Slender Piers Design. Viaduct Over the Alberche River

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1. Introduction

In this paper the design of the supports of the Viaduct over the Alberche river is undertaken, using the method proposed by the FIP Recommendations 1996 (FIP96) [1].

The solution, thus obtained, is then compared with two non-linear analysis, taking into account non-linear material behaviour, as well as non-linearity due to changes in geometry, with different degrees of simplification.

In a first non-linear analysis, the supports are considered as cantilevers in the transversal direction and as simply supported at one end and embedded in the other end in the longitudinal direction. Transversal loads transmitted to the supports by the deck are obtained considering a linear behaviour of the structure. Wind loads acting directly on the columns are also considered. In the longitudinal direction a displacement of 100 mm is imposed on the top of the column to take into account a displacement-limiting device installed at the abutments (the maximum displacement allowed is 100 mm).

In a second non-linear analysis, the whole structure is modelled. In this case, a linear behaviour of the deck has been assumed, while geometric and material non-linearities are considered in the behaviour of the supports.

Finally, the same columns have been designed according to the procedures proposed by Eurocode 2 (EC2) [2] and Model Code 90 (MC90) [3].

2. Description of the Structure and Loads Considered

The Viaduct over the Alberche River is a five span composite structure. The length of the spans are 38.00, 56.00, 66.00, 52.00 and 34.00 m with a total length of 246 m (figure 1). The layout in plan is complex, begin-

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ning with a straight line, followed by a transition curve and ending in a circle with a radius of 350 m. The cross-section is composed of two double T steel beams with a height of 2.30 m connected to a reinforced concrete slab of variable depth from 0.15 to 0.30 m (figure 2).

The four columns have a hollow rectangular cross section of exterior dimensions 4.00x1.80 m^2 and a wall depth of 0.30 m. The columns are ended at the top by a composite steel-concrete structure designed to extend the width of the columns in order to support the deck. The two central columns, P2 and P3 (figure 4) have a height of 40.92 and 44.05 m, respectively. The two exterior columns, P1 and P4, are 23.04 and 22.58 m high.

The deck is fixed in the longitudinal direction to piers P2 and P3, and supported on neoprene bearings at piers P1 and P4 as well as at the abutments.

At the abutments, a special system is installed in order to limit the maximum longitudinal displacement of the deck (and therefore that of the more slender columns as well) to 100 mm (figure 5).

The material properties of the piers, as well as the partial safety factors, are given in table 1.

Material	f_{ck}, f_{yk} [MPa]	E_c,E_s[MPa]	γ
Concrete C-250	25.0	30000	1.50
Reinforcing Steel S-500	510.0	210000	1.15

Table 1. Pier material properties and partial safety factors.

For the design, the loads and load combinations required by the Spanish Standard of Loads on Road Bridges IAP [4] have been considered. In table 2, a brief summary is presented.

Loads	
– Permanents (G)	
Deck self-weight	95.0 kN/m
Pavement, sidewalks and safety barriers	45.0 kN/m
Pier self weight	25.0 kN/m^3
– Free shrinkage of deck slab (G*)	-260 $\mu\varepsilon$
– Variable actions	
Traffic	
Vertical	
uniform load	4.0 kN/m^2
concentrated load	600.0 kN
Horizontal	
braking	2.5 kN/m
centrifugal force	1.2 kN/m
Other external loads	
Longitudinal or transversal wind	±2.0 kN/m^2
Temperature on deck	
concrete	±17.0°
steel	±35.0°

Table 2. Loads considered.

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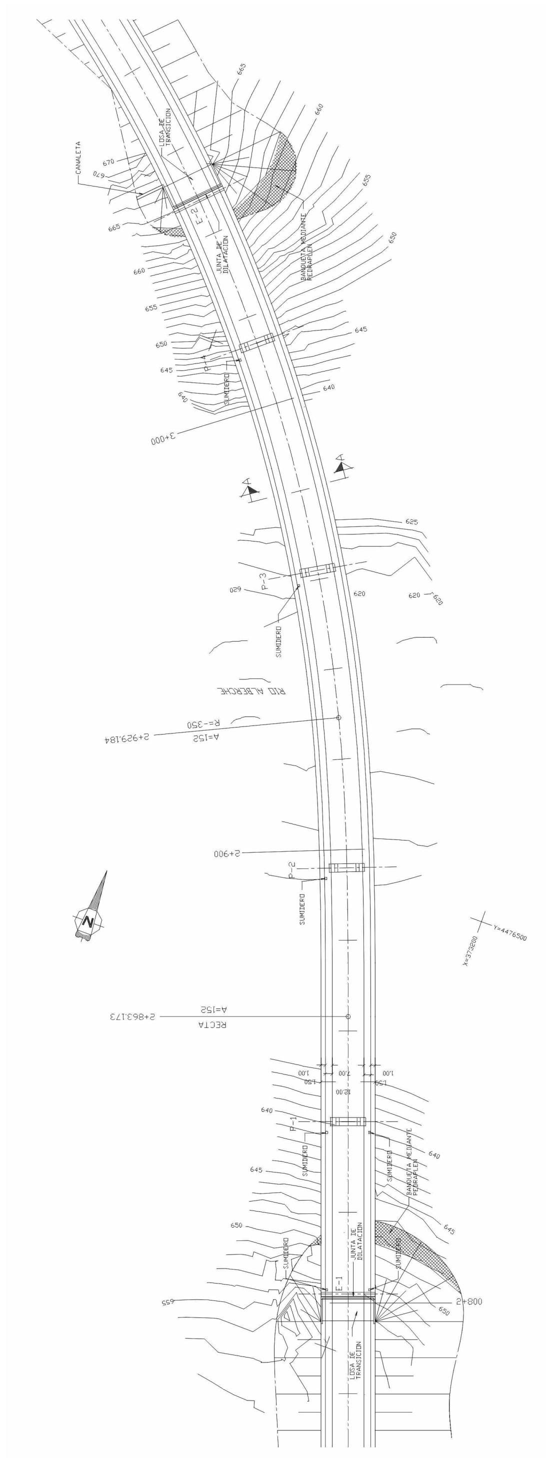

Figure 1. Plan view of the structure.

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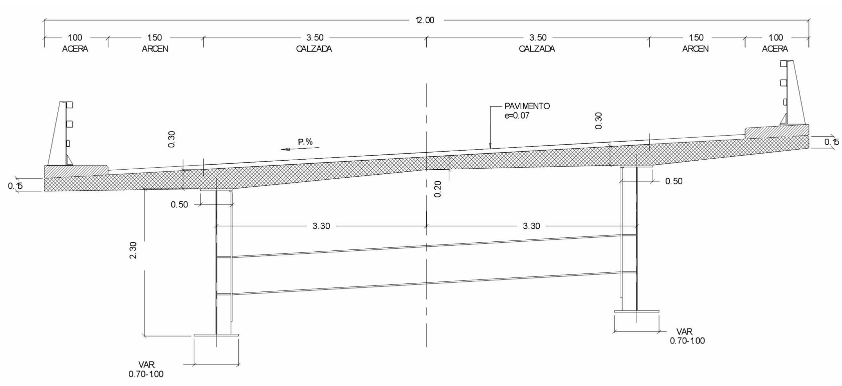

Figure 2. Deck cross-section.

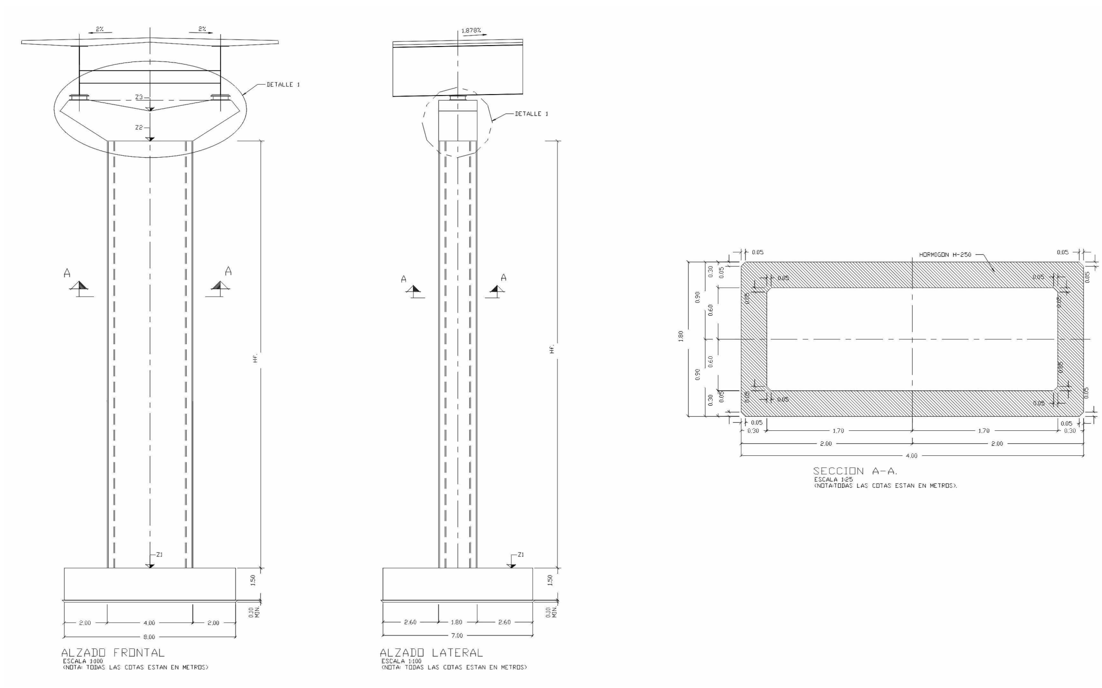

Figure 3. Piers P1 and P4. Elevation and cross-section.

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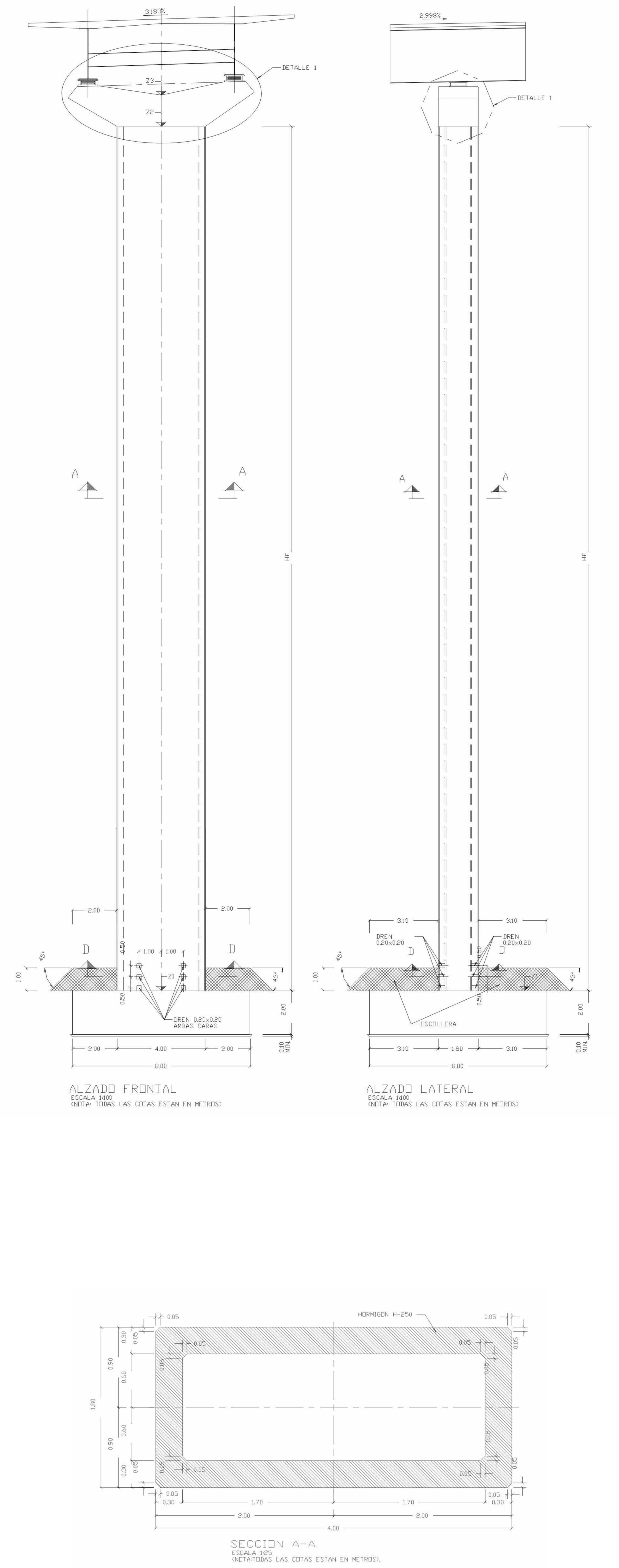

Figure 4. Piers P2 and P3. Elevation and cross-section.

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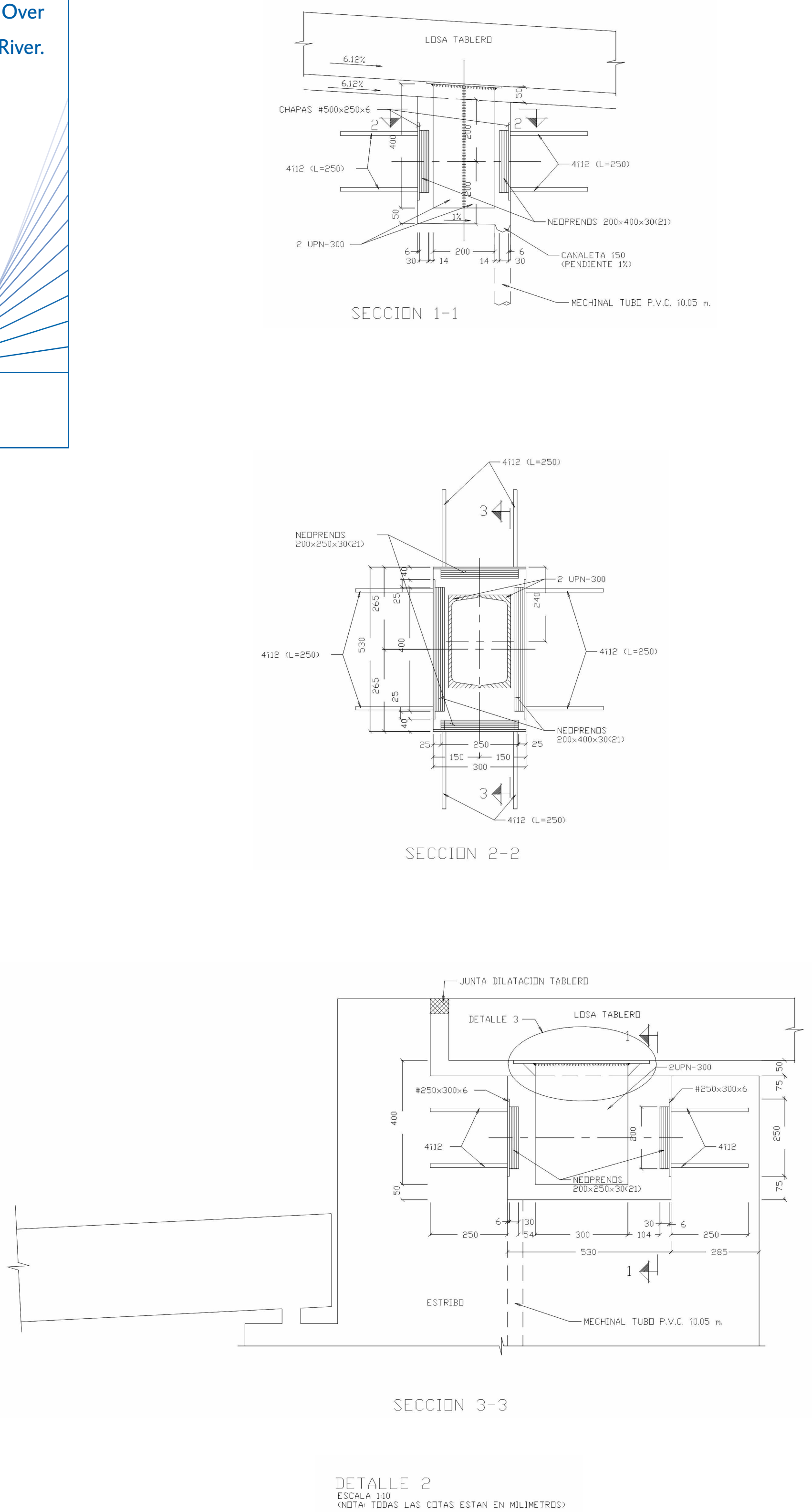

Figure 5. Detail of displacement-limiting system.

The load partial safety factors and the load combinations considered are described in tables 3 and 4

Load	Favourable effect	Unfavourable effect
Permanent	$\gamma=\gamma_{G^*}=1.00$	$\gamma_G=\gamma_{G^*}=1.35$
Variable	$\gamma_Q=0$	$\gamma_Q=1.50$

Table 3. Load partial safety factors.

Combination	Description and combination factors
I	Permanent load, traffic load on half of deck width and transversal wind $1.35(G+G^*)+1.50°\ Q_{\text{traffic load on half of deck width}}+0.45°Q_{\text{transversal wind}}$
II	Permanent load and transversal wind $1.35(G+G^*)+1.50°Q_{\text{transversal wind}}$
III	Permanent load, traffic load on full of deck width and transversal wind $1.35(G+G^*)+1.50°Q_{\text{traffic load on full deck}}+0.45°Q_{\text{longitudinal wind}}$
IV	Permanent load, traffic load on hald of deck width and longitudinal wind $1.35(G+G^*)+1.50°Q_{\text{traffic load on full deck width}}+0.45°Q_{\text{longitudinal wind}}$
V	Permanent load and longitudinal wind $1.35(G+G^*)+1.50°Q_{\text{longitudinal wind}}$

Table 4. Load combinations considered.

3 Design According To FIP Recommendations 1996

FIP 96 establish a simplified procedure for slender supports subject to non-skew bending, which allows the design of reinforcement by adding an additional eccentricity e_2, obtained in a simplified manner, which takes into account second order effects. For skew bending, the use of a simplified interaction diagram is proposed:

$$\left(\frac{M_{sd.x}}{M_{Rd.x}}\right)+\left(\frac{M_{sd.y}}{M_{Rd.y}}\right)\geq 1$$

where:

$M_{sd,x}$ Design bending moment in the x direction, including second order effects

$M_{sd,y}$ Design bending moment in the y direction, including second order effects

$M_{Rd,x}$ Ultimate bending moment in the x direction resisted by the cross section, for the given normal force, N_{sd}.

$M_{Rd,y}$ Ultimate bending moment in the y direction resisted by the cross section, for the given normal force, N_{sd}.

To determine $M_{sd,x}$ and $M_{sd,y}$ the procedure established in paragraph 6.6.6 of FIP 96 is used. In each direction:

$$M_{sd} = M^{o}_{sd} + M_2$$
$$M_2 = N_{sd} \cdot e_2$$
$$e_2 = (1/r) \cdot l_0^2/10$$

where:

M_{sd}^{0}	First order design bending moment
M_2	Bending moment due to second order effects
e_2	Second order eccentricity
l_0	Equivalent support length (buckling length)
$1/r$	Reference curvature defined according to FIP 96.

In the case of bridge piers, the worst design combinations usually involve skew bending. The value of l_0 depends on the type of connection between pier and deck, as well as on the boundary conditions of the structure as a whole, making it difficult to determine this parameter. Furthermore l_0, usually, has a different value in the longitudinal and in the transversal direction. Finally, the forces transmitted to the piers by the deck are a function of the general behaviour of the structure, and, in particular, of the stiffness of the piers.

For this example, the first order forces have been determined assuming a linear behaviour of the structure, using the non-cracked stiffness for both deck and piers and modelling the connections between deck and piers in a realistic way, taking into account the different characteristics of the bearing supports. In tables 5 and 6 the first order forces for each combination group are shown for piers P1 and P4, and piers P2 and P3, respectively. For each group of piers, the worst combination considering each two piers is shown in each case.

In the transversal direction, for the determination of the second order forces, the piers are considered as cantilevers and, therefore, $l_0=2 \cdot l$. In this way, any contribution of the deck to the transversal stability of the piers is neglected.

Load combination	N_{sd}[KN]	$M_{sd,transversal}^{o}$[mKN]	$M_{sd,longitudinal}^{o}$[mKN]
I	14900	11300	1100
II	13250	13500	2150
III	16700	1950	3800
IV	14900	7450	3400
V	12900	200	6100

Table 5. First order forces. Piers P1 and P4.

Load Combination	N_{sd}[KN]	$M_{sd,transversal}^{o}$[mKN]	$M_{sd,longitudinal}^{o}$[mKN]
I	20900	19950	700
II	17700	40950	1250
III	23150	6100	2950
IV	20900	8200	2800
V	17650	750	5000

Table 6. First order forces. Piers P2 and P3.

In the longitudinal direction, in order to take into account that the longitudinal displacement has been limited to 100 mm, e_2 has been taken as 100 mm for both groups of piers, P1-P4 and P2-P3. Besides, the favourable effect of the bending moment, due to the horizontal reaction at the displacement-limiting system, is neglected. In tables 7 and 8, the resulting design forces for the embedded section of the pier are shown. Since the reinforcement is constant along the full length of the piers, the embedding is the critical section.

In order to determine the required reinforcement, an iterative procedure is necessary. A certain amount of reinforcement is firstly proposed. For this amount, M_{Rd} and M_{Sd} are determined for each direction and the condition established by the proposed interaction diagram is checked. The procedure is repeated until the interaction condition is strictly fulfilled.

Load Combination	N_{sd}[KN]	$M_{sd,transversal}^{o}$[mKN]	$M_{sd,longitudinal}^{o}$[mKN]
I	14900	14035	2595
II	13250	15925	3470
III	16700	5045	5460
IV	14900	10160	4890
V	12900	2540	7390

Table 7. Design forces, including second order effects. Piers P1 and P4.

Load Combination	N_{sd}[KN]	$M_{sd,transversal}{}^{o}$[mKN]	$M_{sd,longitudinal}{}^{o}$[mKN]
I	20900	30400	2780
II	17700	51685	3025
III	23150	16830	5255
IV	20900	18650	4890
V	17650	10480	6760

Table 8. Design forces, including second order effects. Piers P2 and P3.

Tables 9 and 10 give the values of M_{Rd} (in each direction) and of ($M_{sdx}/M_{Rdx}+M_{sdy}/M_{Rdy}$) for each group of piers, for the different combinations and for the proposed reinforcement. For piers P1 and P4 the proposed reinforcement A_s = 12480 mm² (ω=0.11) is the minimum reinforcement required by the Spanish reinforced concrete Standard [5]. For piers P2 and P3, the proposed reinforcement is As = 65663 mm² (ω=0.56), results from combination II, as can be seen in table 10.

Load Combination	N_{sd}[KN]	$M_{Rd,transversal}{}^{o}$[mKN]	$M_{Rd,longitudinal}{}^{o}$[mKN]	$\left(\frac{M_{sd.x}}{M_{Rd.x}}\right)+\left(\frac{M_{sd.y}}{M_{Rd.y}}\right)\geq 1$
I	14900	30000	15210	0.64
II	13250	28870	14160	0.80
III	16700	30920	16240	0.50
IV	14900	30000	15210	0.66
V	12900	28595	13935	0.62

Table 9. Interaction diagram. Piers P1 and P4.

Load Combination	N_{sd}[KN]	$M_{sd,transversal}{}^{o}$[mKN]	$M_{sd,longitudinal}{}^{o}$[mKN]	$\left(\frac{M_{sd.x}}{M_{Rd.x}}\right)+\left(\frac{M_{sd.y}}{M_{Rd.y}}\right)\geq 1$
I	20900	57260	31210	0.77
II	17700	57435	31090	0.99
III	23150	56780	30655	0.46
IV	20900	57260	31210	0.48
V	17650	57435	31085	0.40

Table 10. Interaction diagram. Piers P2 and P3.

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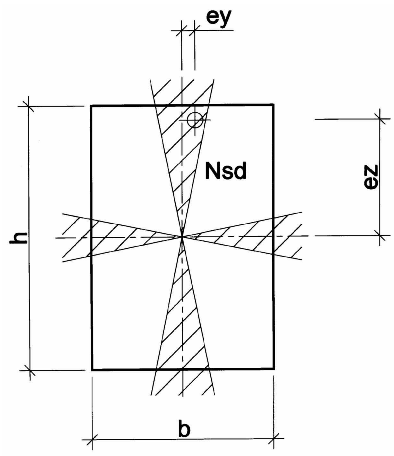
4. Design According To EC2 And CEB-FIP Model Code 90

EC2 and MC90 establish, for the design of slender columns subject to compression and non-skew bending, similar criteria between them. These criteria are also similar to the one proposed by FIP 96.

For skew bending, an independent check for each plane of bending is allowed, only if there is a predominant eccentricity, as shown in figure 6.



$$\frac{e_z/h}{e_y/b} \geq \frac{1}{4} \text{ ó } \frac{e_y/b}{e_z/h} \geq \frac{1}{4} \quad \text{(MC90)}$$

$$\frac{e_z/h}{e_y/b} \geq \frac{1}{5} \text{ ó } \frac{e_y/b}{e_z/h} \geq \frac{1}{5} \quad \text{(EC2)}$$

Figure 6. Condition which must be fulfilled in order to be allowed to check each bending plane separately.

This type of proposal is clearly insufficient for the design of bridge columns since, in many cases, the design combinations involve skew bending without a predominant eccentricity and, therefore do not fulfill the above condition.

In this example, however, load combination number II, which governs the design of piers P2 and P3, is a combination with a predominant eccentricity in the transversal direction, and fulfills the condition established by both EC2 and MC90 . Table 11 shows the amount of reinforcement obtained with EC2 and MC90 . These are compared to the amount of reinforcement determined in paragraph 3[1].

[1] For the design according EC2 and MC90, the piers are supported as explained in paragraph 3. In the transversal direction, the piers are considered as cantilevers and the second order eccentricity in the longitudinal direction has been taken as 100 mm.

	A_s[mm^2]	ω
Eurocode 2	64171	0.55
CEB-FIP Model Code 90	69965	0.60
FIP Recommendations 1996	65663	0.56

Table 11. Design of piers P2 and P3 according to FIP Recommendations 1996, EC2and CEB-FIP Model Code 90.

In this case, where the criteria of EC2 AND MC90 is of application, similar results are obtained.

5. Non-Linear Check

In order to check the reinforcement obtained using FIP 96, a non-linear computation was undertaken. For this check a finite element program was used taking into account both the mechanical non-linearity, due to the non-linear behaviour of the materials, as well as the geometrical non-linearity, due to the effect of the displacements on the forces.

The non-linear behaviour of concrete was modelled using the parabola-rectangle diagram shown in figure 7, with a maximum stress of f_{cd}. It is well known that this diagram was developed in order to determine the ultimate limit state due to normal forces and that it underestimates the stiffness of the cross-section for the lower range of stresses.

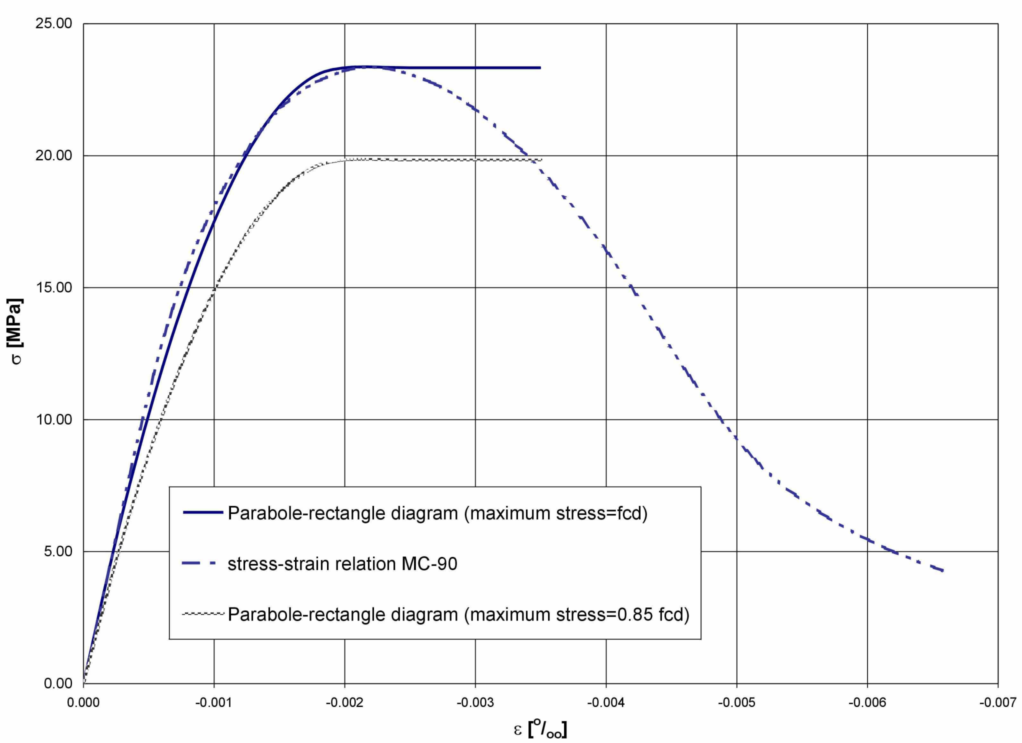


Figure 7. Constitutive relations of concrete.

However, taking f_{cd} as maximum stress, instead of $0.85 \cdot f_{cd}$ (see figure 7), it has been shown (6) that, for the ultimate limit state of instability, the parabola-rectangle diagram leads to adequate results, similar to those obtained with more precise diagrams, such as that also shown in figure 7, proposed in the paragraph 2.1.4.4.1 of MC90.

Tension stiffening is neglected. For steel, a bilinear diagram has been used, considering a maximum stress equal f_{yd}.

In all cases, load combination II, which governs the design of the slender columns, was checked. A first analysis is undertaken with the same loads used to determine the amount of reinforcement. Then, the wind load is increased until the collapse of the structure comes about.

Two different structural systems were studied. The results of table 12 correspond to the analysis of the pier alone, supported in the same manner as considered for the design of the reinforcement, as explained in paragraph 3. In this case, the vertical and the transversal horizontal loads are determined through linear analysis using the non-cracked stiffness of deck and piers. In the longitudinal direction, an imposed displacement of 100 mm is considered, in order to take into account the limit to the displacement allowed by the structural system (see figure 8).

The results show that the system is stable under combination II, and reaches a collapse only after the wind load has been increased by a factor of 1.20.

	N_{sd}[KN]	$M_{sd,transversal}$ [mKN]	$M_{sd,longitudinal}$ [mKN]
First order bending moments	17700	40950	1250
Ultimate load capacity β=1.80	17700	62280	1535

Table 12. Analysis of the ultimate bearing capacity of an isolated column. Piers P2 and P3.

Finally, a non-linear analysis considering the whole system (see figure 9) was undertaken. The results of this analysis are shown in table 13. In this case, a linear behaviour of the deck was assumed, while both the mechanical and the geometrical non-linearities were considered in the piers. Other results [6] show that the non-linear behaviour of the deck is of little importance in the results of this type of structural analysis.

As in the previous analysis, the structures is first checked for the initial loads of Combination II, which proves stable. Then, the wind load is increased until the collapse of the structure is attained. Table 13 shows that the wind load has to be multiplied by 1.80 before the structure collapses.

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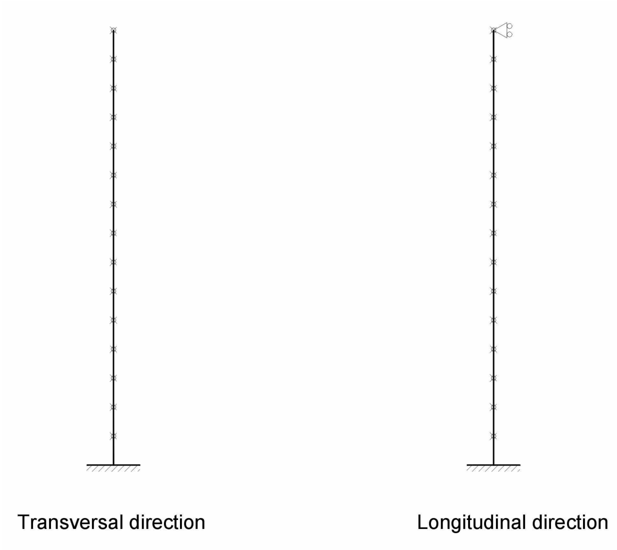

Figure 8. Structural Model. Non-linear analysis of the isolated column.

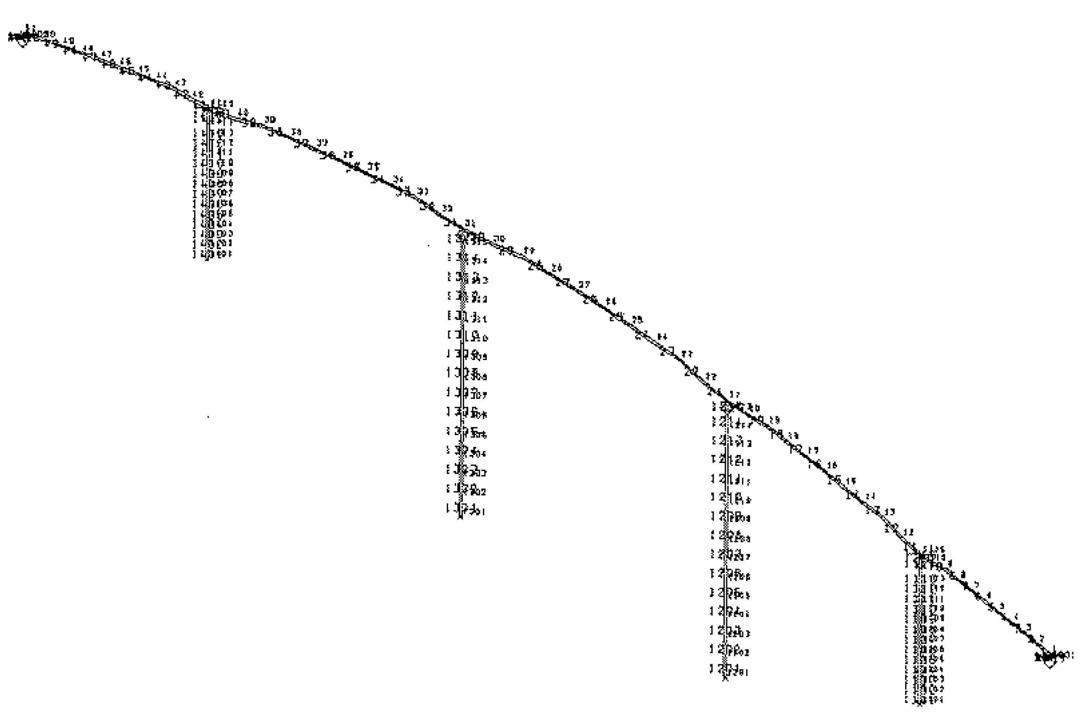

Figure 9. Structural model. Non-linear analysis of the whole structure.

	N_{sd}[KN]	$M_{sd,transversal}$ [mKN]	$M_{sd,longitudinal}$ [mKN]
First order bending moments	17700	40950	1250
Ultimate load capacity β=1.80	17700	60815	1345

Table 13. Analysis of the ultimate bearing capacity of the structure. Piers P2 and P3.

6. Conclusion

The method proposed by FIP 96 allows the study of slender bridge columns, subject to skew bending. This method has been applied to a real structure having very slender piers. In this case, it has been shown that this method leads to results which are both reasonable and on the safe side, according to the more precise non-linear checks carried out.

7. References

[1] FIP Recommendations 1996. ‘Practical Design of Structural Concrete’. International Federation for Prestressing. May 1998.

[2] Eurocode 2 (1989). Eurocode 2, Design of concrete structures. European Committee for Standardisation. 1989.

[3] CEB-FIP Model Code (1990). CEB-FIP Model Code for concrete structures, Euro-International Committe for Concrete, Bulletin n° 213/214, Lausanne, May 1993.

[4] Spanish Code of Loads on Road Bridges (1995). IAP Instrucción sobre Acciones a considerar en el Proyecto de Puentes de Carretera. Ministerio de Fomento. Dirección General de Carreteras. Madrid, April 1997.

[5] Spanish Reinforced Concrete Code (1991). EH-91 Instrucción para el Proyecto y la Ejecución de Obras de Hormigón en masa o armado. Comisión Permanente del Hormigón. Ministerio de Obras Públicas, Transportes y Medio Ambiente. Madrid, 1992.

[6] Working Group ‘Design Examples‘. FIP Commision 3. Meeting in Madrid, on 24th and 25th October 1997.