*Paper No. 16. (quote when citing this article)*
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

# BEHAVIOR OF PRECAST SHEAR WALLS UNDER QUASI-DYNAMIC LOADING MODEL TESTING RESULTS

DIMITRIJEVIC RADOVAN L.

Director for Marketing and Public Relations
IMS Institute 11000 Beograd, Bul.Vojvode Misica 43, YUGOSLAVIA

## ABSTRACT

Shear walls in precast prestressed skeleton are use for acceptance of lateral forces, decreasing of total deformation under its actions and absorbing energy. In this structure shear walls consist of two columns and thin concrete wall between them, all precast. The model investigation of that structure is done under cyclic loading to examine behavior of this kind of shear walls, make choice of analytical model, and compare its values with experimental results.

## KEYWORDS

Precast; prestressed; skeleton; shear wall; ductility; absorption of energy; hysteretics; load bearing capacity.

## INTRODUCTION

Precast prestressed skeleton is widely applied for all type of buildings - dwellings, offices, hospitals, shopping centers, garages and so on, in Yugoslavia and some other countries . During more of 40 years of practice this structure was analyzed and tested for many aspects, institutions, researchers and designers. The main interest was given to its characteristics and behavior under influence of inertial forces during earthquake or similar load. Precast column and floor slabs are unify ones in monolithic skeleton using tendons for prestressing in two orthogonal directions on every floor level. The joint column - floor slab is based on action of friction between two concrete elements and depend proportionally of coefficient of friction (Muravljov,1974,Madaras et all,1990) and intensity of axial force.

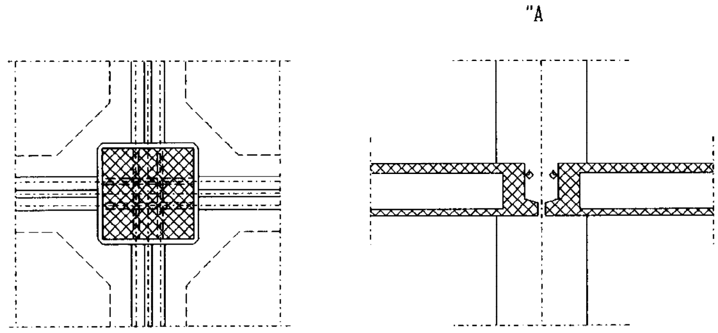

Fig. 1. Column - slabs joint.

Tendons are posed in the free space between two precast floor slabs and through holes in columns. In this phase of assembly, skeleton has characteristics of post tensioning structure. After tensioning cables the space, where they are posed, is cast with concrete of the same quality as columns and floor slabs to protect cables against the corrosion (Dimitrijevic,1992a) and to increase the cross section of the skeleton beams. Than skeleton structure has the behavior of prestressed structure (Petrovic,1965,1978). Columns and floor slabs are main load bearing structural elements for vertical loads and lateral forces, too. To decrease great deformations under lateral forces, which could destroy all other buildings elements except the skeleton (Petrovic,1967), the structure is stiffened by shear walls made of two columns and thin concrete membrane between them Fig. 2. Concrete wall has role to connect columns in structure (Jurkovski et all,1978) which will be able to accept bending moments produced by lateral forces decomposed in two axial forces which load and dissload related columns axially during its action (Dimitrijevic 1984,1994).

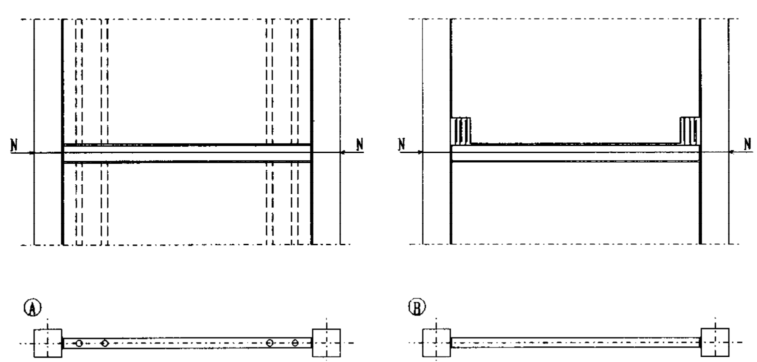

Fig. 2. Shear wall conceptions

The short description of shear walls conception's development in precast prestressed skeleton will show the interaction between theoretic research and practice. First idea was to accept full action of lateral forces by shear walls consisting of two columns and strong reinforced wall. To achieve this goal, shear walls were cast in site with continual reinforcement from foundation to the top of the building. This solution was theoretically good, but the execution takes time and cost. The research was made how to get similar precast structure. Next solution was precast wall with holes for reinforcement which permit to put necessary reinforcement bars after assembly of the structure and to cast those holes. The production of this kind of walls was not easy. Research continued. In the practice some solutions of shear walls position in the lay out of buildings connect two or three shear walls in unity as cross or T, with very stiffened structure. This structure of shear walls accepts strong bending moments and makes troubles how to accept it on level of foundation and through beams of skeleton. The recommendation was to use as much as possible only simple composition of columns and wall, what increases number of shear walls in both directions. It was necessary to permit to designer some opening for doors and windows in the middle of shear walls. Heretic idea than appears - we do not need continuation of walls membrane and its role could be only connection of two columns to achieve commune acceptance of lateral forces actions. It was necessary to test this solution theoretically and experimentally (Dimitrijevic,1992b,), as the idea to use thin concrete membrane between columns as absorber of seismic energy during earthquake. That could be possible because the membrane does not have any load bearing role in acceptance of the vertical load, so some cracks in the membrane can not decrease total structural safety

## MODELS DESCRIPTION

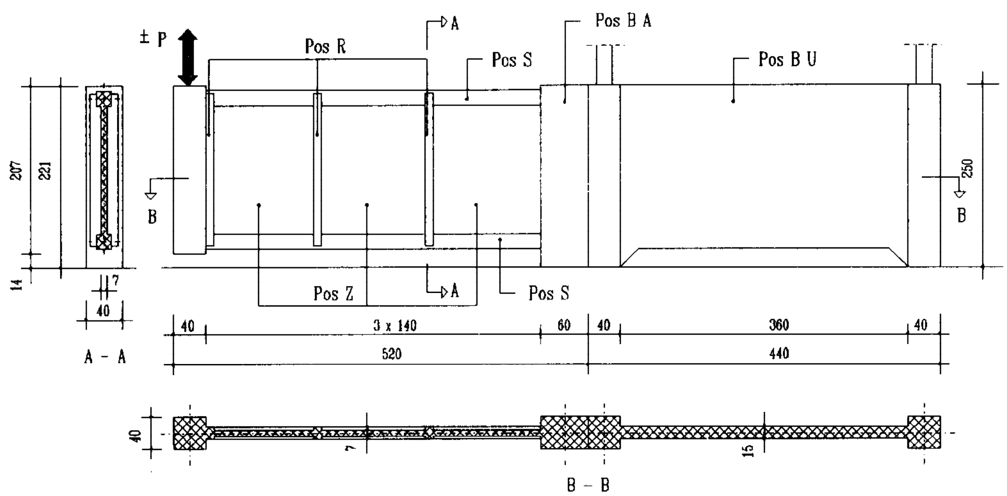


Fig. 3. Models scheme

The model represents a lower half part of six stories buildings shear wall of IMS skeleton system, consisting of two columns and concrete membrane between them on scale 1:2 Fig. 3. The model is composed of precast elements: 3 membranes, 2 tree-stories columns, 1 anchor and 1 loading bloc, 3 pairs of precast skeleton beams which are post tensioned together with columns to get skeleton frame. All elements are connected with very stiffened concrete bloc-wall by longitudinal tensioning of tendons for all 4 tests example. The assemble of model was made in vertical position, as it is done in practice, but loading is made in horizontal position for technical reasons. The vertical load is simulated by longitudinal tension of 4 tendons diameter 12,5 mm, with total force of 238,8 kN. Tensioning of every pair of skeleton precast beams, simulated floor slabs, is done by one tendon diameter of 15,2 mm. System of prestressing is SPB IMS. Tests program considers two series of testing, first with half intensity of axial force in skeleton beams (67,5 kN), and second with full axial force (135,o kN) to get answer how much the unity of columns and membrane depends on the axial forces intensity. In every series of testing the model is loaded until the collapse, than it was repaired by injection of epoxy emulsion and again loaded to see if it is possible to use membrane successfully and make ready for a new loading.

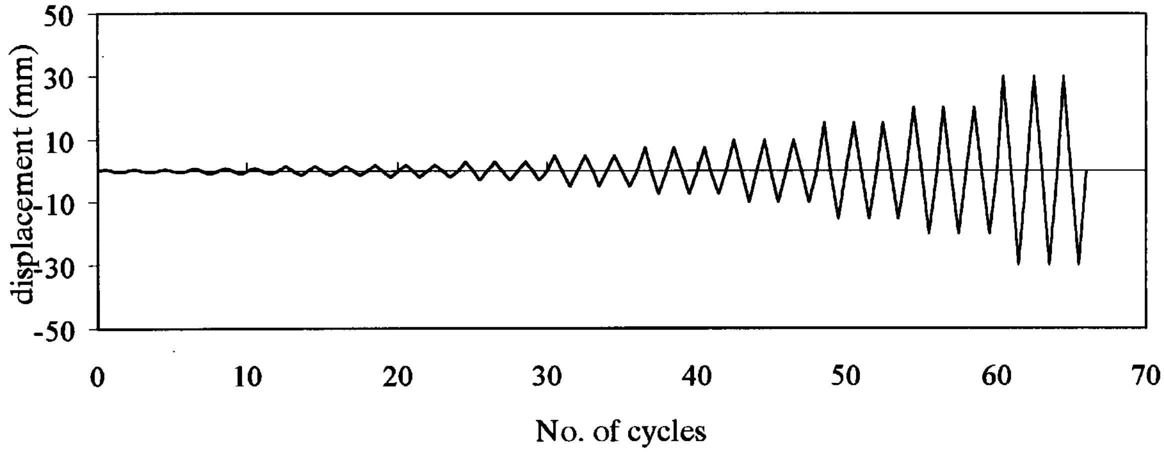

Fig. 4. The loading treatment

Examples 1 and 3 were intact models, and 2 and 4 repaired. The idea of tests was, also, to examine possibility if necessary reinforcement can be added next to columns and cast in site in a small part of membrane to make shear wall of precast elements able for acceptance of bending moments done by lateral cyclic forces. .Those 2 steel bars was connected in anchor bloc simulated foundation with dowel. The same bloc was used for connection of precast columns, 4 bars for each column. The quality of concrete was 40 mPa. The percentage of reinforcement in columns was 1,087 %, in concrete membrane 0,555 %. The loading was made by hydraulic jack managed by controller MTS 406.11. with maximal capacity of force of 1000 kN. All displacements were measured by inductive deformeters HBM and dilatation by strain gauges. Testing results were registered by using amplificator HBM 3073, skinner UPM 60 and PC AT-386.The loading is given on Fig. 4.

## RESULTS OF TESTING

Results of testing give some answers to load bearing capacity, stiffness, deformations and energetic parameters as dissipation of energy and effective ductility.

### Load Bearing Capacity

Analytically was got critical bending moment $M_{cr}$ and horizontal force $H_{cr}$ depending of tension strength in concrete without cracks as were obtained as experimental corresponding values. Models SW2 and SW4 were repaired after collapse, so they had some cracks from the beginning.

| | $M_{cr}$ (kNm) | $H_{cr}$ (kN) |
|---|---|---|
| Theoretical | 250.0 ÷ 369.3 | 58.0 ÷ 83.9 |
| SW1 | 164.6 | 37.4 |
| SW3 | 206.8 | 47.0 |

### Maximal Load Bearing capacity

The software ISDS was used to calculate analytical value for cantilever beam based on 4 hypotheses: A. as a column, B. as a truss, C. as infield frame with wall and D. the same frame, but frame's columns are treated as two different sections - one the column itself and second the reinforced part of concrete membrane cast in site. On those base it was got maximal load bearing capacity for bending moments M*lim* and horizontal forces H*lim*, which can be compared with experimental ones.

| | Theoretical | | | | Experimental | | | |
|---|---|---|---|---|---|---|---|---|
| | A | B | C | D | SW1 | SW2 | SW3 | SW4 |
| M*lim* (kNm) | 831.7 | 935.0 | 871.2 | 435.8 | 1059.1 | 1076.2 | 1038.4 | 1139.6 |
| H*lim* (kN) | 189.0 | 212.5 | 198.0 | 99.0 | 240.7 | 244.6 | 236.0 | 259.0 |

Analytical models A, B and C gave very close results to experimental ones, but the model D where the column is treated as compose of two part the total capacity was much smaller and not real.

<u>Stiffness</u>

We can consider two cases, the stiffness before appearing of cracks (elastic) and the stiffness after (plastic). The measure of stiffness K could be determine as reciprocal value of displacement *f* of the top of the cantilever produced by corresponding horizontal force.
Analytical values of K*e* (elastic) for analytical model of calculation A and C are compared with experimental.
Analytical model determines the force H*e lim* = 25,0kN when first crack for model A appears and we can compare it with experimental results. Displacements after appearing cracks are calculated by using software CREEP for horizontal forces of 200 kN and 250 kN and compared with experimental ones, too, as maximal displacements in the moment of collapse.

| | | Theoretical | SW1 | SW2 | SW3 | SW4 |
|---|---|---|---|---|---|---|
| K*e* (kN/cm) | | A 703.1<br>C 925.9 | 471.7 | - | 809.7 | - |
| *f* (mm) | H=25 kN | 0.53 | 0.53 | - | 0.31 | - |
| *f* (mm) | H=200 kN | 9.15 | 16.5 | 14.0 | 10.1 | 15.4 |
| *f* (mm) | H=250 kN | 14.75 | 30.0 | 35.0 | 17.5 | 20.4 |
| *f* (mm) | Collapse | - | 40.0 | 40.0 | 30.0 | 50.0 |

<u>Ductility</u>

We can consider the effective coefficient of ductility $m$ as relation of maximal deformation in the moment of collapse and deformation in the beginning of creeping. Experimental results are:

| | SW1 | SW2 | SW3 | SW4 |
|---|---|---|---|---|
| $m$ | 5.67 | 2.34 | 4.17 | 5.33 |

For reinforcement RA 400/500 the satisfied coefficient $m = 4 - 5$ . Given experimental data are expected, except for model SW2, and explanation could be the fact that some of strength gauges release, so the beginning of the creeping of reinforcement was not correctly determined.

<u>Dissipation of energy</u>

The measure of dissipated energy can be calculated from hysteretic loop area Fig. 5. from diagram of loading and disloading in relation of force and deformation. If diagram of dissipated energy Fig.

6. show increasing of deformation and dissipated energy, the structure has characteristics for energy absorption under cyclic loading. Diagrams of experimental results show that the increasing of dissipated energy and displacement correspond to the square curve, it means has possibility of dissipation.

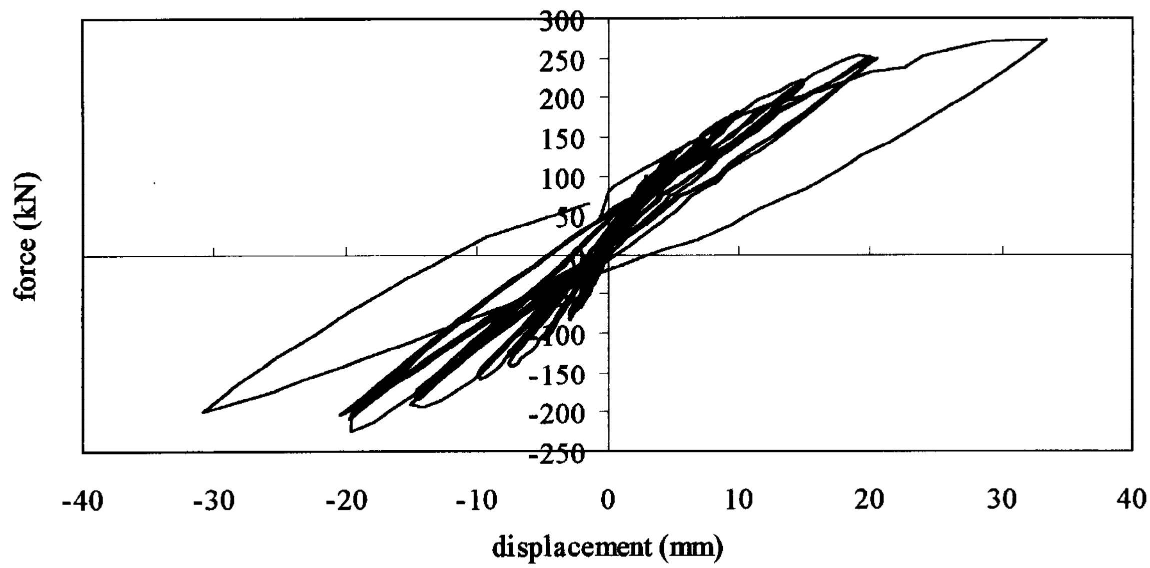


Fig. 5 Hysteretic loop

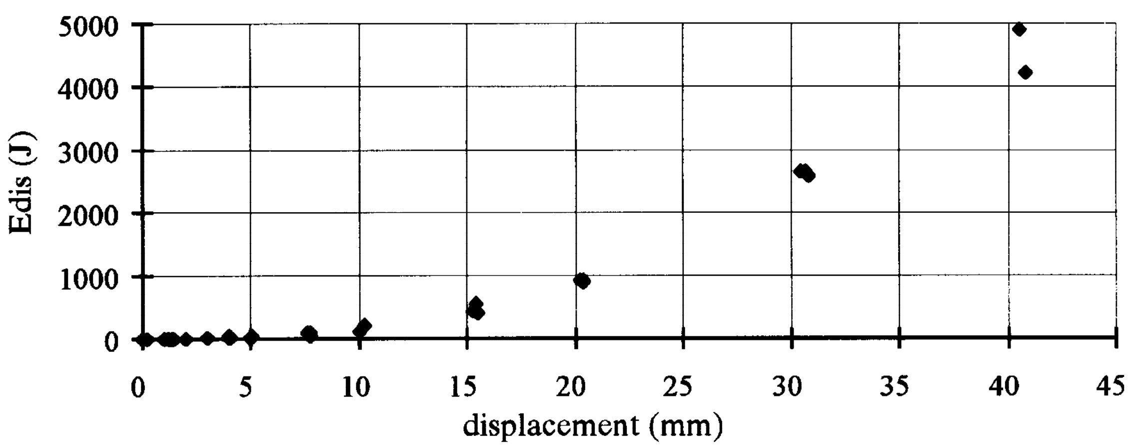


Fig. 6. Diagram of dissipated energy and displacement

## CONCLUSION

a. The load bearing capacity of the shear wall in the moment of first appearance of cracks given by experiment is smaller then analytical one.
b. The experimental maximal load bearing capacity in limit state is a little bigger than analytical, it means that applied analytical methods are useful and correct for this kind of shear walls. For

cantilever structure the stiffness does not have influence on determination of distribution of forces. But the stiffness of the model got by analytical model before appearance of cracks is bigger than experimental one and much more bigger in limit state before collapse. For that reason it is recommended to decrease the stiffness for about 30 - 40 % in calculation, what is on the side of safety.
c. The analysis of stiffness show very important influence of the intensity of axial force - prestressing. The model SW3 which is prestressed with full prestressing force show good agreement between experimental and analytical results. In the same time it approved possibility which designer has playing with the intensity of prestressing force.
d. Deformability of the shear wall depends on the intensity of the axial force, and experimental displacements are bigger than analytical, especially on models with half of the prestressing force.
e. Diagram of dissipated energy and deformation show very positive characteristic of that kind of shear wall to dissipate energy with satisfy ductility.
f. The precast part of the shear wall had a role of energy absorber in all models. The restoring of model was easy and satisfied.

## REFERENCE

Dimitrijevic,R.(1984). Shear walls in prefabricated prestressed skeleton system IMS. Third international symposium on wall structures. Vol. I. pp. 113-115. Warsaw.

Dimitrijevic,R.(1992). Quality control and corrosion and their influence as regards prestressed skeletons. FIP Symposium. Vol. II. pp. 255-262. Budapest.

Dimitrijevic,R.(1992). Behavior of semi-rigid prestressed connections of concrete structural elements under cyclic loading. X world conference on earthquake engineering. Vol. VI.pp. 3127- 3130. Madrid.

Dimitrijevic,R.(1994). Prestressing technology in housing Yugoslavs experience.XII FIP Congress. National report. pp. 65-82. Washington.

Jurkovski,D. et all. (1978). Forced vibration full scale tests on five buildings constructed by industrialize methods. Symposium on research on the field of earthquake resisted design of structure Vol. I. pp. 77-115. Dubrovnik-Cavtat.

Madaras,G. and K.Szilassy.(1990). Characteristics of joints of DUNA TESIT (IMS) structure and possibility of its straightening. Conference on IMS system. Internal report. 36. Pech.

Muravljov,M.(1974). Measuring of friction coefficient in the joint of column and floor slab. Bulletin IMS. 3, 21-22.

Petrovic,B.(1965). Testing of joints between floor slab and shear wall in IMS skeleton system. Internal report of IMS. p. 38. Beograd.

Petrovic,B.(1967). Measuring logarithmic decrement on a building in IMS system. *Nase gradjevinarstvo.* Vol XXI. 5, pp. 97-105. Beograd.

Petrovic,B.(1978). Testing of models of some IMS elements and their joints. Symposium on research on the field of earthquake resisted design of structure. Vol.1, pp.43-76.Dubrovnik-Cavtat.