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Seismic behaviour of steel fiber concrete beam-column

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ABSTRACT: The behaviour of reinforced concrete frames subjected to earthquake motions is much influenced by the behaviour of the joint regions. This is especially true in the case of precast frames. Adequate ductility of the structure, given by elements and their connections, is also ensured by reinforcement, in many cases producing congestion in joints. To avoid steel congestion in beam-column joints, and to improve the seismic behaviour of the structure, the use of the steel fiber reinforced concrete (SFRC) in four full-scale beam-column joint subassemblages was tested. The behaviour of three precast joints is presented and compared with a monolothical one. The experimental results show the effect of the SFRC on the bending moment capacity, cracking resistance, ductility and stiffness degradation of the joint frame.

1 PRECAST REINFORCED CONCRETE FRAMES

The structures designed for the seismic loads recommended by design codes can survive strong ground earthquake motions only if they have sufficient ability to dissipate seismic energy. This energy dissipation is provided mainly by inelastic deformations in critical regions of the structural system and requires adequate ductility of the elements and their connections
(Paulay 1977).

The joints are the regions of greatest seismic design difficulty. The lateral displacements of precast frames may be greater than that of monolithic systems due to the reduction of stiffness in the joints. Hence, particular attention should be given to achieving adequate strength and stiffness of the joints and, in addition, it is most important that the beam-column subassemblages have the necessary ductility. The joint core needs special attention, because of the critical shear and bond stresses that can develop there (Fig.1a). Presenting the shear-resisting mechanisms of beam-column core (Fig.1b,1c), Wong, Priestley and Park (1990) show that the design of the joints according to the seismic codes (Code of Practice for the Design of Concrete Structures 1982), leads to a maze of reinforcements in the joints with negative effects on their quality. The congestion produced by the steel also increases labour and material costs.

In order to decrease the steel congestion in beam-column joints of ductile concrete seismic resistant structures and to ensure that seismic joints maintain integrity through several cycles of reversed flexure, it was assumed that fiber reinforced concrete in the joint region can offer a solution. Recent researches have established that steel fibers produced significant improvements in engineering properties of the concrete and the composite has found its use as a promising material. At the same time, tests on SFRC members (Nagasaka 1988) and subassemblages have been devoted to establishing strength and behaviour under external loads (Olariu, Ioani, Poienar 1987, 1988).

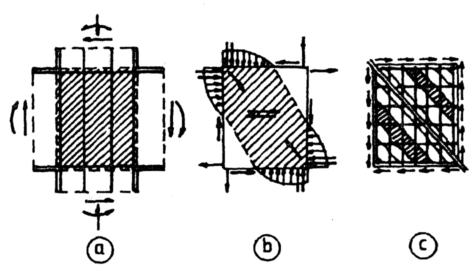

Figure 1. The joint core mechanism.

In the present investigation, the performances of the SFRC joints have been studied by the testing of some precast R/C frame subassemblages.

2 TEST UNITS

During the test programme, four full-scale central joints were tested. The subassemblages represent a part of an 8-storey residential building concrete frame structure (Fig.2). The dimensioning and reinforcement were carried out according to existing Romanian design codes. The geometrical configuration, the loading conditions and the test programme have been identical for all the units.

The joint NM was completely cast in place (Fig.3) and the joint NP (Fig.3) was composed by precast R/C beams and cast in place columns. The concrete used in NM and NP units (beams, columns, joint areas) was the normal one. The other two models consisted of cast in situ inferior and superior columns and two precast beams connected in the joint by steel fiber reinforced concrete (SFRC), as follows: NPF I -the beams are face-to-face with columns (Fig.3) and NPF II -the beams are at a distance equal to the beam height (60 cm.) from the columns (Fig.3). In the case of NPF II, the number of vertical and horizontal hoops, in the cast-in-place joint area, was diminished.

The characteristics of materials used for the experimental elements are outlined in Tables 1 and 2.

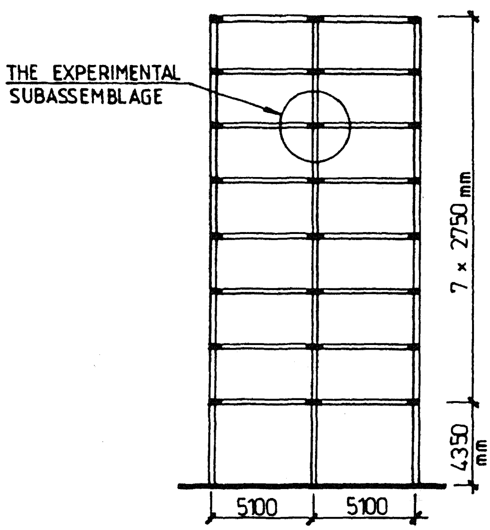

Figure 2. The prefabricated concrete frame.

In a previous paper the authors (1987) have drawn the conclusion that the optimal volumetric reinforcement percentage v_v for SFRC in joints is about 1.5%. Therefore we used plain steel fibers in volume percentage of 1.5%, fibers having an aspect ratio equal to 118 (length = 45mm, diameter = 0.38mm) and an ultimate strength of 1200MPa.

Table 1. Properties of specimens.

Specimen	NM, NP, NPF I, NPF II	
Beam	Top bars	4D25
	Bottom bars	2D25+2D10
	Hoops	$1\phi10+1\phi8/100$
Bottom column	Bars	8D20+4D16
	Hoops	$3\phi10/100$
Top column	Bars	12D16
	Hoops	$3\phi8/100$
Joint area	Hoops	$6\phi10^*$

D diameter of reinforcing bars (mm)
ϕ diameter of hoops (mm)
* NPF II has only $4\phi10$ and vertical hoops $\phi10$ at 20 cm.

Table 2. Material properties of specimens

Specimen	NM	NP	NPFI	NPFII
Steel: f_y	358	358	376	376
Concrete joint				
f_{cube}	27.4	26.3	37.5	32.0
f_{ct}	2.38	2,07	6.67	5.04
E_c	24424	24387	24014	19584

f_y = yield strength of steel reinforcement (MPa)
f_{cube} = concrete compressive cube strength (MPa)
f_{ct} = tensile strength of concrete (MPa)
E_c = modulus of elasticity for concrete (MPa)

3 TEST PROCEDURE FOR UNITS

The supporting system and the force applying were thus chosen to meet as close as possible the corresponding edge conditions of the building structure (Fig.4). Alternate reverse horizontal forces were applied to the top of the column(Fig.5). The loading history represents two major earthquake actions (Fig.6). Different parameters were measured and recorded by strain gauges (concrete strains, stress in steel bars and in hoops) or by mechanical and electronic devices.

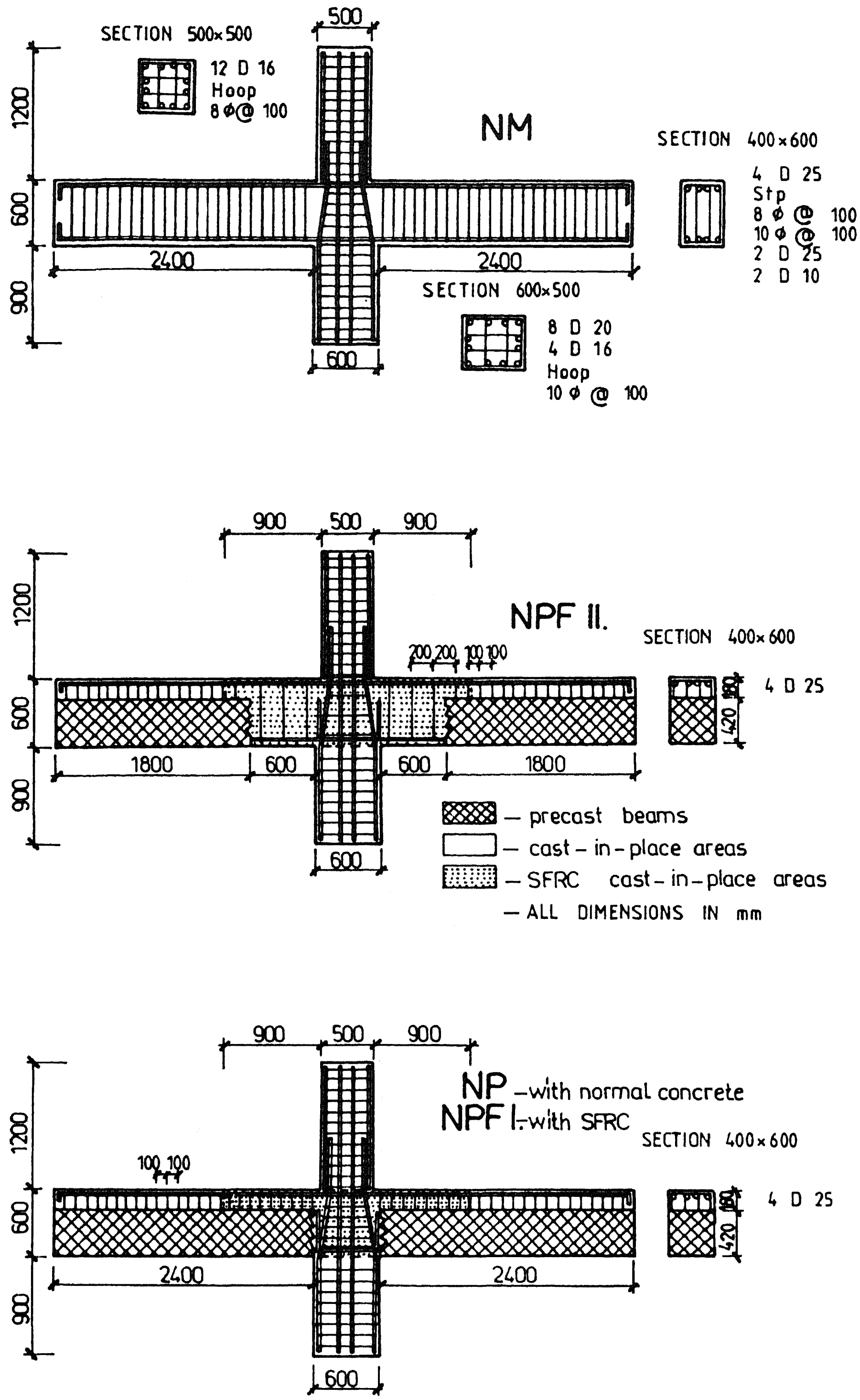

Figure 3. Reinforcement of the test subassemblages.

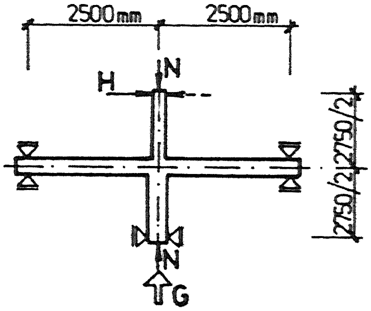

Figure 4. Static test diagram.

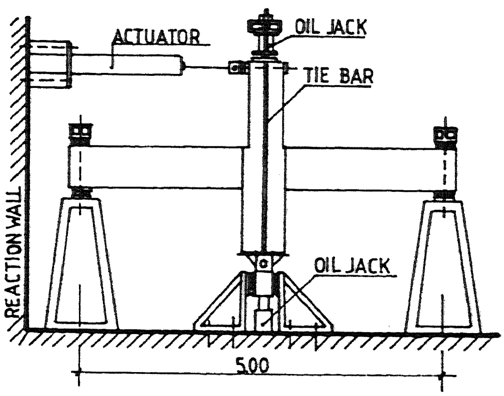

Figure 5. Loading condition.

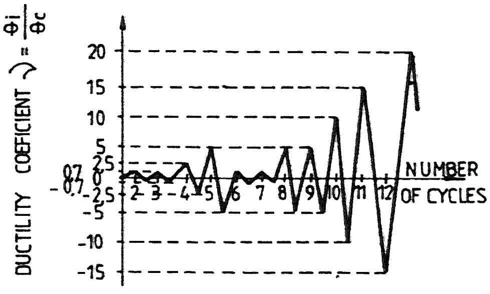

Figure 6. Loading history.

Table 3. Cracking state of units

Unit	No. of cycle	Ductility factor v	Horizontal force KN	%
Type A cracks				
NM	II	0.7	86	126
NP	I	0.5	66	100
NPF I	II	0.7	124	188
NPF II	II	0.7	200	303
Type B cracks				
NM	IV	2.5	170	89
NP	IV	2.5	190	100
NPF I	V	5.0	238	125
NPF II	IV	2.5	245	129

Figure 7. The cracking distribution (NP model).

Figure 8. The cracking distribution (NPF I model).

4 TEST RESULTS FOR UNITS

4.1 The cracking state

The occurring of cracks in the four units had a complex character, determined by the inner stresses developed both in the precast beam-joint interface area and in the core joint. Two types of cracks were studied:

A. Cracks due to bending moment (at the face of the precast beams);

B. Cracks in the central area of the joint, along-side with the theoretical diagonal failure plane of the joint.

The horizontal force (Table 3), when cracking started in the joint core is by 25-29% greater in SFRC precast joints, compared to the classical precast joint (NP).

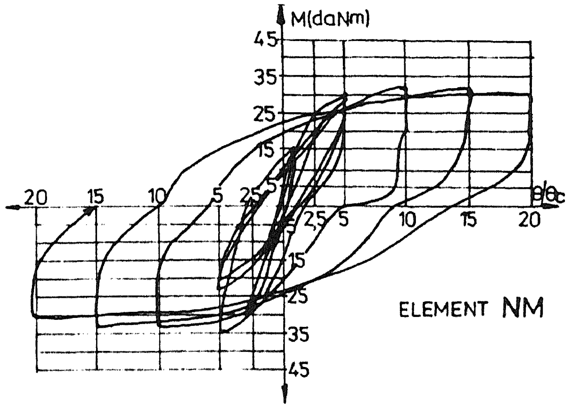

Figure 9. The hysteresis loop (NM model).

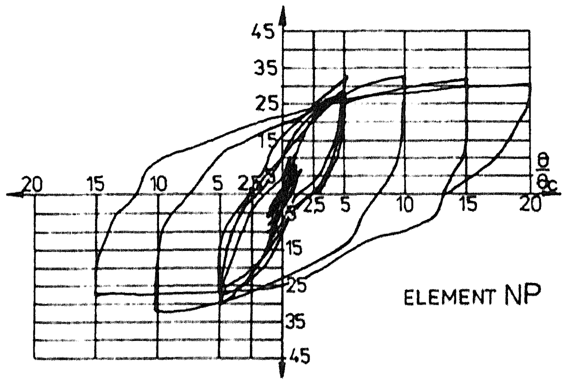

Figure 10. The hysteresis loop (NP model).

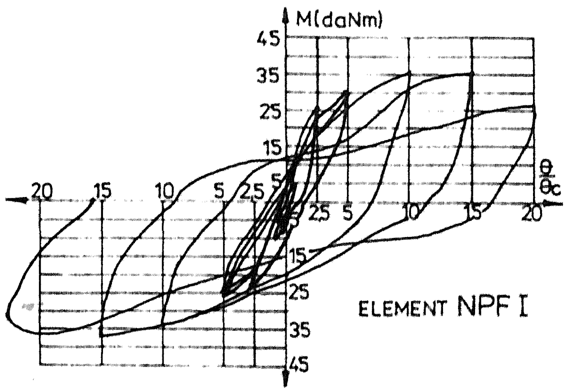

Figure 11. The hysteresis loop (NPF I model).

Maximum width of cracks at the forces prescribed by Romanian design Code were 0.3mm in NM, 0.7mm in NP and only 0.5mm in SFRC precast joint (NPF I). The cracking pattern is different for SFRC joints. The cracks are more numerous, more evenly

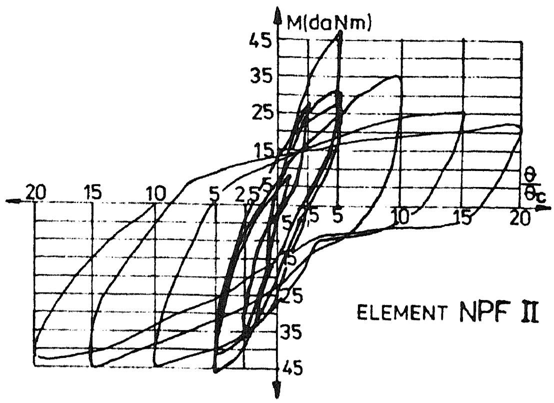

Figure 12. The hysteresis loop (NPF II model).

distributed, with smaller widths than in classical NP and NM joints (Fig.6, Fig.7).

4.2 Strength capacity

Unit strength defined as maximum moment reached during the tests are illustrated in Table 4.

Table 4. Strength capacity for units

Specimen	H_{max}	M_{max}	%
NM	290	295	102
NP	285	290	100
NPF I	364	370	127
NPF II	452	460	158

H_{max} = maximum horizontal force (kN)
M_{max} = maximum flexural moment (kNm)

The increase of the strength capacity of joints with SFRC is due to the disperse reinforcement of the monolithical concrete, to the improvement of longitudinal bar anchorage from the joint core, to reduced crack widths in the joint even after many cycles. The failure of the subassemblage occurred after 13 cycles by the widening of the crack in the beam at the face of the column.

4.3 Energy-dissipating capacities

The displacement-controlled cycles were established according to a $v = \theta/\theta_y$ ductility factor, where θ_y is the theoretical relative rotation between the beam and the column, corresponding to the first yield in the longitudinal reinforcing bars and θ is the

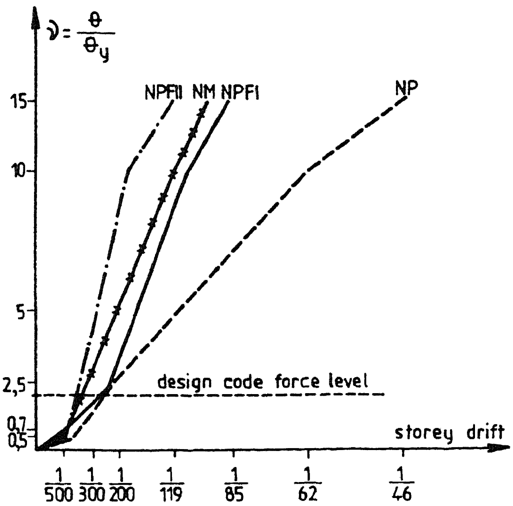

Figure 13. The stiffness degradation.

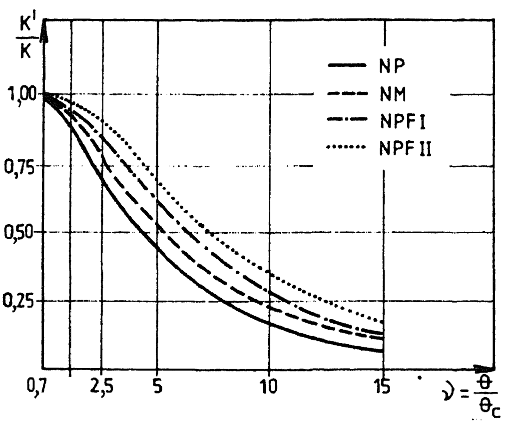

Figure 14. Drift variation.

maximum relative rotation in that cycle. The hysteresis loops for the tested units are shown in Figs.9, 10, 11 and 12.

During the tests, in each cycle and for each element, the secant stiffness (K′) was determined as the ratio of maximum forces H of a cycle to the attained displacement Δ, corresponding to this force.

The stiffness degradation during the cycles for the four units is shown in Fig.13. As it is a design concern, according to the attained ductility, the storey drift was computed (Fig.14).

5 CONCLUSIONS

The test results led to the following conclusions:

1. Precast SFRC joints revealed a superior behaviour over R/C precast or monolithic joints.
2. The steel fiber confines the concrete from the joint core, controlling the cracking and reducing the potential sources of brittle failure. In SFRC joints, the cracks started at a force greater by 25% than in classical units as finer cracks without spalling.
3. The ultimate strength capacity of SFRC precast joints is about 27-58% greater than in R/C joints (NM,NP) due to a superior quality of the concrete in the joint and to a better anchorage of the longitudinal bars in the joint.
4. the energy-dissipating capacity of SFRC joints is greater than in R/C joints by 20% in cycles I-X, as for ductility factors up to $\nu = 5$. The M-θ hysteresis loop has pinching, except the failing cycle XIII. The NPF II variant of precast SFRC joint has the greatest energy-dissipating capacity, by 50% greater than for the precast R/C joint NP.
5. The stiffness degradation is more reduced for SFRC joints. As it was expected (Fig.13), the NPF II joint has the most reduced stiffness degradation.
6. A higher stiffness of the SFRC NPF II joint leads to meeting the design requirements for seismic forces, on condition that the storey drift (Fig.14) is smaller (1/450) than the limit imposed by the NP15/87 standards (1/300). The P100 code limits the storey drift to 1/200, condition which is not met either by the NPF I joint (1/240) or by the monolithic normal concrete joint NM (1/400).

The research allowed the completing of the necessary provisions to implement new types of connections in seismic design, based on this superior material:SFRC.

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