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SQUAT R/C WALLS UNDER INELASTIC SHEAR REVERSALS

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ABSTRACT

The present study addresses the problem of cyclic shear in squat reinforced concrete walls and attempts to assess the validity of current design provisions, in particular those of Eurocode 8. The paper describes the design of a comprehensive experimental programme involving walls with shear ratios of 1.0 and 1.5, detailed to the provisions of EC8; problems in applying these provisions are pointed out. The wall specimens are reinforced against shear either conventionally (orthogonal grids of web reinforcement), or with cross-inclined bars; moreover the effects of using welded bars as web reinforcement, and of the quality of construction joints are investigated. Results from tests on three specimens with shear ratios of 1.5 are discussed. All specimens failed in a predominantly flexural mode, while diagonal shear cracks in the web appeared, together with some sliding at the base. The best performance was observed in the specimen with bidiagonal reinforcement.

KEYWORDS

Reinforced concrete walls; cyclic testing; bidiagonal reinforcement; sliding shear; low slenderness; shear ratio.

INTRODUCTION

The investigation reported herein addresses the problem of cyclic shear in reinforced concrete (R/C) walls with low shear ratios ($a_s = M / Vl_w < 1.5$). The prevailing shear failure modes in such members are either *diagonal tension* or *sliding shear* (Penelis & Kappos, 1996). The new Eurocode 8 (CEN TC 250, 1995) for the Seismic Design of Structures which appeared recently as an ENV (prestandard) for a trial application of three years, is the first document of regulatory character that makes a clear distinction between the different modes of shear failure in R/C walls subjected to seismic loading. The EC8 provisions are largely based on the experimental work carried out by various investigators regarding the strength and deformability of walls subjected to seismic loading. However a review of the available literature showed that these studies do not provide conclusive information with respect to the cyclic shear behaviour of squat (low slenderness) walls, which are common in low-rise construction, particularly in the case that a sliding shear mode dominates. More specifically, only in the work by Paulay, Priestley and Synge (1982) has the effect of bidiagonal reinforcement on delaying sliding shear failure been investigated.

The main purpose of the present study is to assess the validity of current design provisions for cyclic shear in squat R/C walls, in particular those of EC8. The paper describes the design of a comprehensive experimental programme involving walls with aspect ratios of 1.0 and 1.5, detailed to the provisions of EC8; problems in applying these provisions are pointed out. The wall specimens are reinforced against shear either conventionally (orthogonal grids of web reinforcement), or with cross-inclined bidiagonal bars. Results from tests on three specimens with shear ratios of 1.5 are discussed and preliminary conclusions are drawn.

DESIGN OF WALL SPECIMENS

The experimental programme was designed in such a way as to ensure that either a diagonal tension (combined with flexural cracking and spalling) or a sliding shear mode were expected. Two aspect ratios were selected, namely $a_s = 1.0$ and $a_s = 1.5$; these ratios are below and above the limit of 1.3 specified by Eurocode 8 for the applicability of the truss model for shear design. The programme includes a total of 13 specimens, 5 specimens with a_s=1.0 and 8 specimens with a_s=1.5, as some additional parameters (welded bars as web reinforcement, quality of construction joints) are considered in the latter case. All specimens are of the cantilever type, hence the aforementioned aspect ratios coincide with the shear ratio $a_s = M/(V.l_w)$.

Since the number of parameters involved is larger than the available number of equations, an appropriate design strategy had to be devised to ensure a realistic design, respecting the EC8 minimum reinforcement requirements. The dimensions of the specimens were selected taking into account the capacity of the reaction frame (Fig. 2); this resulted in a 100 by 1200 mm wall section, corresponding to a scale of about 1:2.5. In order to achieve the required aspect ratios, specimen heights of 1200 and 1800 mm were selected for a_s equal to 1.0 and 1.5, respectively. The percentage of longitudinal reinforcement in the boundary elements was kept close to the minimum (1%) required by the code.

The intermediate ductility class (DC "M") which is generally adopted in the new national codes of earthquake-prone countries in Europe and elsewhere was used in the present study. The full set of EC8 provisions for shear have been applied to the wall specimens and it was found that the lower limit set to the equation referring to the frictional (aggregate interlock) component of the resistance to sliding shear

$$V_{fd} = \mu_f [(\Sigma A_{sj} f_{yd} + N_{Sd})\, \xi + M_{Sd}/z] \leq 0.25 f_{cd}\, \xi l_w\, b_w \qquad (1)$$

(where ΣA_{sj} the area of vertical bars in the web plus that of any additional specially provided bars in the edge members) is systematically lower (up to 75%) than the values resulting from the basic equation, which implies that either the limit is over-conservative or the equation unconservative (see also Penelis & Kappos, 1996). One of the main objectives of the experimental programme was to clarify this point.

In addition to the design against shear the remaining provisions of EC8 were applied, including those for the confinement of the wall boundary elements; the only code requirement that was not satisfied in some of the specimens was the one relating the spacing of hoops in the boundary elements to the width b_o of the confined core, namely $s \leq b_o/3$, which results at a spacing of only 27 mm, hence creating construction problems.

TEST PARAMETERS AND EXPERIMENTAL SET-UP

Test parameters and specimen selection

The main parameters examined in the experimental programme are the following:

- Effect of *aspect (shear) ratio* (a_s = 1.0 and 1.5) on the failure mode.
- Effect of *bidiagonal bars* on the sliding shear behaviour, considering both the quantity and the distance (l_d) of the intersection point of the bars from the wall base.
- Effect of *horizontal (grid) reinforcement ratio* on the shear behaviour.
- Effect of *axial loading* on the shear response ($N = 0$ and $N = 0.07 f_c A_c$, where A_c is the gross area of the section).
- Effect of using *welded grids* of bars as web shear reinforcement.
- Effect of *construction joints* (with and without roughening) on sliding shear behaviour.

A complete list of the selected specimens is shown in Table 1, where the relevant test parameters are indicated and defined in a legend. Also given as an insert to the table are characteristic cross-sections at the base of specimens with inclined bars intersecting close to the base or at a distance equal to $l_w/2$, and of a specimen without inclined bars. It is pointed out that specimens LSW1, MSW1 have a horizontal (grid) reinforcement ratio equal to the sum of the ratios of horizontal and bidiagonal reinforcement in specimens LSW4, LSW5, MSW4 and MSW5, in order to study the effect of increasing shear reinforcement in the conventional way or through cross-inclined bars. The three additional specimens with a_s=1.5 (MSW6 to MSW8) are used for studying the effect of welded grid reinforcement, and of the quality of construction joint (good - poor). Details of the specimen reinforcement are shown in Fig. 1 for both types of specimens used. Measured yield strengths of the 8 mm bars (Grade 500) used in the edge regions were equal to 630 MPa, and concrete strengths varied from 24 to 26 MPa at the time of testing.

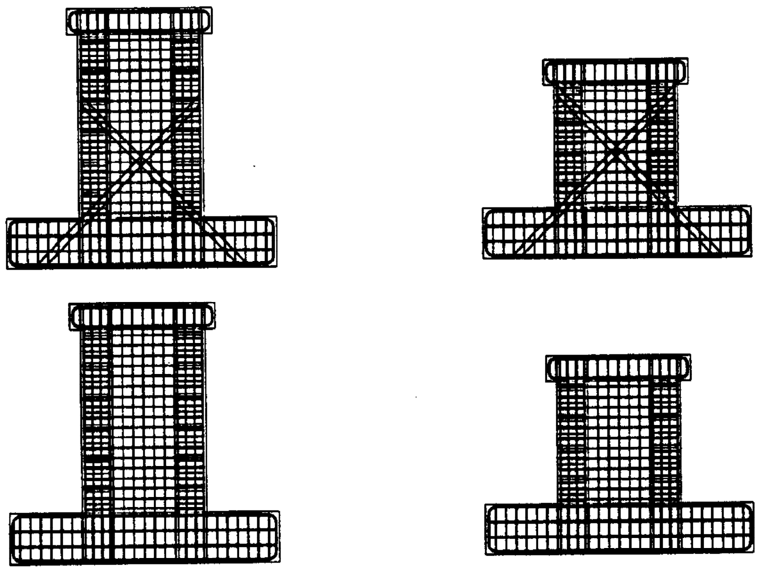


Fig. 1 Reinforcement layout in wall specimens with aspect ratios of 1.0 and 1.5

Experimental set-up

The reaction frame used in the present study is shown in Fig. 2; this steel frame was designed by the authors and constructed under their supervision. The four columns of the frame have a IPB 600 section with additional stiffening plates at their bases. The columns are fixed to the 1.0 m thick prestressed floor of the Laboratory of Concrete Structures, Aristotle University of Thessaloniki. Resting on two of the columns is a horizontal beam to which one of the hydraulic actuators is attached; this beam can be displaced vertically at intervals of 150 mm, thus allowing the application of loading at various heights. The axial loading is applied by a vertical actuator attached to a beam system as shown in Fig. 2. The horizontal actuator has a capacity of +1000 and -600 (tension) kN, and the vertical one a capacity of +250 or -250 kN. The data acquisition system consists of six internal control and recording channels (three for each actuator) and a total of six channels for monitoring data from external instruments (LVDTs, strain gages).

The anchorage block of the specimen (see Fig. 1, 2) is fixed to the laboratory floor with prestressing bars having a total prestressing force of 700 kN, which prevent uplifting of the specimen during the application of the horizontal loading. Moreover, a blocking mechanism involving two tubes eccentrically fitted within each other is used for preventing the horizontal sliding of the specimen. Measured displacements due to uplifting and to sliding were found to be negligible (less than 0.08 mm).

The horizontal loading was applied at the top beam of the specimens (see Fig. 2) wherein two metal plates were attached, connected through a prestressing force of 300 kN to avoid possible elongation of the bars. The use of a stiff beam at the top of wall specimens has been questioned by some researchers (Lopes & Elnashai, 1990) who pointed out that it tends to overestimate the shear capacity of the walls; it is a fact however that in most practical situations beams are framing into the walls at the level of floor slabs, although these beams are usually less stiff than those used in test specimens.

Table 1 Details of test specimens

No	Specimen	Section	Sect. Area	Height	Span Rat.	Acc	ρ_h	ρ_v	ρ_{cc}	ρ_w	ρ_d	l_d
		$b_w \times l_w$		h_w	$a_s=h_w/l_w$	$l_c \times b_w$						
		cm × cm	cm^2	cm		cm^2	%	%	%	%	%	cm
1	LSW1	10 X 120	1200	120	1	240	0.565	0.277	1.3	1.1	_	_
2	LSW2	10 X 120	1200	120	1	240	0.277	0.277	1.3	1.1	N≠0	_
3	LSW3	10 X 120	1200	120	1	240	0.277	0.277	1.3	1.1	_	_
4	LSW4	10 X 120	1200	120	1	240	0.277	0.277	1.3	1.1	0.286	0
5	LSW5	10 X 120	1200	120	1	240	0.277	0.277	1.3	1.1	0.286	$l_w/2$
6	MSW1	10 X 120	1200	180	1.5	240	0.565	0.277	1.3	1.1	_	_
7	MSW2	10 X 120	1200	180	1.5	240	0.277	0.277	1.3	1.1	N≠0	_
8	MSW3	10 X 120	1200	180	1.5	240	0.277	0.277	1.3	1.1	_	_
9	MSW4	10 X 120	1200	180	1.5	240	0.277	0.277	1.3	1.1	0.286	0
10	MSW5	10 X 120	1200	180	1.5	240	0.277	0.277	1.3	1.1	0.286	$l_w/2$
11	MSW6	10 X 120	1200	180	1.5	240	0.277	0.277	1.3	1.1	_	_
12	MSW7	10 X 120	1200	180	1.5	240	0.277	0.277	1.3	1.1	_	_
13	MSW8	10 X 120	1200	180	1.5	240	0.277	0.277	1.3	1.1	_	_

$A_{cc}=l_c \times b_w=\max\{2 \times b_w, 0.2 \times l_w\} \times b_w$

$\rho_h=A_{sh}/(b_w \times l_w)$, $\rho_v=A_{sv}/(b_w \times l_w)$, $\rho_{cc}=A_{sc}/A_{cc}$,

$\rho_w=(l_{stir.} \times A_{s\,stir.})/(l_c \times b_w \times s)$, $\rho_d=A_{sd}/(b_w \times l_w)$

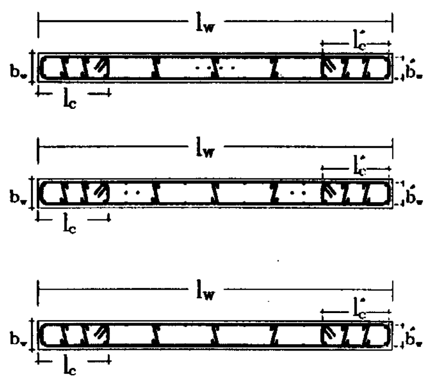
A_{sd}: Area of total diagonal reinforcement.

A_{sc}: Long. reinforcement in boundary element.

A_{sh}: Horizontal web reinforcement.

A_{sv}: Vertical web reinforcement.

Typical cross sections:



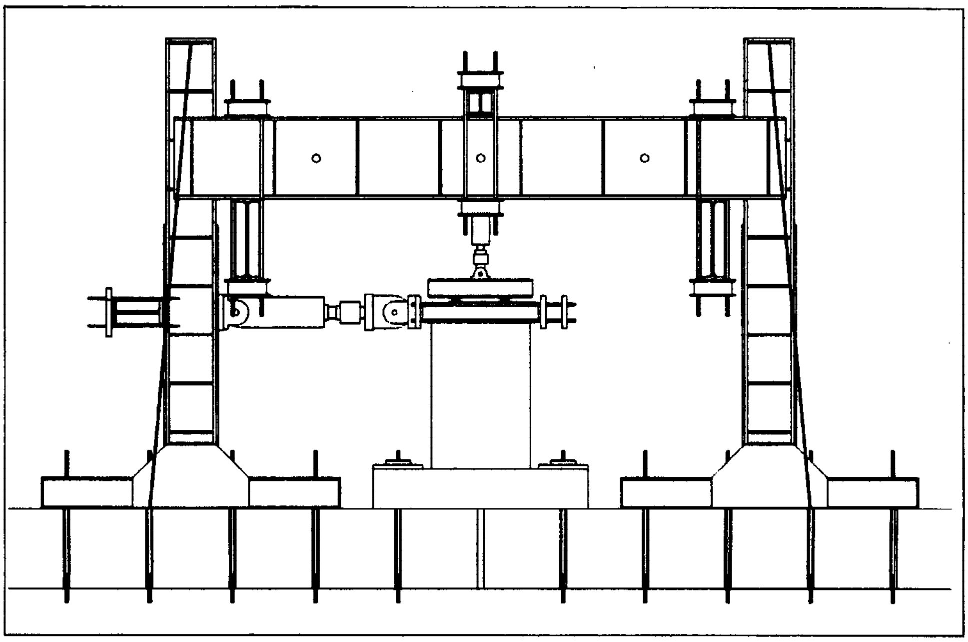


Fig. 2 Side view of reaction frame with specimen in position

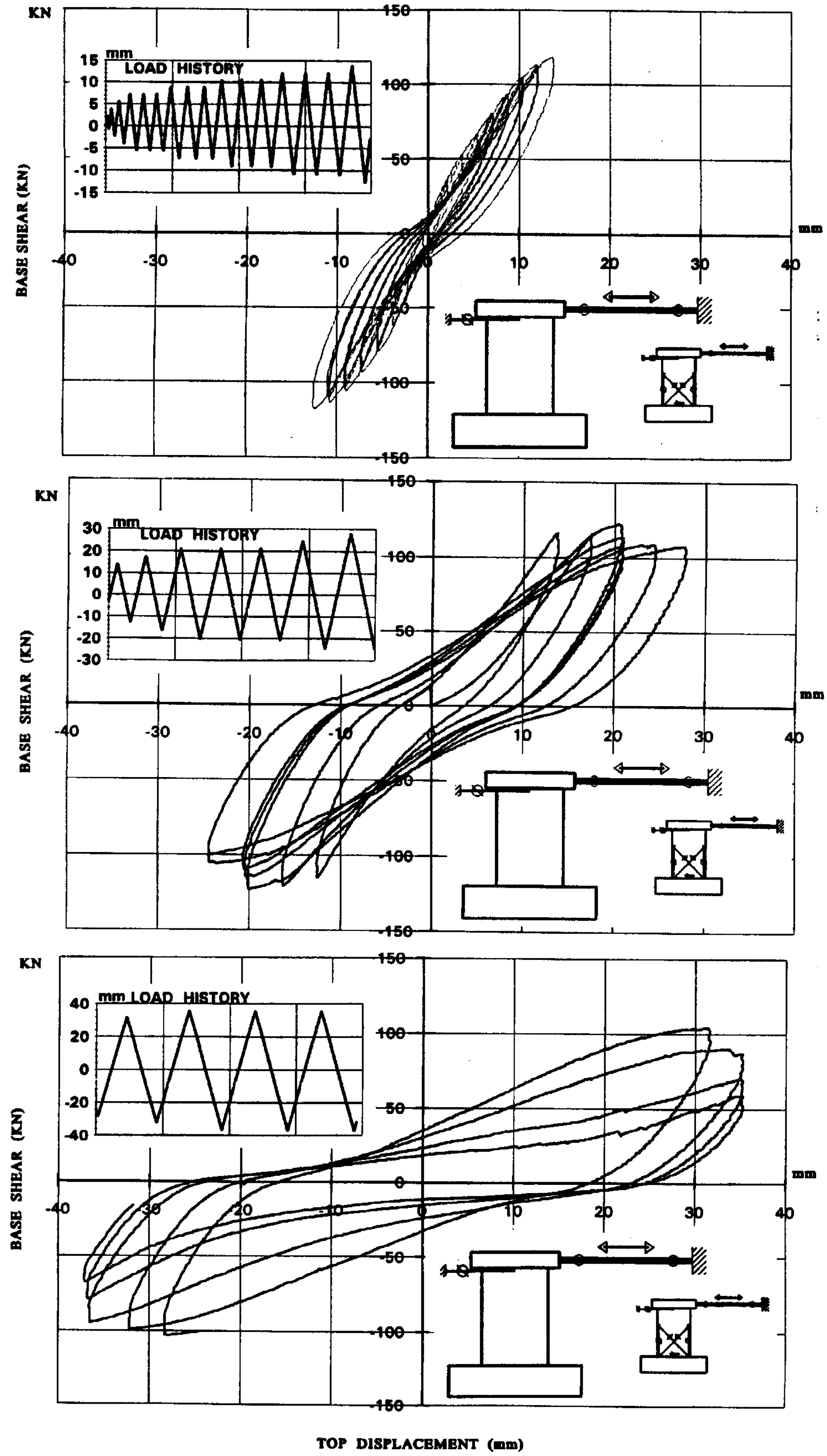


Fig. 3 Load-displacement curves for specimen MSW3 subjected to various levels of cyclic loading

specimen, and of prestressing the plates at the top of the specimen, a *finite element study* of the specimen under various loading conditions at the top was carried out; the 2-D shell elements incorporated in the *SAP 90* code were used for constructing the model; both static and dynamic analyses (using a spectrum which gave the same base shear as in static analysis) were carried out. The analysis of the two specimen types for various loading conditions has shown that although the stress distribution is quite different at the top of the specimen, depending on the position of the loading point, the stress pattern at the base is very similar in all cases, both dynamic and static; as expected, dynamic stresses are slightly lower than the static ones. It was thus concluded that so long as the critical region ("plastic hinge") is the one close to the base, the load may be applied either at the edge or at the middle of the specimen head; for reasons of practicality the former solution was preferred. Details of the FE study may be found in Salonikios et al. (1995).

As the effect of loading history was not selected as a test variable, the typical procedure of applying three loading cycles at each ductility level until failure, was used in the present study (see inserts in Fig. 3). Displacement control was used throughout the test, with the exception of the first cycle in the elastic range.

DISCUSSION OF RESULTS

Shown in Fig. 3 are base shear - top displacement hysteresis loops for specimen MSW3 (see Table 1), subjected to increasing levels of deformation, without axial load. The well-known characteristics of R/C members subjected to cyclic loading, such as decreasing unloading and reloading stiffness and pinching of hysteresis loops can be clearly seen in the figure. At a displacement of 35 mm (which is equal to 2% the wall height, and corresponds to a displacement ductility of about 3.5), the strength degradation is significant, in particular during the second and third loading cycle at this displacement; hence failure of the specimen is considered to occur at this stage, although some further cycling is possible (at reduced strength) due to the applied displacement control. While diagonal cracking of the web appeared, the *failure mode* of the specimen was primarily *flexural*, involving spalling of cover concrete, followed by buckling of bars and crushing of concrete in the boundary members of the wall, as shown in Fig. 4; the asymmetric distribution of damage in the symmetrically reinforced specimen is worth pointing out. It appears that if the EC8 limitation on hoop spacing in the boundary members had been followed, a higher ductility might have been achieved.

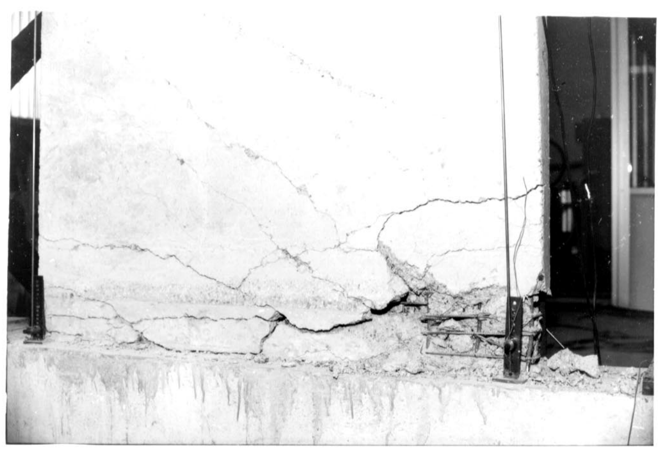


Fig. 4 Concrete crushing and reinforcement buckling at the edges of the MSW3 wall

Measured sliding displacements at the fixed end of specimen MSW3 increased with the level of inelasticity, reaching a maximum of 13% the corresponding top displacement. Shear deformations exhibited the well-known strongly pinched shape indicating sliding along the shear cracks; a characteristic load-shear deformation diagram in the advanced inelastic range is shown in Fig. 5 (deformation was measured along the inclined LVDT indicated in the insert to the figure).

Specimen MSW2 was similar to the previous one, but a constant axial load of 200 kN ($=0.07A_c f_c$) was applied at its top during cycling. As shown in Fig. 6, the strength of the wall increased (as expected) due to the presence of the compressive axial load, but the ductility was inferior to that of MSW3. Significant

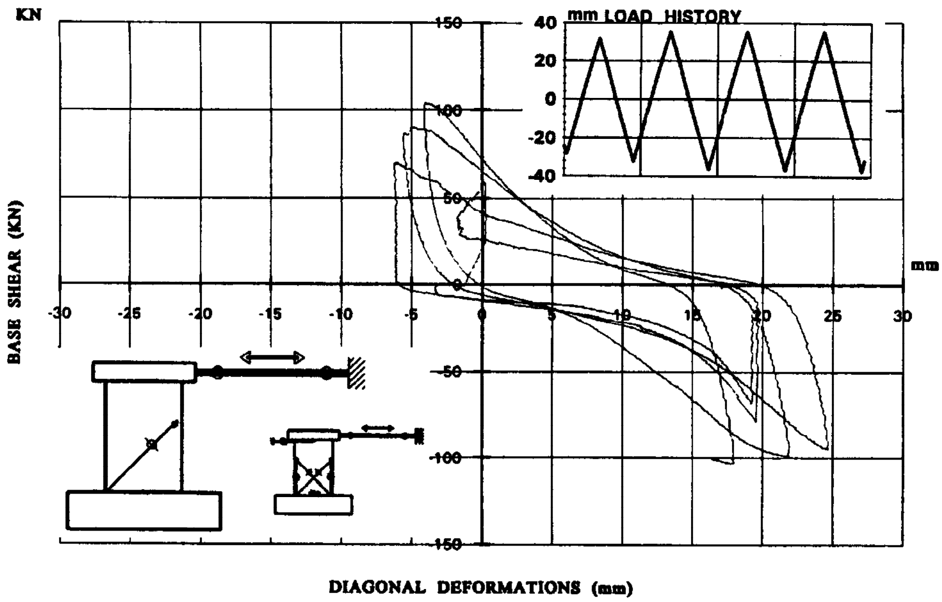


Fig. 5 Load - diagonal deformation loops for specimen MSW3 at the cycles close to failure

strength degradation occurred at a displacement of 25 mm (1.4% the specimen height), following extensive concrete crushing and reinforcement buckling at the edges; further cycling led to eventual fracture of some buckled bars. Hence, the behaviour of this axially loaded wall was dominated by flexure, and not by the diagonal shear cracks that developed in the web. Again using the EC8 recommended spacing (27 mm) of hoops in the boundary members (instead of the 42 mm provided) would have improved the ductility of the wall. Sliding at the fixed base was found to be smaller than in the case of the MSW3 specimen.

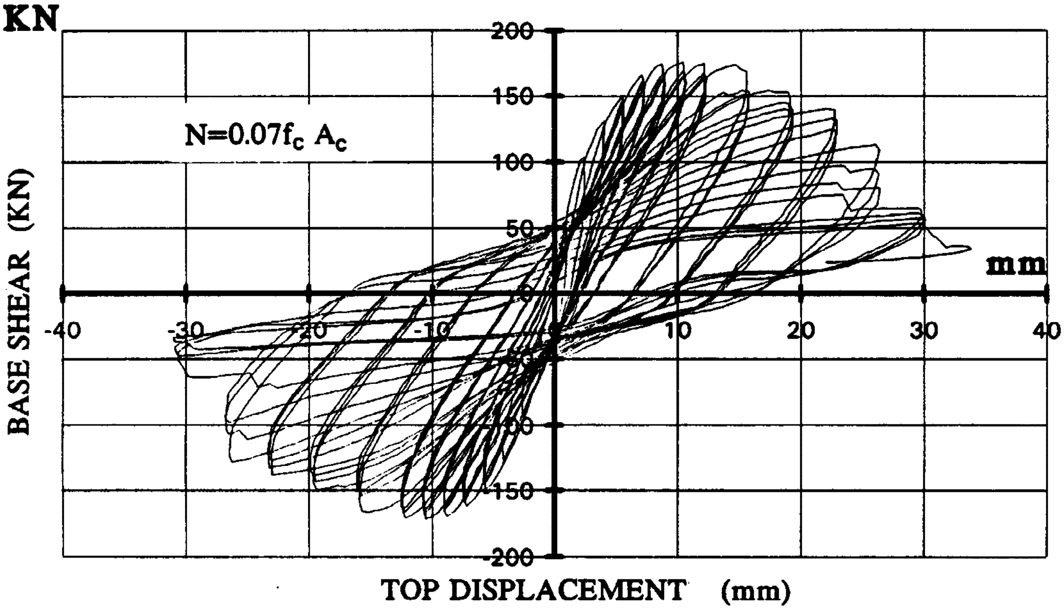


Fig. 6 Load-displacement curves for specimen MSW2

Shown in Fig. 7 are the hysteresis loops recorded for specimen MSW5, which was similar to MSW2, but in addition it contained the bidiagonal reinforcement indicated in Fig. 1 and Table 1. Comparison with Fig. 6 shows that the strength of specimen MSW5 was roughly the same as that of MSW3 which was subjected to axial loading; this was expected since the inclined bars are positioned close to the member edges in the region near the base and contribute to the flexural strength of the wall (as also recognised by EC8). In addition to the strength increase, the bidiagonal reinforcement contributed to reducing pinching of the hysteresis loops, thus resulting in increased energy dissipation. Moreover the ductility of the specimen was superior to that of MSW2, the significant strength degradation occurring at a displacement of about 30 mm (1.7% the specimen height). Diagonal tension cracks appeared in both directions, however the failure mode was a flexural one, similar to that of specimen MSW3. It has to be pointed out here that the increased strength due to diagonal bars intersecting away from the wall base may also have unfavourable effects, since it increases the shear stress in the member; this was not the case with specimen MSW5, where the maximum shear stress was of the order of $0.25\sqrt{f_c}$, well below the limits defining shear-dominated behaviour (Penelis and Kappos, 1996).

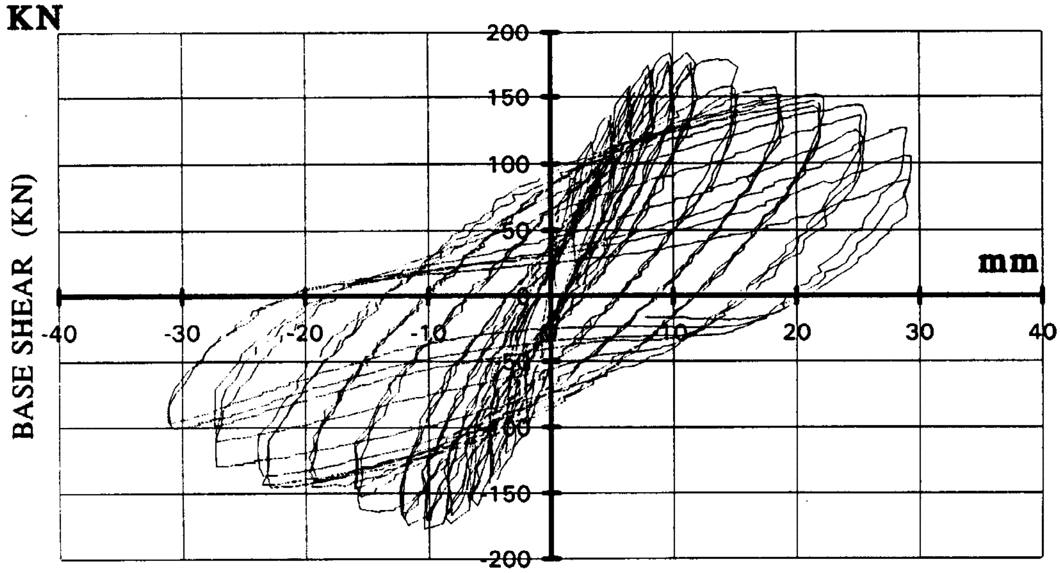


Fig. 7 Load-displacement curves for specimen MSW5

CONCLUSIONS

Currently available experimental data concerning the behaviour of squat R/C walls subjected to high cyclic shear are rather inconclusive, especially with respect to the sliding shear failure mode and the role of inclined bars in preventing it. The experimental programme presented herein aimed at clarifying these points and at shedding new light on the understanding of cyclic shear behaviour of low slenderness walls, as well as to contribute to the calibration of the EC8 provisions for shear design. Preliminary conclusions drawn so far refer only to walls with aspect ratio 1.5 (the EC8 limit for "squat" walls is 2.0).

All specimens tested to date failed in a predominantly flexural mode, characterised by concrete crushing and reinforcement buckling at the confined edges. Moderate diagonal cracking of the web and sliding at the fixed base were also observed, but did not significantly affect the failure mode. From the limited data discussed herein it appears that walls with a_s=1.5, although classified as squat in EC8, may be flexure dominated (at least for the low edge reinforcement ratios typical in low-rise walls), and the critical design parameter is not shear reinforcement, but confinement of the boundary regions. This does not appear to be the case with walls with a_s=1.0, whose testing is currently being completed and results will be reported at the conference.

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