STEEL FIBRE REINFORCED CONCRETE PRECAST SEGMENTS FOR VERTICAL SHAFTS

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Abstract

The use of steel reinforced precast concrete segments for soil supporting in vertical shafts is a common practice. For this application, steel reinforcing bars have been used up to date to reinforce the concrete during the demoulding operations, stocking, transport, placing, as well as under service conditions. In this sense, in the great majority of the applications, the need of reinforcement is minimum and responds to the requirement of ductile behaviour.

In this context, the use of structural fibres for substituting the traditional steel bars may be a viable alternative. Moreover, the use of the suitable amount of fibres may guarantee the ductile behaviour required.

This work aims at presenting a design procedure to assess the optimum amount of fibres needed to reach the structural requirements fixed in the project. This procedure is applied in three different vertical shafts in the metropolitan area of Barcelona.

Keywords: shaft, support, segment, fibre, optimum.

1. Introduction

Fibre reinforced concrete (FRC) is a composite material which it use in precast segments leads to an improvement of its mechanical response [1-2] increasing its tenacity, its fire and impact resistance and its performance facing concentrated tensions that may occur in certain loading situations (demoulding, stocking, transport, manipulation, thrust of the jacks, among others; see Fig.1). Likewise, the use of FRC allows the partial or even total replacement of the traditional passive reinforcement bars, thus improving the efficiency of the production and ensuring the competitiveness of FRC in this application. [3].

Currently there are numerous experiences in which precast fibre reinforced concrete segments (PFRCS) were used with structural purposes (see Tab. 1) in highway tunnels

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(RT), railway tunnels (RWT), water supply (WTT), gas transport (GPT) and for services (ST). Some of these are still under construction (u.c).

Fig. 1: (a) Internal transport; stocking (b) in plant and (c) in site and (d) placing

It may be observed in Tab. 1, that the slenderness (λ), ratio between the inner diameter of the tunnel (D_i) and the thickness of the segments (h), ranges from 12.0 to 36.0. In this sense, for small values of λ, the sensibility of the ring versus the bending moment is low and predominantly works under compression in service. Therefore, in these cases the reinforcement consists of a minimum amount placed so as to avoid a brittle failure mode that might occur during the transitory loading situations (see Fig.1). On the contrary, the larger λ is, the higher the deformability of the ring as well as its sensibility when facing asymmetries of the soil load. Usually, for large values of λ, the service conditions use to be the most unfavourable and govern both the amount and configuration of the reinforcement.

This contribution aims at presenting a design method for designing PFRCS. To achieve this goal, the bases of the design philosophy are presented firstly. Finally, the proposed design procedure is applied for designing the reinforcement of the PFRCS of three shafts planned to be constructed in Barcelona.

2. Proposed design philosophy of fibre reinforced concrete segments

The main longitudinal reinforcement utilized in the precast segments (see Fig. 2) is placed in order to guarantee the required capacity for facing both the tensile and compressive stresses that occur during construction stages and in service. Additional reinforcement use to be located at certain points in the areas of contact between the segments and the jacks (this is not included in Fig. 2 for being local and non-relevant in this study). Likewise, auxiliary reinforcement is placed where the lifting bolts are inserted.

Tab.1: Excavated tunnels with TBM in which FRC segments have been applied

Name	**Year**	**Count.**	**Func.**	D_i (m)	λ []
Fontsanta-Trinitat Tunnel	2010	ES	WTT	5.2	26.0
The Clem Jones Tunnel	2010	AUS	RT	11.3	28.3
Ems-Dollard Crossing	2010	DE-NL	GPT	3.0	12.0
Cuty West Cable Tunnel	2010	AUS	EP	2.5	12.5
Adelaide Desalination Plant	2010	AUS	WT	2.8	14.0
Extension of the FGC in BCN	2010	ES	RWT	6.0	20.0
Brightwater East	2011	USA	WTT	5.1	19.6
Brightwater Central	2011	USA	WTT	4.7	14.2
Brightwater West	2011	USA	WTT	3.7	14.2
East Side CSO Tunnel	2011	USA	WTT	6.7	18.6
Victorian Desalination Plant	2011	AUS	WTT	4.0	17,4
Monte Lirio Tunnel	2012	PAN	WTT	3.2	12.8
Lee Tunnel Sewer	u.c.	UK	WTT	-	0.0
Line 9 of Barcelona Metro	u.c.	ES	MT	10.9	31.1
Brenner Base Tunnel	u.c.	ITA-AUT	RT	5.6	28.0
The Wehrhahn Line	u.c.	DE	MT	8.3	18.4
Crossrail	u.c.	UK	RWT	6.2	20.7

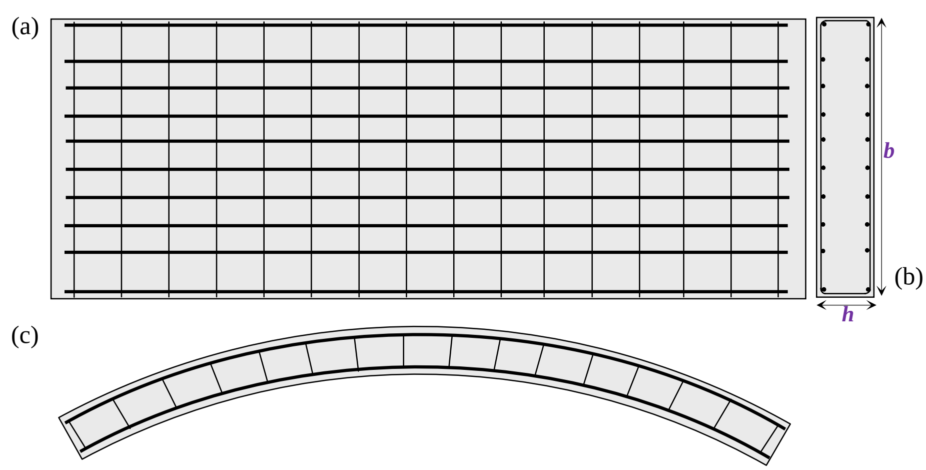

Fig. 2: View (a) on floor, (b) transversal and (c) frontal of the configuration of a reinforced concrete segment.

This study focuses on the main reinforcement (see **Fig. 2**). However, it must be highlighted that other studies and experiences reveal that the local reinforcements previously mentioned might also be replaced given the 3-D resistant mechanisms provided by the fibres.

In most cases, the amount of reinforcement responds to the need of guaranteeing the ductile behaviour in hypothetic scenarios of cracking. In such cases, the minimum amount of reinforcement established has to be placed. Likewise, a minimum amount of reinforcement (minimum related to the cross area of concrete) has to be used in order to control the opening of the cracks that usually appear during the curing phase of the concrete due to shrinkage in fresh state or other thermal action.

To evaluate the amount of minimum mechanic reinforcement ($A_{s,min}$) in case of using traditional steel bars, the concrete standards suggest expressions that allow guaranteeing the ductile behaviour in ULS. These expressions do not consider the contribution of the fibres in the mechanical response of the FRC, moreover it is known that these lead to conservative values of $A_{s,min}$ [4]. Thus, in order to assess the optimum quantity of fibres (C_f) that guarantee the ductile response of the section, it is necessary to consider some additional hypothesis that permit addressing the problem stated.

In this respect, considering a criterion of ductile behaviour in case of failure, the most unfavourable section must be capable of bearing its cracking bending moment (M_{cr}). In short, the ultimate bending moment of the section (M_u), which depends on its geometry (h and b, see Fig. 2) and the C_f, must be higher than M_{cr}. In Fig. 3 a diagram moment – curvature (M - χ) is shown wherein the type of response of a generic cross section of the segment in function of C_f can be observed.

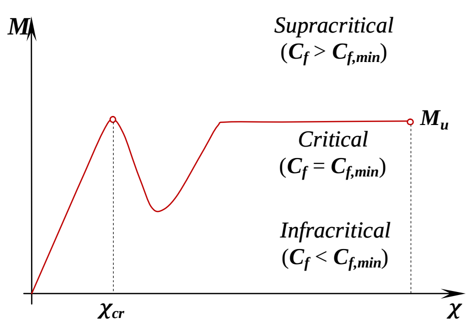

Fig. 3: Modes of failure depending on the amount of $\boldsymbol{C_f}$

Fig.3 reveals that situations in which the C_f used is lower than $C_{f,min}$ lead to unsafe and undesirable brittle failure modes.

For that matter, at a design level, in cases in which the design bending moment M_d is not higher to M_{cr} ($M_d < M_{cr}$), the equation $M_u = M_{fis}$ must be solved to obtain the $C_{f,min}$ that guarantees the ductile behaviour of the cross section. On the contrary, if $M_d \geq M_{cr}$, the design condition $M_u = M_d$ should be imposed to obtain the value of C_f that meets with the bearing capacity required at the most unfavourable section. In any case, these equations are nonlinear and, therefore, they must be solved by resorting to some numerical method.

Generally, this phase of the design is conducted when there is still no information related to the mechanical behaviour of the FRC that will be used for the production of the segments. Hence, with the aim of defining one of the constitutive laws gathered in the regulations, it is necessary to set some values of the flexural residual strength of the FRC ($f_{R,i}$). These values of $f_{R,i}$ will allow assessing the direct tensile stresses (σ_i) and their respective strains (ε_i). These parameters will define the constitutive law for simulating the FRC subjected to tension.

For instance, the design method for FRC elements proposed in the MC2010 [5] consist of establishing the post – cracking residual strength values $f_{R,1}$ and $f_{R,3}$ necessaries to resist the design loads N_d and M_d. Thus, the process is based on the obtaining of these values $f_{R,1}$ and $f_{R,3}$ and, subsequently, fixing the type and the amount of fibres C_f that leads to such values of $f_{R,i}$.

Having defined these first steps, the M - χ diagram of the section and, in consequence, the associated M_u related to each C_f value can be obtained. The flow diagram in Fig.4 highlights the purposed process to design PFRCSs.

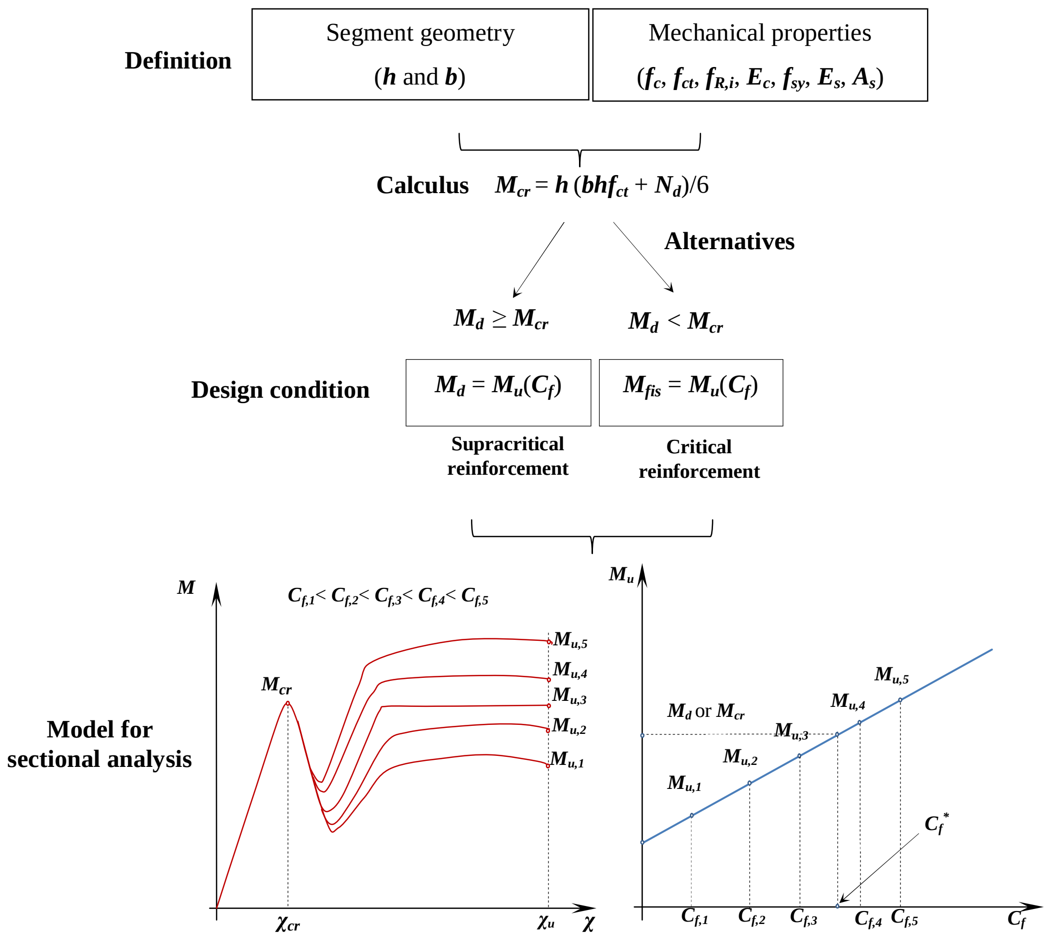

Fig. 4: Procedure proposed for the optimum design of PFRCSs

The design procedure suggested on Fig. 4 concludes with the assessment of an estimated value for the optimal amount of fibres C_f^* that allows to satisfy the design requirements. Nevertheless, this value of C_f^* is obtained by assuming specific values of $f_{R,i}$ that generally refers to previous experimental campaigns for a determined type of fibres. Subsequently, after having set the type of structural fibres to be used, prismatic specimens must be moulded with the same concrete and amount of fibres C_f estimated in the first stage of the design. These specimens will be subjected to a control test specified in the current standards to characterize the mechanical behaviour of the FRC. If the response obtained is equivalent or enhanced in reference to that derived from the design procedure proposed here, the value of C_f* can be reduced in view to further optimization. On the contrary, the values of $f_{R,i}$ must be adjusted based on the experimental results and the value of C_f^* updated (increased) if the results do not fit the requirements.

The simulation of the behaviour of the segment can be addressed resorting to finite element models considering its entire geometry and the most complex constitutive models. Nevertheless, the configuration of the longitudinal reinforcement (see Fig. 2) is governed by the response of the most critical section (the midspan) in the different stages which the segment is subjected to.

Therefore, the design requires the analysis of the stress-strain response of such section with a numerical model capable to simulate the post-cracking of the FRC.

In this study, the model of Analysis of Evolutionary Sections (AES) [6] was used to obtain the associated M_u for different values of C_f. This permits the deduction of the M_u - C_f curves and the determination the value of C_f^* that meets the mechanical requirements established for each segment, following the process described in Fig. 4.

3. Application example

3.1 Introduction

For the construction of the High Speed Railway in Barcelona, various vertical shafts should be excavated. Particularly, the vertical shafts of Urgell street (Φ = 9,2 m and h = 41,0 m), of Nápols street (Φ = 9,2 m and h = 42,3 m) and Independència street (Φ = 9,2 m and h = 44,0 m) are planned to be supported with precast concrete (C 40/45) segmental linings with 400 mm of height.

Initially, the reinforcement of these segments was based on the use of traditional reinforcement, being this reinforcement the minimum to guarantee the ductility in case of failure during the manipulation and transport of the segments. In this sense, the rings are expected to be completely compressed in service when these are subjected to the earth loads (Tab. 2 gathers the design loads obtained by using the commercial software STATIK® to simulate the mechanical behaviour of the most loaded rings of each shaft).

Tab.2: Design efforts affecting the precast segments for each shaft

Shaft	N_d (kN/m)	M_d (mkN/m)
Urgell	4776	241
Nápols	6900	347
Independència	2235	113

Notice that the design loads presented in Tab. 2 are predominantly of compression due to the low ratio $M_d/N_d \approx 0.05$; thus, implicating that the cross sections of the segments are fully compressed (maximum compressive stresses of 21,0 N/mm^2). However, although the concrete is capable to resist these compression levels (f_{ck} = 40 N/mm^2), there is a need of supplying a minimum amount of reinforcement to face these stresses as well as those derived from restrained shrinkage, thermic actions, among others.

Likewise, during manipulation and transport operations of the segment, bending forces (M_d) are expected to appear. Nevertheless, these M_d are not likely to reach even the design cracking moment of the cross section ($M_{cr,d}$ = 52,0 mkN/m). Thus, a minimum amount of reinforcement is required to guarantee the ductile behaviour in a hypothetic case of failure.

Taking into account all abovementioned, a reinforcement strategy based on two layers with 10 each one of steel 16 mm – diameter reinforcing bars (20 bars in each section) was proposed in the initial project to face all the load stages. Nonetheless, the use of a suitable amount of structural fibres can replace all this reinforcement and lead to technical and economic benefits.

3.2 Mechanical performance simulation of the material

In the great majority of the cases, when facing the design of any FRC structural element, the technician does have available neither the relation $f_{Ri} - C_f$ nor the type of fibre to be used. Therefore, following the design strategy established in the MC2010 for FRC, the values $f_{R,1}$ and $f_{R,3}$ that permits guaranteeing the load requirements, should be fixed and, subsequently, once the type of fibre is decided, assess experimentally the value of C_f.

Alternatively, for this work and with pre – design purposes, the relation $f_{Rm,i} - C_f$ proposed by Barros et al. 2005 [7] (Eqs. (1) and (2)) were used to assess the optimum value of C_f. This equations were derived from an extensive experimental campaign for which fibre type DRAMIX® 80/60 were used.

$$f_{Rm,1} = 0.0945 C_f + 0,702 \quad (1)$$

$$f_{Rm,4} = 0.926 C_f \quad (2)$$

In the Eqs. (1) and (2), $f_{Rm,i}$ is expressed in N/mm^2 and C_f in kg/m^3. In this sense, values of C_f ranging between 20 kg/m^3 (minimum to consider the structural contribution of the fibres) and 60 kg/m^3 (maximum regarding to economic reasons in this particular application) were considered.

In Tab. 3 the main variables involved in the simulation of the post – cracking behaviour of the FRC are listed. With this purpose, the tri – linear constitutive equation (see Fig. 5) suggested in [8] was implemented in the model AES [6].

For the Ultimate Limit State analysis, the characteristic values of the parameters were obtained by multiplying the average ones by a factor 0.7 ($\boldsymbol{f_{Rk,i}} = 0.7\boldsymbol{f_{Rm,i}}$). Finally, the design values were derived from the characteristic by dividing by a partial safety factor of 1.5 ($\boldsymbol{f_{Rd,i}} = \boldsymbol{f_{Rk,i}}/1.5$).

Tab.3: Mechanical parameters to define the post-cracking response of FRC

C_f (kg/m^3)	$f_{Rm,1}$ (N/mm^2)	$f_{Rm,4}$ (N/mm^2)	σ_1 (N/mm^2)	ε_1 (‰)	σ_1 (N/mm^2)	ε_1 (‰)	σ_1 (N/mm^2)	ε_1 (‰)
20	2.592	2.400	3.476	0.122	0.761	0.222	0.580	25.000
30	3.537	3.275	3.476	0.122	1.039	0.222	0.791	25.000
40	4.482	4.150	3.476	0.122	1.316	0.222	1.002	25.000
50	5.427	5.025	3.476	0.122	1.594	0.222	1.214	25.000
60	6.372	5.900	3.476	0.122	1.871	0.222	1.425	25.000

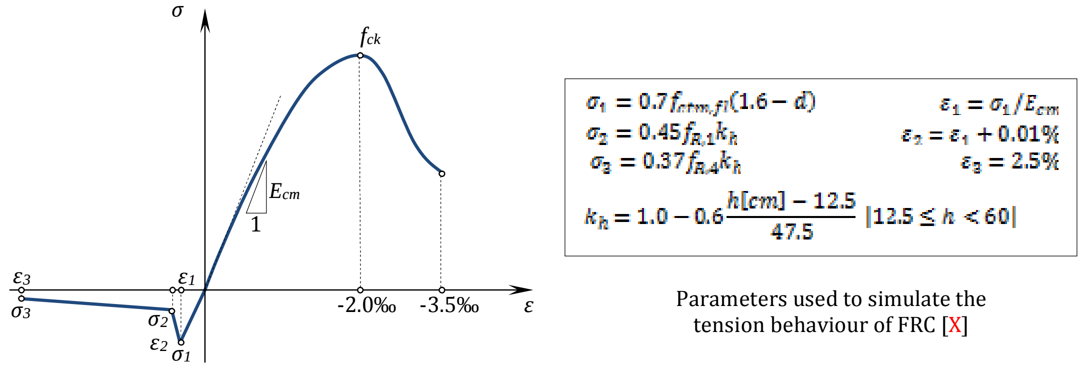

Fig. 5: Constitutive equation adopted to simulate the post – cracking behaviour of the FRC

3.3 Results obtained

3.3.1 Simple bending (stages prior service)

By using the numerical model AES and considering the Eqs. (1) and (2), the ultimate bending moment (M_u) for each amount of fibres C_f was derived (see Fig. 6).

As it can be seen in Fig. 6, the optimum amount of fibres C_f* to guarantee a ductile behaviour in case of failure of the segment is 48 kg/m^3 (50 kg/m^3 for technical and production purposes). This C_f* has been obtained considering a specific type of fibres and a mechanical performance (see Eqs. (1) and (2)).

In any case, before the production of these segments, an experimental campaign with the final type of fibre should be carried out to determine the correct value of C_f. This value might be almost equal to $\boldsymbol{C_f}$* provided the type of fibre finally used is similar to the one adopted for this analysis.

3.3.2 Compression combined with bending (service)

Having estimated the optimum value of C_f (50 kg/m^3), the N_u – M_u interaction diagram was calculated with the model AES (see Fig. 7). This curve can be used to verify if the external forces generated by the earth loads over the critical sections of the segments during service, could be resisted with the amount of fibres derived from the previous analysis.

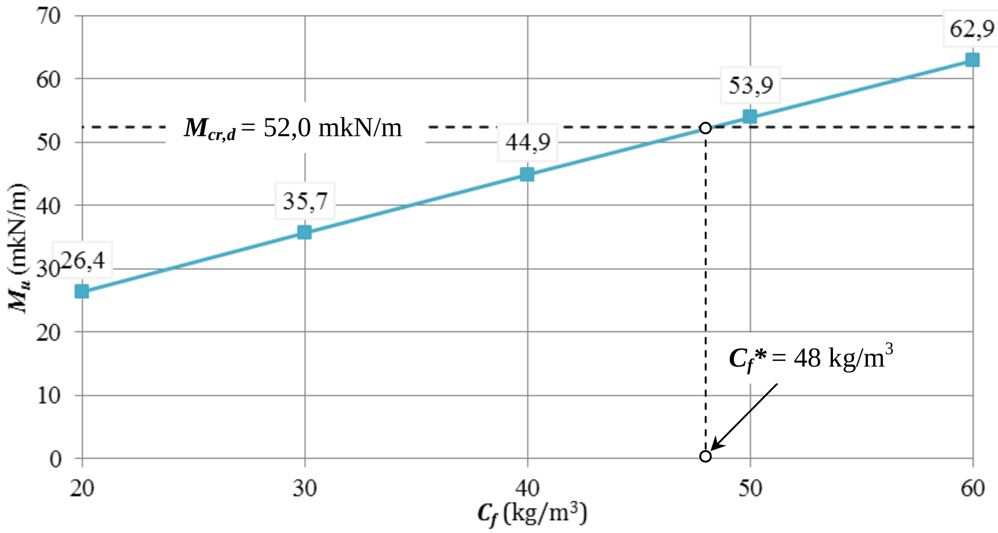

Fig. 6: $\boldsymbol{M_u} - \boldsymbol{C_f}$ curve obtained with AES for the precast segment analysed.

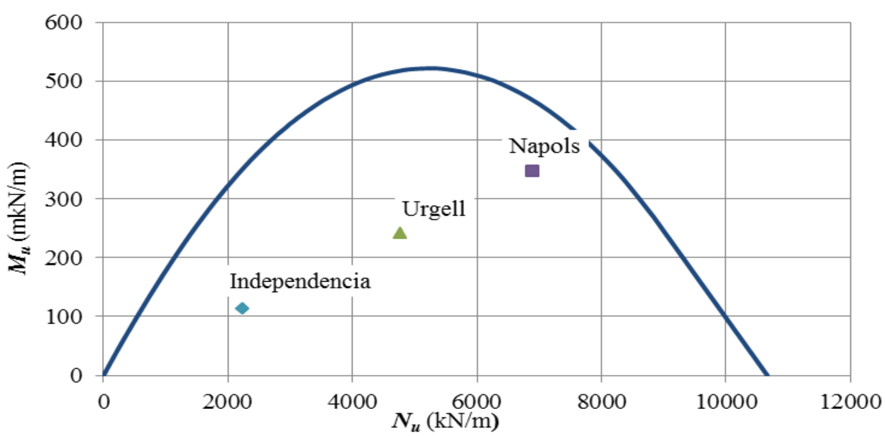

Fig. 7: $\boldsymbol{N_u} - \boldsymbol{M_u}$ interaction diagram of the section with 50 kg/m^3 of fibres.

The results gathered in Fig. 7 highlight that the most unfavorable sections of the precast segments of each shaft, could resist the external loads during service considering and amount of 50 kg/m^3 of steel fibres.

4. Conclusions

The global conclusions derived from this work can be summarized in the following points:

1. Fibres can substitute the total amount of reinforcing steel bars in those PFRCSs submitted to low – moderate loads in service conditions (usual situation in tunnels or shafts excavated with TBM). For these elements, the most unfavourable load conditions use to appear during manipulation, transport and other intermediate production phases.
2. The method proposed can be understood as a pre – design method to assess the amount of fibres C_f necessary to reach the design mechanical requirements. Having this value of C_f, the technician could analyse the suitability of the substitution of the

traditional reinforcement by fibres (or a combination) resorting to technical, economical and other reasons.

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