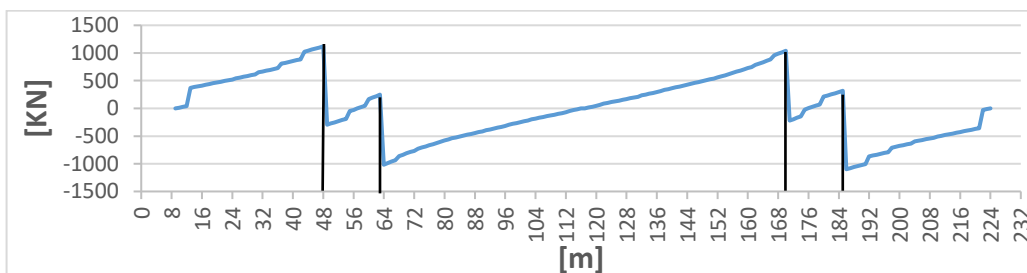
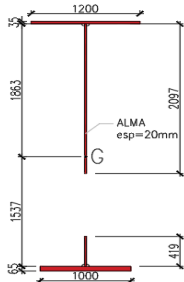


Figure 6.2 - Transversal stresses in the main beams of the deck - Fundamental Combination of Actions

**Table 6.1 - Vertical reactions in the provisional supports - Fundamental Combination of Actions**

		Vertical reaction in the supports [KN]
<i>Position in the longitudinal span</i>	48,0m	1612
	63,0m	1952
	168,0m	1952
	183,0m	1612

For the conditioning section, its classification is made according to EN 1993-1-1, for negative bending moments. The bottom flange (compressed) is found to be class 2 and the web of class 4. The corresponding effective section, determined in accordance with the provisions of EN 1993-1-5, is shown in Figure 6.3. For the elastic analysis the effective properties of this section are those given in Table 6.2.



**Table 6.2 - Characteristics of the effective section and relevant data**

	$I_{yy}$ [m <sup>4</sup> ]	$Z_g$ [m]	$W_{eff,y}$ [m <sup>3</sup> ]	$M_{y,Ed}$ [KNm]	$\sigma_{y,Ed}$ [MPa]
<i>Upper fibre</i>	0,366744	-1,898	-0,1932	-24242	125,5
<i>Bottom fibre</i>		1,602	0,2289		-105,9

**Figure 6.3 - Dimensions of the effective steel section in the conditioning zone [mm]**

Table 6.2 also shows the tensions acting on the extreme fibers of the conditioning section, which shows that there are no stresses higher than the yield stress of the S355 steel.

The interaction between transverse stress and bending moment is discussed in EN 1993 1-5, clause 7.1, where it is stated that this interaction should only be considered if the following two conditions are not verified:

$$\begin{cases} V_{Ed} \leq 0.5 \cdot V_{bw,Rd} \\ M_{Ed} \leq M_{f,Rd} \end{cases} \quad (6.1)$$

Table 6.3 summarizes the relevant results:

**Table 6.3 - Values relevant for the calculation of  $V_{bw,Rd}$**

$a$	$h_w$	$t_w$	$f_{yw}$
6,0m	3,4m	0,02m	355MPa

Thus  $V_{bw,Rd} = 5332$  KN, and given that  $V_{Ed} = 1119$  KN,  $\bar{\eta}_3 = V_{Ed}/V_{bw,Rd} = 0,21 < 0,50$ . It is concluded from this that the resistance to the transverse stress of the web is much higher than the maximum transverse stress acting, so that it is not necessary to make any correction of the value of the resistant bending moment due to the simultaneous occurrence of the transverse stress.

The model of verification of local resistance of the web, taking into account the concentrated load applied to its plane, is defined in EN 1993 1-5, in section 6. In order to calculate the section resistance at a concentrated force, the following input data are required:

**Table 6.4 - Input data for the calculation of  $F_{Rd}$**

$a$	$d$	$t_w$	$f_{yf}$	$f_{yw}$	$b_f$	$t_f$	$S_s$	$E$
6,0m	3,4m	0,02m	335MPa	355MPa	1,0m	0,065m	0,3m	210GPa

It should be noted that the value of the variable  $S_s$  is defined by the Projector and represents the minimum length at which the concentrated force is applied. In Table 8.5 are the intermediate results of the calculation.

**Table 6.5 - Intermediate data from the calculation of**

$k_F$	$F_{CR}$	$m_1$	$m_2$	$l_y$	$\lambda F$	$\chi_F$	$L_{eff}$
6,64	2954KN	50	54,72	1,76m	2,06	0,24	0,43m

This gives the value of  $F_{Rd} = 3037\text{KN}$  for the resistant strength of the web to the local buckling. The acting value is  $F_{Ed} = 1612\text{KN}$  corresponding to the vertical reaction in the bearing. Thus, EN 1993 1-5, in clause 6.6, requires the following verification:

$$\eta_2 = \frac{F_{Ed}}{f_{yw} \cdot L_{eff} \cdot t_w / \gamma_{M1}} = \frac{F_{Ed}}{F_{Rd}} = \frac{1612}{3037} = 0,531 < 1 \quad (6.2)$$

Clause 7.2 of EN 1993 1-5 mentions the interaction between concentrated forces in the web with bending moments and axial stresses. Considering the possibility of interaction between the global effect of the bending moment  $\eta_1$  with the local effect on the web  $\eta_2$ , we have:

$$\eta_1 = \frac{M_{Ed}}{f_y \cdot W_{eff} / \gamma_{M0}} = \frac{24242}{355 \cdot 10^3 \cdot 0,2289 / 1,0} = 0,298 \quad (6.3)$$

$$\eta_2 + 0,8 \cdot \eta_1 = 0,769 \leq 1,4 \quad (6.4)$$

It is ensured security in the most requested section in the launch phase.

## 6.2 Verification of the concreting phase of the slab – without consideration on the effect of shrinkage

In this constructive phase, the safety of the deck is evaluated when the slab is concreted, after the steel structure has been completely launched and is supported on the definitive support devices. Effective deck sections, steel and mixed, were adopted in the safety verification calculations. In this way, in the case of the mixed section, the structure is modelled using the total inertia of the effective sections, homogenized for short duration actions. The loads are defined by the specific weight of the concrete slab, the pre-slabs and the constructive overload. In order to obtain the stress diagram at the end of each phase, it is necessary to add all the diagrams of the previous phases, while being careful to assign to the sections, steel and mixed, the stresses that actually request them in each phase. The phase-to-phase analysis of the obtained stresses allows us to conclude that the bending stress diagrams representative of the most conditioning situations for each of the two mentioned cases - steel resistant section and mixed resistant section - are those shown in Figure 6.4 and Figure 6.5.

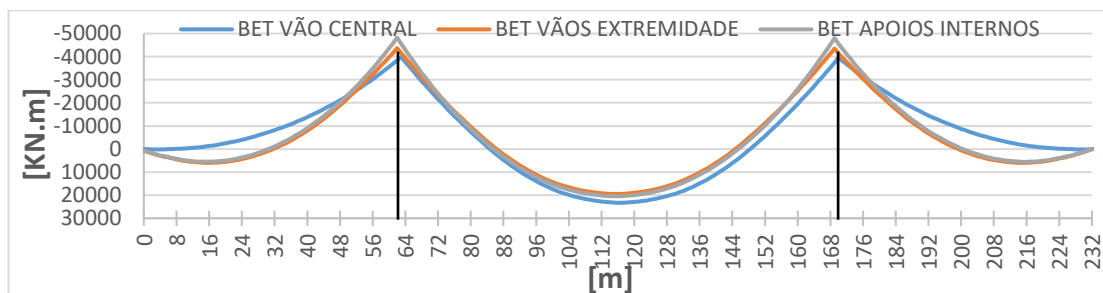


Figure 6.4 - Bending moments of steel sections

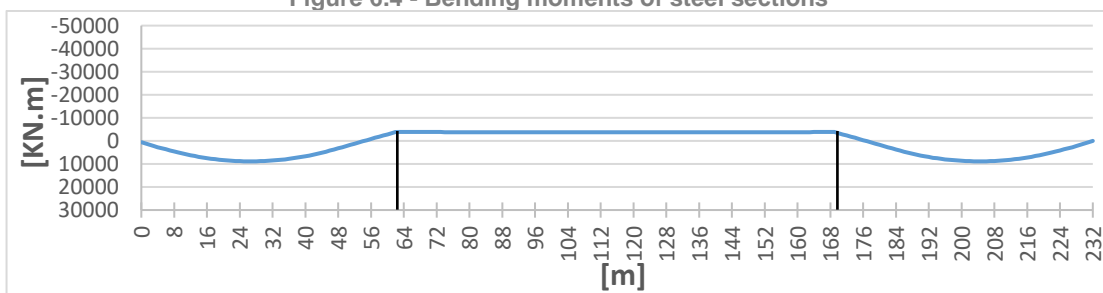


Figure 6.5 - Bending moments of mixed sections

The maximum values recorded in these phases are listed in Table 6.6:

Table 6.6 - Maximum bending moments in the concreting phase

	Steel section	Position in span [m]	Mixed section	Position in span [m]
$M^{max}$ [KNm]	23262	115,5	-	-
$M^{min}$ [KNm]	-48266	63,0	-3795	115,5

During the deck construction, the steel part of the section must always remain elastic and, on the other hand, it must also be ensured that there is no cracking of the concrete slab. For the conditioning steel section, corresponding to a position in the longitudinal span located 63.0m from the end, the classification is made according to EN 1993-1-1, for negative bending moments. The lower flange is found to be class 1 and the web of class 4. The corresponding effective section, determined in accordance with the provisions of EN 1993-1-5, is shown in Figure 6.6. For the elastic analysis, the effective properties of this section are as in Table 6.7.

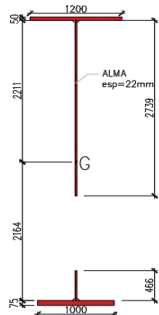


Table 6.7 – Maximum elastic moment of the steel section – internal support zone

	$M_{y,Ed}$ [KNm]	$I_{yy}$ [m <sup>4</sup> ]	$Z_g$ [m]	$W_{eff}$ [m <sup>3</sup> ]	$\sigma_{y,Ed}$ [MPa]
Upper fiber	-48266	0,78229	-2,2612	-0,345962	139,5
Bottom fiber			2,2388	0,3494237	-138,1

Figure 6.6 - Effective steel section dimensions – internal bearing zone [mm]

Table 6.7 also shows the tensions acting on the extreme fibers of the conditioning section, which shows that there are no stresses higher than the yield stress of the S355 steel.

The maximum bending moment for the mixed section is negative and occurs midway through the central span. It is important to ensure that this stress does not cause cracking of the concrete slab during this constructive phase. This bending moment is expressed in Table 6.6.

When assessing the safety of the beams, it is necessary to calculate the geometrical properties of their cross-sections by homogenizing the cross-section in a single material. This homogenization depends on the nature of the action. The homogenization of the section is discussed in section 5.4.22 of EN 1994-2. For the present case of long-term actions, we have:

$$n_L = n_0 \cdot (1 + \psi_L \cdot \varphi_c(t, t_0)) = \frac{210}{32,3} \cdot (1 + 1,1 \cdot 2,1) = 21,50 \quad (8.11)$$

By dividing the effective width of the slab by the homogenization coefficient, the homogeneous width of the slab is obtained, as shown in Figure 6.7.

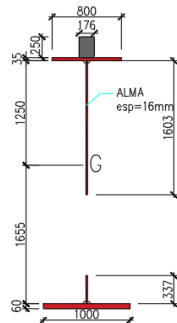


Table 6.8 – Calculation of elastic resistant bending moments

Fiber	$M_{y,Ed}$ [KNm]	$I_{yy}$ [m <sup>4</sup> ]	$Z_g$ [m]	$W_{y,eff}$ [m <sup>3</sup> ]	$\sigma_{y,Ed}$ [MPa]
Concrete	-3795	0,3256804	-1,535	-0,212169	1,3
Upper flange			-1,285	-0,253448	14,9
Bottom flange			1,715	0,189901	-19,9

Figure 6.7 – Mixed homogenised section for short duration factors

As can be seen in Table 6.8, it is verified that the tensile stress in the extreme fiber of the concrete slab is lower than the characteristic value of the average tensile strength ( $f_{ctm}$ ), which, for the concrete used, acquires a value of 2.80MPa. This ensures that there is no cracking of the concrete slab for this constructive phase, where the central zone of the central span already has a mixed resistant section.

## 7 Comparison of results

Figure 7.1 shows the results of the vertical displacements in the main beams of the deck, at the end of the evolutionary sequence of the various phases that make up its constructive process - blue diagram - and in the hypothetical case the deck is soon installed in its final position, without the consideration of the constructive phase - orange diagram - and in the latter case the loads due to the weight of the steel structure, the pre-slabs and the concrete slab were considered as actions in the main beams with the intensities referred, and all to act simultaneously in the three spans.



In the graph of Figure 7.1, we can see that, by comparing the two cases, the value of the maximum vertical displacement, in the central zone of the deck, that is obtained is superior when considering the construction phase. These results are justified by the fact that, in the case where the constructive phase is considered, the additional actions imposed by the effects of shrinkage and creep introduce vertical displacement increments into the main steel beams. For the case where the installed deck is admitted, from the beginning, in its definitive position, all the loads act simultaneously on the beams, so that the resulting deformed configuration does not contemplate a concreting sequence, not registering the displacement increments from rheological effects.

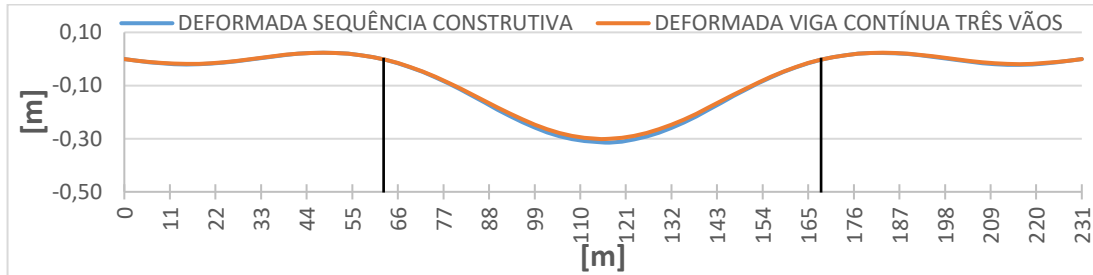


Figure 7.1 – Comparison of vertical displacements acting on the main beams of the deck

The values of the maximum normal tensions acting on the extreme fibres of the welded sections of the main steel beams are then compared for the analysis in which the construction phase of the deck is considered, and in the case where it is not considered. Table 7.1 illustrates the normal tension acting on the extreme fibres of the welded section located on the inner supports of the deck. The stresses were calculated for the maximum negative bending moments acting in these zones, in cases where the constructive phase of the deck is considered or not. The corresponding effective section for determination of the effective flexural modulus has already been illustrated in Figure 6.6.

Table 7.1 – Comparison of the tensions acting on the extreme fibers in the zone of the internal supports

	$M_{y,Ed}^{máx}$ [KNm]	$I_{yy}$ [m <sup>4</sup> ]		$z_g$ [m]	$W_{y,eff}$ [m <sup>3</sup> ]	$\sigma_{y,Ed}$
Construction sequence	-44962	0,78229	Upper fibre	-2,2612	-0,345962	130
			Bottom fibre	2,2388	0,3494237	-128,7
Continuous beam	-42060	0,78229	Upper fibre	-2,2612	-0,345962	121,6
			Bottom fibre	2,2388	0,3494237	-120,4

The following two figures - Figure 7.2 and Figure 7.3 - illustrate the stress distributions to which the slab is subjected, at the end of two distinct concreting sequences. Figure 7.2 shows a continuous concreting sequence, that is, the first section to be concreted corresponds to the extreme span, the next section being concreted is the internal support, followed by a consecutive and continuous sequence until the span of opposite extreme span of the deck. Figure 7.3 shows the results for the concreting sequence already discussed.

These two figures - Figure 7.2 and Figure 7.3 - are only intended to demonstrate that the adoption of a continuous concreting process leads to maximum stresses, acting on the slab, higher than the maximum stresses obtained in the case of a concreting process divided by sections. In fact, the maximum stresses reached in the first case are in the order of magnitude of 4800KPa, while in the second case the values are in the order of 2700KPa. The comparative analysis between the two sequences allows to conclude that the concreting sequence defined for this work is clearly advantageous because it avoids the premature cracking of slab concrete unlike the continuous concreting sequence, where the maximum tensions reached at the end are clearly greater than the value of  $f_{ctm}$ .

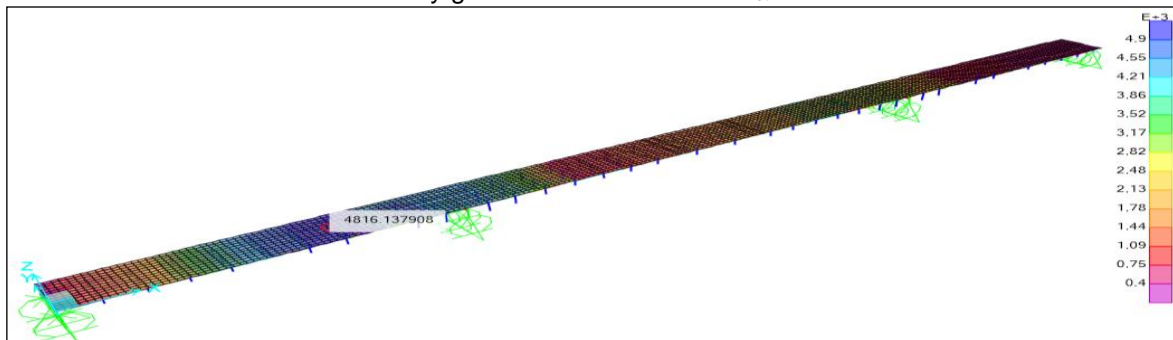


Figure 7.2 – Stress distribution – Continuous concreting

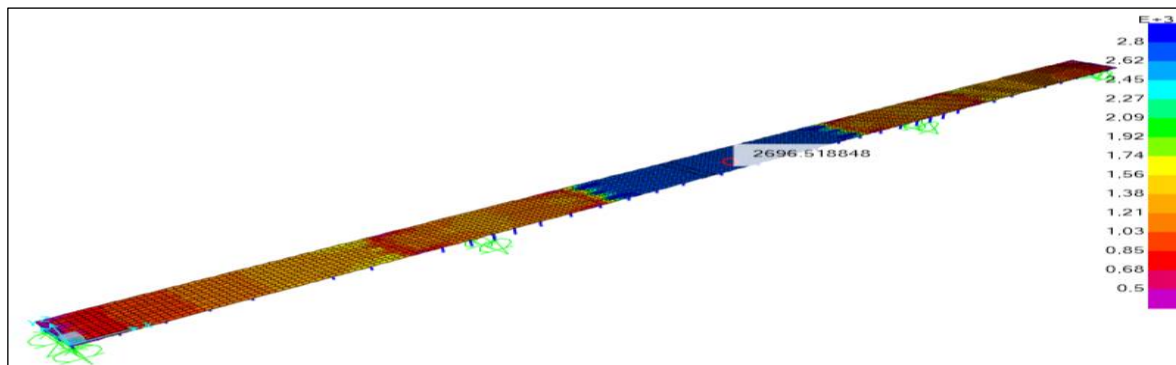


Figure 7.3 – Stress distribution – Sequential concreting

## 8 Conclusions

In the present dissertation are presented the results of the study of the influence of the process of assembly of the steel structure and concrete sequence in the tension state of a composite steel-concrete deck, continuous in three spans. The constructive process considered is the incremental launching of the steel structure and segmented sequential concreting.

The constructive phase of the deck was modelled, considering the evolution of its geometry and the applied loads, followed by the respective structural analyses. The relevant results were extracted at each phase of the process - at the level of stresses, vertical displacements and tensions - which allowed to identify the main conclusions of this work that are listed below:

1. The analysis and evaluation of the stresses and deformations induced in each one of the phases of the process of assembly of the steel deck is a task of great importance, because during these phases are made several operations of adjustment of the deformations and the installed tensions. This analysis allows to define and quantify, for example, the counterweights to be applied in the phases of isostatic behaviour of the deck, the manoeuvres of lifting and/or lowering of the provisional supports of the deck, the operations of removal and installation of new provisional supports.
2. It has been proven that the main beams of the steel deck comply with the safety conditions during the launching stages of the steel structure, and no reinforcements are necessary.
3. It is ensured, in the concreting positions, that there is no yielding of the profiles of the steel main beams. On the other hand, in the phases in which the main beams already have composite behaviour, the maximum negative moments generated - which cause tensile stresses on the slab - are lower than necessary to cause cracking of the slab concrete.
4. Observing the vertical displacements that occur along the longitudinal span of the deck, it is concluded that its maximum value - registered in the middle of the central span in both analyses - is 5% higher in the case in which the constructive phase is considered.
5. The values of the maximum normal tensions acting on the (top and bottom) extreme fibres of the welded sections of the main steel beams are higher in the case where the constructive phase is considered, both in the zones of the internal supports as in the middle of the central span. The largest difference recorded in the values of normal tensions correspond to the upper fibre of the welded section of the central span, in which the consideration of the constructive phase introduces an increase of 19%.
6. The adoption of a step-by-step concreting process is clearly advantageous - as shown in Chapter 7 of this work - in terms of the level of stresses that are installed in the concrete slab at the end of the sequence, instead of the option of continuous concreting. The results obtained allow us to conclude that, at the end of the step-by-step concreting process, the maximum values of tensile stresses installed in the concrete slab are lower than the value of the tensile strength of the concrete ( $f_{ctm}$ ), thus avoiding the cracking of the concrete during this phase. On the other hand, in continuous concreting, the maximum tensile stress values are clearly greater than  $f_{ctm}$ . It is also concluded that the maximum values of tensile stresses at the end of the sequential concreting appear at the central span (zone of positive moments), and that these maximum values are 44% lower in this case compared to that based on a continuous concreting.

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