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SEISMIC BEHAVIOUR OF FIBRE REINFORCED CONCRETE FRAMES

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SUMMARY

In the present paper the behaviour of framed structures realised by using high strength fibre reinforced concrete and subjected to lateral forces, also including P-δ effects, is analysed. The design of moment resisting frames is based on capacity design criteria: overstrength of the columns with respect to the beams and overstrength of the joints with respect to the columns to prevent brittle and premature failure in concrete. To obtain these performances, especially when high strength concrete is utilised, a large amount of transverse reinforcement is required, often very problematic to place in cast; to overcome this drawback the use of short fibres randomly distributed in the concrete is suggested. In order to estimate the effectiveness of the combined use of reinforcing fibres and steel transverse reinforcement a nonlinear analysis of framed structures under controlled lateral displacements is performed; the analysis is based on a finite element method in which the members are discretised into segments in order to taking into account of different moment-curvature relationships. The latter are calibrated on the basis of diagrams obtained by using analytical models for stress-strain compressive law of the concrete, fitting experimental results, also presented here. The results obtained show that is possible to reach comparable performance utilising very high percentage of transverse reinforcement or less amount of transverse steel coupled with fibre reinforced concrete. Moreover particular care should be paid to the influence of the P-δ effect in member subjected to very high axial forces.

INTRODUCTION

The evaluation of the ultimate bearing capacity of framed structures under actions due to severe earthquake, as suggested by recent European and International Codes, requires a nonlinear analysis, for which the complete constitutive laws of the materials are required for both monotonic and cyclic response. To ensure adequate dissipative capacity of the structural system and high ductility values, particular attention should be paid to the design of longitudinal and transverse steel reinforcements, especially when high strength concrete (HSC) members are used, avoiding brittle failure due to shear in the beams or in the joint regions. In fact, although high strength concrete offers, with respect to normal strength concrete, several advantages like more strength and durability, it is characterised by high brittleness in the post-peak response. To reduce this effect and to ensure adequate local ductility of the members a larger amount of transversal reinforcement is required with respect to normal strength concrete, especially in members subjected to axial load and bending moment when high values of compressive strain are involved. Recent studies [Bentur and Mindess, 1990] have shown that the use of discontinuous short fibres, randomly distributed in the matrices in adequate percentages and shapes, and coupled with traditional steel transverse reinforcement (spirals, stirrups, etc.), allows: i) to obtain high values of curvature ductility in the column cross-sections also in the presence of high values of axial loads [Campione et al., 1998]; ii) to avoid brittle shear failure in the beams ensuring the making of flexural mechanism [Campione et al., 1999a]; iii) to increase the shear strength also in the core of the beam-column joints in which high values of stress are concentrated [Filiatrault et al., 1995]. In this way it is possible to reduce, for these critical regions, the amount of transverse reinforcement required, generally difficult to place in cast. The aim of the present work is to show the advantages above mentioned in the case in which fibre reinforced high strength concrete is employed to partially fabricate end portions of beams and columns and joint regions of reinforced concrete frames, using

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normal concrete in the other portions of structure. To show these effects a nonlinear static analysis was carried out by modelling the experimental cyclic response in compression of fibre reinforced high strength concrete, also in the presence of transverse reinforcement. The analysis is carried out using a finite element method in which the members are discretised into segments, for which it is necessary to define the significant parameters of the flexural behaviour. For this raison a preliminary analysis is carried out in order to define the moment-curvature diagrams for different axial load, based on analytical laws in compression of the fibre reinforced high strength concrete fitting experimental results and on the constitutive law for the longitudinal steel taking into account the Baushinger and the strain-hardening effects. The results obtained show that a ductile behaviour, implying general large amount of transverse reinforcement in the critical regions, can also be reached decreasing the confining reinforcement but integrating the concrete with reinforcing fibres, avoiding in this way the congestion of the steel reinforcements.

EXPERIMENTAL INVESTIGATION IN COMPRESSION

In the present paragraph the experimental results of compression tests carried out by the authors in a previous study [Campione et al., 1999b] are briefly mentioned. The types of concrete investigated were: plain concrete, fibre reinforced concrete, plain concrete with steel spirals, fibre reinforced concrete with spirals. Different types of matrices were utilised: normal strength (NSC), middle strength (MSC) and high strength concrete (HSC), having characteristics shown in Table 1. In all the cases examined the use of superplasticizer in the percentage of 2 % by weight of cement was necessary to ensure good workability, which is reduced because of the high volume fraction of fibres and because of the use of silica fume in the case of HSC .

Table 1: Characteristics of the matrices investigated

Matrices	Cement (Kg/m^3)	Water (Kg/m^3)	Coarse aggregate (Kg/m^3)	Sand (Kg/m^3)	Silica fume (Kg/m^3)	Superplasticizer (%)
NSC	300	180	1050	850	/	2
MSC	300	164	1044	738	/	2
HSC	400	150	1050	720	55	2

The characteristics of the fibres investigated are shown in Table 2: L_f is the length of the fibres, d_f the equivalent diameter, f'_t the tensile strength and E the Young modulus. Experimental results [Campione et al., 1999b] regarding specimens reinforced with different percentages of fibres (1.5, 2, 3 %), have shown that the percentages of 2 % ensures especially for HSC best performances. Results referring to the coupled use of steel spirals and fibres are also presented here. The steel spirals of 5 mm diameter, yielding stress of 550 MPa, were placed in the moulds with pitches of 50 and 25 mm, corresponding to a volumetric ratio of ρ_s = 1.765% and 3.530 % respectively.

Table 2: Types and properties of the fibres

Fibre type	Shape	Length L_f (mm)	Equivalent diameter d_f (mm)	Tensile strength f'_t (MPa)	Young modulus E (MPa)
Polyolefin		25	0.80	375	12000
Carbon		20	0.78	800	100000
Hooked steel		30	0.50	1115	207000

Compression tests were carried out using a universal testing machine with open loop, by imposing a slow rate of displacement (0.2 mm/min) and recording the load-deformation curves, utilising LVDT's and a data acquisition system. In Figure 1 stress-strain curves for matrices at different strength are shown. It is interesting to observe that, with the increasing in strength, more brittle behaviour and higher values of initial tangent modulus were noticed. The stress-strain curve for HSC has a sharp peak and a steeper descending branch after the peak load. The research here mentioned focused mainly how to improve the compressive ductility of plain high strength concrete.

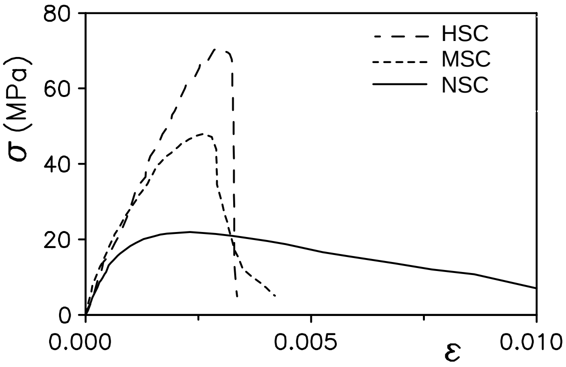

Figure 1: Results in compression for plain concrete at different strength

In Figure 2 the stress-strain curves for MSC and HSC, reinforced with different volume percentages of transverse reinforcement, are plotted. It was observed that when the maximum stress in compression is achieved the steel spirals are close to the yielding phenomenon; after, the coupled failure of the concrete core and of the steel spirals leads to specimen failure. At this stage a sudden drop in the stress-strain curve in compression, measured in the softening branch, is recorded and then no residual strength is available. It is interesting to observe that in the case of HSC, because of more brittle behaviour with respect to MSC, also the strain corresponding to the failure of steel spirals is reduced, so, more lateral steel reinforcement is required in order to obtain the same available ductility.

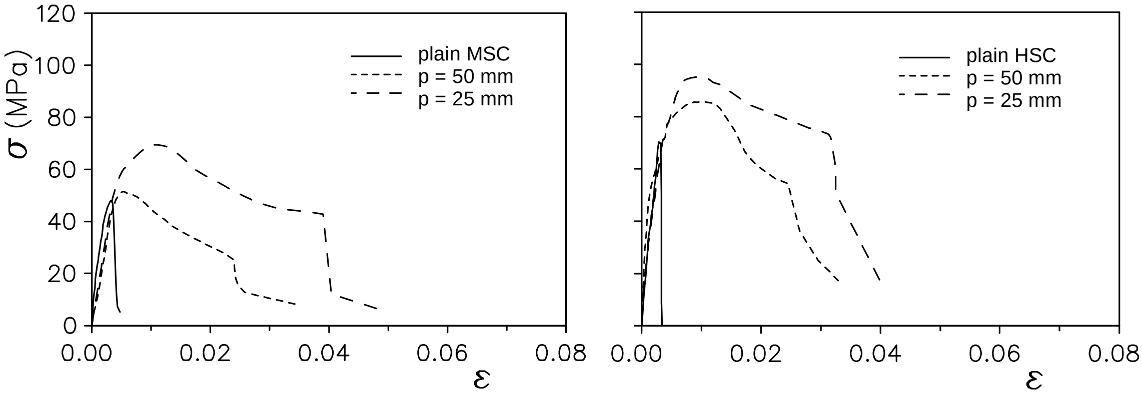

Figure 2: Stress-strain curves of plain concrete ($V_f = 0$) with steel spirals

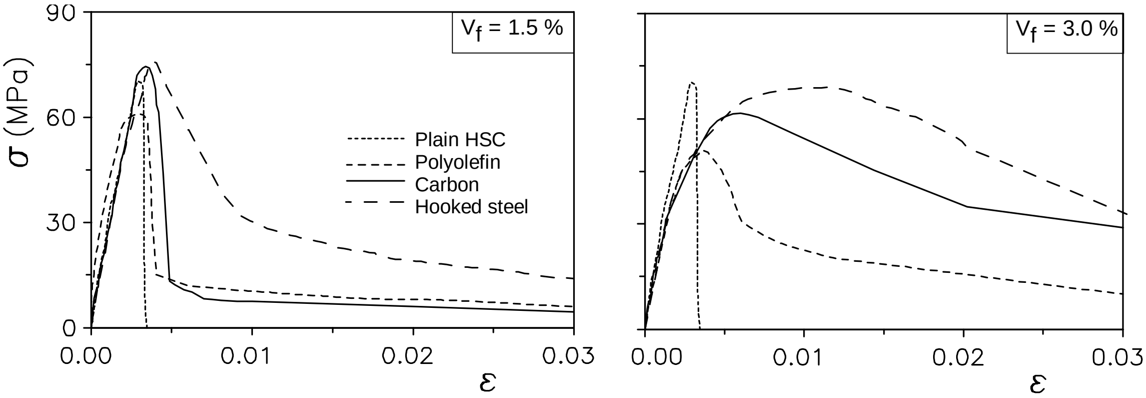

Figure 3: Stress-strain curves of fibre reinforced concrete at different types and percentages

Figure 3 shows experimental stress-strain curves obtained for HSC with different types of fibres at 1.5 and 3.0 % by volume. The adding of fibres in the matrices improves the residual strength and the energy absorption capacity with respect to plain concrete, but no significant variations in the maximum stress are observed.

Figure 4 shows the effect of carbon fibres at 2 % by volume percentage on the MSC and HSC matrices reinforced with steel spirals at pitches of 50 and 25 mm. This fibre type was subject of numerous studies previously carried out by the authors. The improvements due to the presence of fibres were: a lower steep of the descending branch; an increase in the ultimate strain corresponding to the failure of the steel spiral; and finally a significant residual strength after the spiral failure [Campione et al., 1999b].

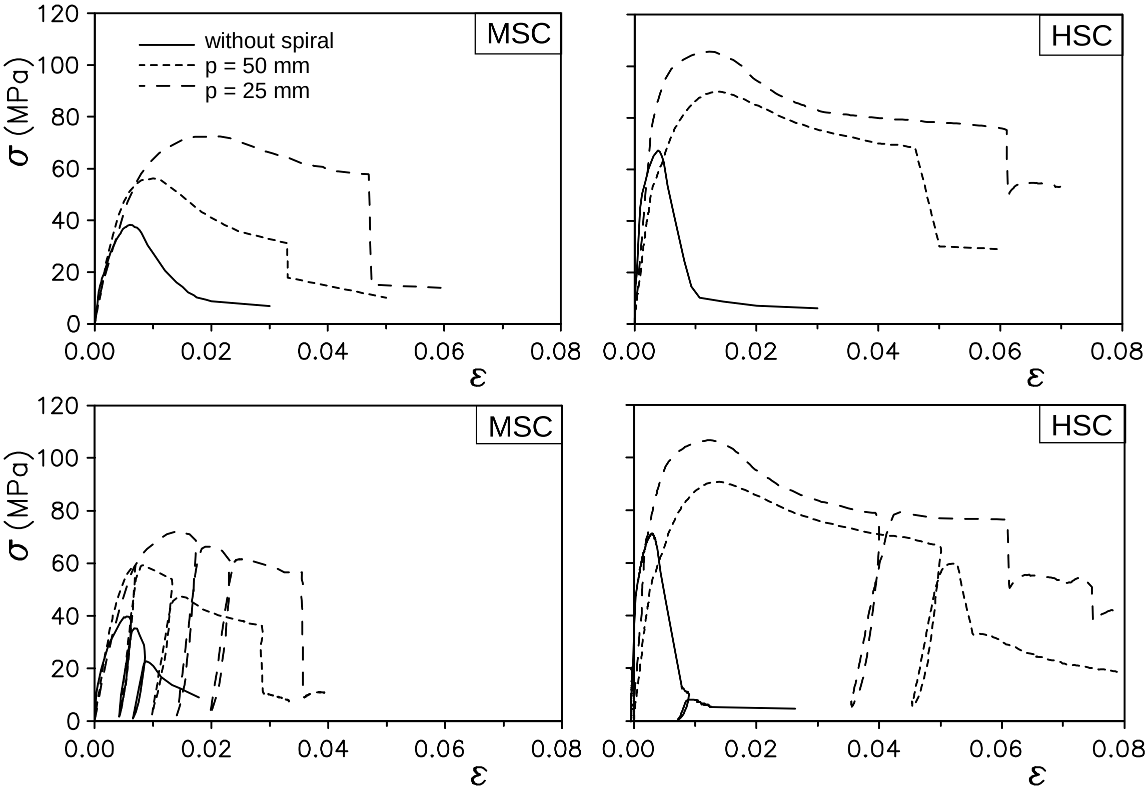

Figure 4: Monotonic and cyclic σ–ε curves of carbon fibre (V_f = 2%) reinforced concrete with steel spirals

In the same figure the cyclic behaviour of MSC and HSC with carbon fibres at 2 % by volume percentage and steel spirals is shown. In all cases examined, the cyclic envelope curve is very close to the monotonic response.

ANALYTICAL MODELLING FOR CONCRETE IN COMPRESSION

As stressed in previous studies regarding high strength concrete [Campione et al., 1998], due to the brittle nature of the matrix, it is possible to model the monotonic response by using the expression proposed by Mander et al. [1988], but with two different values of the β coefficients: β_1 for the ascending branch and β_2 for the softening branch. The β_1 values are depending on tangent and secant module; the β_2 coefficients are obtained fitting the experimental results in the softening branch. For the cyclic response it is possible to model the unloading and reloading branches utilising the analytical laws proposed for confined concrete by Mander et al. [1988] and confirmed by the authors for normal and high strength fibre reinforced concrete confined with steel spirals. The tangent modulus of elasticity can be evaluated adopting the following expression suggested by the Canadian Code [CSA 1994] that is in good agreement with the experimental data :

$$E_t = \left(3300 \cdot \sqrt{f'_c} + 6900\right) \cdot \left(\frac{\gamma_c}{2300}\right)^{1.5} \quad \text{in MPa} \tag{1}$$

being γ_c the density of concrete in Kg/m^3, that for the HSC considered here is equal to 2400 Kg/m^3. In Table 3 the fundamental parameters to model HSC in compression are given: the peak stress f'_c and the corresponding strain ε_0; the tangent modulus of elasticity E_t deduced by Eq. (1); the β_1 and β_2 coefficients and the ultimate strain ε_{cu} corresponding to steel spiral failure. These values will be utilised in the applications that follow.

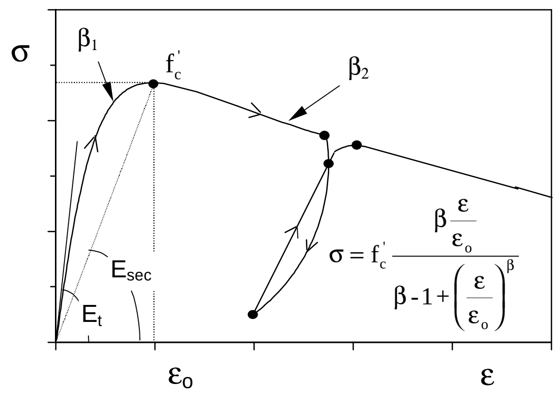

Figure 5: Analytical σ-ε curves

Table 3: Characteristic values for HSC in compression

ρ_s (%)	V_f (%)	f'_c (MPa)	ε_0	E_t (MPa)	β_1	β_2	ε_{cu}
0	0	70.23	0.0029	36833	3.62	14.00	/
0	2	67.30	0.0038	36212	1.40	4.50	/
1.765	0	85.77	0.0102	39931	1.18	2.50	0.023
1.765	2	90.06	0.0140	40736	1.50	1.65	0.045
3.530	0	95.26	0.0102	41687	1.25	1.55	0.030
3.530	2	105.71	0.0122	43520	1.28	1.50	0.064

FLEXURAL CYCLIC BEHAVIOUR

In order to point out the advantages due to the presence of the fibres and to verify the increase in the dissipative capacity of the sections in which the plastic hinges occur, the cyclic moment-curvature diagrams are deduced. For the fibre reinforced concrete, the carbon fibre type is considered. The results refer to a square cross-section made of HSC subjected to a cyclically variable bending moment combined with an axial load that is assumed to be constant. The approach adopted is the well known method based on a discretisation of the section into a finite number of elements having constant value of stress deduced by the analytical law presented in the previous paragraph. The following data are assumed: c/h=0.136, being c the cover and h the distance from the top of the section to the centroid of the bottom steel; $\mu=\mu'=A_s/(bh)=0.89$ %, being $A_s=A'_s$ the longitudinal steel in tension (or in compression) and b the base section; $f_y=440$ MPa the yielding stress of the steel; $E_s=200000$ MPa and $E_h=0.01\cdot E_s$ the Young modulus and the hardening modulus, respectively. The constitutive laws assumed for the material are: the σ–ε relationship proposed in the previous paragraph for the concrete in compression (the tensile strength of the concrete is neglected) and the model proposed in Menegotto and Pinto [1973] including also the Baushinger effect for the longitudinal steel.

The curves showed in Figure 6, obtained for a constant value of axial load N, corresponding to a dimensionless value of 0.21, are in terms of $M/(bh^2)$-φh, M being the bending moment evaluated with respect to the centroid of the cross-section, and φ the curvature. The diagrams contain the second cycle (when the cover is spalled) for three different types of concrete shown in the same figure, depending on the confinement level; the spalling strain for the cover is assumed equal to 0.004 and 0.006 for plain and fibre reinforced concrete respectively. The curves are obtained by considering the maximum strain value for the concrete corresponding to the spiral failure ε_{cu}, verifying that the maximum elongation in the steel is less than 9%, limit value provided by EC8 [Eurocode 8, 1994] for critical regions designed for high ductility. The following values of curvature ductility are reached: φ_{max}/φ_y =15.90, 16.63, 20.76 for type material a), b) and c) respectively (Figure 6). In the same diagram, with thin solid line, the cycle obtained by using material type c) with ε_{max}=0.03 is showed in order to point out the equivalent performance obtained by using a concrete containing half percentage of transverse reinforcement steel ratio and carbon fibres at 2% by volume, having this material further deformation capacity.

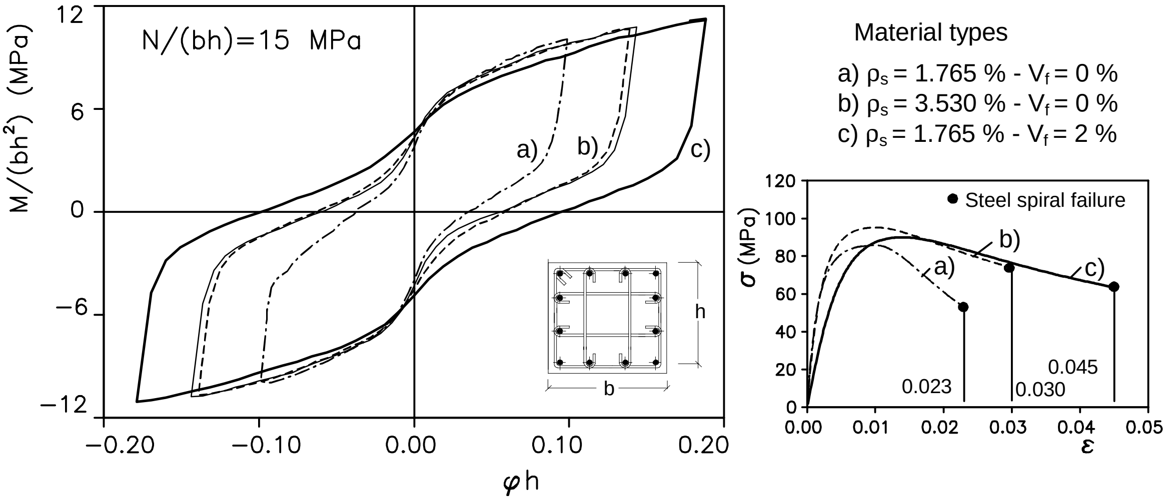

Figure 6: M/(bh^2) - φh curves with variation in the material type

CODE REQUIREMENTS

Referring to members subjected essentially to axial load and bending moment, several International and European Codes [ACI 1995, EC8 1994], in order to ensure local ductility allowing a dissipative global mechanism, propose for normal strength concrete some geometrical limitations and a minimum amount of transverse reinforcement. Moreover limitations on the total longitudinal steel reinforcement ratio are given: Eurocode 8 provides a minimum value 1%, and a maximum value 4%; ACI 318-95 Code suggests the same minimum limitation but a maximum value equal to 8%, that can imply a congestion of the reinforcement, especially in critical regions.

Recently Canadian Code [CSA 1994] and the Provisions ACI 318-95 [Ghosh, 1997] introduced the use of high strength concrete up to 80 MPa, allowing in this way to reach high value of bearing capacity for the columns, by imposing obviously a minimum amount of transverse reinforcement.

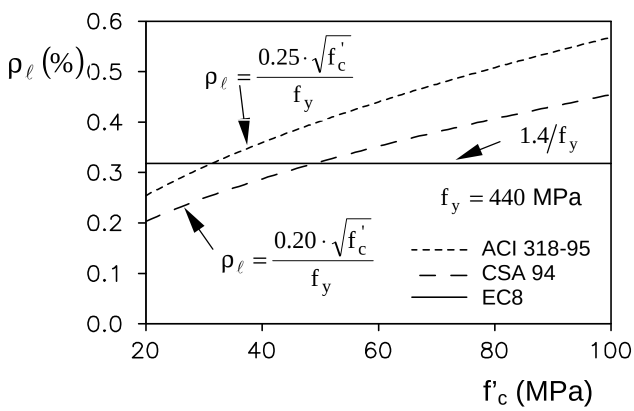

Figure 7: Minimum flexural reinforcement

For the HSC beams the above mentioned codes prescribe a minimum percentage of tension steel reinforcement, related to the strength of the concrete and to the yielding stress of the longitudinal steel, as shown in Figure 7, to ensure a ductile flexural response avoiding that the flexural capacity being lower than the cracking moment calculated using the modulus of rupture. In the same figure the minimum value proposed by EC8, as a function of f_y only, is contained. In the numerical applications that follow, referring to HSC with f'_c =70.23 MPa, the more strict limitations are adopted.

NUMERICAL APPLICATION

The numerical application refers to a one floor reinforced concrete frames designed utilising high strength concrete in which the members (beams, columns, joints) were designed in accordance with the following capacity design criteria: overstrength of the columns with respect to the beams and overstrength of the joints with respect to the columns to prevent brittle and premature failure. The examples refer to three different types of frames denoted as: A, B and C in which longitudinal reinforcements are the same, but different amount of transverse reinforcement and concrete types are considered.

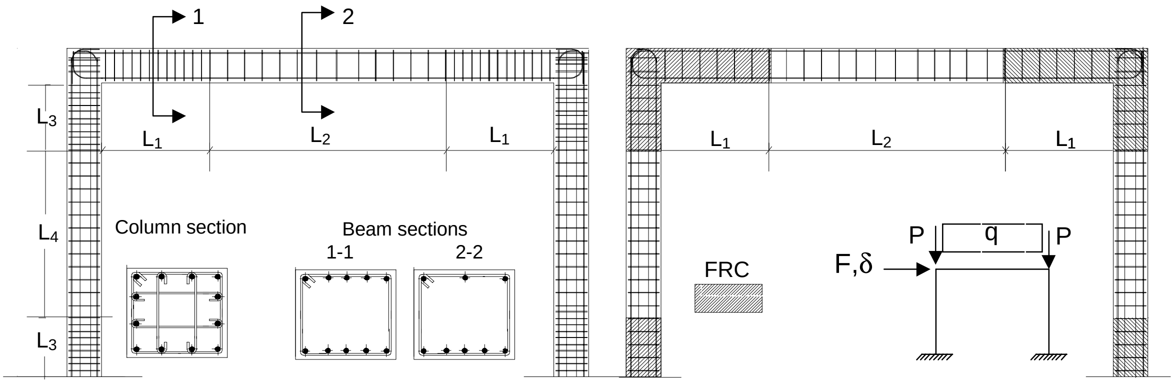

Figure 8: Structural scheme

The structural scheme is shown in Figure 8 in which the calculus model is also enclosed. In type A, HS plain concrete is utilised; the hoops in critical regions of the columns, having pattern as in Figure 8, are arranged with spacing equal to 90 mm, corresponding to a reinforcement ratio ρ_s=1.765 % evaluated with reference to the effective confined core according to Mander et al. [1988]; for the beams a minimum amount of stirrups are considered according to high ductility beams EC8 requirements. The type B is obtained varying only, with respect to the type A, the spacing of the stirrups in the critical regions of the columns, assumed equal to 40 mm, corresponding to ρ_s=3.530 %; in this case all requirements contained in EC8 for high ductility performance are satisfied, circumstance that is not fully verified for type A. Finally type C is designed as type A, but fibre reinforced concrete (FRC) in the critical regions is employed, being in this way possible to decrease also the number of stirrups in the critical regions of the beams, as shown by the authors in previous study [Campione et al., 1999a].

The nonlinear analysis is carried out by using DRAIN-2DX program assuming the following data: square-cross section for columns and beams with side 400 mm; longitudinal reinforcing steel bars of 20 mm diameter arranged as shown in Figure 8; L_1=0.8 m, L_2=2.9 m, L_3=0.6 m, L_4=1.8 m. The frames were subjected to: - a lateral displacement monotonically increasing; - a vertical load on the beam q=30 KN/m; - a concentrated vertical load P=5000 KN for each column. The beams are divided in three elements to take into account the different reinforcement adopted at the top of the section; the column is considered as an element. Each element is modelled with elastic beam with two end plastic hinges idealised through bilinear model preliminarily calibrated on the basis of the effective moment-curvature diagrams [Filiatrault et al., 1998a,b]. In order to take into account the axial load, the N-M yield surface also is defined.

The results of the static push-over analysis are presented in Figure 9a), in which the global mechanism, the same for all types examined, is shown and the sequence of plastic hinging is indicated. In Figure 9b), the effective $M/(bh^2)$-φh curves for the different plastic hinges considered at the base column subjected to higher value of axial load are considered. The maximum lateral displacement corresponds to the failure of the concrete $\varepsilon = \varepsilon_{cu}$. At this step of analysis the maximum elongation of the steel is less than the limit value prescribed by EC8 for high ductility performance. The comparison between the curves shows that the advantages due to the presence of the fibres, stressed in the analysis of the cross-section, are reflected into translation ductility of the frames. In the same Figure 9a) the results of the analysis considering the P-δ effect are also included. It can be observed that the use of HSC allows to realise bearing elements with reduced dimensions but the second-order effects can be very significant. For the cases examined, the base-shear corresponding to the making of global mechanism is reduced by 20 % about and the deformation capacity of the plastic hinges is not fully exploited.

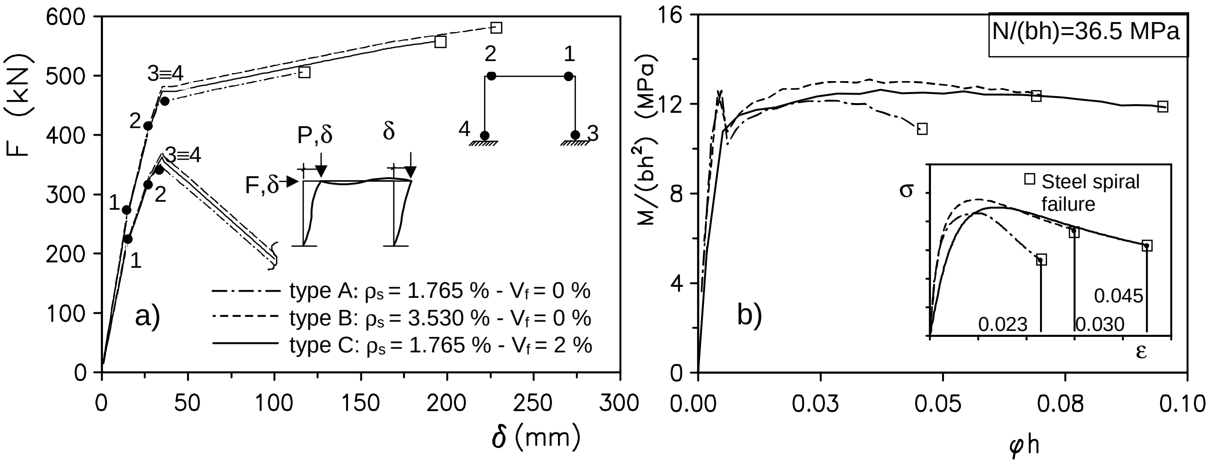

Figure 9: Collapse mechanism for different confining levels : a) base shear-lateral displacement curves ;

b) plastic hinges at column base sections

CONCLUSIONS

The nonlinear analysis for framed structures has stressed that comparable ductile behaviour can be obtained by using in the critical regions less amount of transverse reinforcement but integrating the concrete with reinforcing fibres. Results have shown that also in the case of HSC members it is possible to achieve a dissipative collapse mechanism in presence of very high values of axial loads, but particular attention must be paid on the P-δ effect that can significantly reduce the bearing capacity and the available ductility of the frames. The analysis carried out here, needs however to examine further aspects as the buckling problem for the longitudinal steel bars, the fixed end rotation effects and the dowel action in the beams.

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