STRUCTURAL DESIGN OF REINFORCED CONCRETE RUNWAY BEAMS OF OVERHEAD TRAVELLING CRANES IN UNDERGROUND POWERHOUSES

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ABSTRATCT

A very important structure in underground powerhouses is the support beam of the runway rail of a overhead travelling crane that will be used for numerous tasks, namely during the construction phase and in the final phase of the structure. This dissertation will focus on the study of a reinforced concrete solution of the support beam based on the geometric definition of an existing underground powerhouse.

After quantifying the actions to which the structure will submitted, the amount of anchorages was quantified for the safety verifications in the construction phase. It is necessary to check the equilibrium of the cross section of the beam under several load cases that will occur in this phase.

In the final phase, after the implementation of the calculation model, safety was verified to all the relevant ultimate limit state. As it is a beam with a cross-section with greater dimensions than that those found in buildings structures, the cracking was evaluated with detail, taking advantage of a specific software. The safety of the columns and wall supporting the beam have also been verified, ensuring that their dimensions are adequate, and that the design results in normal amounts of reinforcement. Subsequently, all the design of the required reinforcement was performed.

KEYWORDS: Reinforced concrete, bridge crane, structural design, underground powerhouses.

1 INTRODUCTION

1.1 BACKGROUND

Powerhouses are major projects in the field of civil engineering. In addition to transportation and installation of the turbine, the bridge crane can also serve as support for the construction and maintenance.

Powerhouses are divided into two types: conventional surface and underground. An example of a practical case where a underground powerhouse is found is the Salamonde project, Salamonde II, a dam located on the Cávado river in the municipality of Vieira do Minho.

1.2 MAIN GOALS

Based on the initial geometric definition of a structural solution of a support beam of the crane track of a bridge crane, this dissertation aims to quantify the strength of the anchorages for the security verifications in the construction phase, and to design and verify the safety to all relevant ultimate limit state in the definitive phase of the reinforced concrete beams to support the crane track of the bridge crane.

2 BASIC DATA AND CASE STUDY

2.1 LOCATION AND STRUCTURE DEFINITION

It is considered that the structure is located in the Cávado-Rabagão basin, in the municipality of Vieira do Minho, near the Peneda-Gerês National Park. The geometry of the structure is shown in Figure 2.1.

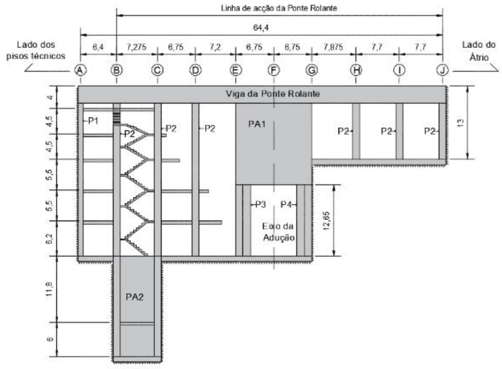

Figure 2.1: *Structure geometry*

2.2 MATERIALS

The structural material used is concrete C25/30 with steel bars reinforcement A500NR, with the characteristics in Table 2.1. The characteristics of the bedrock are also mentioned, taken from [4]. Concrete has the following specifications:

NP EN 206-1: C25/30 XC2 (P) Cl 0,40 D_{max}22 S3.

Table 2.1: *Materials properties*

Concrete C25/30			
f_{ck} $[MPa]$	f_{cd} $[MPa]$	f_{ctm} $[MPa]$	E_c $[GPa]$
25,0	16,7	2,6	31,0
Steel Bars A500NR			
f_{yk} $[MPa]$	f_{yd} $[MPa]$	ε_{yd}	E_s $[GPa]$
500,0	435,0	2,18E-03	210,0
Bedrock: Granite			
f_{md} $[MPa]$		μ	
13,0		0,7	

2.3 ACTIONS

The considered actions are its own weight and the action of the bridge crane, which will be the only variable action. In Figure 2.2 and Figure 2.3 are two-section crane load schemes.

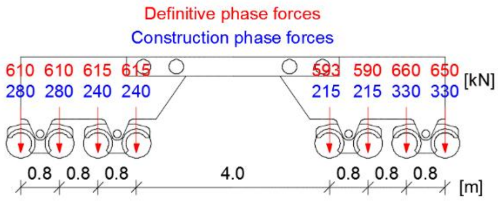

Figure 2.2: *Bridge crane and vertical forces*

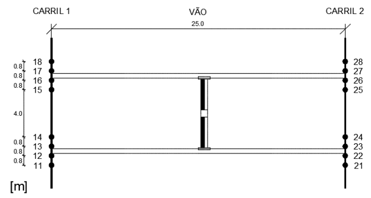

Figure 2.3: *Bridge crane plant [m]*

The operation of the bridge crane will cause vertical and horizontal reactions to the support beam. The vertical reactions are caused by the weight of the entire structure of the bridge crane as well as the materials and machines that the carried in its operation. The horizontal reactions can be divided into reactions transverse to the direction of the crane track and reactions in the direction of the crane track. In the transverse direction the reactions will be caused by the braking of the car (case B1), the inertia forces (case B2) and the forces due to the bridge crossing (case B3). In the rolling direction the two cases are identical (case C) and may occur due to the braking of the crane or the collision forces in the damper to impose the crane stop. In the definitive phase the bridge crane supports the maximum of 450 tons and in the construction phase the bridge supports the maximum of 120 tons. These data were obtained from a supplier and the dynamic coefficients are in accordance with EN13001-3.

All cases of horizontal forces are represented in Figure 2.4 and his maximum values are in Table 2.2.

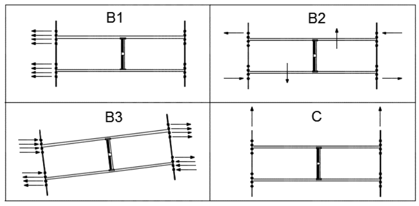

Figure 2.4: *Bridge crane horizontal reactions*

Table 2.2: *Maximum values of forces*

Phase	Vertical $[kN]$	$B1$ $[kN]$	$B2$ $[kN]$	$B3$ $[kN]$	C $[kN]$
Defin.	660	3	0	95	23
Const.	330	3	15	106	23

2.4 Safety Verifications and Combinations of Actions

For this project the safety verifications were adopted according to NP EN1990 [6], verification to the Ultimate Limit States (ELU) and the serviceability limit sate (SLS).The cracking is checked using a GaLa Reinforcement 4.0 software [3].

3 ANALYSIS OF THE STRUCTURE DURING THE CONSTRUCTION STAGE

This chapter will analyze the operation of the support beam for the bridge crane during the construction phase. The solution found is to make use of several anchorages along the beam. The objective of the study of this phase is to design the anchors necessary to equilibrium the forces in the beam.

$$F_{T,H} = \left(P_{viga} + F_{PR,V} - F_{A,V}\right) * \sin\left(\alpha_{sup}\right) + \left(F_{A,H} - F_{PR,H}\right) * \cos\left(\alpha_{sup}\right) \tag{3.1}$$

$$F_{T,V} = \left(P_{viga} + F_{PR,V} - F_{A,V}\right) * \cos\left(\alpha_{sup}\right) - \left(F_{A,H} - F_{PR,H}\right) * \sin\left(\alpha_{sup}\right) \tag{3.2}$$

$$x_T = \frac{P_{viga} * x_G + F_{PR,V} * x_{PR} + F_{A,H} * y_A - F_{PR,H} * y_{PR} - F_{A,V} * x_A}{F_{T,H}} \tag{3.3}$$

3.1 CROSS-SECTION

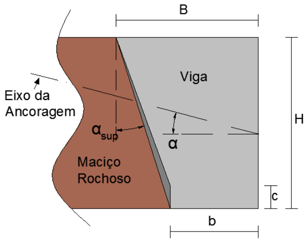

***Figure 3.1:** Generic Cross-Section*

3.2 FREE BODY DIAGRAM

In order to analyze the beam in the construction phase, the equilibrium of the free-body diagram of the beam has to be analyzed. The acting forces on the section and the respective application points are shown in Figure 3.2. There are three forces unknowns ($F_{T,H}, F_{T,V}, x_T$), which are determined using the three fundamental equilibrium equations ((3.1), (3.2) and (3.3)).

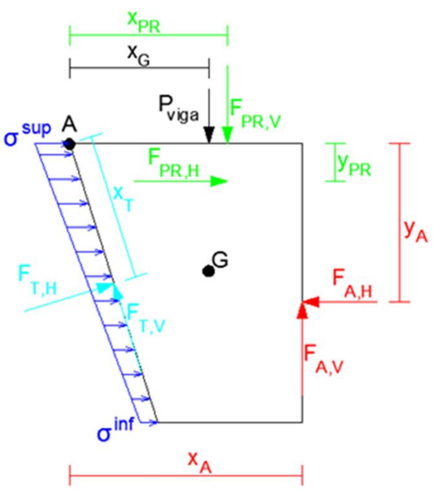

***Figure 3.2:** Acting Forces on cross-section*

Given the three unknowns of the free-body diagram of the beam, it can then calculate the normal and tangential stresses on the contact surface with the rock. For the case of the resultant being inside the central core, the resulting contact stresses are shown in Figure 3.3.

To analyze the stresses, the resultants calculated at the distance x_T from point A are moved to the center of the contact surface. For these forces to be statically equivalent there will appear a moment (M_T),. Thus, the moment is defined by equation (3.4) and the upper and lower normal stresses are defined by expressions (3.5) and (3.6), where the negative sign represents compressions

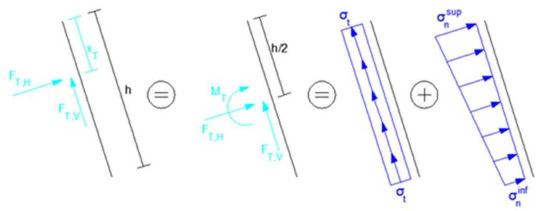

***Figure 3.3:** Contact stresses*

$$M_T = F_{T,H} * \left(\frac{h}{2} - x_T\right) \tag{3.4}$$

$$\sigma_n^{sup} = -\frac{F_{T,H}}{h} - \frac{M_T}{\frac{h^2}{6}} \tag{3.5}$$

$$\sigma_n^{inf} = -\frac{F_{T,H}}{h} + \frac{M_T}{\frac{h^2}{6}} \qquad (3.6)$$

3.3 SAFETY VERIFICATION

The safety check used in this stage will be safety to slide. This verification consists of comparing the maximum resisting friction force (F_a), which results from the multiplication of the resultant of the normal force by the coefficient of friction between the concrete and the bedrock (μ), with the resultant of the tangential stresses on the same surface, calculating a safety factor (F. S.), is intended to be greater or equal to 2.

$$F.S. = \frac{F_{T,H} * \mu}{F_{T,V}} \qquad (3.7)$$

3.4 DESIGN EXAMPLE

In the study case of this dissertation, the dimensions of the beam and the slope of the anchors, are represented in Figure 3.1, and the point of application of the forces, are represented in Figure 3.2 and are indicated in Table 3.1.

Table 3.1: *Main Dimensions*

$H\ [m]$	3,00	$x_{PR}\ [m]$	0,80
$B\ [m]$	2,50	$x_G\ [m]$	1,50
$b\ [m]$	1,55	$y_{PR}\ [m]$	0,00
$c\ [m]$	0,40	$y_A\ [m]$	1,70
$\alpha\ [°]$	15,00	$x_A\ [m]$	2,50
$\alpha_{sup}\ [°]$	17,57	$h\ [m]$	3,15

The actions values to which the beam is subject are shown in Table 3.2.

Table 3.2: *Actions values*

$P_{viga}\ [kN/m]$	147,13
$F_{PR,H}\ [kN/m]$	95,00
$F_{PR,V}\ [kN/m]$	660,00

In order to determine the strength required in the anchorages to verify the safety of the support beam in this phase, the beam subjected to the action of the own weight and the action of the bridge crane is analyzed. To calculate the $P_{útil}$ of the required anchorages, the variation of the safety factor with the variation of values was analyzed. This variation is represented in Figure 3.4, as well as the value of the desired safety factor.

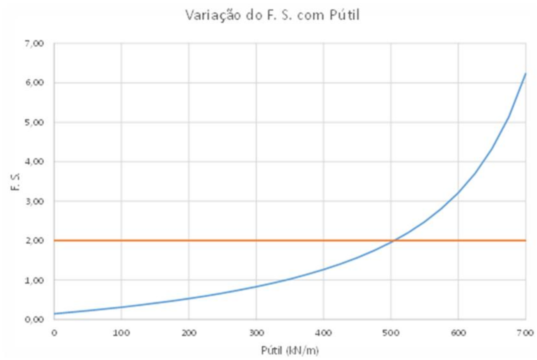

Figure 3.4: *F.S. variation with* $\boldsymbol{P_{útil}}$

This $P_{útil}$ is 505 kN/m, in this way we will conservatively use a $P_{útil}$ equal to 600 kN/m in order to check the safety to the slide. This force will correspond to a force of 1200 kN for each Anchorage, considering the influence length of each anchorage.

Once the load of each anchorage has been defined, the safety in the anchorage zones consists of limiting the compressive stresses located in the concrete as well as designing the reinforcement to absorb the tensile forces.

The reinforcement steel bars were designed considering a maximum tension of 300 MPa. This measure is intended to ensure the control of cracking. In the transverse direction, 4 x 4ϕ12//0.25 ($A_s = 18.08\ cm^2/m$) were centered on each anchor. In the longitudinal direction

$4\phi 20$ ($A_s = 12.57\ cm^2/m$) are placed along the beam, as shown in Figure 3.5.

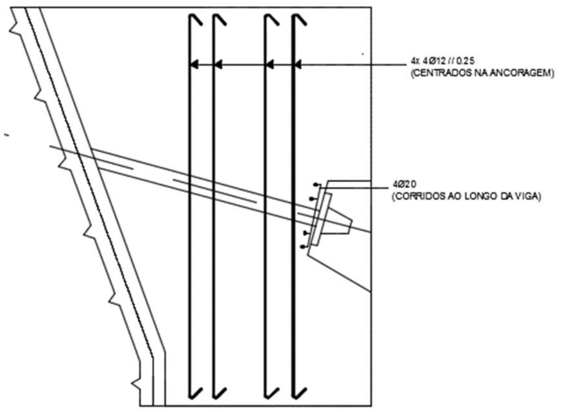

Figure 3.5: *Reinforcement in the Anchorage zone*

4 ANALYSIS OF THE STRUCTURE DURING THE DEFINITIVE PHASE

4.1 Modelling

The calculation of the inner forces caused by the different actions will be done using the software SAP2000, so the calculation model will be built in this program. The calculation model is shown in Figure 4.1. As it is represented in the geometry, Figure 2.1, the crane moves between the axis B and the J, so to define properly its action, this was considered in the software like a moving load

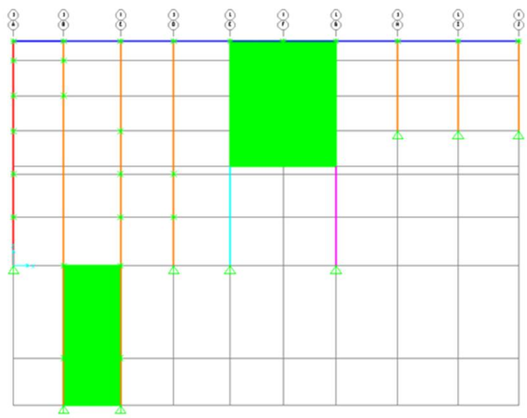

Figure 4.1: *SAP2000 Model*

4.2 Design Bending Moments and Shear

The relevant inner forces of the beam are the bending moment and shear caused by the action of the bridge crane and its own weight.

The envelope of the bending moment diagram (BMD) along the beam for the ULS is shown in Figure 4.2. In Figure 4.3 the DMF envelope in the ELS is represented, in order to analyze the crack in the beam. The envelope of the shear force diagram (SFD) in the beam ULS, Is shown in Figure 4.4.Table 4.1 shows the maximum values of BMD, ULS and SLS, and the ULS, SFD presented above

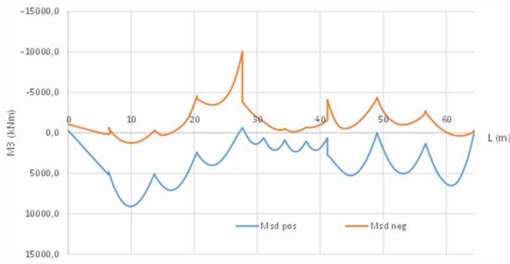

Figure 4.2: *BMD along the beam for the ULS*

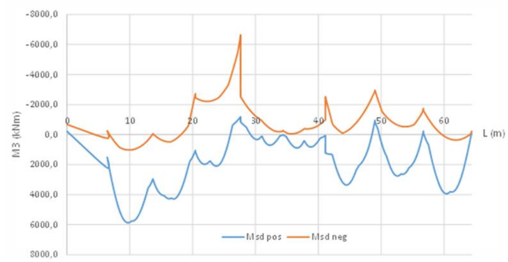

Figure 4.3: *BMD along the beam for the SLS*

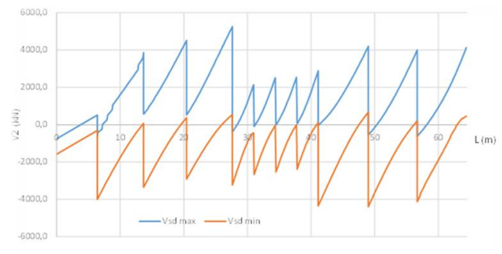

Figure 4.4: *SFD along the beam for the ULS*

Table 4.1: *Bending moments and shear for ULS and SLS verifications*

Max	ULS		SLS
	$V_{sd}\ [kN]$	$M_{sd}\ [kNm]$	$M_k\ [kNm]$
Posit	5253,4	9100,6	5868,7
Neg	-4396,4	-10034,5	-6620,6

In this phase is only intended to verify the safety in the zone of connection between the beam and the columns, so only the maximum and minimum values of the bending moments and transverse stress at the intersection in the ULS are presented. As for the normal force, the value in the base was considered since it is the maximum value and the one that is most relevant in the design.

Table 4.2: *Maximium values for the column design*

N $[kN]$	$V3$ $[kN]$	$M2$ $[kNm]$	$V2$ $[kN]$	$M3$ $[kNm]$
-6346,5	9,1	47,8	-420,9	1117,8

In relation to the wall the absolute maximum of the bending moments in the connection of the wall with the beam was considered. These values are shown in Table 4.3.

Table 4.3: *Maximium values for the wall design*

ULS	N $[kN]$	$M11$ $[kNm/m]$	$M22$ $[kNm/m]$
	18336,4	31,06	69,80

4.3 Beam Safety Verifications

For the ULS bending moment, the conditioning sections for the negative and positive bending moment were dimensioned.

Obtaining the value of the, which is equal to the positive and negative moment, In the *Table 4.5* we calculated the resistance moment for the use of the minimum reinforcement in the beam, thus knowing which areas of the beam that will need reinforcement beyond the zones of maximum moment.

Table 4.4: *Design ELU bending moment*

Section	Middle Span	Support
$M_{sd}\ [kNm]$	9100,6	-10034,5
μ	0,043	0,048
ω	0,045	0,049
$A_s\ [cm^2]$	75,48	83,47
Adopted	21 ϕ 25	20 ϕ 25
$A_{s,adopt}\ [cm^2]$	103,08	98,17

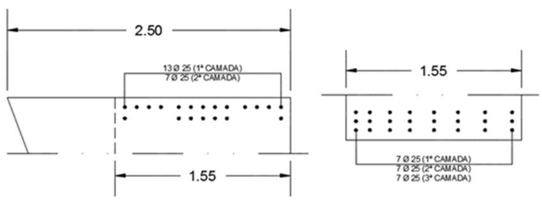

Figure 4.5: *Reinforcement bars for bending moment*

Table 4.5: *Adopted reinforcement*

Minimum	Superior	Inferior
$A_{s,min}\ [cm^2]$	59,72	
Adopted	14 ϕ 25	13 ϕ 25
$A_{sd}\ [cm^2]$	68,72	63,81

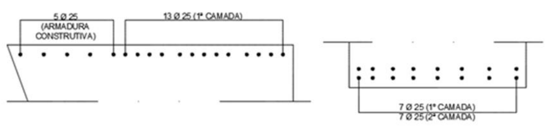

Figure 4.6: *Minimium Reinforcement*

Thus, in Figure 4.7 we can see the envelope of the bending moments with the resistant bending moments of the beam, considering already the reinforcement adopted in the entire beam.

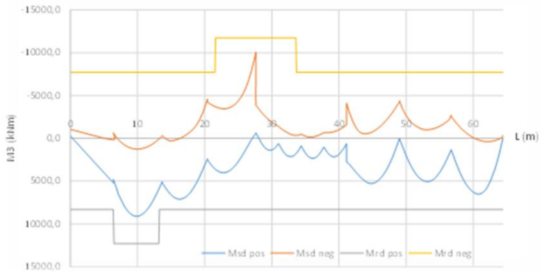

Figure 4.7: *Resisting bending moments for ULS*

For the ULS shear force, the conditioning sections were dimensioned. From the values the absolute maximum of the value of the transverse effort was obtained, analyzing the section subject to this effort.

Table 4.6: *Shear force design*

Section	
$V_{sd}\ [kN]$	5253,4
$\left(\frac{A_{sw}}{s}\right)\ [cm^2/m]$	23,15
Outer stirrup	ϕ 16 // 0,25
Inner stirrup	ϕ 12 // 0,25

Due to the dimensions of the section it is necessary to provide an outer and an inner stirrup.

For the cracking limit states it was determined that the conditioning sections coincide with the values of the maximum acting moment. Since it is a beam that is more than 1,0 meters in height, the minimum reinforcement of the web must be calculated. This armature is important because the cracking can be transmitted from the lower zone, or higher depending on the moment, to the zone of the web

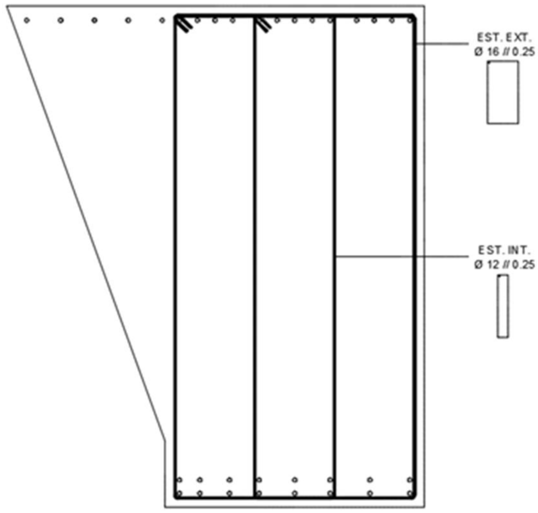

Figure 4.8: *Transverse reinforcement*

Table 4.7: *Web reinforcement*

$b\ [m]$	1,55
$k * k_c$	0,50
$A_{ct}\ [m^2/m]$	0,78
$f_{ct.ef}\ [MPa]$	2,60
$\sigma_s\ [MPa]$	500,00
$A_{s,min}\ [cm^2/m/face]$	20,15
Adopted	ϕ 20 // 0,15

With the determined reinforcement, the crack width (w_k) was then calculated by the expression (7.8) of EC2.

Table 4.8: *Cracking width with EC2*

Crack width	Mid span	Support
$s_{r,max}\ [m]$	0,330	0,338
$\varepsilon_{sm} - \varepsilon_{cm}$	5,54E-04	7,47E-04
$w_k\ [mm]$	0,18	0,25
$w_{k,max}\ [mm]$	0,30	0,30

Due to the great complexity of analyzing the cracking in the beam and due to the simplifications made by EC2, we used the GaLa software [3] to determine the cracking in the beam more accurately. In this calculation tool it is possible to analyze the most conditioning sections taking into account the contribution of all the longitudinal reinforcement and with the rigorous geometry of the section of the beam obtaining a more accurate and closer to reality. The comparison of the results are presented in Table 4.9.

Table 4.9: *Cracking widths*

Crack width	EC2 $[mm]$ (simp. Method)	GaLa $[mm]$	Diference [%]
Mid span	0,18	0,23	20,2
Support	0,25	0,19	-25,3

The difference in values, both in the mid-span as well as in the support, is around 20%. This difference can be explained because of the program considers the actual distribution of reinforcement in each section. Therefore, the values obtained with the program are closer to reality.

Based on EC2 a limit of L/250 was established for the total deformation in the mid-span due combination of quasi-permanent actions and a limit of L/1000 for the mobile load, these restrictions generally guarantee the full functioning of the bridge.

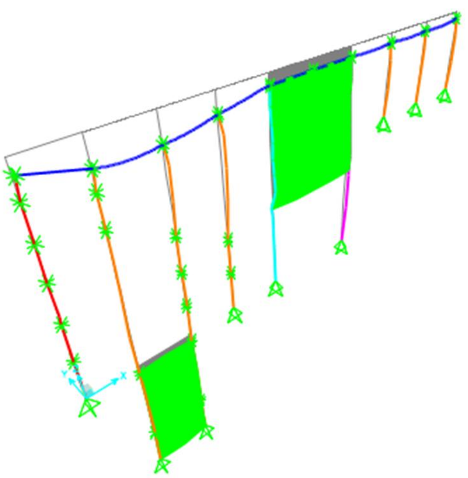

Figure 4.9: *Deformation for the SLS*

It is verified that the maximum deformation occurs in the middle of the second span of the beam, to be observed from left to right, in the zone of maximum positive moment, as expected.

Table 4.10: *Deformation values*

Max Deformation	Mobile load	Combination
$L\ (m)$	7,275	
$\Delta\ (mm)$	1,50	4,38
$\Delta_{max}\ (mm)$	7,275	29,10

In order to complete the safety checks of the beam, it is necessary to check the action of the concentrated force of a bridge crane wheel on the beam surface. This verification will be analogous to that made for the anchorage zone at the construction phase. Thus, in the direction transverse to the force were placed 2φ12//0.25 ($A_s = 9{,}04\ cm^2/m$).. In the longitudinal direction 5φ16 ($A_s = 10{,}05\ cm^2/m$) were centered and run under the rail along the beam.

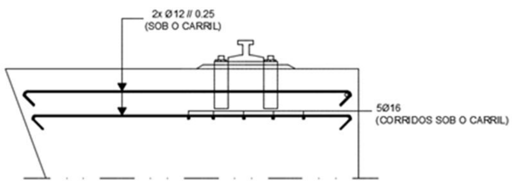

Figure 4.10: *Reinforcement under the bridge crane*

4.4 Column Safety Verifications

It has been found that the minimum reinforcement is sufficient to verify safety on all sections. A regular distribution of ϕ25 bars spaced 0.10 meters on each side of the columns is adopted, as shown in Figure 4.11. In Figure 4.12 is represented the stirrups, this stirrups will be important for securing the concrete, preventing localized buckling of the bars and keeping the longitudinal reinforcement bars in position during placement of concrete. For the structural wall we chose to create two embedded, at the end with 1.25 meters side, and capable of resisting the bending moment (M11).

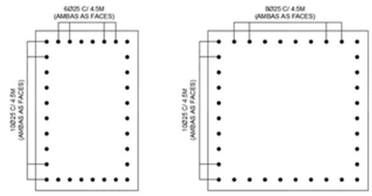

Figure 4.11: *Columns longitudinal reinforcement (stirrups not reprensented)*

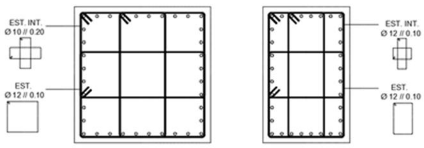

Figure 4.12: *Reinforcement of columns*

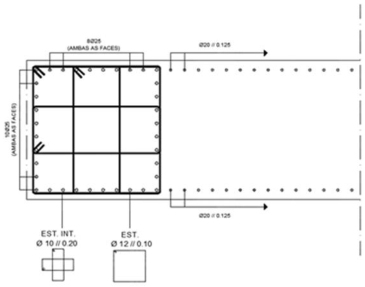

Figure 4.13: *Wall reinforcement (Stirrups not represented)*

5 CONCLUSION

The main objective of this dissertation was to verify the safety of the reinforced concrete support beam of the crane track of a bridge crane in underground powerhouses, both in the construction phase and in the final phase. This objective was fulfilled with the quantification of amount of anchorage, in the construction phase, and with the design and detailing of reinforcement, in the definitive phase.

At the construction stage, the amount and force at each anchorage, were determined to verify the safety to the sliding, $P_{útil}$ = 1200 kN/anchorage. In the definitive phase all the necessary reinforcements were dimensioned for the beam to verify the safety to all the relevant limit states. With respect to the columns and the structural wall, the minimum reinforcement is sufficient to resist all actions, since they have high dimensions for the inner forces to which they will be subjected.

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