Paper No. 381. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

SEISMIC BEHAVIOUR OF R.C. SHEAR-WALLS: AN EXPERIMENTAL STUDY

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ABSTRACT

Seismic behaviour of r.c. shear-walls has been experimentally studied. Five pairs of reinforced-concrete shear-walls with various arrangements of steel reinforcement have been tested by subjecting them to simulated seismic loading. Amount and distribution of steel, as well as the level of imposed vertical load, under which the walls have been tested, determined the lateral resistance of the walls. The behaviour of walls, subjected to low level of compression, was more ductile than the behaviour of walls tested at high level of vertical load. Confinement of vertical steel improved the ductility of the walls at high level of vertical load.

KEYWORDS

Reinforced-concrete shear-wall; seismic behaviour; lateral resistance; ductility; reinforcement.

INTRODUCTION

Shear-wall structures have drawbacks regarding structural configuration that hindered their wider use in the past. For example, they do not permit such architectural flexibility of buildings as frame structural systems. When subjected to horizontal loads, their behaviour is governed by shear, what sometimes causes brittle failures. Because of their rigidity, fundamental periods of vibration of shear-wall structures are short. Resulting resonance effects when subjected to short period earthquakes cause higher design seismic forces than in the case of flexible frame structures of comparable size, and increase the amount of steel reinforcement.

However, what seemed to be drawback in the past has become an advantage at present time: the rigidity of shear-wall structures reduces possible damage to non-structural elements. As the past earthquakes prove, shear-walls can be both strong and ductile, if adequately conceived and reinforced. Despite their predominant shear behaviour, shear-walls possess adequate energy dissipation capacity needed to withstand strong earthquakes. This has been indicated by post-earthquake observations as well as by experimental research (Wood, 1989; Wood, 1990; Lefas et al., 1990; Lefas and Kotsovos, 1990; Wallace and Moehle, 1992).

In Slovenia, r.c. shear-wall structures are commonly used in residential construction. Rectangular sections (free-edged walls) are typical, reinforced with distributed web reinforcement and concentrated steel at vertical edges. Shear-walls with boundary elements (barbell sections) are rare. In the design of shear-wall buildings, design requirements of former Yugoslavia's seismic code have been used until recently. In 1995, however, Eurocode 8 has been adopted by a National Application Document. As was the case of Yugoslavia's code, some specific requirements of Eurocode 8 regarding the amount and detailing of reinforcing steel in free-edged shear-walls are lacking experimental support. In order to make a contribution to experimental data base, ten shear-walls have been recently tested also at National Building and Civil Engineering Institute (ZAG) in Ljubljana. This paper discusses the results of experiments.

DESCRIPTION OF EXPERIMENTS

Typical residential shear-wall building in Slovenia is 6 to 12 storeys high, with storey height varying between 2.6 and 2.8 m. As the distance between the walls is 6.0 m to 6.6 m, and minimum thickness of the walls 15 cm, the resulting net area of the walls in each principal direction of the building varies between 2.5 % and 3.5 % of the gross floor area, which is more than required by the code (1.5 %). The specimens tested within this study represented shear-walls with rectangular section, located centrally in the lower-most two storeys of a typical shear-wall building. Because of the limited capability of testing facilities in the laboratory, model walls, constructed at 1:3 scale, have been tested.

The requirements of former Yugoslavia's code, still used in Slovenia (Technical norms, 1981), and European pre-standard Eurocode 8 (Eurocode, 1995), have been used in the design of specimens. When conceiving the project, draft requirements have been taken into account, as the development of Eurocode 8 progressed. In order to study the influence of vertical loads on the seismic behaviour, specimens of the same type have been tested at two levels of vertical load. When conceiving the tests, it has been assumed that concrete is fully utilised in the case of a 12-storey building, whereas working compressive stresses are 2-times smaller in the case of a 6-storey structure. Test matrix is given in Table 1.

Table 1. Characteristics of tested walls

Type and designation	Hor. web steel	Vert. web steel	Boundary steel	Confi- nement	Vertical load
Type 1: SW00N1	0.26 %	0.26 %	-	no	0.1 β_B
Type 1: SW00N2	0.26 %	0.26 %	-	no	0.2 β_B
Type 2: SW23N1	0.26 %	0.26 %	2.3 %	no	0.1 β_B
Type 2: SW23N2	0.26 %	0.26 %	2.3 %	no	0.2 β_B
Type 3: SW23C1	0.26 %	0.26 %	2.3 %	yes	0.1 β_B
Type 3: SW23C2	0.26 %	0.26 %	2.3 %	yes	0.2 β_B
Type 4: SW60N1	0.38 %	0.38 %	6 %	no	0.1 β_B
Type 4: SW60N2	0.38 %	0.38 %	6 %	no	0.2 β_B
Type 5: SW60C1	0.38 %	0.38 %	6 %	yes	0.1 β_B
Type 5: SW60C2	0.38 %	0.38 %	6 %	yes	0.2 β_B

Note: β_B is equal to 0.7 of compressive cube strength.

The amount of distributed horizontal and vertical web reinforcement varied from 0.26 % (3.0 mm diameter bars at 59 mm distance, yield stress/tensile strength 478/531 MPa) in the case of walls Type 1, 2, and 3, and 0.38 % (3.8 mm diameter bars at 59 mm distance, 469/516 MPa) in the case of walls Type 4 and 5. 0.26 % is minimum, as required by former Yugoslavia's seismic code (0.25 % - Technical norms, 1981). 0.38 % was calculated in order to prevent shear failure in the case of walls with strong vertical boundary steel. Walls Type 1 without concentrated boundary vertical steel represented referential walls.

In the case of weaker walls Type 2 and 3, percentage of concentrated steel (2.3 % - four 6 mm diameter bars, 538/581 MPa) is slightly greater than required by former Yugoslavia's seismic code (minimum 1.5 % with regard to the area of boundary section, 1/10 of the wall's length long). In the case of stronger walls Type 4 and 5, however, 6 % (six 8 mm diameter bars, 465/693 MPa) represents maximum amount of steel, permitted for r.c. columns by former Yugoslavia's code for r.c. structures (Technical Norms, 1987). Boundary steel of walls Type 3 and 5 was confined over the lower half of walls' height. Stirrups (1.4 mm diameter wire, 427/514 MPa) have been placed so that the volumetric mechanical ratio of confining steel according to one of the previous drafts of EC 8 has been ω_{wd} = 0.24. Dimensions of specimens and arrangement of distributed web and concentrated vertical reinforcement is shown in Figs. 1 and 2.

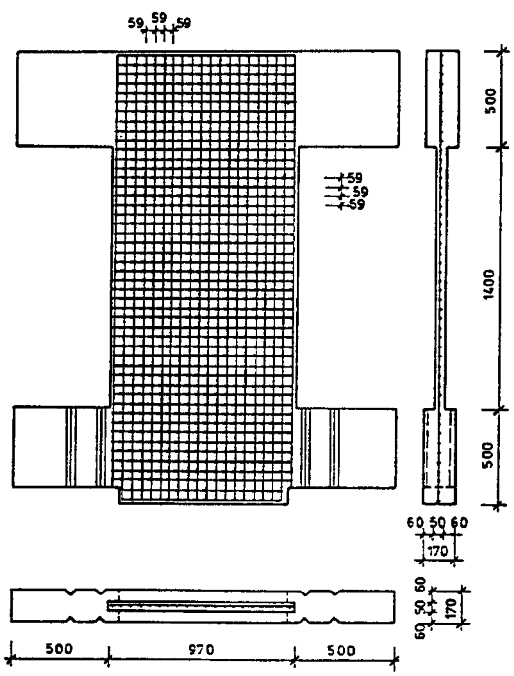

Fig. 1. Dimensions of walls and distribution of web reinforcement

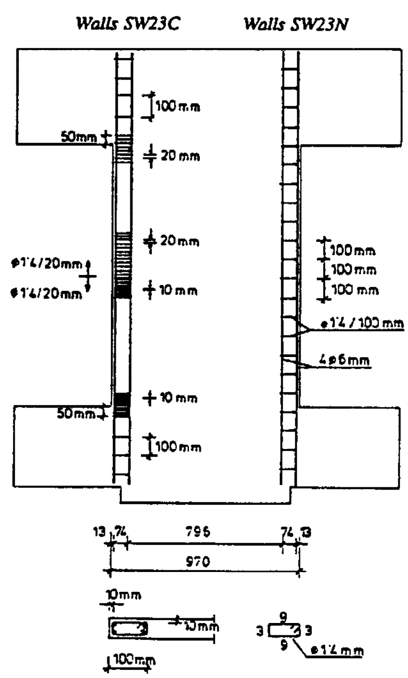

Fig. 2. Distribution of vertical concentrated steel and confinement in the case of walls SW23

A specially designed concrete mix, prepared in a concrete plant and delivered to laboratory, has been used to cast the specimens. Grade MB 30 concrete (cube strength β_k = 30 MPa) was mixed with gravel aggregate, 0 - 10 mm in diameter. Actual average compressive strength, determined after testing the walls, was 41.1 MPa (standard deviation 3.1 MPa, coefficient of variation 0.07). It was, however, equal to 54.8 MPa in the case of walls SW23N1 and SW23N2.

A centrally located wall, where no significant changes in axial loads are expected, has been tested. Hence, the magnitude of axial load depended only on gravity loads. According to the results of parametric analysis,

TEST RESULTS

Test results are summarised in Table 2, where for each wall the values of relative storey displacement and corresponding horizontal reaction force are given at characteristic points of hysteresis envelopes, characterised by yield limit (H_y, d_y), maximum resistance (H_{max}, d_{Hmax}), and maximum displacement (H_{dmax}, d_{max}).

Table 2. Parameters of lateral resistance and deformability

Designation	H_y (kN)	d_y (mm)	H_{max} (kN)	d_{Hmax} (mm)	H_{dmax} (kN)	d_{max} mm
SW00N1	36.8	3.6	39.0	8.4	29.3	19.2
SW00N2	53.7	2.4	62.5	6.7	60.6	8.5
SW23N1	54.7	3.6	64.1	7.8	25.6	22.7
SW23N2	68.8	2.4	85.6	7.2	63.9	8.4
SW23C1	50.8	3.6	64.2	9.6	33.3	17.0
SW23C2	61.5	2.4	83.4	8.4	81.2	10.8
SW60N1	82.1	6.0	105.1	15.6	54.7	18.0
SW60N2	77.3	3.6	110.2	9.0	110.2	9.0
SW60C1	85.0	6.0	109.7	16.8	109.4	19.6
SW60C2	85.0	3.6	117.1	10.2	108.0	11.0

Average values obtained at loading in positive and negative direction are given in Table 2. In Figs. 6 and 7, lateral load - lateral displacement hysteresis envelopes are presented for specimens tested at low and high level of axial load, respectively.

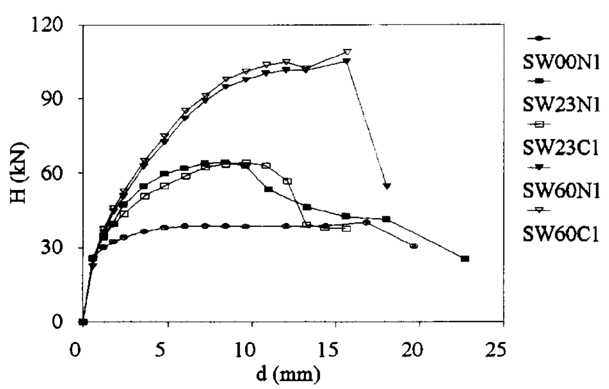

Fig. 6. Lateral load - displacement hysteresis envelopes - low axial load

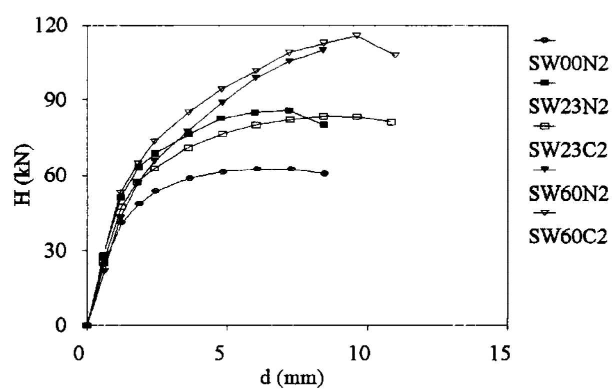

Fig 7. Lateral load - displacement hysteresis envelopes - high axial load

Figures 8 and 9 show moment - curvature relationships at the bottom section of the tested walls, obtained on the basis of the measured values of strain of vertical reinforcement at the bottom section of the walls, forces measured at the actuators, and known geometry of the walls. Average values of curvature, evaluated by taking into account the measured values of strain of outer and inner pair of vertical bars in the boundary section of the walls, are plotted in Figs. 8 and 9.

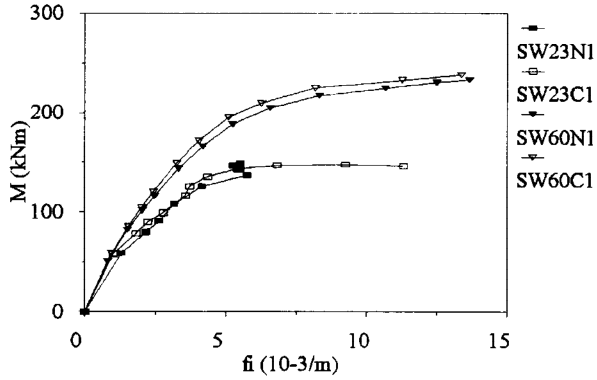

Fig. 8. **Moment - curvature relationship. Bottom section, low axial load**

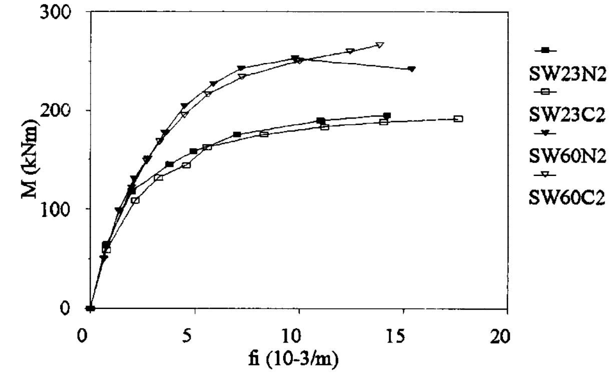

Fig 9. **Moment - curvature relationship. Bottom section, high axial load**

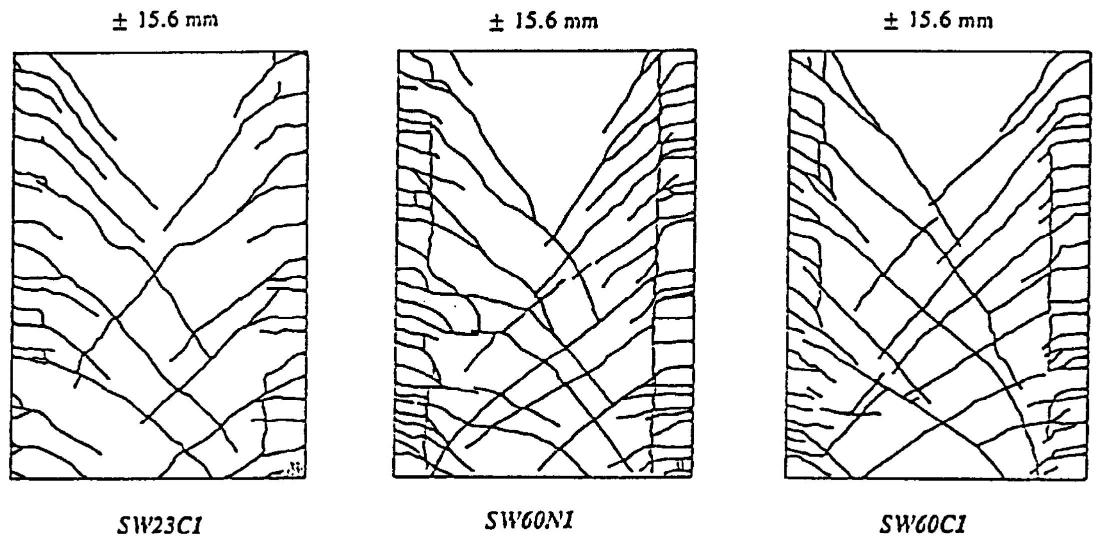

Fig. 10. Typical crack patterns at ultimate state of walls SW23C1, SW60N1 and SW60C1

Crack patterns, developed in typical walls at ultimate state, are compared in Fig. 10. As a measure of energy dissipation capacity, the amount of input energy at characteristic points of hysteresis envelopes, defined as cumulative work of actuators from the beginning of test to the observation point, is compared for the walls tested at low and high level of vertical load, in Table 3. On the basis of the analysis of test results, the following observations can be made:

- The amount of concentrated vertical reinforcement and level of axial load influenced the lateral resistance of walls. Predominant flexural behaviour has been observed, as expected, with yielding of steel at tensioned and crushing of concrete at compressed side of walls.
- Generally, the behaviour of walls tested at low level of axial load was more ductile than the behaviour of walls, tested at high axial load. Brittle collapse took place almost immediately after the attained maximum resistance in the case of the walls, subjected to high axial load. Resistance slowly degraded in the case of the walls, tested at low level of vertical load.
- Rupture of extreme tensioned vertical reinforcement took place in the case of walls, tested at low level of axial load, resulting into severe strength degradation. In the case of high axial load, crushing of concrete and buckling of concentrated steel caused instantaneous collapse.
- Although it obviously prevented buckling of compressed steel, confinement did not improve lateral resistance and ductility of the walls tested at low level of axial load. Fact that slightly larger values of both

CONCLUSIONS

Whereas some of the observed phenomena could have been expected, the observed behaviour and measurements indicate that further experimental research is needed to support, or relax some requirements of seismic codes, regarding the arrangement of steel in r.c. shear-walls.

The height of the critical region of the wall, as defined by Eurocode 8, seems conservative. Namely, according to Eurocode 8, the height of the critical region, where confinement of boundary steel is required to achieve high ductility class, should be equal to the greater value of the length of the wall or 1/6 of the total wall height. The value, however, is related to storey height h_s and should not exceed h_s or 2 h_s, depending on the height of the building. As indicated by these experiments, confinement of concentrated vertical steel proved efficient only in the bottom-most parts of the walls.

In order to prevent buckling of confined edge part of the wall as a whole, it seems that confinement should be in part anchored into the web. Also, as indicated by the observed behaviour and crack patterns, in order to ensure ductile behaviour of shear-walls, attention should be paid to the length of the edge area, where vertical concentrated steel is distributed. In the particular case studied and especially in the case of heavy vertical reinforcement, 10 % of the wall's length, as required by Yugoslav code, was too little. In this regard, the limit of 0.2 % compressive strain, or a minimum of 15 % of the wall's length, as proposed by Eurocode 8, seems adequate.

ACKNOWLEDGEMENTS

The research discussed in this paper has been financed by the Ministry of Science and Technology of Republic of Slovenia under Grant No. P2-5208-227.

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