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Cyclic tests on normal and lightweight concrete beam-column subassemblages

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ABSTRACT: Ten half-size interior beam-column subassemblages were tested under quasi-static earthquake-type loadings, five in normal and five in lightweight concrete. The basic specimen was designed according to EC8 code. In the others different parameters were varied: a) the amount of joint transverse reinforcement, b) the diameter of the beam's longitudinal bars, and c) anchorage details. The behavior of the specimens was compared in terms of lateral load-displacement hysteretic response. In some cases detailing arrangements differing from those given in EC8 resulted in equally satisfactory performances. Besides, it was assessed that the behavior of lightweight concrete specimens is comparable to that of normal concrete specimens.

1 INTRODUCTION

This work is part of an experimental and theoretical research project which aims to ascertain the extendability of EC8 rules for reinforced concrete structures to lightweight concrete structures. Comparative studies on the materials (Nuti and Pinto 1987) and on member elements (Baggio et al. 1988) have already been carried out. At present, attention is focused to the seismic behavior of beam-column subassemblages (Monti and Nuti 1992).

2 BACKGROUND

The design of r.c. buildings in seismic regions according to EC8 code (1988) follows the well-established capacity design criterion, by which a "strong column-weak beam" behavior is sought. This latter implies that: a) the beams' ends become plastic hinges, b) the columns remain preferably elastic, and c) the joints remain rigid. This configuration leads to a highly favorable structural mechanism in which: the beams ensure the dissipation of the energy input at the base, the columns cannot develop "soft-storey" mechanisms and the joints act as links between the beams and the columns. The beams should therefore have a highly stable hysteretic behavior in order to undergo significant plastic rotations without any substantial loss of strength or energy-dissipation capacity: this is obtained by means of accurate reinforcement detailing. The columns should be designed to resist the actual maximum flexural strength of the beams' end cross-sections framing in the joint, in order to remain in the elastic range. The task of the joints is therefore to transmit the high horizontal and vertical shear forces between the beams and the columns. Adequate transverse reinforcement should be supplied in order to resist to such shear forces and to provide confinement to the concrete core and sufficient anchorage to beam longitudinal reinforcement.

This paper examines the effectiveness of EC8 provisions and the effects of different reinforcement configurations in the design of joints. Besides, it tries to determine whether the use of lightweight concrete could result in an equally reliable response of such critical regions.

3 EXPERIMENTAL STUDY

3.1 Test specimens

Ten half-scale interior beam-column subassemblages were tested, five in normal reinforced concrete (N) and five in lightweight reinforced concrete (L).

The dimensions of the specimens are shown in Fig. 1. The half-scale specimen is considered as part of a typical moment-resisting frame with beam span of about 5.0 m and interstorey of about 3.5 m. On the column top two hydraulic jacks applied the vertical load N_d and the lateral force F reproducing the effects of a seismic action. The applied vertical load N_d, set to 20% of the axial strength load, was not a parameter of concern (Uzumeri 1977), therefore it was kept constant in all the tests. All the specimens were instrumented with 50 transducers.

In the basic specimens, NA1 and LA1 (i.e., those designed according to EC8 code, ductility class "High"), the continuous longitudinal reinforcement in the 200x400 mm beams consisted of

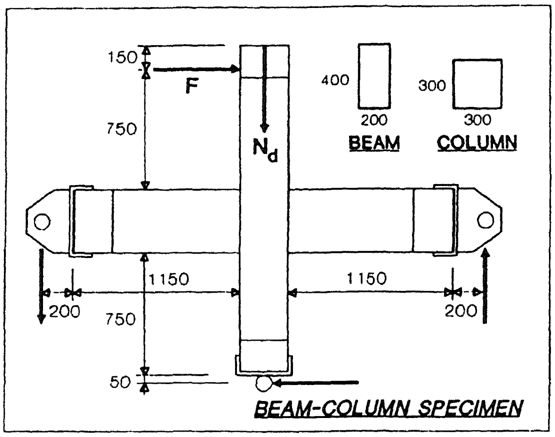

Figure 1. Test specimen.

three $\phi16$ bars at the top and two $\phi16$ bars at the bottom; the beam web reinforcement was provided by means of $\phi8$ square hoops at 80 mm spacing. The continuous longitudinal reinforcement in the 300x300 mm columns consisted of four $\phi20$ bars at each side; the column and joint transverse reinforcement consisted of $\phi8$ square hoops at 50 mm spacing.

In the other specimens, the following parameters were modified: amount of transverse reinforcement in the joint (specimens NB1, LB1, ND1, LD1, with $\phi8$ square hoops at 100 mm spacing); different anchorage details of beams' longitudinal bars (NC1, LC1, ND1, LD1, with two top and two bottom bars anchored inside the joint); different diameter of beams' longitudinal bars (NE1, LE1, with four $\phi14$ bars at the top and one $\phi12$ plus two $\phi14$ bars at the bottom).

The design concrete compressive strength was 20 MPa. The longitudinal and transverse reinforcement design strength was 382 MPa.

Table 1. Material properties, joint transverse reinforcement ratios, shear stress ratio and bar diameter-core depth ratio

TEST	f_{cm} (MPa)	f_{ym} (MPa)	ω_j	ρ_j (%)	τ_j (%)	δ
NA1	38.2	473	.31	1.15	18.8	1/18
LA1	35.8	461	.33	1.15	18.2	1/18
NB1	26.3	576	.23	0.58	29.3	1/18
LB1	35.0	569	.17	0.58	25.1	1/18
NC1	22.5	636	.53	1.15	37.1	1/18
LC1	32.3	552	.37	1.15	24.4	1/18
ND1	30.3	545	.20	0.58	24.0	1/18
LD1	34.8	553	.17	0.58	24.4	1/18
NE1	28.9	520	.42	1.15	27.1	1/22
LE1	31.3	570	.38	1.15	26.0	1/22

In Table 1 the mechanical properties of the materials employed were determined by standard laboratory tests (f_{cm}= mean concrete strength and f_{ym}= mean steel yield strength) and are listed, along with the joint transverse reinforcement ratios (ω_j mechanical and ρ_j geometrical), the actual shear stress ratio τ_j in the joint concrete core and the bar diameter-core depth ratio δ.

The actual shear stress ratio is defined as

$$\tau_j = \frac{1}{b_c h_c}\left[\frac{2}{3}(A_{s1}+A_{s2})f_{ym} - V_c\right] \qquad (1)$$

where b_c,h_c= joint's width and depth; A_{s1},A_{s2}= top and bottom beam's longitudinal reinforcement cross-sections; V_c= column shear when both beams yield.

3.2 Loading history

The top column displacement history for each specimen is depicted in Fig. 2.

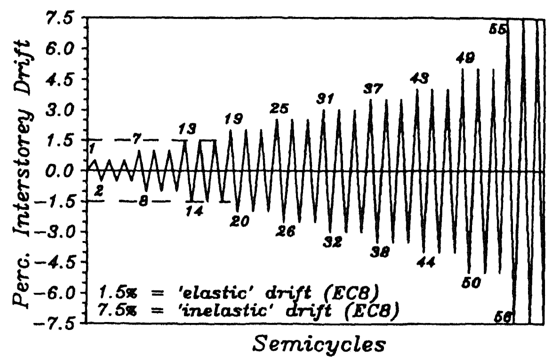

Figure 2. Top column displacement history.

Displacement increments were imposed equal to one-third of the design maximum interstorey drift allowed in EC8, i.e. 1.5% of the storey height. It is worth noting that a 150 mm top displacement corresponds to a relative column end displacement of 7.5% the interstorey height. This is exactly the "inelastic" value which can be found according to EC8 with a behavior factor of 5 (ductility and regularity class "High").

Each increment (10 mm) corresponded to about 50% of the displacement δ_y by which both beams yield. The maximun displacement δ_{MAX}= 150 mm corresponded to a monotonic displacement ductility level of 7.5 and to a cumulative displacement ductility demand D_n of about 190. Here, D_n is conventionally evaluated as follows (for symmetric cycles)

$$D_n = \frac{2}{\delta_y}\left(\frac{\delta_{MAX}}{2} + \sum_{i=1}^{n} n_c \delta_i\right) \qquad (2)$$

where n_c is the number of (consecutive) cycles with displacement δ_i and n is the number of displacement increments.

3.3 Test results and influence of parameters

In all the tests performed, the columns remained elastic while wide cracks opened in the beams, mainly at the interface with the column.

Fig. 3 presents the results of specimens NA1 and LA1, i.e. those designed following EC8 code prescription. It is noted that the overall performance of the lightweight concrete specimen is better in terms of dissipated energy (pinching less severe), even though a more accentuated strength degradation is observed. The pinching in the diagrams can be regarded as an indicator of the amount of the slippage of the rebars through the joint. In both tests, the beams' longitudinal reinforcement slipped through the joint core after few semicycles (7-10), as was also observed in previous experimental tests on lightweight beam-column joints (Hanson 1983). Exception to this behavior is given by specimens NE1 and LE1, i.e. those with lower bar diameter-core depth ratio (δ=1/22), where slippage was observed only after semicycle 13. Early slippage was also observed in NB1 and LB1 specimens, i.e. those with lower joint transverse reinforcement ratio (ρ_j=0.58%). In these tests, the overall behavior was substantially stable, even though the joints crushed completely when displacements reached 150 mm, so that this value appear to be a lower bound for this tests. Further tests are needed to assess the reliability of joint transverse reinforcement ratios higher than this value, but lower than that given in EC8 code.

Fig. 4 presents the results of specimens NC1 and LC1, i.e. those with the beam's longitudinal bars anchored inside the joint with a 90-degree hook. This arrangement resulted in a shortening of the beams' plastic hinge, but the performance of the subassemblages has evidently improved. This is due to the widening of the compression strut inside the joint core, which assures the shear transfer mechanism (Milburn and Park 1982). The well-confined concrete core (ρ_j=1.15) allowed the development of such a mechanism. In specimens ND1 and LD1 the joint core was not sufficently confined (ρ_j=0.58) thus resulting in the complete crushing of the concrete core and in a poor overall performance.

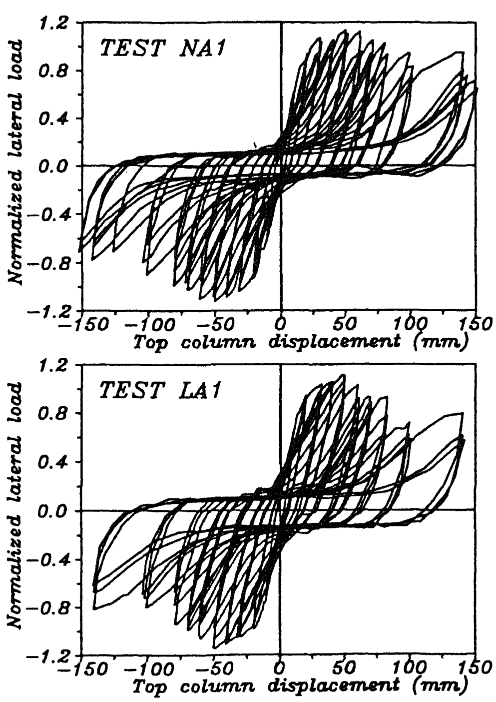

Figure 3. Lateral force-top column displacement diagrams for specimens NA1 and LA1.

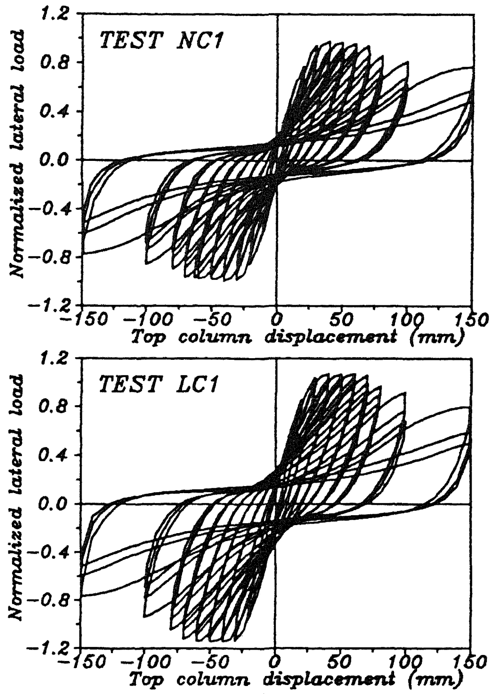

Figure 4. Lateral force-top column displacement diagrams for specimens NC1 and LC1.

4 CONCLUSIONS

Ten beam-column subassemblage specimens were tested. Five of them were in normal concrete and five in lightweight concrete. The parameters of concern were: the amount of joint transverse reinforcement, the anchorage details of the beam reinforcement and the diameter of the beam longitudinal reinforcement. It was found that:

1- lightweight concrete specimens exhibit essentially the same behavior as the normal concrete ones.

2- low transverse reinforcement ratios in the joint result in poor overall behavior if the main reiforcement is anchored inside the joint core; they seem sufficient if the longitudinal reinforcement passes through the joint. This suggests that the rules given in EC8 code could be over conservative.

3- the specimens with the beam reinforcement anchored inside the joint showed a good overall response, but the joint was extremely damaged while the plastic hinge in the beams was shortened.

4- the specimens with lower bar diameter-core depth ratio showed a better initial behavior, because slippage is delayed.

Extensive results are presented elsewhere (Monti and Nuti 1992) also in terms of beams' moment-rotation and moment-crack width response, along with an evaluation of the performances of the specimens in terms of energy-dissipation capacity, stiffness and strength degradation.

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