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Influence of Distribution of Reinforcement on Seismic Behaviour of R.C. Shear-Walls: An Experimental Study

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ABSTRACT

Seismic behaviour of 10 reinforced-concrete free-edged shear-walls, designed to fail in bending, has been experimentally investigated. Tests of five different types of specimens tested at two levels of vertical load indicated that the amount of steel and level of imposed vertical load determine the lateral resistance of the walls. The behaviour of walls, subjected to low, was more ductile than the behaviour of walls tested at high level of vertical load. However, confinement of vertical steel improved ductility of walls at high, but did not influence their ductility at low level of vertical load.

INTRODUCTION

In the last decades, seismic behaviour of shear-walls with boundary elements and concentrated vertical reinforcement (barbell sections) has been primarily investigated. Experimental research in shear-walls with rectangular sections (free-edged walls) was relatively scarce. Even after the earthquake of Chile of 1985, where good behaviour of r.c. shear-wall buildings without boundary elements has been observed, the number of experimental investigations did not increase significantly. Since the construction of r.c. shear-wall structures in Slovenia is quite popular, a series of r.c. shear-walls has been tested also at the Institute for Testing and Research in Materials and Structures (ZRMK) in Ljubljana, Slovenia, in order to further investigate possibilities for improving their seismic behaviour.

DESCRIPTION OF SPECIMENS

The specimens tested within this study represented shear-walls with rectangular section, located in the lower-most two storeys of a r.c. shear-wall building, which is typically 6- or 12-storey high. Typical distance between the walls is 6.0 - 6.6 m, with storey height varying between 2.6 and 2.8 m. As the minimum thickness of the walls is 15 cm, the resulting net area of the walls in each principal direction varies between 2.5 % and 3.5 % of the gross floor area in most practical cases. Model walls, constructed at 1:3 scale, have been tested. This made possible relatively simple testing, and has not influenced the results (observed failure mechanism and physical parameters, which define the seismic behaviour of r.c. shear-walls).

When designing the specimens, the requirements of technical Codes, still used in Slovenia (former Yugoslavia's Codes), model Eurocode 8, as well as some recent experimental research results have been considered. In order to study the influence of vertical loads on seismic behaviour,

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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 utilized in the case of a 12-storey building, whereas working 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	Horizontal steel	Vertical steel	Vertical conc. steel	Vertical load	Pred. type of failure
Type 1 *SW00N1*	3.0/59 mm 0.26 %	3.0/59 mm 0.26 %	- -	0.1 $ß_B$	flexure
Type 1 *SW00N2*	3.0/59 mm 0.26 %	3.0/59 mm 0.26 %	- -	0.2 $ß_B$	flexure
Type 2 *SW23N1*	3.0/59 mm 0.26 %	3.0/59 mm 0.26 %	4ϕ6mm 2.3 %	0.1 $ß_B$	shear/flex.
Type 2 *SW23N2*	3.0/59 mm 0.26 %	3.0/59 mm 0.26 %	4ϕ6mm 2.3 %	0.2 $ß_B$	flexure
Type 3 *SW23C1*	3.0/59 mm 0.26 %	3.0/59 mm 0.26 %	4ϕ6mm 2.3 %	0.1 $ß_B$ confinement	shear/flex.
Type 3 *SW23C2*	3.0/59 mm 0.26 %	3.0/59 mm 0.26 %	4ϕ6mm 2.3 %	0.2 $ß_B$ confinement	flexure
Type 4 *SW60N1*	3.8/59 mm 0.38 %	3.8/59 mm 0.38 %	6ϕ8mm 6 %	0.1 $ß_B$	shear/flex.
Type 4 *SW60N2*	3.8/59 mm 0.38 %	3.8/59 mm 0.38 %	6ϕ8mm 6 %	0.2 $ß_B$	flexure
Type 5 *SW60C1*	3.8/59 mm 0.38 %	3.8/59 mm 0.38 %	6ϕ8mm 6 %	0.1 $ß_B$ confinement	shear/flex.
Type 5 *SW60C2*	3.8/59 mm 0.38 %	3.8/59 mm 0.38 %	6ϕ8mm 6 %	0.2 $ß_B$ confinement	flexure

Note: $ß_B$ is equal to 0.7 of compressive cube strength.

Walls Type 1, 2 and 3 have been reinforced with minimum amount of distributed web reinforcement, as required by former Yugoslavia's code for construction of buildings in seismic zones (0.25 % - *Technical Norms 1981)*. This is slightly more than required by the new Eurocode 8 document (0.20 % - *Eurocode 1993*). Walls Type 1 without concentrated vertical steel represented referential walls.

In the case of weaker walls Type 2 and 3, percentage of concentrated steel (2.3 %) 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, percentage of concentrated steel represents the maximum amount of steel, permitted for r.c. columns by former Yugoslavia's code for r.c. structures (maximum 6 % - *Technical Norms 1987*). To prevent predominant shear behaviour, the amount of distributed horizontal and vertical web reinforcement (0.38 %) in that case was determined by calculation. Concentrated vertical reinforcement of walls Type 3 and 5 was confined over the lower half of walls' height. Stirrups 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. Concentrated vertical reinforcement of walls Type 2 and 4, however, has not been confined in the critical zone. Dimensions of specimens and arrangement of distributed web and concentrated vertical reinforcement is shown in Figs. 1 and 2.

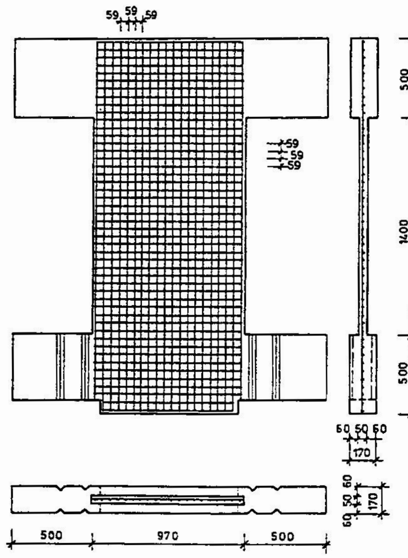

Fig. 1: Dimensions of walls and distribution of mesh reinforcement

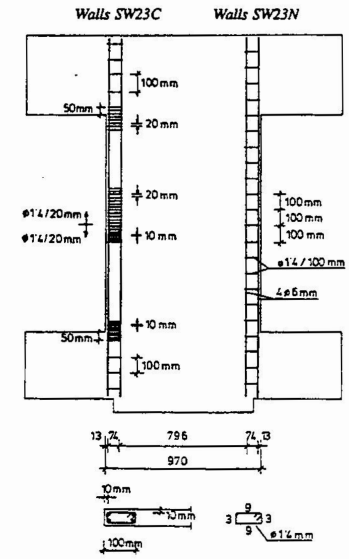

Fig. 2: Distribution of vertical concentrated steel and confinement in the case of walls SW23

Table 2: Mechanical properties of reinforcing steel

Nominal diameter (mm)	Actual diameter (mm)	Yield stress (MPa)	Tensile strength (MPa)	Elongation at rupture (%)
8	7.4-7.8	465	693	20.0
6	5.7-6.0	538	581	7.0
3.8	3.8	469	516	19.5
3.0	3.0	478	531	23.3
1.4	1.4	427	514	-

Commercially available reinforcing steel has been used to reinforce the specimens. Deformed steel bars $\phi 6$ and $\phi 8$ mm have been used for concentrated vertical reinforcement, and annealed wire

$\phi 3.0$ in $\phi 3.8$ mm for web reinforcement. 1.4 mm diameter annealed wire has been be used for confining the concentrated reinforcement in the case of walls Type 3 and 5. Mechanical characteristics of reinforcing steel are given in Table 2.

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), mixed with gravel aggregate, 0 - 10 mm in diameter, has been seen used. Actual average 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 (standard deviation 1.7 MPa, coefficient of variation 0.03) in the case of walls *SW23N1* and *SW23N2*.

TESTING PROCEDURE AND MEASUREMENTS

Since the simulation of changes in axial load during earthquakes due to overturning effects would be relatively difficult, the wall located in the middle of the building's in plan, where no significant changes in axial loads are expected, has been tested. Hence, the magnitude of axial load depended only on gravity loads. As has been found by parametric analysis, elastic restraints of coupling beams can be simulated by adding a constant bending moment along the height of vertical cantilever. The test set-up has been conceived accordingly (Fig.3).

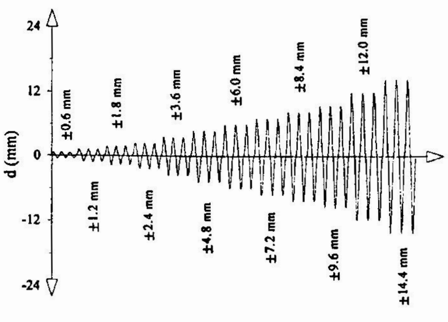

Fig. 4: Lateral displacement program

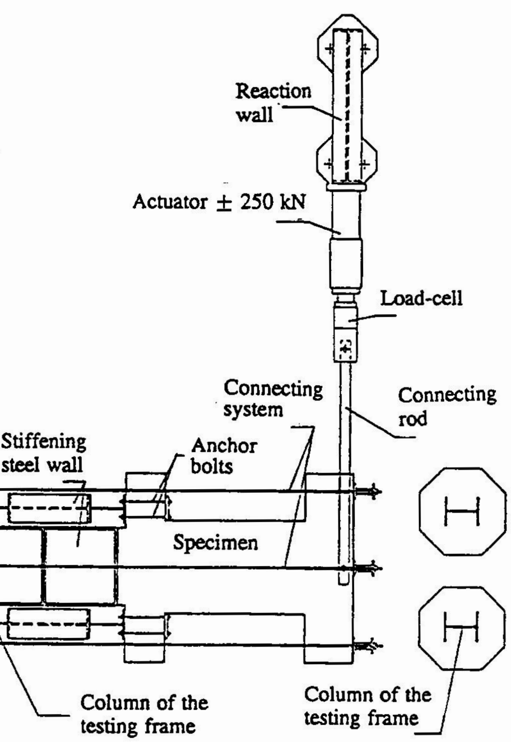

Fig. 3: Layout of test set-up in plan

Forces at connections between actuators and specimens have been measured by means of load cells, and LVDT-s have been used to measure the displacements. Changes in strains of reinforcement have been measured with strain-gauges, glued on the vertical and horizontal bars of concentrated boundary and distributed web reinforcement. Typical instrumentation of walls with LVDT-s and strain-gauges is shown in Figs. 5 and 6, respectively.

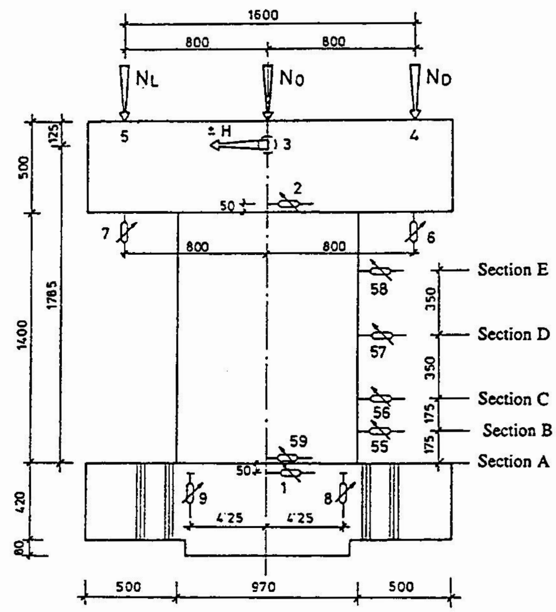

Fig. 5: Instrumentation of walls with load cells and LVDT-s

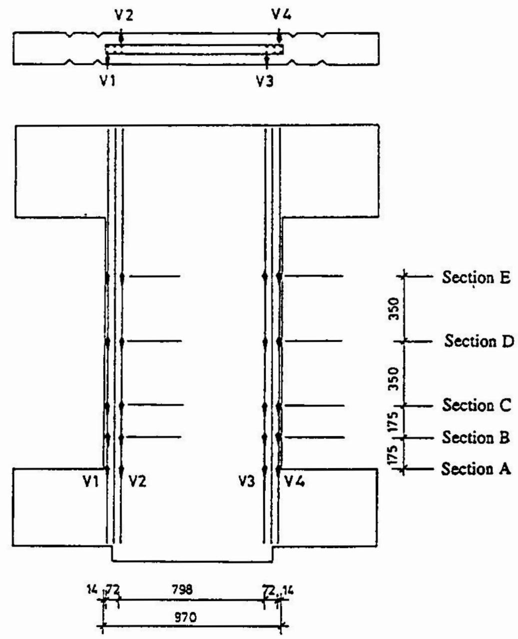

Fig. 6: Typical instrumentation of concentrated vertical steel with strain-gauges

TEST RESULTS

Test results are summarized in Table 3, where values of relative storey displacements and corresponding horizontal reaction forces are given at characteristic points of storey hysteresis envelopes, characterized by yield limit (H_y, d_y), maximum resistance (H_{max}, d_{Hmax}), and maximum displacement (H_{dmax}, d_{max}). Average values obtained at loading in positive and negative direction are given in the table. Lateral load - lateral displacement hysteresis envelopes are presented in Figs. 7 and 8 for specimens tested at low and high level of axial load, respectively. Typical lateral load - strain in vertical reinforcement hysteresis loops, measured in bottom two sections of walls, tested at low and high level of axial load, are shown in Fig. 9. In Fig. 10, however, typical crack patterns developed in the walls at ultimate state, are compared. On the basis of 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 as predicted by calculations. Predominant flexural behaviour has been observed, as expected, with yielding of steel at tensioned and crushing of concrete at compressed side of walls.

Table 3: Parameters of lateral resistance and deformability

Desig-nation	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

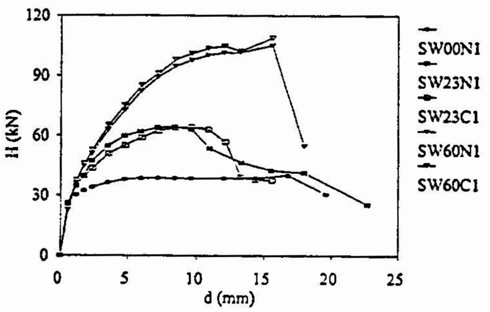

Fig. 7: Lateral load - displacement hysteresis envelopes - low axial load

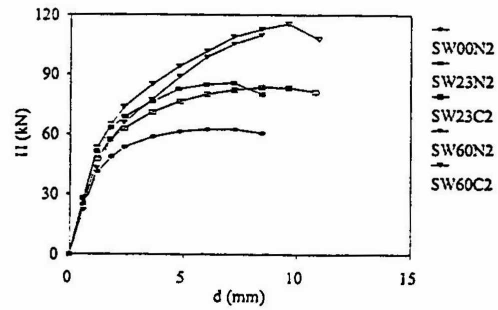

Fig 8: Lateral load - displacement hysteresis envelopes - high axial load

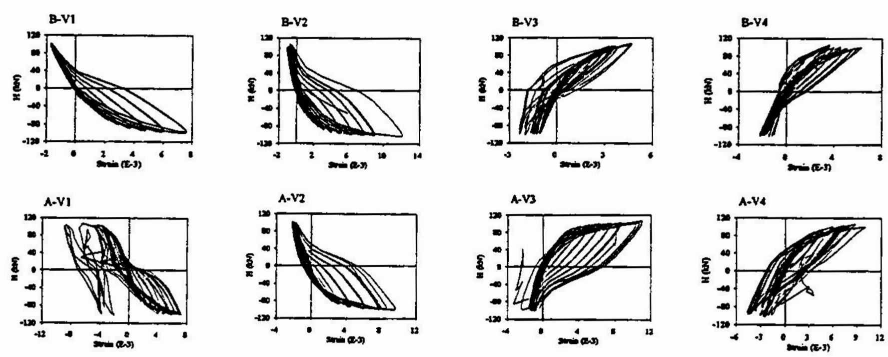

Fig. 9: Typical lateral load - strain in vertical reinforcement hysteresis loops in bottom two sections of Wall SW60N1

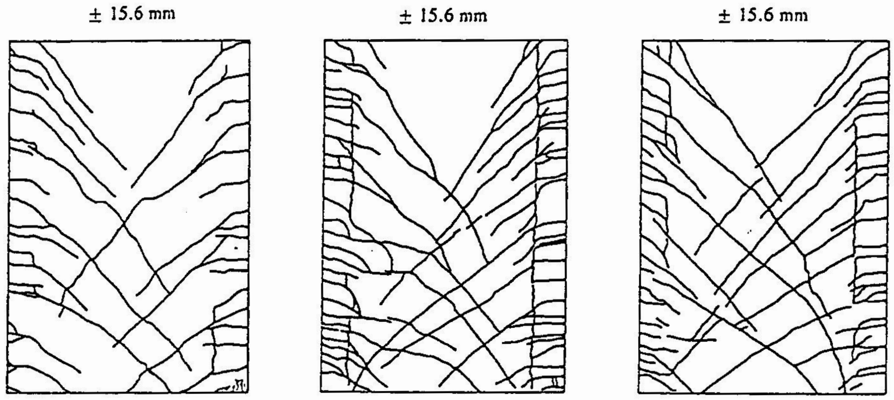

Fig. 10: Typical crack patterns at ultimate state of walls SW23C1, SW60N1 and SW60C1

• Generally, the behaviour of walls tested at low level of axial load was more ductile than behaviour of walls, tested at high axial load. Whereas collapse took place almost immediately after the attained maximum resistance in the case of high axial load, resistance slowly degraded in the case of walls, tested at low level of lateral load. In the latter case, in no case walls actually collapsed, although the lateral resistance severely degraded due to successive rupture of tension steel and, hence, reduced effective area of the bottom section.

• 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. The process of crushing of concrete and buckling of steel in compression was not of explosive character. 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 parameters have been obtained by testing the walls without confinement in this particular case, cannot be the reason of any conclusion.

• Confinement obviously improved the behaviour of walls, tested at high level of axial load. As indicated by the rupture of stirrups in some cases, stirrups were fully activated and prevented buckling of individual bars. In some cases, however, the complete column, consisting of confined concentrated vertical reinforcement, buckled as one single element.

• As indicated by the analysis of crack propagation and final crack pattern, vertical boundary elements formed after the attained maximum resistance, and prevented propagation of diagonally oriented cracks towards the edges of walls. It seems that formation of these elements, which confined the inner part of the wall, separated by diagonal cracks, was more pronounced in the case of walls, reinforced with larger amount of vertical concentrated reinforcement, confined and tested at high level of axial load.

CONCLUSIONS

Whereas some of the observed phenomena could have been expected, the observed behaviour and measurements indicated 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.

In that sense, the length of critical height of the wall as defined by *Eurocode 8* seems conservative. 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 observed behaviour and crack patterns, in order to ensure ductile behaviour of walls, attention should be also paid to the size 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 seems too little. It can be concluded, that ductility of walls would be improved, if reinforcement would have been distributed within the 15 % of the wall's length as proposed by *Eurocode 8*.

It is expected that by detailed analysis of a large number of measured data, and subsequent correlation of damage propagation with observed deformations of the walls and distribution of strain in the reinforcement, a good basis will be also given for developing improved mathematical models and practical methods for seismic resistance analysis and design of r.c. shear-walls.

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REFERENCES

Eurocode No.8. Structures in Seismic Regions - Design, Part 1, General and Building, (1993) Doc. TC250/SC8/N57A.

Lefas, I.D., Kotsovos, M.D. Ambrasseys, N.N. (1990). "Behaviour of reinforced concrete structural walls: strength, deformation characteristics, and failure mechanism." *ACI Struct. Journal*, 87 (1), pp.23-31.

Lefas, I.D., Kotsovos, M.D. (1990) "Strength and deformation characteristics of reinforced concrete walls under load reversals." *ACI Struct. Journal*, 87 (6), pp.716-726.

Technical norms for the construction of building structures in seismic zones (1981). Official Gazette of SFRY, no.31. Beograd, Yugoslavia (in Slovene).

Technical norms for concrete and reinforced-concrete (1987). Official Gazette of SFRY, no.11. Beograd, Yugoslavia (in Slovene)

Wallace, J.W, Moehle, J.P. (1992). "Ductility and detailing requirements of bearing wall buildings." *J. Struct. Eng. ASCE*, 118 (6), pp.1652-1643.

Wood, S.L. (1989). "Minimum tensile reinforcement requirements in walls." *ACI Struct. Journal*, 86 (4), pp.582-591.

Wood, S.L. (1990). "Shear strength of low-rise reinforced concrete walls." *ACI Struct. Journal*, 87 (1), pp.99-107.