



SG-7

## DUCTILITY EVALUATION OF CONCRETE WALL PANELS

Michele CALVI and Giorgio MACCHI

Dipartimento di Meccanica Strutturale, Universita' di Pavia, Italy

### SUMMARY

This paper presents the first results of an experimental and analytical research which has the objective of characterizing the cyclic behavior of concrete shear walls. The effects of percentage and quality of reinforcement, the possible failure mechanisms, the ductility that can be expected and the consequent evaluation of the behavior factors to be used in static equivalent analyses are discussed. The experimental results are presently limited to cyclic diagonal compression tests.

### INTRODUCTION

The results presented in this paper have to be regarded as the first part of a research project whose objective is to characterize the seismic answer of concrete shear wall buildings as a function of concrete strength, steel grade, percentage and details, shear ratio, flexural to shear strength ratio, and other parameters which will prove to be important.

The first part of this project is based on six tests on concrete panels subjected to cyclic diagonal compression, with a variation only in the reinforcement. The experimental and analytical work will be finalized to the evaluation of behavior factors to be used in seismic equivalent static analyses. The importance and actuality of the general topic does not need any comment [1,2,3,4,5]; these first tests and analytical interpretations allow interesting observations on the possible failure mechanisms and on the available ductility levels.

### MATERIALS AND METHODS

Tests have been performed on six concrete panels, 1 m by 1 m, 80 mm thick, with two 200 mm by 300 mm beams at the top and bottom edges, for a total height of 1.4 m. A concrete cubic strength of 30 MPa has been very well approximated in all the cases.

The reinforcement was always provided by a single net of 6 mm bars spaced at 200 mm in both directions; the properties of the steel are summarized in table 1. Note that the main differences are in the available elongation and in the reinforcement type.

The tests have been performed defining a "cycle displacement" ( $v_c$ ) as the value of the displacement at the 75% of the cracking load ( $V_c$ ); the nth cycle in both directions has then been loaded up to a horizontal displacement of n

times  $v_c$ .

The local ductility has been defined as the ratio between the ultimate displacement and the cycle displacement, so that the available ductility is substantially equal to the number of cycles performed.

Table 1 - Reinforcement Properties

Panel Type	Reinforcement Type	$f_{sy}$ (MPa)	$f_{su}$ (MPa)	E (%)
A	Welded Wire	548	638	10
B	Welded Wire	519	690	18
C	Rebars	550	637	20

$f_{sy}$  = yield strength  $f_{su}$  = ultimate strength E = elongation on 10 diameter

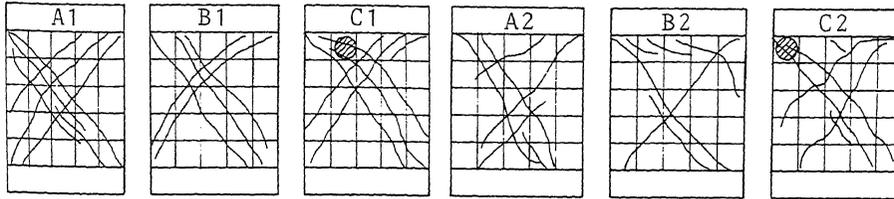


Fig. 1 Crack patterns and crushed zones of the six panels at the end of the tests

## RESULTS

The main results are summarized in table 2 and in figures 1 and 2. The following observations can be made:

- there is a wide dispersion of the results for all the variables observed; the coefficient of variation is larger for the cracking load ( $V_c$ ) and for the ultimate displacement ( $v_u$ ) and smaller for the failure load ( $V_u$ ) and for the cycle displacement ( $v_c$ );
- there is no significant difference between maximum load and ultimate load, or in other words no softening is recognizable in the envelope of the load vs. displacement curves;
- the cracking patterns are apparently similar in all the cases, but it will be stressed how a small difference in the number of main cracks or in their distance may be important.

The main topics to be discussed are related to the failure mechanisms, to the available ductility and to the effects of the different reinforcements.

Table 2 - Summary of the experimental results

Panel type	A1	A2	B1	B2	C1	C2	Average
Cracking load $V_c$ (kN)	353	256	412	481	441	333	379
Failure load $V_u$ (kN)	461	422	514	446	549	432	471
Failure mode	horizontal steel			concrete			
Number of cycles	10	5	5	7	4	5	6
Cycle displ. $v_c$ (mm)	0.40	0.50	0.60	0.48	0.56	0.40	0.49
Failure displ. $v_u$ (mm)	3.2	2.7	3.1	2.9	2.8	1.8	2.75
Local ductility	8.0	5.4	5.2	6.0	5.0	4.5	5.6

**Failure Mechanisms** The failure is always started by some cracking originated in the central region of the panel, with a cracks' orientation approximately matching the diagonal of the panel. The reinforcement interested by the cracks immediately yields because of its small section with respect to the concrete

area crossed by the crack.

All the subsequent cycles may then increase the length and/or the number of the cracks (but the crack pattern becomes usually stable after two or three cycles).

The final failure may be reached according to three different ways, as follows:

1. A crack crosses completely a panel, from side to side, and all the horizontal steel reaches the failure;
2. In some region close to the corner of the panel the failure domain of concrete is reached, with a prevalence of the compression stress;
3. The compressed strut buckles because of its reduced geometrical section (due to the cracks parallel to the load), because of its reduced stiffness (due to the cracks perpendicular to the load), and because of its high

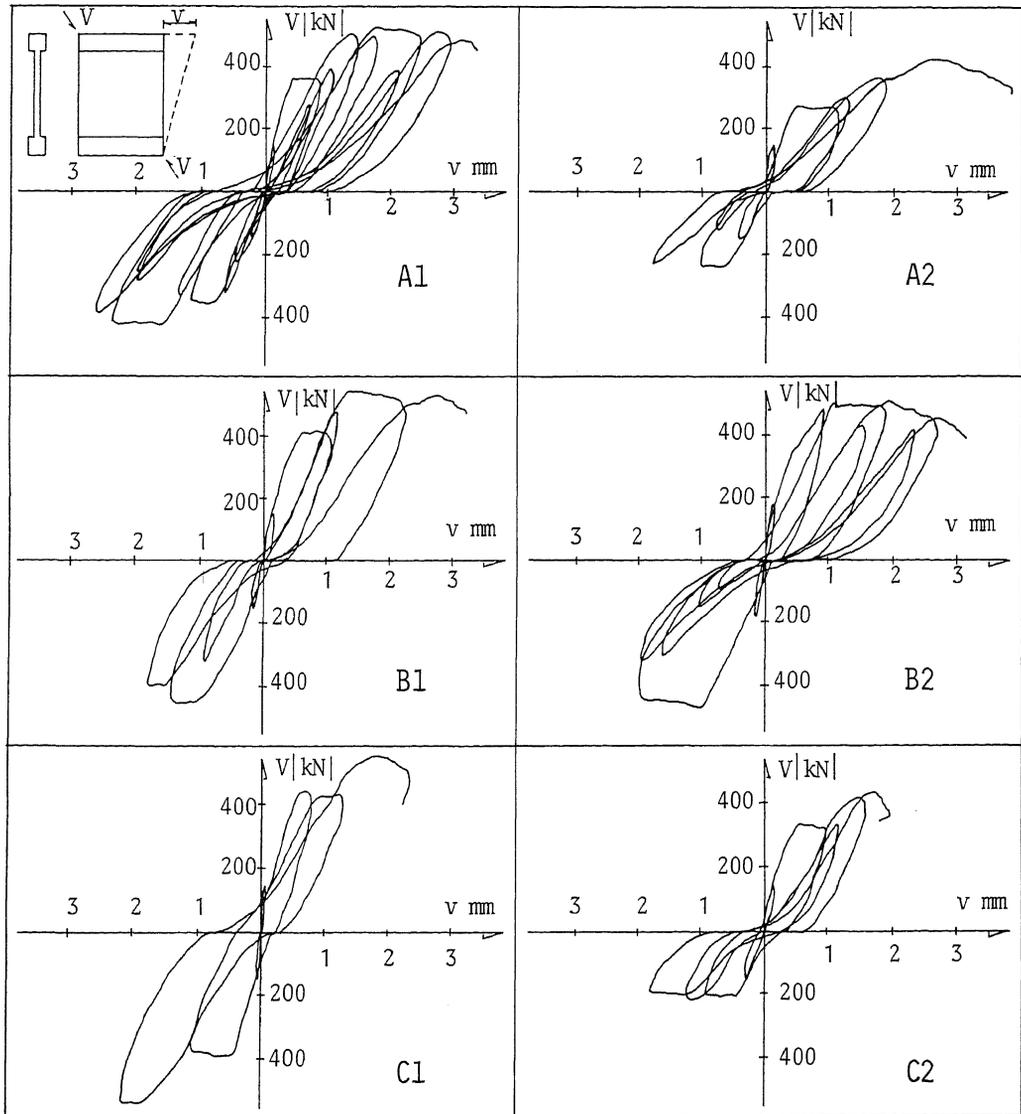


Fig. 2 Total force vs. horizontal displacement curves for the six panels

initial slenderness (see ref. 5).

Failure modes 2. and 3. are clearly strictly related and difficult to separate.

In order to set some quantitative relation among the possible way and location of the final failure some simple linear F. E. analyses have been run, using plane stress eight node element. Figure 3 summarizes some results, showing the concrete failure domain in the tension-compression region, the stress paths followed in some point of the panel, and some lines connecting stress points reached at the same time on different paths.

As shown in figure 3 some cracking in the central region of the panels is theoretically obtained at a load of 362 kN (experimental average 379 kN), while if the panel is supposed to behave linearly a load of 546 kN is required to reach a compressive failure in the corner region. As a crack opens some steel immediately yields, because of the low reinforcement percentage (the steel should take a stress of about 1000 MPa to absorb the force released by the opening of a diagonal crack). Unloading immediately follows, since the imposed displacement of the first cycle is reached.

The behavior at the next cycle in the same direction is of course a function of the real crack pattern. If only one crack 300 mm long is

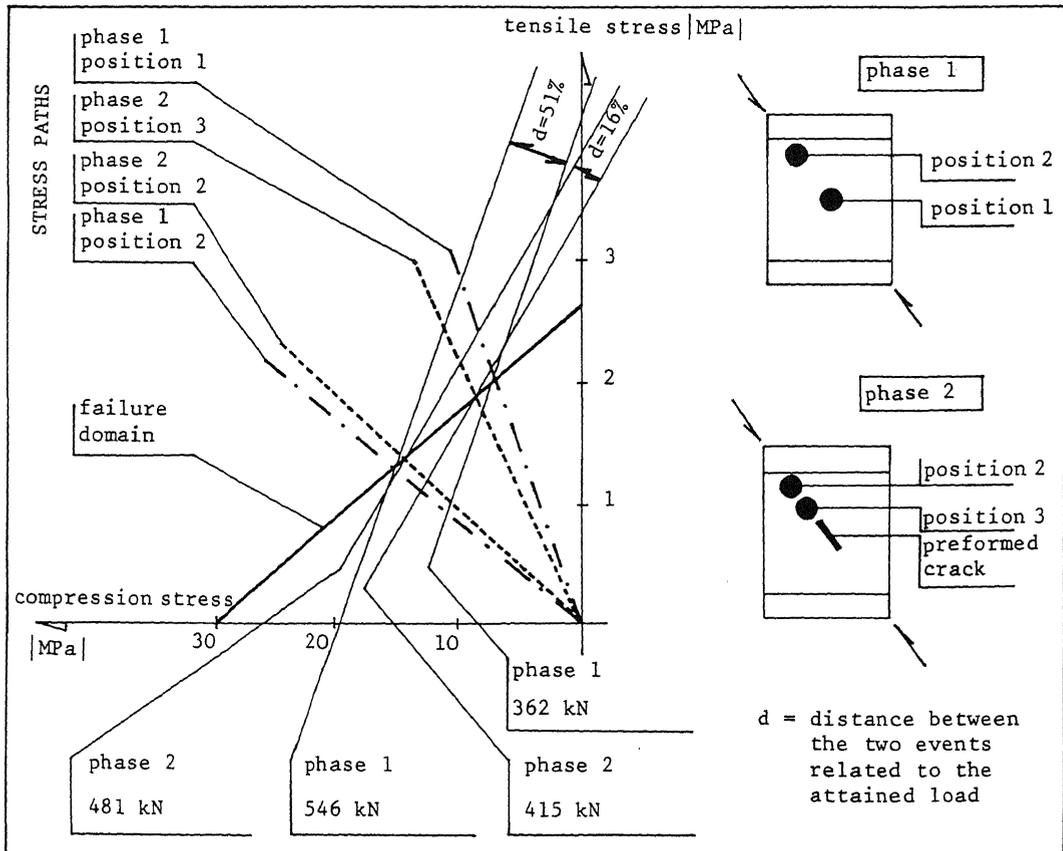


Fig. 3 Stress paths during the loading history, as a function of the location in the panel and of the initial conditions (with or without a preformed crack), obtained from numerical analyses.

considered, and the remaining part of the panel is supposed to behave linearly, the theoretical load at which a new crack or a compression failure are obtained are 415 kN and 481 kN, so that the "distance" between the two possible events is reduced from 51% to 16%, and the experimental average failure load (471 kN) seems to be approximated from the two sides.

From a different point of view, if the compressed strut is considered, the ratio between the unsupported length and the thickness is about 18, so that a reduction of about 30% of the bending moment capacity might be assumed [6], even if it is not easy to quantify the effects of the cracks perpendicular to the load. This reduction may be important if a certain amount of accidental eccentricity has to be considered. However the effects of the cracks parallel to the loading direction seem to be more effective in reducing the load capacity because of the reduction in the section of the strut. As a matter of fact the two panels which failed in compression show a narrowing in the compression strut close to the corner.

Ductility As already stated the displacement corresponding to the 75% of the cracking load has been selected as a yielding equivalent displacement in order to define a measure of the local ductility of the panels. The values obtained are summarized in table 1, the average ductility being 5.6.

It can be observed how it is hard to recognize the failure mechanisms from the number of cycles, from the maximum displacement, from the amount of ductility and even from the force versus displacement curves (fig. 2).

In other words even an explosive failure of the concrete strut can not be considered brittle, since it has to be triggered by the opening of cracks in a certain number of cycles. This fact has to be considered a consequence of the way of testing, which allows the different failure mechanisms previously discussed to be at the same level of probability of occurrence.

The average ductility of 5.6 has anyhow to be regarded as a pretty low value with respect to the behavior factors currently suggested for concrete buildings based on shear wall systems; applying different and well established criteria in order to pass from local ductility to global ductility to behavior factors [7], values around 2 are obtained for the latest, while most of the codes allow to assume behavior factors of the order of 4.

Nevertheless it has to be recognized that shear failures are not easy to be obtained in shear walls, because the increasing of vertical reinforcement at the edges to avoid flexural failure is also increasing the shear resistance, particularly in the case of large diameter bars.

Effect of reinforcement type and grade As already noted in the comments on ductility, it is hard to recognize any relation between reinforcement properties and panels' behavior. More precisely, a considerable increase in the steel ductility did not show in the present case any important improvement in the ductility of the panel. Nevertheless, nothing is proved for the case of brittle steel (ultimate uniform elongation less than 6%), for which a worse behavior of the panel can be expected.

According to the authors' opinion these data seem to indicate the existence of a limit value for the elongation capacity beyond which the number and pattern of the cracks are much more significant than the elongation capacity of the bars crossing each crack, and the crack pattern can be considered a random result of local characteristics of concrete and steel. This may again be a consequence of the way of testing and particularly of the small number of cracks involved in the test.

It should also be underlined that the small diameter of the bars, and moreover the welded joints for the case of wire meshes, allow a perfect anchorage very closely to the crack edges, so that it is possible to take advantage from the elongation capabilities only on very short bar lengths.

The performances of smooth bars, when small diameters bars have to be used in shear walls, could be usefully explored, since a spreading on a wider

length of the yielded region would be highly effective in increasing the ductility.

#### CONCLUSIONS

While on one side the suggestion of using at least the reinforcement percentage able to sustain the force released at the opening of a crack is certainly advisable, on the other side the danger of concrete compressive failure would become higher. Under this respect the properties of high strength concrete, where the tensile strength does not grow proportionally to the compressive strength, seem to be very interesting. Of course the influence of second order effects become more important and have to be accurately checked.

The behavior factors allowed by many codes for static seismic equivalent analysis of shear wall systems are not justified by the tests performed. It has to be recognized, however, that the number of tests is small, the way of testing unique, and the parameters taken into account limited.

A serie of experimental tests on 4 by 2 m panels is now being performed, adopting as variable parameters the concrete strength, the reinforcement percentage and details, the shear ratio and the way of loading [8].

#### ACKNOWLEDGEMENTS

The research described in this paper has been partially supported by the Italian Research Council (C.N.R.), by the Italian Ministry of Education (MPI) and by the Ferriere Nord S.p.a.

The authors wish to express their gratitude to Professor T.P. Tassios for the helpful criticism and suggestions.

#### REFERENCES

1. Hiraishi, H., Nakata, S., Kitagawa, Y., Kaminosono, T., "Static Tests on Shear Walls and Beam-Column Assemblies and Study on Correlation between Shaking Table Tests and Pseudo-dynamic Tests", EQ Effects on R.C. Structures, ACI SP-84, Detroit, (1985)
2. Paulay, T., Priestley, M. J. N., Syngé, A. J., "Ductility in Earthquake Resistant Squat Shearwalls", ACI Journal, Vol. 79, No. 26, 257-269, (1982)
3. Mau, S. T., Hsu, T. T. C., "Shear Design and Analysis of Low-Rise Structural Walls", ACI Journal, Vol. 83, No. 33, 306-315, (1986)
4. Vecchio, F., Collins, M. P., "The Response of Reinforced Concrete to in-Plane Shear and Normal Stresses", Publication No. 82-03, Department of Civil Engineering, University of Toronto, (1982)
5. Vallenás, J. M., Bertero, V. V. and Popov, E. P., "Hysteretic Behavior of Reinforced Concrete Structural Walls", Report No. UCB/EERC 79/20, University of California, Berkeley, (1979)
6. Mac Gregor, J. G., Breen, J. E., Pfrang, E. O., "Design of Slender Columns", ACI Journal, Vol. 67, No. 1, 6-28, (1970)
7. Calvi, G. M., Macchi, G., "Behavior Coefficients for Evaluation and Retrofitting", Proc. of the U.S.A.-Italy Workshop on Masonry Structures, Washington D.C., (1987)
8. Calvi, G. M., "A 3-DOF Testing Machine for in-plane Behaviour of Structures", Materials and Structures, (to appear, 1988)