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EXPERIMENTAL STUDY OF THE EFFECT OF REINFORCEMENT STABILITY ON THE CAPACITY OF REINFORCED CONCRETE COLUMNS

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SUMMARY

The paper presents the results of an experimental program focusing on the effect of buckling of reinforcing bars on the post-peak behavior of reinforced concrete columns in seismic regions. Full-scale, square cross-section column specimens were tested with stirrup spacing (4-, 6-, 8- and 12-bar diameters) being the main variable. All specimens were subjected to displacement cycles of increasing amplitude with constant axial force. Test results show that specimens behave satisfactorily to the imposed displacement history up to the occurrence of bar buckling. Column behaviour after bar buckling is conditioned by stirrup spacing and peak tensile strain of the bars at the instant of reinforcement buckling.

INTRODUCTION

Considerable effort has been devoted in the past in investigating the effects of bar instability in reinforced concrete columns. The majority of past studies focused on bar instability through testing individual bars under compression (i.e. Rodriguez et al [1], Monti et al [2]) and mainly focusing on calibrating material models for reinforcement buckling. Many researchers have, however, recognized that the mechanics of buckling in concrete is quite different than in individual bars tested under compression and the problem should be studied in column specimens under cyclic loading (i.e. Moyer [3], Pantazopoulou [4]). There exist a small number of tests on column members under concentric compression (i.e. Moyer et al [3], Ooya et al [5], Dhakal et al [6]) and under cyclic loading (i.e, Dhakal et al [6], Gomez et al [7], Honda et al [8], Attolico et al [9], Suda et al [10]) usually involving small number of specimens.

The results of an experimental program on the response of RC columns with varying transverse reinforcement spacing are presented herein. The program focuses on the post-buckling behaviour of reinforced concrete columns and investigates the whether code-specified dense stirrup spacing results in higher strain at steel bars after buckling leading reinforcing bars to fracture on the subsequent deformation cycle when they straighten-up.

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EXPERIMENTAL PROGRAM

The experimental campaign included the testing of four column specimens dimensioned according to the provisions of Eurocode 8. The specimens simulated part of a real column from the point of inflection (assumed at mid-height) to the point of maximum moment (column base or top). A 250mm-square cross-section was selected for the column to obtain a flexure-dominated specimen (high shear-span-ratio L/h=6.4) while keeping also axial load needs within the capacity of the host laboratory. The specimen was founded on a heavy foundation base, used to anchor the specimen on the laboratory strong floor, and reinforced longitudinally with four 16-mm ribbed bars (S500). Transverse reinforcement consisted of properly anchored (135-degree hooks) 10mm-diameter stirrups (S500) placed at distances varying among the columns tested: considering the requirements of Eurocode 8 for the DCH class (s_{max}=*min*[$8d_{bL}$, 175mm, $b_o/2$]), stirrup spacing ranged between 4- and 12-bar diameters. Thus, transverse reinforcement was placed at $4D_{bL}$(specimen C60), $6D_{bL}$(specimen C100), $8D_{bL}$ (specimen C130) and $12D_{bL}$ (specimen C190), i.e 64mm, 100mm, 128mm and 1902mm, respectively. The 16mm-diameter vertical bars had a yield stress of were 514MPa and a tensile strength 659MPa, while the corresponding values for the stirrups were 542MPa and 657MPa, respectively. The strain hardening of the 8-mm bar used was estimated from tensile tests on coupons approximately equal 15%. Concrete compressive strength (measured on 150-by-300mm cylinders) at the time of testing was in all specimens equal to 24.7 MPa.

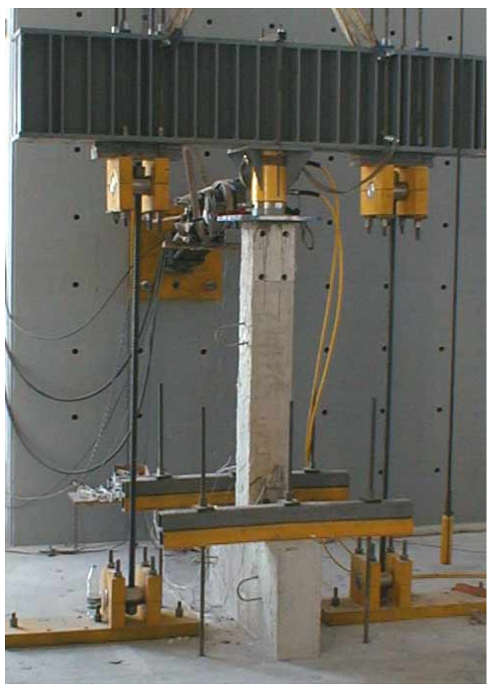


Figure 2 Test setup

The behavior of the columns was studied under cycling of horizontal displacements along the E-W direction and with the amplitude increasing in 5mm steps, at the presence of constant axial force. The load history with closely spaced single cycles was chosen over the usual protocols of 3 cycles at few displacement ductility levels, to study better the cyclic behavior of the specimen up to failure. An axial load of approximately 500kN was applied through a jack at the top, along the member longitudinal axis, corresponding to a normalized mean axial load ratio, $\nu=N/A_cf_c$, during the test equal to 0.33. The jack acted against vertical rods connected to the laboratory strong floor through a hinge (Figure 2). With this setup the P-Δ moment at the base of the column, which is added to that due to the horizontal force, is equal to the axial load, times the tip deflection of the column, times the hinge distance ratio from the base and the top of the column (i.e. times 0.5/1.6=0.3125).

Strains at several positions in the lower region of the specimens were monitored via strain gage measurements. Eight, high elongation, strain gages were installed in each specimen, all on vertical bars and at mid-distance between the base of the column and the first stirrup and also between first and second stirrup from the base. Additionally, each column was instrumented with two pairs of displacement transducers to monitor the deformation of the column on the two section faces normal to the direction of loading. From the recorded measurements, the mean deformation over the zone covered by each displacement transducer and member rotation at two different levels with respect to the foundation base can be obtained.

TEST RESULTS

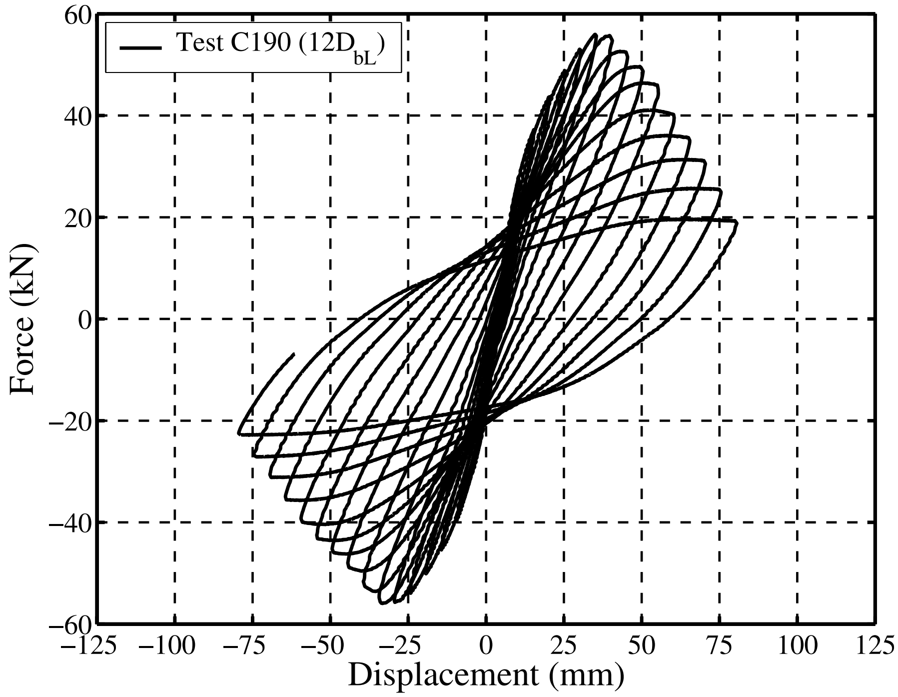
Results in the form of force-displacement loops for the specimen with the largest distance between stirrups i.e. C190 ($12D_{bL}$) are presented in Figure 4(a). The behavior of the specimen during testing was in flexure, reaching displacement ductility approximately equal to 2.5. The concrete cover and a large part of the core concrete at the lower 200mm of the column disintegrated and all bars buckled between the base of the column and the first stirrup above it. Loss of concrete cover due to spalling initiated at approximately 1.8% drift while maximum resistance was reached at 2.2% drift. Member conventional failure - defined as the deformation level corresponding to not more than 80% of maximum resistance - occurred at 3.4% (Table 1). In the absence of considerable lateral confinement from transverse reinforcement (large stirrup spacing) longitudinal bar buckling preceded ‘failure’ (drift 5%). The direction of buckling was towards the external surface of the column in an inclined plane with respect to the loading axis and rather towards the diagonal of the section. None of the bars fractured during the test.

Table 1 Characteristics of specimens

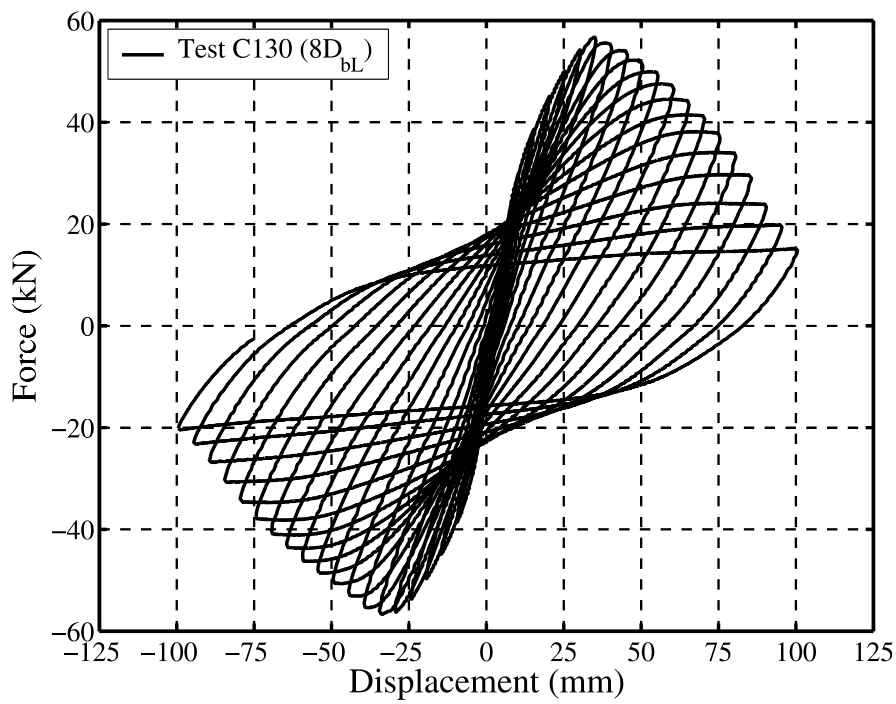
Specimen	Concrete strength (MPa)	Stirrup spacing	Axial load ratio, $v=N/A_cf_c$	Drift at “failure”* (%)	Maximum drift during test (%)
C190	24.7	$12D_{bL}$- 190mm	0.33	3.4	5.0
C130	24.7	$8D_{bL}$- 130mm	0.33	4.0	6.2
C100	24.7	$6D_{bL}$- 100mm	0.33	4.4	6.6
C60	24.7	$4D_{bL}$- 60mm	0.33	5.6	6.9

Similar behavior was obtained in the specimen with $8D_{bL}$=130mm stirrup spacing (C130, Figure 4(b)). The specimen responds with stable hysteresis loops attaining the same maximum resistance as specimen C190 and at similar drift, 2.2%. After reaching peak force, stable strength degradation is observed (albeit at a lower rate than in column C190), while instability of the first bar was visually observed to occur at 4% drift. A slight change of the rate of strength degradation is observed after buckling. Neither column damage not bar instability evolved symmetrically on both specimen sides (Figure 5(b): on the E-side concrete core disintegration and subsequent buckling of both corner bars occurred between the first and the second stirrup from column base. On the opposite one (W-side) concrete cover and part of the core was damaged between first and second stirrups while one of the corner bars buckled within the first 190mm from the base and the other between 190mm and 380mm (i.e. between first and second stirrups). No stirrup opening or vertical bar fracture was observed. The conventionally defined “failure” was estimated at 4% drift, but member sustained further loading up to 6.2%.

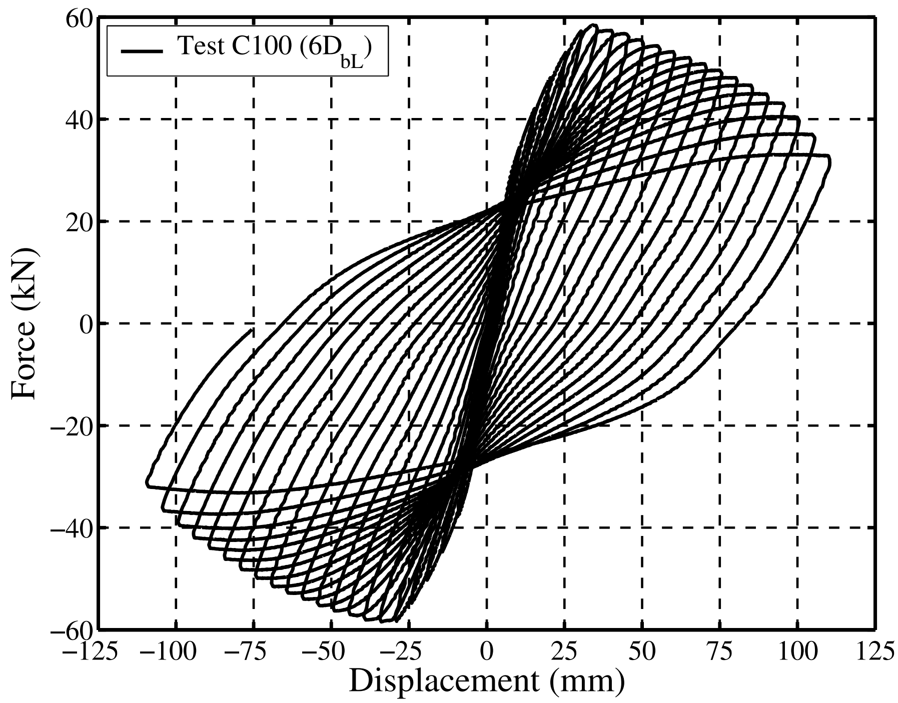
Figure 4(c) depicts the force deformation loops of specimen, C100 in which transverse reinforcement was placed according to the code specified spacing, i.e. $6D_{bL}$ or 100mm. Column exhibits the same flexural behavior and resistance as the previous ones and with slightly smaller rate of strength degradation than specimen C130. Concrete cover spalled off at the lower part of the column corresponding to approximately 100mm to 200mm. Further loading increased concrete spalling and damage penetrated in the core of the section as well, leading to bar buckling observed at 4.4% drift. Reinforcement instability occurred in all corner bars and between the first and the second stirrups from the base (Figure 5(c)).



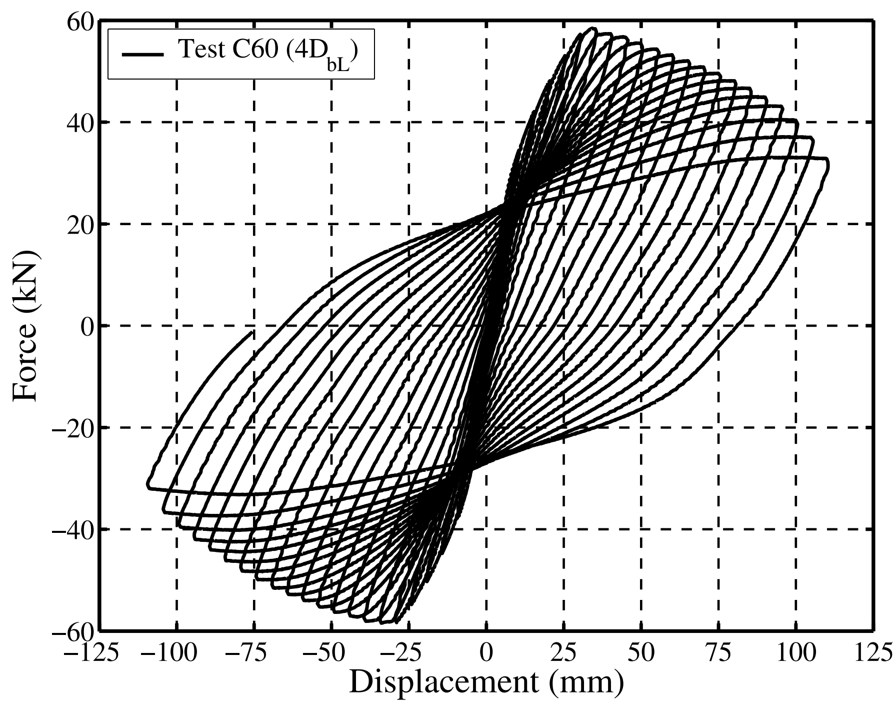
(a)



(b)

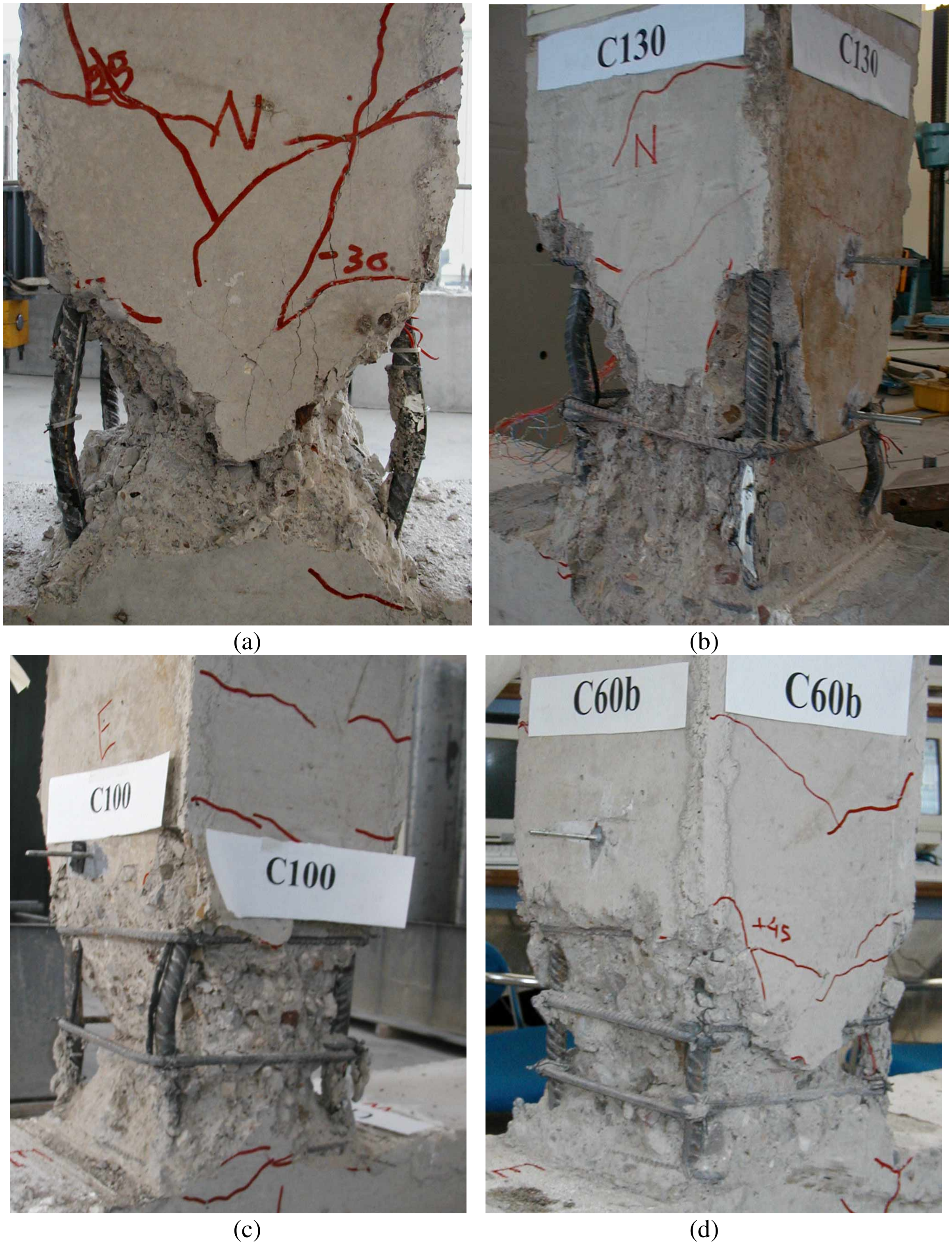


(c)



(d)

Figure 4 Force-deformation loops of tested columns: (a) Specimen C190 ($12D_{bL}$), (b) Specimen C130 ($8D_{bL}$), (c) Specimen C100 ($6D_{bL}$), (d) Specimen C60 (D_{bL})

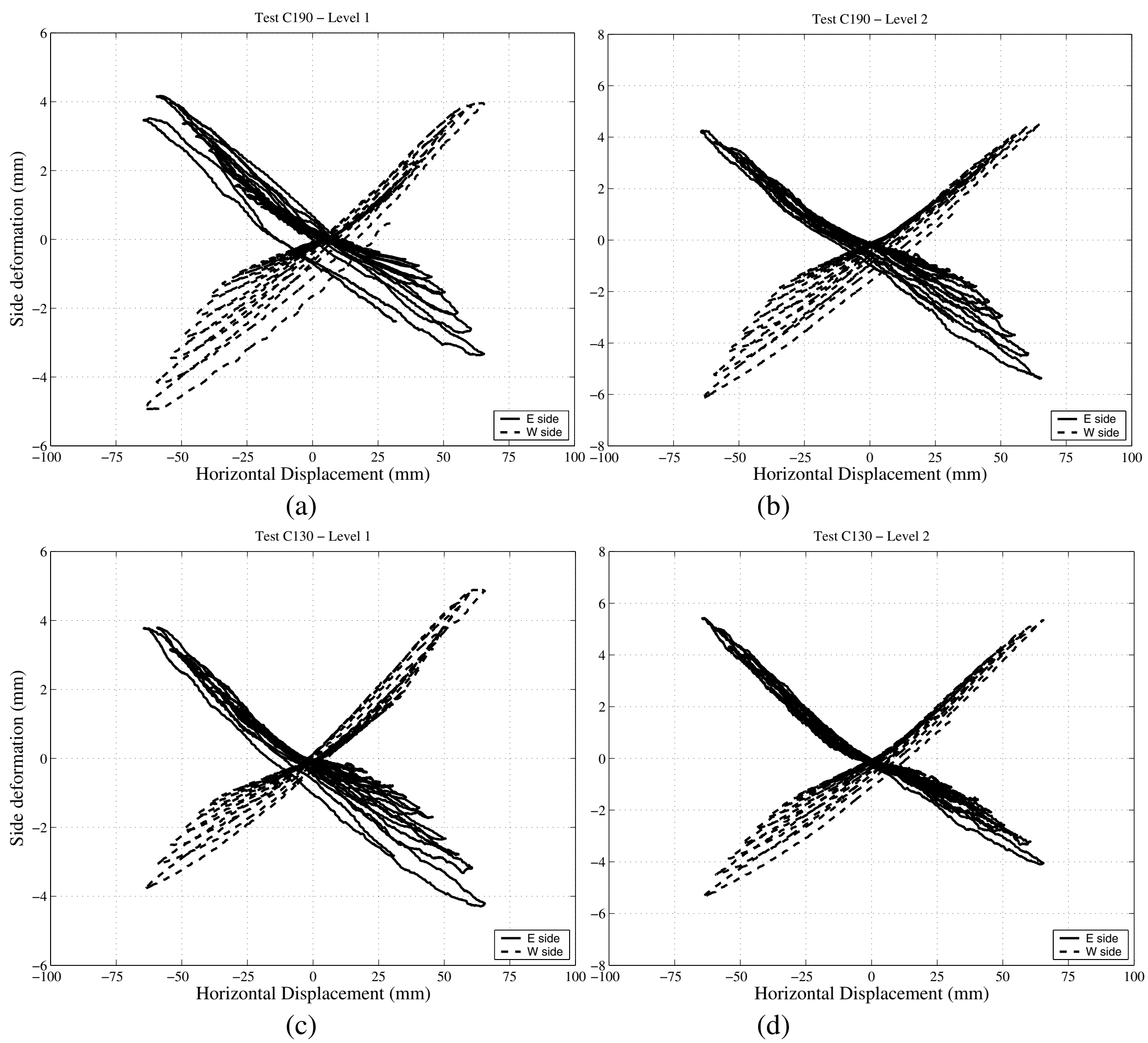


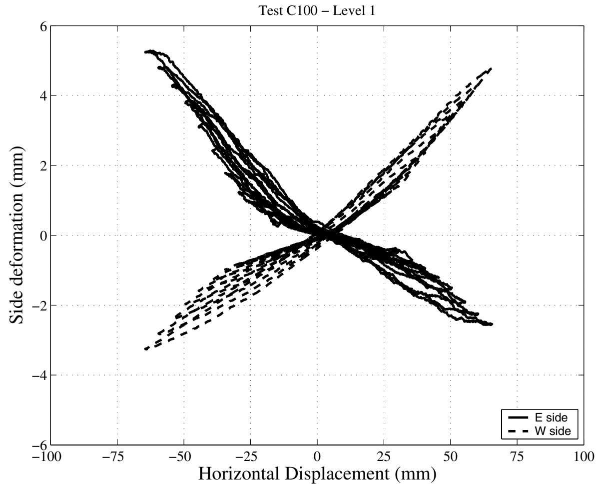
(a) (b) (c) (d)

Figure 5 Buckling of reinforcement and specimen failure: (a) Specimen C190 ($12D_{bL}$), (b) Specimen C130 ($8D_{bL}$), (c) Specimen C100 ($6D_{bL}$), (d) Specimen C60 ($4D_{bL}$)

Figure 4(d) presents the force-deformation loops obtained from specimen C60, i.e. the one with stirrup spacing approximately equal to $4D_{bL}$ or 60mm. Member resistance and overall performance resembled that of previous specimens except that after spalling of concrete cover over the lower 200mm of the specimen, damage continued inside the core of the section on only one of the sides (E-side). Both vertical bars of that side and within the zone between the first and second stirrup buckled at approximately 5.6% drift. Instability of the vertical bars led to a change (increase) in the rate of strength degradation from that point onwards.

The pairs of displacement transducers monitoring the deformation of the column on the two faces of the column normal to the direction of loading were located at 125mm (Level 1) and 250mm (Level 1) from the column base. In Figure 6, deformations measured on both column sides and at both levels are shown.





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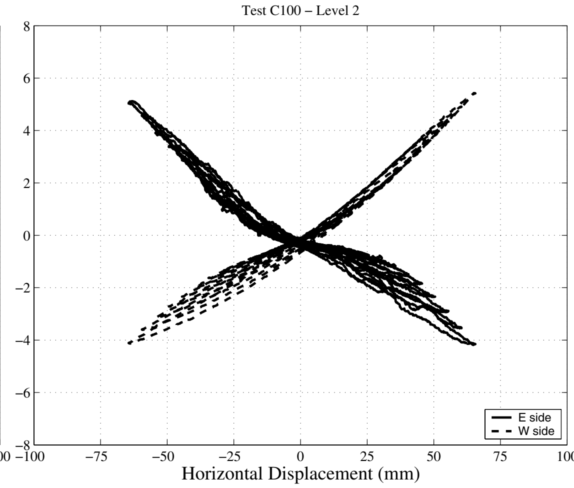

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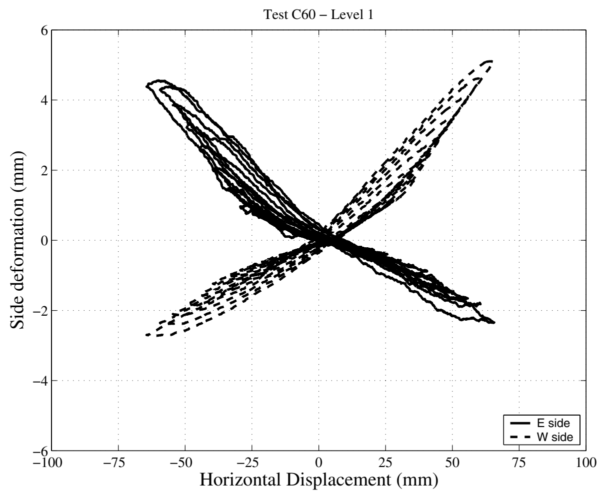
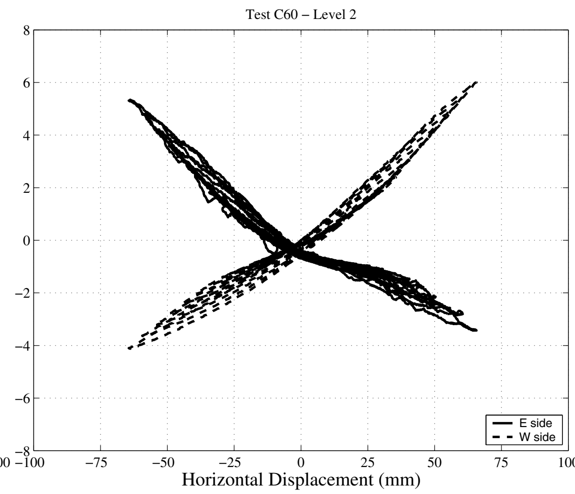


(g)



(h)

Figure 6 Deformations on column side of specimens: (a) C190, (b) C130, (c) C100 and (d) C60

DISCUSSION

Instability of longitudinal reinforcement is considered an important factor for the behavior of RC members which are designed and expected to undergo large deformations in the non-linear range of their response. After concrete crushing in the compression zone reinforcing bars constitute the only source of stability, assuming that cracks are still open. The further the member enters into the inelastic range of response the higher the accumulated strains and bar instability becomes highly probable, especially with the aid of significant axial force. Subsequently, the buckled bar is strained by the mean strain due to buckling superimposed to the additional deformation due to bending (tensile on one bar side and compressive on the other). If the bar buckles in the zone between two successive stirrups and stirrups are closely spaced, then the length over which buckling develops is short with respect to the bar diameter, then the total strain on the tension side of the bar may even exceed the ultimate strain and the bar fractures in the subsequent cycle of column lateral displacement. As in the column tested no bar fracture was observed, this hypothesis was not verified.

Tests on buckling of individual bars (Monti et al [2]) indicated that instability of reinforcing bars develops for values of the ratio of stirrup spacing divided by bar diameter larger than 5. A common requirement for stirrup spacing (derived for reinforcing bars with high ultimate-stress-to- yield-stress ratio) is $6D_{bL}$ (Priestley et al [12]). As Tempcore steel yields low values of the f_u/f_y ratio, a more conservative value was proposed:

$$s \leq \left[3 + 6\left(\frac{f_u}{f_y} - 1\right)\right] D_{bL} \tag{1}$$

which, for f_u/f_y=1.15 (as in this study), yields s/D_{bL}=3.9 for precluding buckling.

In assessing member deformation capacity the model developed by Moyer and Kowalsky [3] was employed. The model is based on the fact that bar buckling and peak tensile strain are directly related and was calibrated from a series of tests on circular columns with $4D_{bL}$ stirrup spacing. Under the assumption that the residual bar strain at neutral column position is half the peak tension strain and parametric studies conducted by section analysis, an equation for predicting the curvature ductility factor at the onset of buckling was proposed:

$$\mu_{\Phi} = 2\left(\frac{K \cdot s}{\Phi_L}\right)^{-2.5} Z \text{ with } Z = ([260 + 325 v_d] + [20 - 25 v_d] * [\rho_L - 0.5]) \tag{2}$$

where s is the stirrup spacing, v_d the normalized axial load, ρ_L the reinforcement ratio, and K a parameter defining whether buckling develops between two stirrups or employs a larger number of stirrups. The latter parameter was equal to 1, as all bars in the tests described above buckled between two stirrups. The equation above is valid for $\mu_{\Phi}>4$, $0.5\%<\rho_L<4\%$, $0<v_d<0.4$, all of which are fulfilled by the tests presented herein. The predictions of the model by Moyer and Kowalsky [3] are compared in Table 1 with results derived from the experimental measurements and with the predictions of the equation for the curvature ductility factor proposed by Pauley and Priestley [11]:

$$\mu_{\phi} = 1 + \frac{\mu_{\Delta} - 1}{3(l_p / l)[1 - 0.5(l_p / l)]} \text{ with } l_p = 0.08 l + 0.022 \Phi_L f_y \tag{3}$$

The comparison (Table 1) shows that the model proposed by Moyer et al approximates the experimentally obtained values of curvature ductility factor, except for the case of the dense stirrup spacing.

Table 1 Comparison of curvature ductility factors

Specimen	Curvature ductility derived from (Eq. 3)	Curvature ductility from tests	Curvature ductility derived from (Eq 2)
C190	3.9	3.5	< 4
C130	4.9	5.0	4.1
C100	6.3	9.5	7.8
C60	8.0	20	28.2

CONCLUSIONS

The experimental results of tests on reinforced concrete columns are presented. Test results show that specimens behave satisfactorily to the imposed displacement history up to the occurrence of bar buckling. The behaviour after bar buckling is conditioned by transverse reinforcement spacing. Despite that in the specimen with the dense stirrup spacing buckling appeared, no bar rupture occurred. The expression proposed by Moyer et al. for determining the curvature ductility factor at which buckling is expected to occur seems promising but more tests are required for further calibration.

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