Design of a roadway cable-stayed bridge

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Abstract

The detail design is presented for a roadway cable-stayed bridge. The pre-design was made dividing the deck in three different structural systems: two roadway viaducts and a cable-stayed bridge on the center.
Two different solutions for deck's cross-section are proposed: a prestress reinforce concrete deck (Solution A) and a composite steel-concrete deck (Solution B).
The design constrains, the materials, the actions/loads and design criteria are first defined. The structural analysis is performed, modeling the structure with the software SAP2000. With the results of these models, both Ultimate Limit State and Service Limit State verifications are performed, following the recent Eurocodes Standards.
The main conclusions of the design are that the design changes introduced to the initial structural system improve the behavior of the deck under static and especially seismic actions. It is concluded also that the composite deck is preferred for the cable-stayed region of the deck, namely due to the reduction of dead load that can be achieved. However, the suspended composite deck is particularly flexible to eccentrically loads, and presents a very low first torsional frequency.
Finally, the two solutions are detailed in specific drawings for each solution and compared in terms of construction feasibility and economics terms, being the concrete deck a more economical solution.

Key-words

Cable-stayed bridges; bridge design; axial suspension; pre-stressed reinforced concrete deck; composite steel-concrete deck

1. INTRODUCTION

The detail design is presented for a roadway cable-stayed bridge, in which two solutions were studied. The bridge is in Santo Tirso, in the north of Portugal, where the seismic action has a small magnitude. The foundation design was based on geotechnical survey executed by bore wall made at 30m spanning. All design procedures follow the Eurocodes Standards.
The present design is based on a pre-design; its development aims to improve the structural behavior for several actions and loads, which were assessed for the concrete deck shown on **Figure 1**. To achieve the best structural system for both solutions was used the software SAP2000 to do the numerical models and simulate the bridges behavior under several loads.
On the other hand, the study of deck prestress cables for concrete solution is also covered on this document. An alternative composite steel-concrete deck was also studied, with the goal of a comparative assess with the first RC deck solution.

2. PRE-DESIGN SOLUTION

The pre-design was developed for the deck configuration shown on **Figure 1**, with three separated structural systems: two roadway viaducts and a cable-stayed bridge at the center, separated by expansion joints. This solution presented several drawbacks; at it was decided to have a continuous deck throughout the viaducts and the main bridge. This decision allows increasing the length of the spans that make the transition from the viaducts to the main bridge and reduce the deformability of the central stayed span.
For the composite solution, the spans and the support system are kept identical.

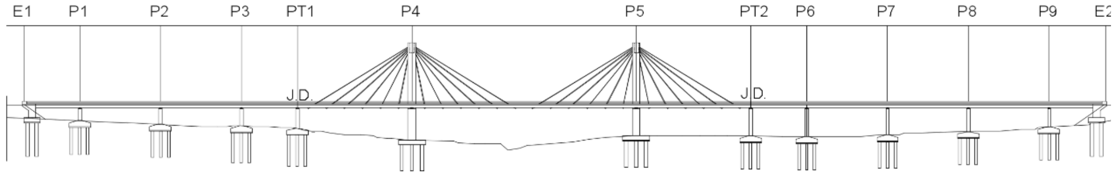

Figure 1 – Front view - scheme design of concrete solution

3. STRUCTURAL SYSTEM

3.1. Deck – Solution A

For the concrete deck, the solution is a slab supported by two ribs with voids at the span region to reduce the self-weight of the deck (**Figure 2**).

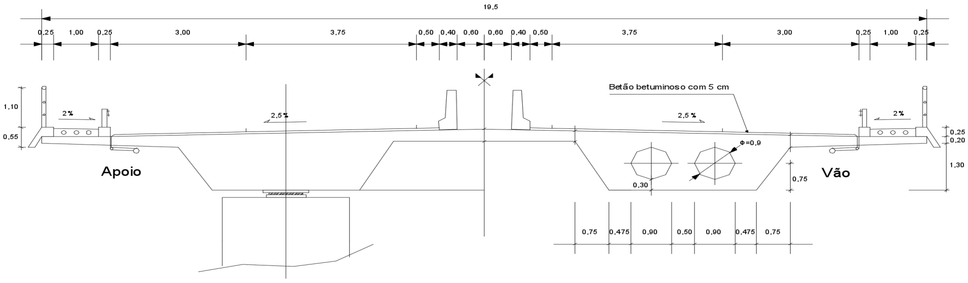

Figure 2 – Deck's cross section – Solution A

3.2. Deck – Solution B

For the composite deck, the solution is a slab supported by four longitudinal steel plate girders and cross-girders spaced at 4 m. The spans are the same span as in solution A (**Figure 3**).

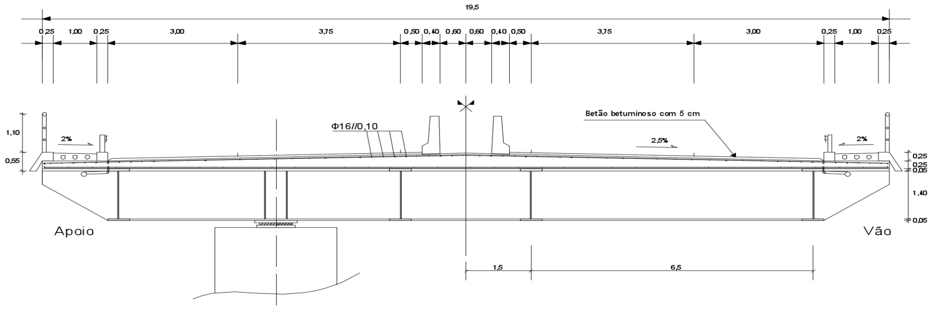

Figure 3 – Deck's cross-section – Solution B

3.3. Suspension system

The suspension system is shown in **Figure 4**. Cables are spaced at 8 m for design A and B. Cable's lengths and number of strands are given by **Table 1**.

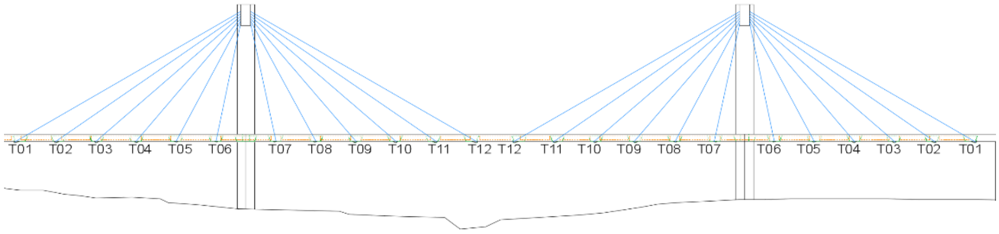

Figure 4 – Suspension system – front view

Table 1 – Cables proprieties

Solution A				Solution B			
Cable	**Strains**	**F_{GUT} [KN]**	**d [m]**	**Cable**	**Strains**	**F_{GUT} [KN]**	**d [m]**
T01	0,6"-61	17019	-46	T01	0,6"-43	11997	-46
T02	0,6"-61		-38	T02	0,6"-37	10323	-38
T03	0,6"-55	15345	-30	T03	0,6"-37		-30
T04	0,6"-55		-22	T04	0,6"-37		-22
T05	0,6"-55		-14	T05	0,6"-31	8649	-14
T06	0,6"-55		-6	T06	0,6"-31		-6
T07	0,6"-55		6	T07	0,6"-31		6
T08	0,6"-55		14	T08	0,6"-31		14
T09	0,6"-55		22	T09	0,6"-37	10323	22
T10	0,6"-55		30	T10	0,6"-37		30
T11	0,6"-61	17019	38	T11	0,6"-37		38
T12	0,6"-61		46	T12	0,6"-43	11997	46

3.4. Piers

Piers solution is shown in the **Figure 5,** which uses two rectangular cross-sections and the piles with circular cross-section of 1.2 m of diameter. This solution supports the viaducts deck; For the cable-stayed bridge a unique pier supporting the towers is adopted.

Both geometrical and mass proprieties are presented in **Table 2**.

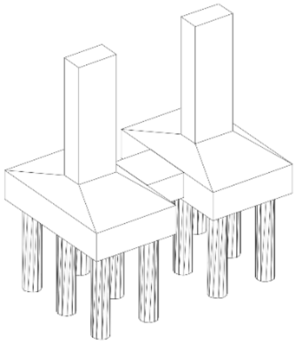

Figure 5 – Piers and piles

Table 2 – Geometrical and mass proprieties of the Piers

Piers	P1	P2	P3	P4	P5	P6	P7	P8	P9	P10	P11
H_{Pier} [m]	5	6,5	7,8	9,5	12,8	12,8	12,5	12,9	11,3	10	8
$b_{section}$ [m]	1,50				3,50		1,50				
$h_{section}$ [m]	2,80				6,00		2,80				
$A_{section}$ [m^2]	4,20				21,00		4,20				
I_L [m^4]	2,74				63,00		2,74				
I_T [m^4]	0,79				21,44		0,79				
W_L [m^3]	1,05				12,25		1,05				
W_T [m^3]	1,96				21,00		1,96				

4. Structural Materials

4.1. Concrete

The concrete proprieties for each solution are given by **Table 3.**

Table 3 – Concrete proprieties

Solution	Concrete	f_{ctm} [MPa]	f_{cd} [MPa]	f_{ck} [MPa]	f_{cm} [MPa]	E_{cm} [GPa]	E_c [GPa]
A	C30/37	2,9	20	30	38	33	33
B	C30/37	2,9	20	30	38	33	34,65
	C40/50	3,5	26,7	40	48	35	36,75

4.2. Steel

The steel proprieties for each solution and for each element are given by the **Tables 4, 5** and **6**

Table 4 – Steel proprieties for reinforcement – Solution A and B

Steel	f_{yd} [MPa]	f_{yk} [MPa]	E_s [GPa]
A500NR	435	500	210

Table 5 – Steel proprieties for pre-stress – Solution A

Steel	$f_{p0,1k}$ [MPa]	f_{pk} [MPa]	E_s [GPa]
High resistance 0,6"S	1670	1860	195

Table 6 – Steel proprieties for beams – Solution B

Steel	t [mm]	f_y [MPa]	f_u [MPa]	E_s [GPa]
S355 NL	$t < 40$	355	490	210
	$40 < t < 80$	335	470	210

5. Actions

5.1. Permanent actions

The self-weight for each solution and for each element are presented in **Table 7.** The Super Imposed Dead Loads for each solution and for each element are given by the **Table 8.**

Table 7 – Deck Self- weight

Solution	Element	Analysis	
		Transversal [kN/m²]	Longitudinal [kN/m]
A	Cantilever	13,75	41,25
	Central slab	8,125	32,50
	Hollow section	-	267,95
	Full section	-	331,57
B	Main beam	-	25,41
	Cross beam	-	17,79
	Slab	6,25	121,88

Table 8 – Super Imposed Dead Loads

Solution	**Element**	**Analysis**	
		Transversal	**Longitudinal**
A	**Unities**	**[kN/m²]**	**[kN/m]**
	Walkway	3,6	7,2
	Bituminous layer	1,2	17,4
	Units	**[kN/m]**	**[kN/m]**
	Fascia Beams	2,25	4,5
	Hand rails	0,75	1,5
	Safety barriers	0,5	1
	Curbs	1,25	2,5
	New jerseys	10	20
B	**Units**	**[kN/m²]**	**[kN/m]**
	Walkway	3,6	7,2
	Bituminous layer	1,2	17,4
	Units	**[kN/m]**	**[kN/m]**
	Fascia Beams	2,25	4,5
	Hand rails	0,75	1,5
	Safety barriers	0,5	1
	Curbs	1,25	2,5
	New jerseys	10	20

5.2. Creep and Retraction

According the procedures given by EN 1992-1-1 the equivalent temperature used to simulate the creep and retraction for each solution are given by the **Table 9.**

Table 9 – Equivalent temperature – creep and retraction

Solution A	Viaduct - $\Delta T_{eq} = 48{,}3°C$
	Cable-stayed bridge - $\Delta T_{eq} = 63{,}9°C$
Solution B	$\Delta T_{eq,ret} = 33{,}4°C$

5.3. Variable actions

5.3.1. Road traffic actions

The road traffic actions for both solutions are defined in **Table 10.**

Table 10 – Road traffic loads

Lane	**TS [kN]**	**UDL [kN/m²]**
1	600	9
2	400	2,5
3	200	2,5
Rest	0	2,5

5.3.2. Wind actions

For design proposes two wind action scenarios are considered: deck loaded and unloaded. For these two situations and for each solution the wind action is given by **Table 11.**

Table 11 – wind action

Solution A	$F_{w,unloaded} = 2{,}24\ kN/m$
	$F_{w,loaded} = 4{,}03\ kN/m$
Solution B	$F_{w,unloaded} = 2{,}62\ kN/m$
	$F_{w,loaded} = 4{,}31\ kN/m$

5.3.3. Temperature actions

According the procedures given by EN 1992-1-5 the vertical component of differential temperature (heat and cool) are given in **Table 12.**

Table 12 – Temperature action

Solution A	$\Delta T_{M,heat} = 15°\text{C}$
	$\Delta T_{M,cool} = -5°\text{C}$
Solution B	$\Delta T_{M,heat} = 15°\text{C}$
	$\Delta T_{M,cool} = -15°\text{C}$

5.3.4. Seismic actions

According the procedures given by EN 1998-2 the seismic action definition are given by the **Table 12.** Regarding to the behavior coefficient for the solution A, it was used as 1.0 (due to the existence of the oleo-dynamic devices linking the deck to both abutments); and for solution B it was adopted a 1.5 coefficient. Class IV importance was used by Owner request.

Table 13 – seismic action definition

Direction	ag_r [m/s^2]	Class	γ_I	ag [m/s^2]	S_{max} [m/s^2]	T_B [s]	T_C [s]	T_D [s]
AS1-1.3	0,35	IV	1,95	0,68	1,35	0,1	0,6	2
AS2-2.3	0,80	IV	1,50	1,20	1,35	0,1	0,25	2

5.4. Load combinations

Following EN1990 – 2 the load combinations covered on this project design are given in **Table 14**

Table 14 Load combinations

ULS– Deck (Fund.)	$1{,}35\times(SW+OPA)+1{,}00\times CBL+1{,}20\times HP+1{,}35\times(UDL+TS+0{,}4FCT)$
ULS – Piers (UDL +VT principals)	$1{,}35\times(SW+OPA)+1{,}35\times(UDL+TS+0{,}4FCT)+1{,}5\times(0{,}6Q_w)$
ULS – Piers (Q_w principal +loaded deck)	$1{,}35\times(SW+OPA)+1{,}5\times Q_w+1{,}35\times(0{,}4UDL+0{,}75TS+0{,}4FCT)$
ULS – Piers (Q_w princ. + unloaded deck)	$1{,}35\times(SW+OPA)+1{,}5\times Q_w$
ULS – Piers (Seismic)	$(SW+OPA)+F_E$
ULS – Fundations AC1 – C	$(SW+OPA)$
ULS– Fundations AC1 – C1	$1{,}35\times(SW+OPA)+1{,}5\times(UDL+TS+FCT+Q_w)$
ULS – Fundations AC1 – C21	$(SW+OPA)$
ULS – Fundations AC1 – C2	$(SW+OPA)+1{,}3\times(UDL+TS+FCT+Q_w)$
SLS– deck and Piers - Frequent	$(SW+OPA+CBL+PE)+0{,}4\times UDL+0{,}75\times TS+0{,}5\times\Delta T_d$
SLS – Deck and Piers -Quasi-permanent	$(SW+OPA+CBL+PE)+0{,}5\times\Delta T_d$
SLS – supports – rare	$SW+OPA+CBL+PE+CREEP+RET+UDL+TS+\Delta T_d$
ULS– cables (Fund.)	$1{,}35\times\left((SW+CBL)+(UDL+TS+FCT)\right)+0{,}6\times\Delta T_d$
SLS – Cables – Characteristics	$(SW+CBL)+(UDL+TS+FCT)+0{,}6\times\Delta T_d$

6. Numerical model analysis

On this section, the analysis is based on the finite element software SAP 2000. This program allowed to create tridimensional models where could be simulated the actions quantified on the last section. Since there are two different solutions with some particularities were created two different models. On this modeling were simulated the geometrical properties of all elements which include the deck's cross section, cross beams, pillars, towers and cable-stayed

6.1. Concrete solution

The concrete deck solution is modeled as shown in the **Figure 6.**

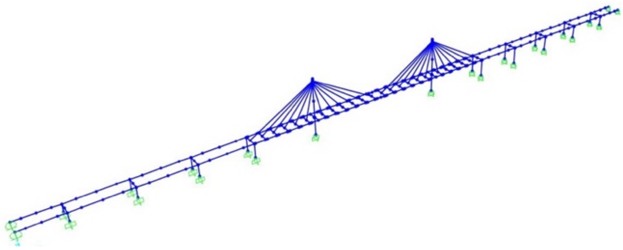

Figure 6 – Concrete Solution – Tridimensional Model

Since the model is defined with two frames along the bridge, **Table 15** resumes the ½ cross section properties.

Table 15– Concrete Solution –½ Cross section properties

	L [m]	h_{CS} [m]	A_{CS} [m^2]	I_{yy} [m^4]	I_{zz} [m^4]
Compact Section	9,75	1,5	8,08	1,80	29,30
Voided section	9,75	1,5	6,81	1,67	28,61

The loads are applied on the deck is according with section 5, which includes the permanent and variable actions, defined as distributed loads, thermal gradients, UDL as a distributed moving load and TS as a point moving load, and prestress tendons. Deck cross beams at supports are modeled using properties from **Table 16** with the self-weight defined as a distributed load along the transversal frames. Pier properties are according to the section 3.4 and the loads are defined as a concentrated forces depending of the height of the element. Foundations are modeled as a rigid.

Table 16 - Concrete Solution – Cross beam properties

	L [m]	h_{CS} [m]	A_{CS} [m^2]	I_{yy} [m^4]	I_{zz} [m^4]
Full Section	2,5	1,3	3,0	1,625	0,3552

7.2. Composite solution

The composite deck is modeled according with **Figure 7.** The analysis of the composite deck for different actions is executed with four different models as described on **Table 17.**

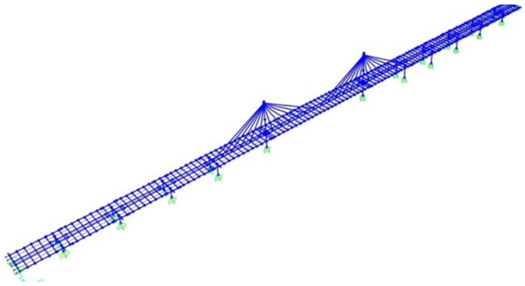

Figure 7 – Composite Solution – Tridimensional Model

Table 17 – Composite Solution –SAP 2000 Models

Model	Deck	Actions	E_s [GPa]	E_c
0	Steel	Steel self-weight	210	-
		Cable-stayed (Pre-design)		
		Concrete slab self-weight		
1	Composite	OPA	210	$E_C' = {E_c}/{(1+\psi\varphi)}$ $\psi = 1{,}10 \ \varphi = 2{,}50$
		Cable-stayed (Final design)		
2	Composite	Retraction	210	$E_C' = {E_c}/{(1+\psi\varphi)}$ $\psi = 0{,}55 \ \varphi = 2{,}50$
3	Composite	Leading Loads	210	$E_C' = {E_c}/{(1+\psi\varphi)}$ $\psi = 0 \ \varphi = 2{,}50$
		Temperature		

The steel deck is made by four main beams each one with 494 m. To improve the stiffness of the structure were created cross beams with the same properties of the main beams described on **Table 18** with the self-weight defined by γ. The concrete slab is 0,25 cm thick and its self-weight is defined by a distributed loads. These properties are only used for model 0.

Table 18 - Composite Solution – Cross section properties

	A_{CS} [m^2]	I_{yy} [m^4]	I_{zz} [m^4]
Structural Steel Frame	0,0076	0,0265	1,04E-03

To simulate the composite structure on *SAP2000* are defined three different models, each one with a different homogeneous section defined by $\eta = E_s/E'_c$. **Table 19** resumes the different sections defined for each model. Pier properties are according to the section 3.4 and the loads are defined as concentrated forces depending on the height of the element. Foundations are executed with piles and modeling as rigid.

Table 19 – Composite Solution – Homogeneous cross section properties

Model	η	Section	A_{CS} [m^2]	I_{yy} [m^4]	I_{zz} [m^4]
1	22,50	Cracked	0,138	0,055	0,067
		Non-cracked - Interior	0,176	0,068	0,002
		Non-cracked - Exterior	0,165	0,065	0,001
2	14,25	Cracked	0,138	0,055	0,067
		Non-cracked - Interior	0,217	0,077	0,003
		Non-cracked - Exterior	0,199	0,074	0,002
3	6	Cracked	0,138	0,055	0,067
		Non-cracked - Interior	0,369	0,095	0,025
		Non-cracked - Exterior	0,327	0,091	0,015

7.3. Cables

The method used to determinate the pre-stress to apply to stayed-cables is based on the displacement minimization for the permanent load. It is applied a uniform temperature variation of -1000 °C to each cable. Then are defined two matrixes, one that translate the influence of the applied temperature on the displacement control points $[\delta]$, and the other the axial force on each cable $[N]$. The displacements for the permanent loads on the same control points are defined on vector $\{\delta_{PA}\}$.

$$[\delta]\{coef\} = \{-\delta_{PA}\} \tag{1}$$

$$[N]\{coef\} = \{P\} \tag{2}$$

Solving the **Equations 1** and **2** allow to define the coefficient that multiplied by the temperature off -1000 °C on each cable, allow to reduce to zero the displacements of the control sections for the permanent action.

7. Service Limit State

For the verification of the SLS are considered the quasi-permanent action combination to verified the decompression of the section and the frequent action combination to verify the concrete cracking. In this section are verified the cross section on the longitudinal and transversal direction, the cable-stayed and also de piers and the towers.

7.1. Solution A

Cross section Verification

The transversal verification is about to assure that cracks does not exceed the limit (w<0,3mm for bridges) as show on the **Table 20**.

Table 20 - Concrete solution – Cracking verification

Parameters	Cantilever	Central Slab
ϕ_{design}	16	16
σ_s [MPa]	221	158
ϕ_{max}	32	32
σ_c [MPa] < 20 MPa	11,70	9,30

The longitudinal verification is guaranteed by the prestress cables layout and values along the cross section that assure that deck cross sections do not exceed the limit of cracking (2,9 MPa) and the decompression

(0 MPa) for the respective combination. The prestress cables solution is show on the **Table 21**. Due to the high compression, the cable stayed bridge deck needs also transversal pre stressed cables as in **Table 22**.

Table 21 - Concrete solution – Pre stress cables design

Structure	Strains [inches]	Nº Strains	Nº Cables - Support	Nº Cables - Span	P_∞ [kN]
Orient viaduct	0,6’’	19	10	10	30400
Occident Viaduct			10	10	30400
Cable-stayed bridge			7	10	21280/30400

Table 22 – Concrete Solution – Transversal pre stress on the cross beams

Cable-stayed	T01	T02	T03	T04	T05	T06	T07	T08	T09	T10	T11	T12
Strains 0,6’’ - 19	4	5	5	6	6	7	7	7	6	5	5	4
P_∞ [kN]	12160	15200	15200	18240	18240	21280	21280	21280	18240	15200	15200	12160

Cable-Stayed Verification

For SLS, when loaded to maximum tension, the cable stayed shouldn’t exceed the 0,5Fgut defined on section 3.3. All stayed verified this limit.

Piers and towers verification

Table 23– Concrete Solution – Piers and towers cracking verification

Pier	P1	P2	P3	P4	P5	PT5	P6	PT6	P7	P8	P9	P10	P11
σ^+ [MPa]	0,0	0,7	1,5	1,2	-2,9	-16,5	-2,1	-13,7	2,2	4,4	3,1	2,6	1,4
σ^- [MPa]	-4,6	-5,5	-6,8	-4,0	-3,1	-17,3	-4,0	-19,8	-5,0	-10,0	-8,1	-7,7	-6,1
f_{ctm} [MPa]	2,9												
f_{cd} [MPa]	20												

7.2. Solution B

Cross section Verification

The transversal and longitudinal verification is about to assure that the cracks on the slab do not exceed the limit (w<0,3mm for bridges) as show on the **Table 24**.

Table 24 - Composite solution – Cracking verification

	Transversal		Longitudinal	
Parameters	Support	Central span	Support	Central span
ϕ_{design}	12	10	16	12
σ_s [MPa]	227	234	259	293
ϕ_{max}	32	32	32	32
σ_c [MPa] < 20 MPa	18,21	12,95	16,51	11,78

Cable Stayed Verification

For SLS, when loaded to maximum tension, the stays should not exceed the 0,5Fgut defined on section 3.3. All stay-cables verified this limit.

Piers and towers verification

Table 25 – Composite Solution – Piers and towers cracking verification

Pier	P1	P2	P3	P4	P5	PT5	P6	PT6	P7	P8	P9	P10	P11
σ^+ [MPa]	0,0	0,4	0,8	0,8	-1,4	-6,6	-0,2	-2,2	1,4	2,6	1,9	1,5	0,8
σ^- [MPa]	-2,7	-3,3	-3,9	-2,5	-1,8	-8,4	-3,0	-12,8	-3,3	-5,9	-5,0	-4,6	-3,6
f_{ctm} [MPa]	2,9												
f_{cd} [MPa]	20												

8. Ultimate Limit State

For the verification of the ULS are considered the fundamental combinations. In this section are verified the cross section on the longitudinal and transversal direction, the cable stayed deck and also de piers and the tower sections.

8.1. Solution A

Cross section verification

The transversal verification is about to reinforce the section to verify the M_{Ed}. In terms of longitudinal verification are defined four critical sections that allow to calculate reinforcements

Table 26 – Concrete solution – Transversal Reinforcements

Section	M_{Ed} [kN.m/m]	$A_{S,\ desing}$ [cm^2/m]		M_{Rd} [kN.m/m]
Cantilever support	-206,16	ϕ16//0,10	20,11	262,31
Central slab support	-101,57	ϕ16//0,10	20,11	218,59
Central slab mid span	34,03	ϕ10//0,20	3,95	42,96

Table 27 - Concrete solution – Longitudinal Reinforcements

Section	M_{Ed} [kN.m/m]	$A_{S,\ eq\ PE}$ [cm^2]	$A_{S,\ desing}$ [cm^2]	
S1 – Viaduct Support	-43743	951,90	14ϕ32	112,59
MS1 – Viaduct Mid Span	42060	951,90	12ϕ32	96,51
S2 – Cable stayed bridge support	7412	666,33	14ϕ32	112,59
MS2 - Cable stayed bridge mid span	54582	951,90	44ϕ32	353,87

Table 28 - Concrete solution –Shear Reinforcements

Section	V_{Ed} [kN.m/m]	$A_{S,\ eq\ PE}$ [cm^2]	$A_{S,\ desing}$ [cm^2]
S1 – Viaduct Support	59,68	6R ϕ12//0,10	67,86
MS1 – Viaduct Mid Span	23,95	6R ϕ12//0,20	33,93
S2 – Cable stayed bridge support	34,56	6R ϕ12//0,20	33,93
MS2 - Cable stayed bridge mid span	20,03	6R ϕ12//0,20	33,93

Cable Stayed Verification

For ULS, when loaded to maximum tension, the cable stayed shouldn't exceed the 0,7Fgut defined on section 3.3. All cable stayed verified this limit.

Piers and towers verification

Table 29 - Concrete Solution – Pier Reinforcements

Pier	$A_{SL,\ design}$ [cm^2]		$A_{ST,\ design}$ [cm^2]	
Viaduct Pier	52ϕ25	255,32	56ϕ25	274,96
Cable stayed bridge Pier	160ϕ32	1286,4	160ϕ32	1286,4
Cable stayed bridge tower	88ϕ32	707,52	100ϕ32	836,16

Table 30 - Concrete Solution – Shear Reinforcements

Pier	$A_{SL,\ design}$ [cm^2/m]		$A_{ST,\ design}$ [cm^2/m]	
Viaduct Pier	8R ϕ12//0,2	45,20	8R ϕ12//0,2	45,20
Cable stayed bridge Pier	14R ϕ12//0,2	79,10	14R ϕ12//0,2	79,10
Cable stayed bridge tower	8R ϕ12//0,2	45,20	8R ϕ12//0,2	45,20

8.2. Solution B

The transversal and longitudinal verification is about to reinforce the slab to verify the M_{ed}. In terms of longitudinal verification of the steel section are defined five critical sections that allow calculating the extensions and respective tension distribution. All the sections verify the limits.

Table 31 - Composite solution – Transversal Reinforcements

Section	M_{Ed} [kN.m/m]	$A_{S,\ desing}$ [cm^2/m]		M_{Rd} [kN.m/m]
Support	-223,70	ϕ12//0,15	7,53	288,39
Mid span	118,80	ϕ10//0,20	3,95	151,21

Table 32– Composite solution – Longitudinal Reinforcements

Section	M_{Ed} [kN.m/m]	$A_{S,\ desing}$ [cm^2/m]		M_{Rd} [kN.m/m]
Support	-173,02	ϕ16//0,10	20,1	192,36
Mid span	83,30	ϕ12//0,125	9,04	86,51

Table 33– Composite solution – Section Tension

Section	N_{Ed} [kN]	M_{Ed} [kN.m]	σ_{Steel} [MPa]	$\sigma_{Reinforcements}$ [MPa]	σ_c [MPa]
S1 – Viaduct Support	904	-15281	-311	198	-
MS1 – Viaduct Mid Span	-1060	12386	234	-	7
S2 – Cable stayed bridge support – P5	-1956	-13372	-276	-	140
S2 – Cable stayed bridge support – P6	-7189	-4132	-101	-	-12
MS2 - Cable stayed bridge mid span	-2096	9539	176	-	-6

Cable Stayed Verification

For ULS, when loaded to maximum tension, the cable stayed shouldn't exceed the 0,7Fgut defined on section 3.3. All cable stayed verified this limit.

Piers and towers verification

Table 34 - Concrete Solution – Pier Reinforcements

Pier	$A_{SL,\ design}$ [cm^2]		$A_{ST,\ design}$ [cm^2]	
Viaduct Pier	56ϕ25	274,96	52ϕ25	255,32
Cable stayed bridge Pier	160ϕ32	1286,4	160ϕ32	1286,4
Cable stayed bridge tower	48ϕ25	235,68	44ϕ25	216,04

Table 35 - Concrete Solution – Shear Reinforcements

Pier	$A_{SL,\ design}$ [cm^2/m]		$A_{ST,\ design}$ [cm^2/m]	
Viaduct Pier	8R ϕ12//0,2	45,20	8R ϕ12//0,2	45,20
Cable-stayed bridge Pier	14R ϕ12//0,2	79,10	14R ϕ12//0,2	79,10
Cable-stayed bridge tower	8R ϕ12//0,2	45,20	8R ϕ12//0,2	45,20

9. Conclusions

On this detail design two different structural deck systems were studied for the same bridge, first a concrete solution and second a composite solution. This study allowed a comparison in terms of structural behavior economic viability.

Regarding to the construction method for concrete solution, the proposed method is falsework for the viaducts and cantilever construction method for the cable-stayed deck.

In other hand, the construction method for the composite steel-concrete solution it is used the incremental launching of the steel structure, not only because that it is a faster method but also because there is no limitations related with the height of the deck.

The main conclusions of this work are following mentioned:

- The changes with respect the pre-design solution, with the elimination the transition piers and of the dilation joints associated, allowed to improve the structure global behavior;
- Due to the fact that the main span has only 100 m, it is not evident the benefits from using a composite deck when compared to de concrete deck.
- The concrete solution global cost is estimated in 5.740.000€, which is about 595€/m^2 per deck square meter; only 17% of the total cost is relative to the deck.
- The composite solution global cost is estimated in 7.810.000€, which is about 890€/m^2 per deck square meter; about 31% of the total cost is relative to structural steel.
- The construction method cost was not taken in account because it depends on the Contractor equipment. This factor can change the global cost of each solution.

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