

POST-ELASTIC BEHAVIOUR OF PLAIN MASONRY SHEAR WALLS

A BERNARDINI, A CASELLATO, C MODENA, R VITALIANI Assistant Professors
Istituto Costruzioni, Ponti e Strade, University of Padova, Italy

ABSTRACT A numerical simulation of the non linear behaviour of a multistory shear wall, subjected to increasing lateral, in plane, actions is presented. Considering the influence of the wall openings, the results indicate that the coupling effect of the horizontal plain masonry beams is active, when extended crack conditions are attained too. The conclusions are discussed with reference to some simplified structural models and Code provisions for shear walls design.

1. INTRODUCTION

The resistance of masonry bearing walls systems connected by the floor slabs and subjected to lateral actions (e. g. wind or earthquake actions) and to a quasi-permanent vertical load, is essentially dependent on the strength of the walls loaded in their middle plane.

This statement is generally true for modern masonry building constructions, whose floor slabs can be considered rigid in their middle plane. In the oldest existing buildings more frequently the in plane deformability of the floors (e. g. timber floors) is not negligible, and furthermore their connections to the walls is lacking or not reliable (e. g. when friction effects are predominant).

In such cases a flexural out of plane mode of failure of the walls can be expected, particularly in seismic situation. However, using some well known, not very expensive, strengthening techniques (e. g. steel ties connecting the walls at the floor levels) this type of failure can be easily prevented.

The evaluation of the in plane shear (1) and flexural (2) resistance of a single not perforated wall can be performed using some experimentally tested failure criteria, both for one-directional slow actions and for cyclic, low or high frequency, actions (3) (4).

On the contrary, when the wall is perforated with the door or window openings, a safe and realistic estimate of the corresponding strength decrease is not easy to obtain.

In the cases of "regular" building constructions, the structural model of each multistory wall corresponds, at every storey, to a system of piers coupled by the floor slab and by the masonry spandrel beams over and under the openings.

A) A first very simple model may be obtained disregarding the coupling effects. For example, the CIB W23A International Code (5) for masonry structural systems recommends an analysis procedure based on the linear elastic behaviour of the vertical masonry strips acting as cantilevers.

For high-rise buildings such model frequently leads to flexural effects at the ground level not consistent with the strength of plain masonry sections of the walls; moreover such model do not agree with the experimental observation of the mode of failure, as can be watched in building constructions of such type, subjected to relevant earthquake ground motions.

B) For this reasons, the safety of low-rise masonry buildings in seismic zones is frequently checked considering the hypothesis of a rigid coupling of the piers (6), eventually controlling the shear resistance of the spandrel beams (7).

C) This procedure has been criticized (8), particularly with regard to the flexural resistance of the beams, and consequently some non linear procedures based on frame models of the vertical walls, connected to the horizontal strips and with an elastic-brittle behaviour of the coupling beams, have been proposed. However the resulting strength, obtained using a complex step-by-step procedure, is not very different from the levels determined by means of the simpler procedure indicated in A), and therefore the real resistance of such building constructions seems to be under-estimated.

The present paper describes a numerical simulation to verify the effective behaviour of a high-rise multistory perforated wall, particularly to demonstrate the coupling efficiency of the spandrel beams in the cracked stages. The considered case-study is a masonry wall of an existing building in Naples, seven storeys high, built up using natural volcanic stone ("tufo") units.

2. FINITE ELEMENTS MODELS

The numerical analyses have been performed using an original

finite element program (CRACK (9)), developed to examine the mechanical behaviour of plane or axisymmetric structures, taking into account the non linear material properties of concrete or masonry assemblages and reinforcing steel bars.

In Fig. 1 the uniaxial stress-strain constitutive law for the indicated masonry type is displayed. Some available experimental data (10) have been employed, and the procedure based on the "equivalent uniaxial strain" has been used for the biaxial states of stress (11).

The geometrical dimensions of the wall and a first finite elements mesh are shown in Fig. 2. This model has been used to obtain qualitative informations about the overall behaviour of the wall, but it cannot be employed to analyse correctly the stress-strain distributions in the piers and in the coupling beams. To this purpose a more refined model of the more interesting part of the wall (the first and second storeys) has been considered (Fig. 3).

Assuming the hypothesis of a constant value, at every point, of the acceleration response to the seismic action, the model indicated in Fig. 2 has been employed to determine the nodal forces to be applied to the upper nodes of the model indicated in Fig. 3 (12). The force intensities correspond to the effects of the dead-weight of the wall and of the vertical quasi-permanent load applied to the floors (action W) and to a value of the acceleration response equal to 0.04g (action P).

Maintaining constant the action W, the action P has been increased until the level 9P, corresponding to the acceleration response equal to 0.36g.

The characteristic value of the mean acceleration response to assess the reliability of the existing masonry buildings in Naples (using a ultimate limit-state procedure) is equal to 0.16g.

The program out-put gives the stress levels at the 9 Gauss integration points of each element, and the displacements of each node.

3. NUMERICAL RESULTS

Some numerical results regarding the stress distributions are summarized and displayed in Figs. 4,5,6,7,8,9,10,11.

Figs. 4 and 5 show the stress distribution on the middle horizontal section of each pier of the first storey; Figs.6, 7

the shear, normal and bending parameters on the pier sections, obtained by means of numerical integration.

Figs. 8 and 9 display the compressive principal stress distribution, crack pattern and nodal displacement at the higher loading levels.

At the lower loading levels, the stress distribution is influenced in the lateral piers by the not negligible shear effect due to the vertical W action. For $P=0$ the wall is uncracked, except some flexural cracks at the middle sections of the spandrel beams.

Increasing the lateral action to the level $1P-2P$, extensive flexural crack conditions of the spandrel beams are observed; at the level $5P-6P$ the diagonal (shear) cracking is diffused on the piers of the first storey, and the wall stiffness decreases rapidly (Fig.12). At the level $8P-9P$ an ultimate failure condition is attained, due to the compressive yielding and crushing (flexural failure) of the lower sections of the piers n.3,4,5 of the first storey.

It is very important to observe that the coupling effects of the spandrel beams are not decreasing in the cracking phases, but on the contrary increasing, when the diagonal cracks reduce significantly the stiffness of the piers (see in fig. 7a the distributions of the normal forces in the piers, and particularly, in fig. 11, the distribution of the bending moments at the bases of the piers and of the resisting moment due to the variation of the normal forces in the piers, equilibrating the external overturning moment M_r).

The distribution of the principal compressive stresses (Fig.9) clearly indicates the formation of diagonal struts in the coupling beams and in the piers, due to the compressive stresses active in the finite elements with one directional cracks (Fig. 10).

The distributions of the shear forces H in the piers (Fig. 6) is proportional to their elastic stiffness in the first stages; at higher loading levels on the contrary the shear H (i.e. the horizontal components of the diagonal compression force in the pier struts) assumes the maximum value in the pier n.4 in the first storey, in the pier n. 3 in the second storey.

The corresponding values of the bending moments M (Fig. 7b), are rather low, and the adimensional ratio $2M/hH$ decreases to values near to 2 at the first storey (Fig. 10), where different restraint conditions between the upper and the lower sections of the piers occur. At the second storey the corresponding values

are less than 2 ,demonstrating a contra-flexure mode of deformation.

4. CONCLUSIONS

A numerical analysis of the mechanical behaviour of a high-rise miltisory wall subjected to a in plane lateral action shows that the coupling effect of the spandrel beams is active in extended cracked conditions too.

The results confirm the possibility to control the safety of the considered building under seismic action using a simplified procedure (POR (6); Fig. 12) wich assumes rigid coupling of the piers. It is worthnoting that the model here considered disregards the contributions of the floors slabs and/or steel ties to the stiffness and strength of the coupling beams; the contribution of the transversal walls is also disregarded.

Considering the general features (total and inter-storey height, spandrel beams slenderness) of the examined wall, the conclusion can be applied to the major part of the buildings of the indicated typology, for a wide range of the values of the involved mechanical and geometrical parameters; of course more extended numerical analyses are required in order to find out the limits of the ranges.

The numerical results seems to encourage the research on a simplified analysis procedure based on truss models of the wall, particularly on the diagonal compressive strut models of the spandrel beams and of the piers (13).

The in plane resistance level of the wall indicated by the numerical analysis seems to be adeguate to the design seismic required forces. More research however is necessary to appropriately take into account:

- the effects of cycling on local strength, when extended cracking conditions are attained;
- the real distribution of the inertial forces in the dynamic inelastic response;
- the interaction between in plane and out of plane components of the seismic action in the walls.

5. REFERENCES

- (1) A Bernardini, C Modena, V Turnsek, U Vescovi, "A comparison of three laboratory test methods to determine the shear

- resistance of masonry walls", Proc. VII WCEE, Vol. 7, Istambul, 1980
- (2) V Turnsek, S Terceelj, P Sheppard, "The flexural resistance of masonry walls to combined horizontal and vertical loads", Proc. 6th ECEE, Dubrovnik, 1978
- (3) R L Majes, R W Clough, P A Hidalgo, H D Mc Niven, "Seismic research on multistory masonry buildings. University of California, Berkeley, 1972 to 1977", Proc. North American Masonry Conf., Boulder, Colorado, 1978
- (4) S Terceelj et al. "The influence of frequency on shear strength and ductility of masonry walls in dynamic loading tests", Proc. VI WCEE, New Dely, 1977
- (5) C.I.B., "International Recommendations for Masonry Structures", Pubbl. n. 58, 1980
- (6) Regione Aut. Friuli-Venezia Giulia, "Raccomandazioni per la riparazione strutturale degli edifici in muratura", DT2, 1978
- (7) D Benedetti, M Tomazevic, "Sulla verifica sismica delle costruzioni in muratura", Ingegneria Sismica, n. 0/1984
- (8) F Braga, M Dolce, "A method for the analysis of antiseismic masonry multistory buildings", Proc. 6th IBMaC, Roma, 1982
- (9) A Bernardini, P Rossetto, A Sproccati, R Vitaliani, "A numerical model of plain or reinforced masonry behaviour in post-cracking stages", Proc. 6th IBMaC, Roma, 1982
- (10) A Bernardini, R Mattone, C Modena, G Pasero, M P Pavano, G Pistone, R Roccati, F Zaupa, "Determinazione delle capacità portanti per carichi verticali e laterali di pannelli murari in tufo", Atti II Congr. Naz. ASS.I.R.C.CO, Ferrara, 1984
- (11) D Darwin, D Pecknold, "Non linear biaxial stress-strain law for concrete", Journ. of the Struct. Div., ASCE, V. 103, n. EM2, 1977
- (12) R De Iaco, "Analisi non lineare di una parete in muratura sotto azione laterale", Istituto di Costruzioni, Ponti e Strade - University of Padova, Tesi di Laurea, A.A. 1982/1983
- (13) B Calderoni, M Gulisano, M Pagano, "The method of bonded diagonals", Proc. 3rd Int. Symp. on Wall Structures, Warsaw, 1984

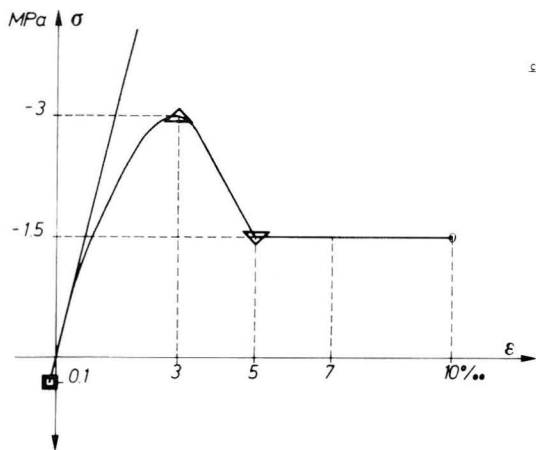
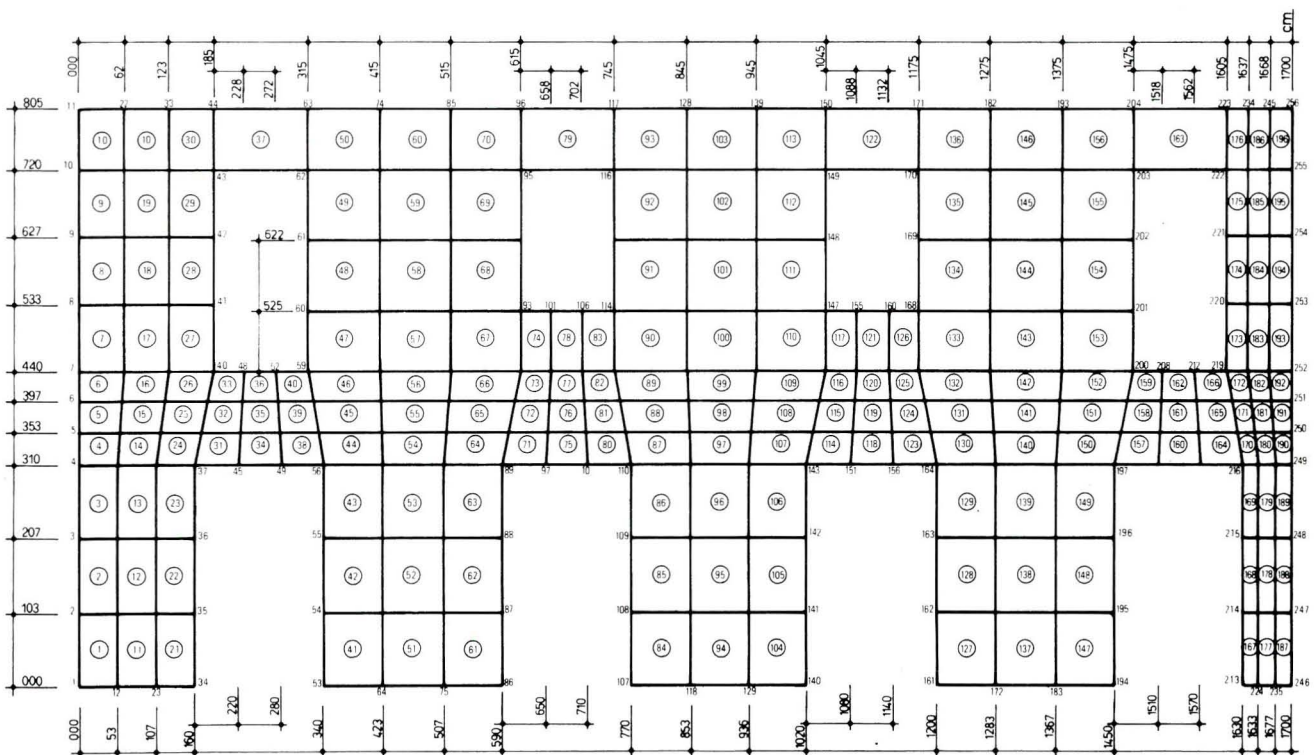
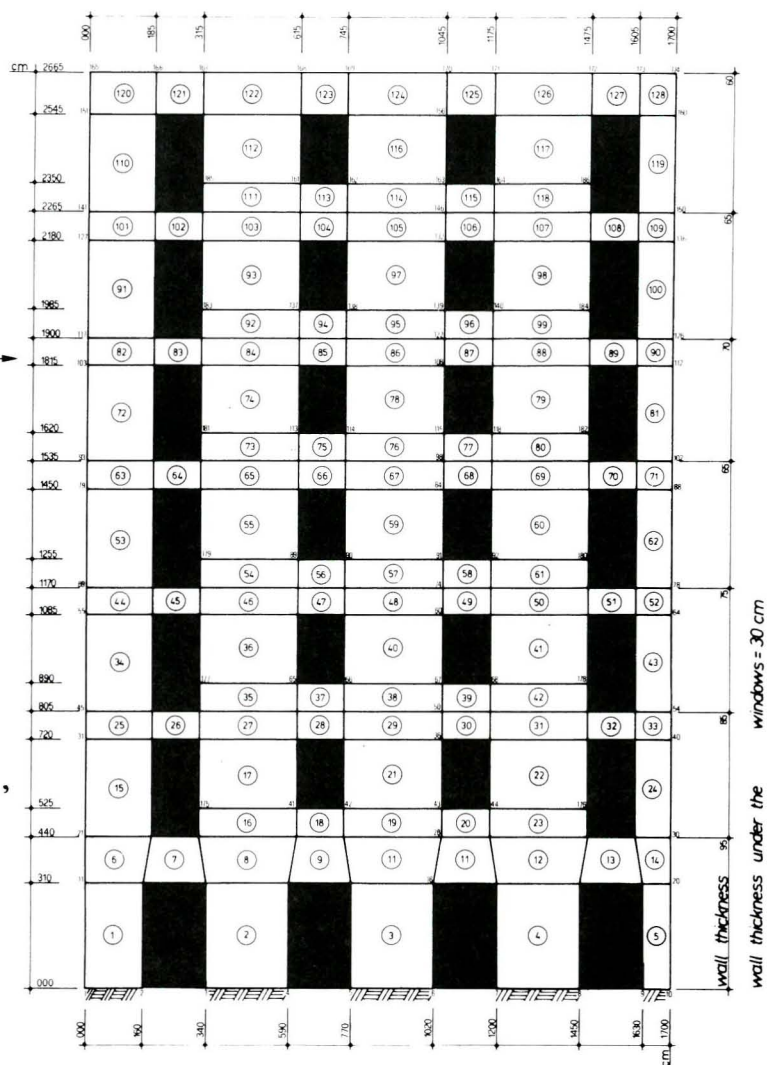


Fig.1 Masonry constitutive law

Fig.2 Geometry and finite element mesh of the overall wall (128, 4 nodes, 9 Gauss points, isoparametric elements, 308 d. of fr.)

Fig.3 Geometry and finite element mesh of the lower 2 storeys (196, 4 nodes, 9 Gauss p., isoparametric elements, 352 d. of fr.)



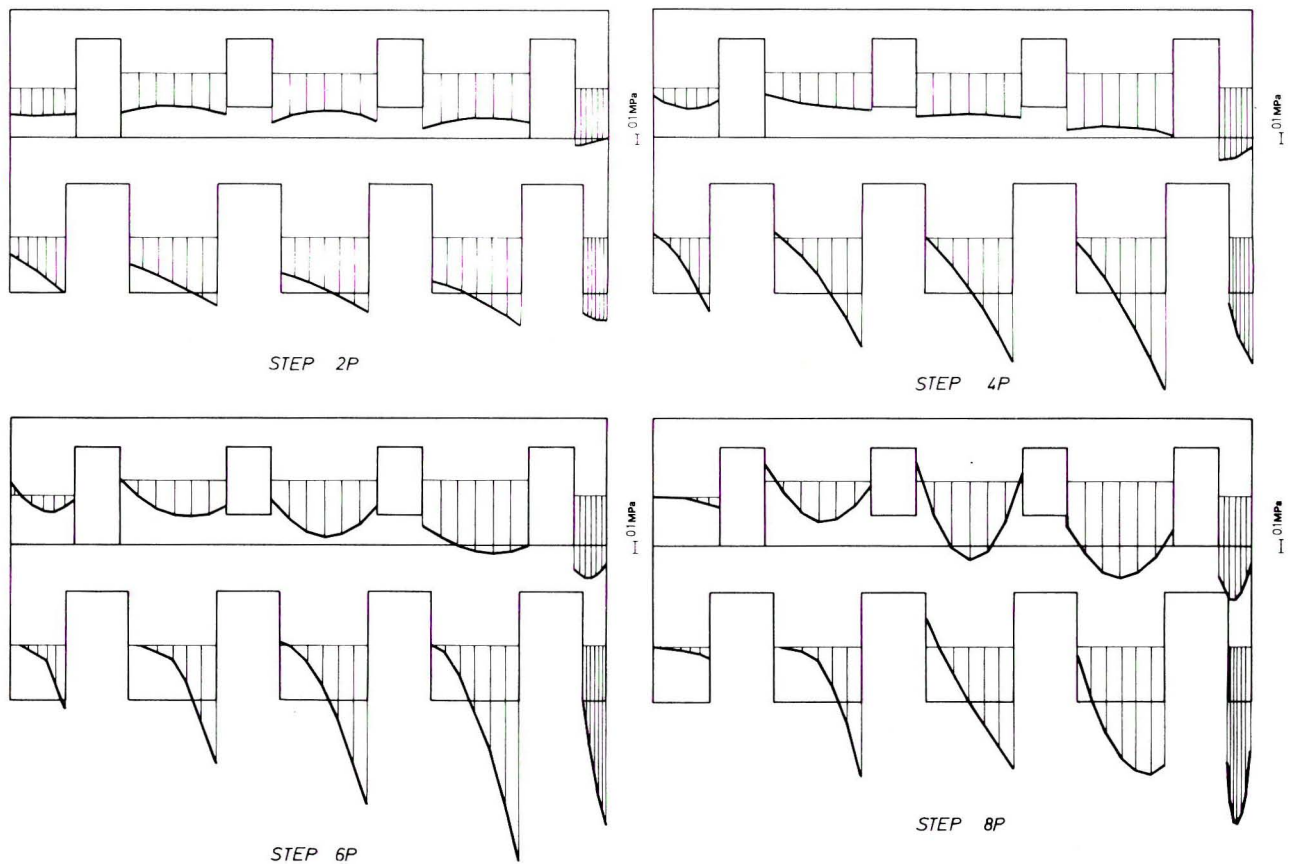


Fig. 4 Vertical stress distributions at the half-height sections of the piers

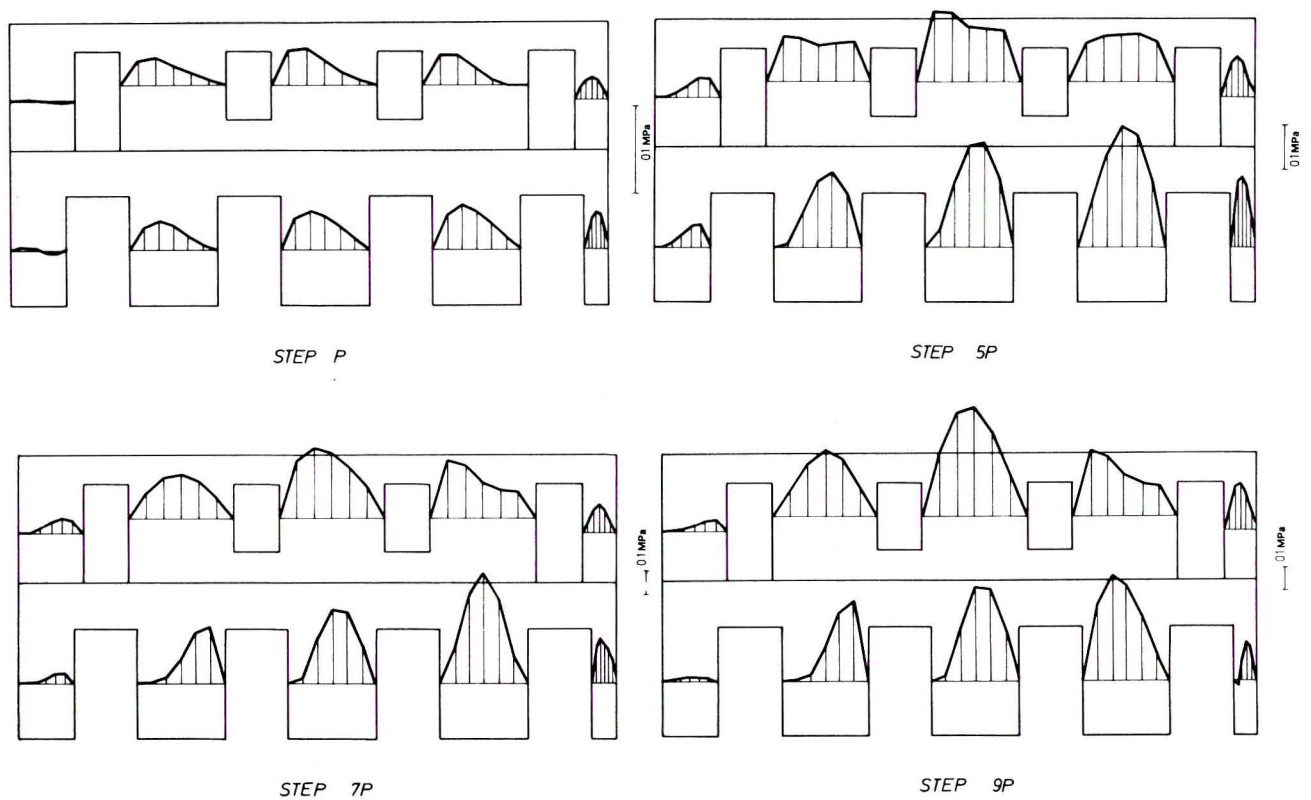
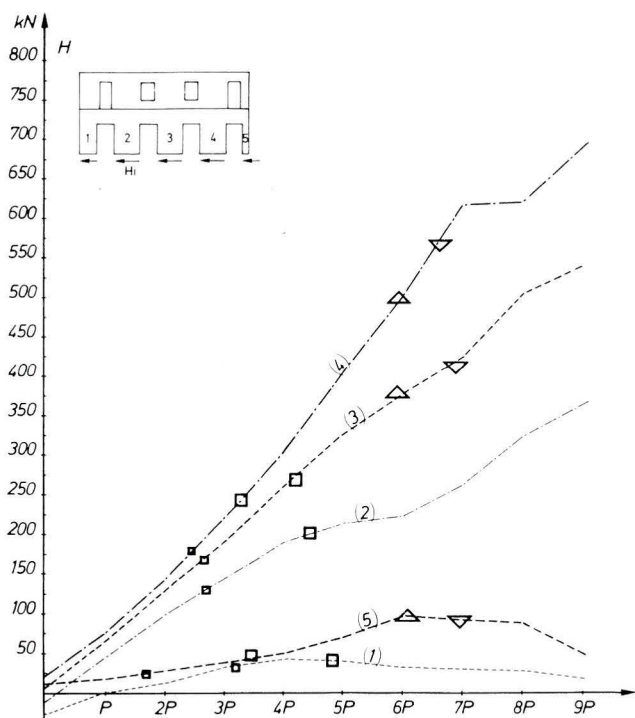
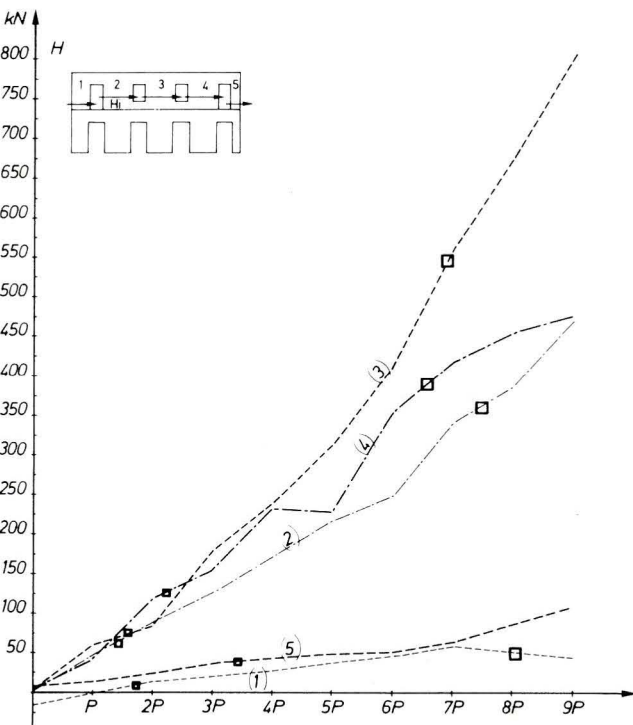


Fig. 5 Shear stress distributions at the half-height sections of the piers



6a

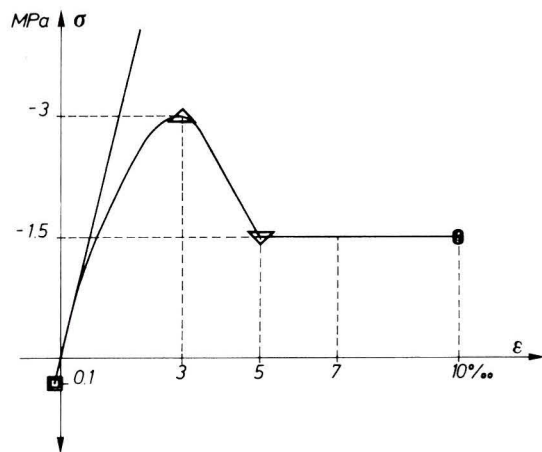


6b

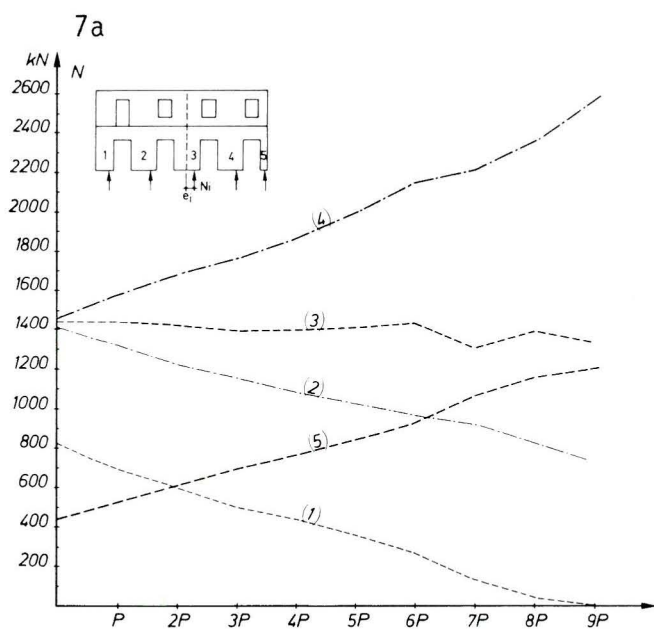
Fig. 6 Shear distributions on the piers

- flexural cracking
- diagonal cracking
- ▲ compressive strength limit
- ▼ compressive yielding limit
- crushing failure

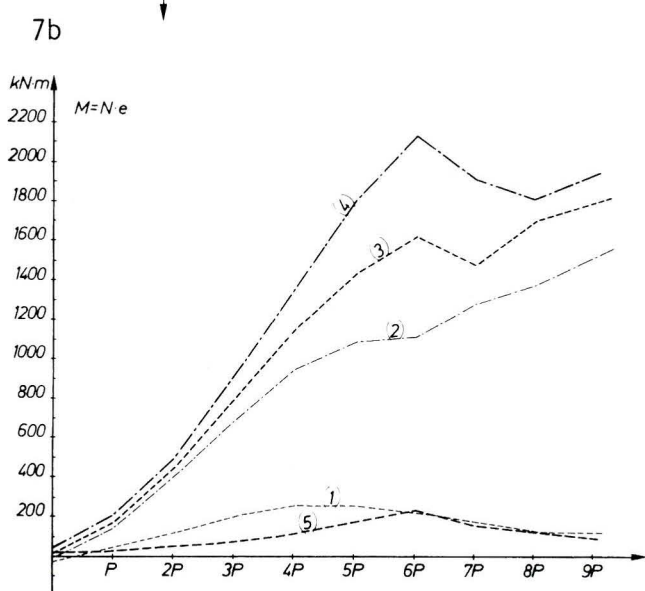
6b



6c



7a



7b

Fig. 7 Normal forces and bending moments (lower sections)-1st storey

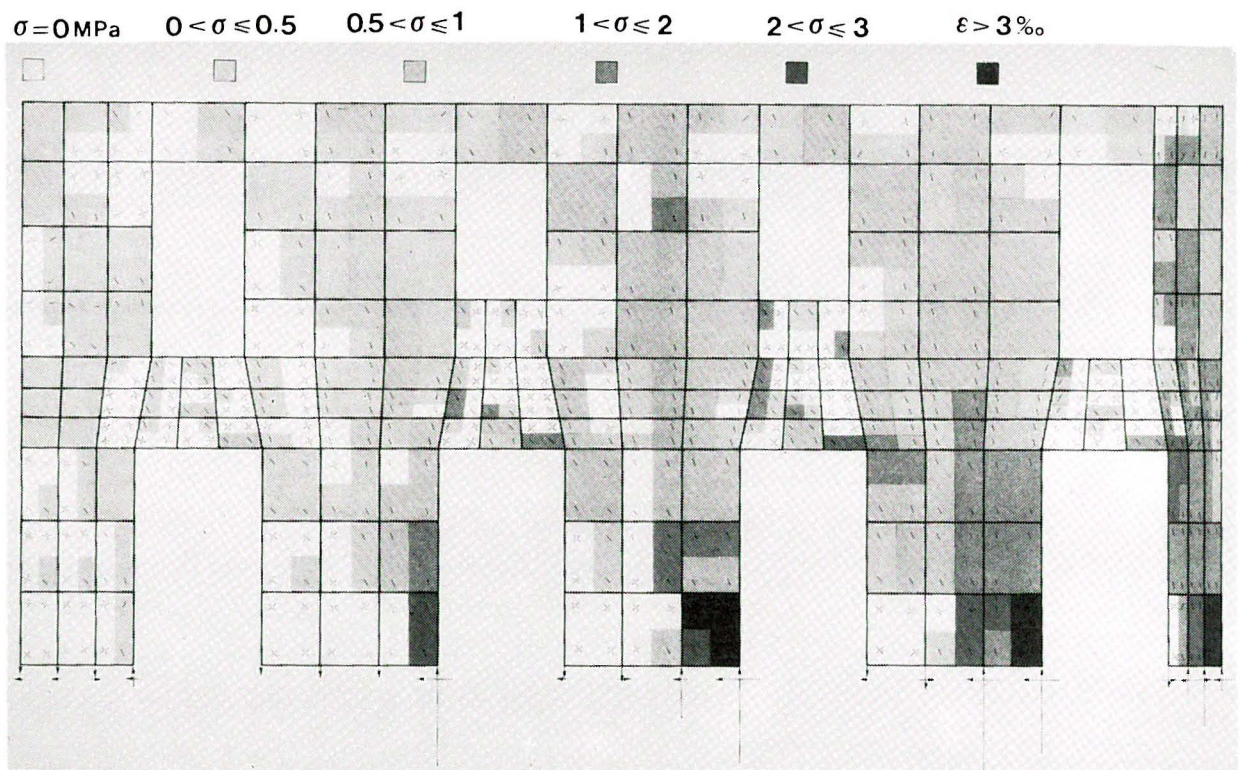


Fig. 8 Support reactions, crack pattern and compressive principal stress levels at the step 7P

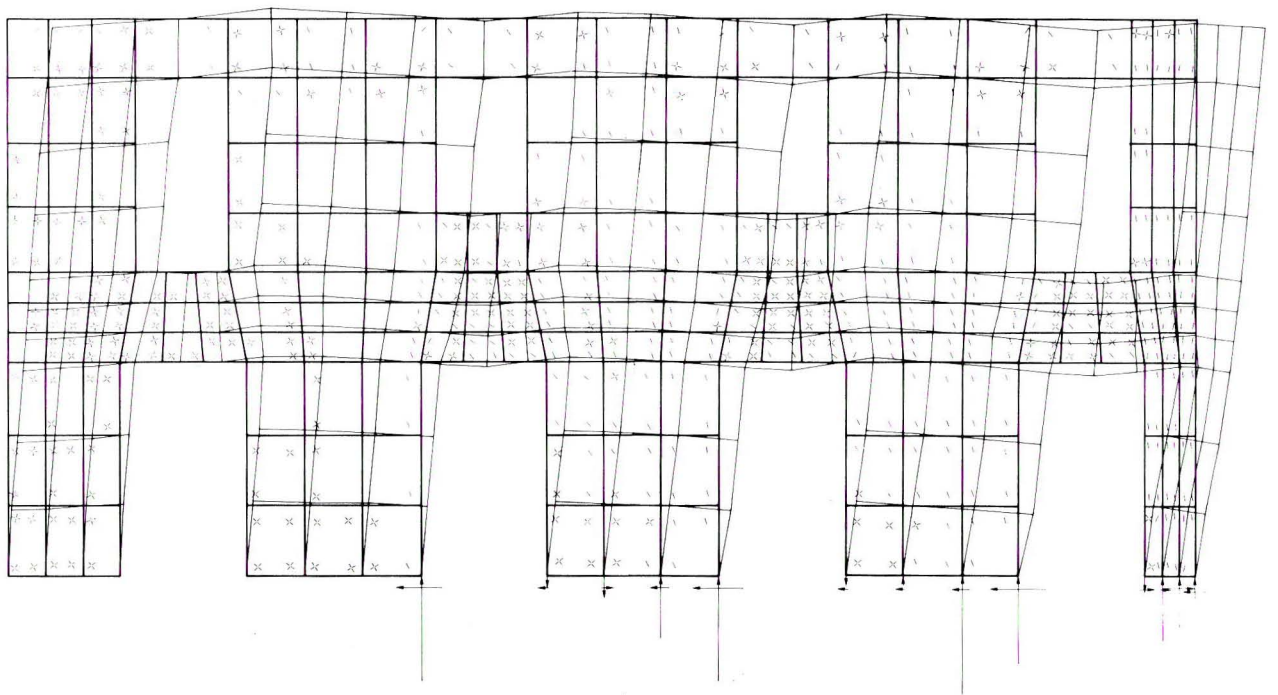


Fig. 9 Support reactions, crack pattern and deformations of the wall at the step 8P

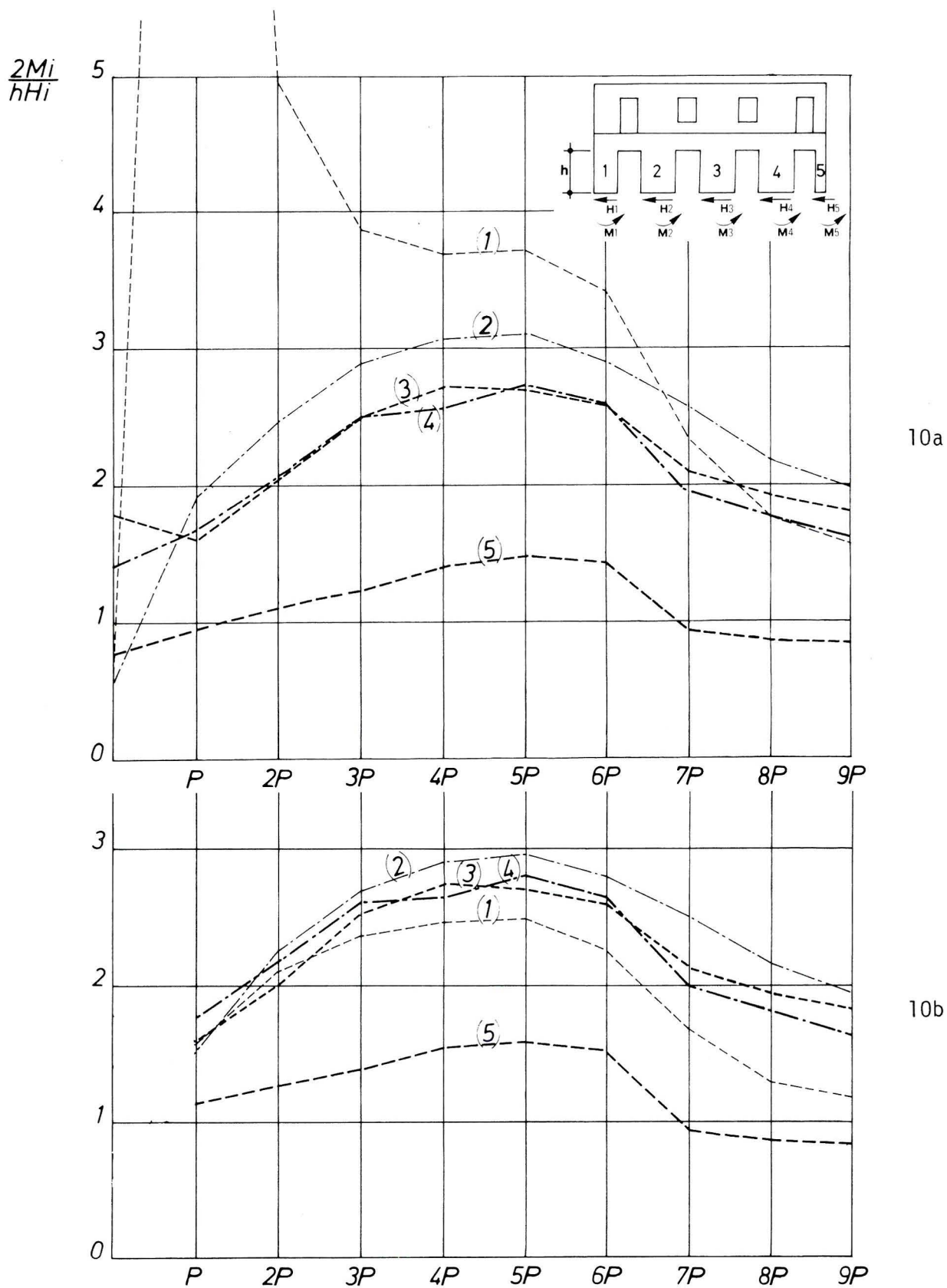


Fig. 10 Distributions of the adimensional parameters $2M/hH$:
a) : $W + nP$ actions b) nP actions ($n = 1$ to 9)

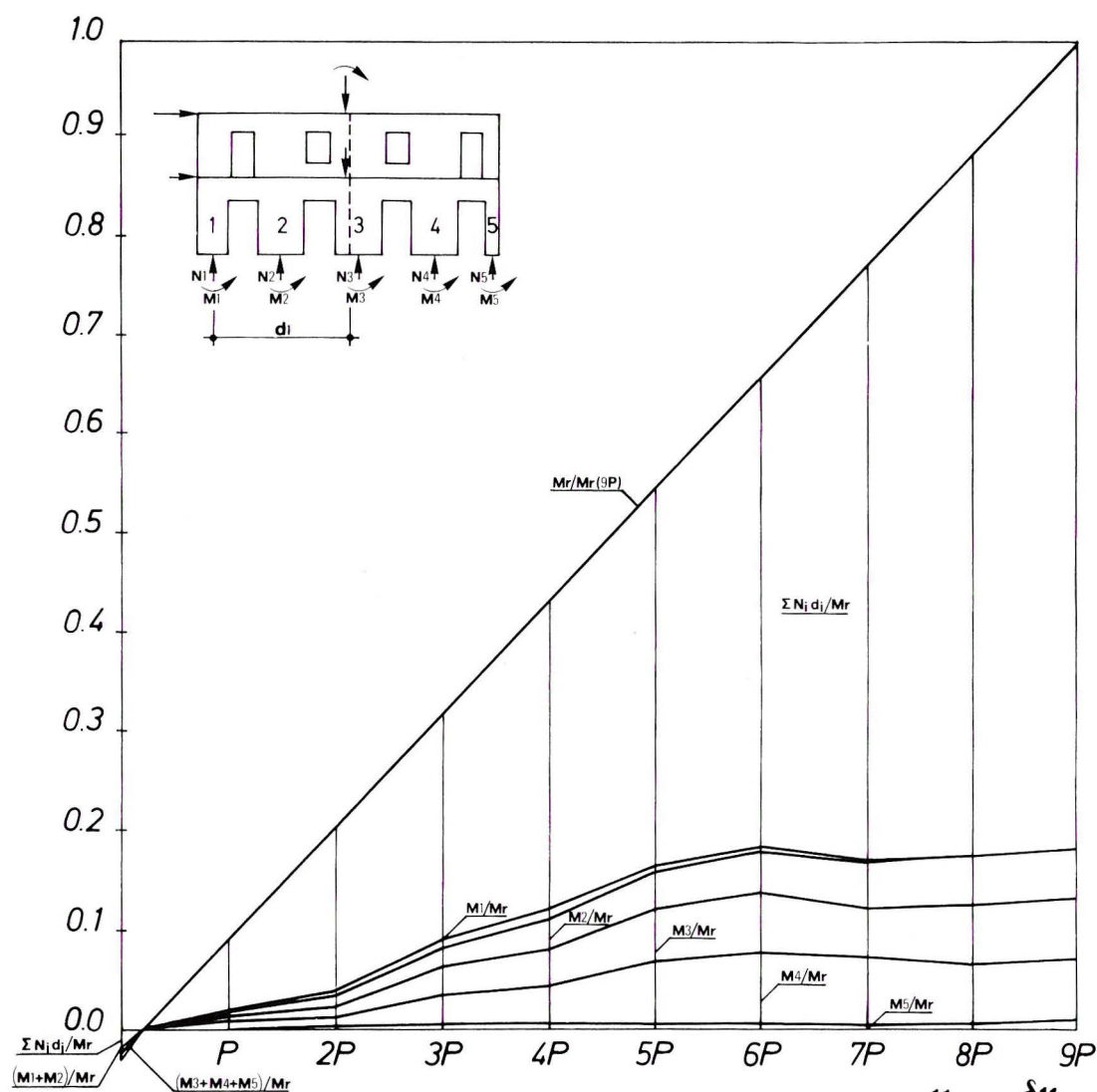


Fig. 11
Distributions
of the bending
moments and
normal forces
equilibrating
the overtur-
ning moment
 M_r .

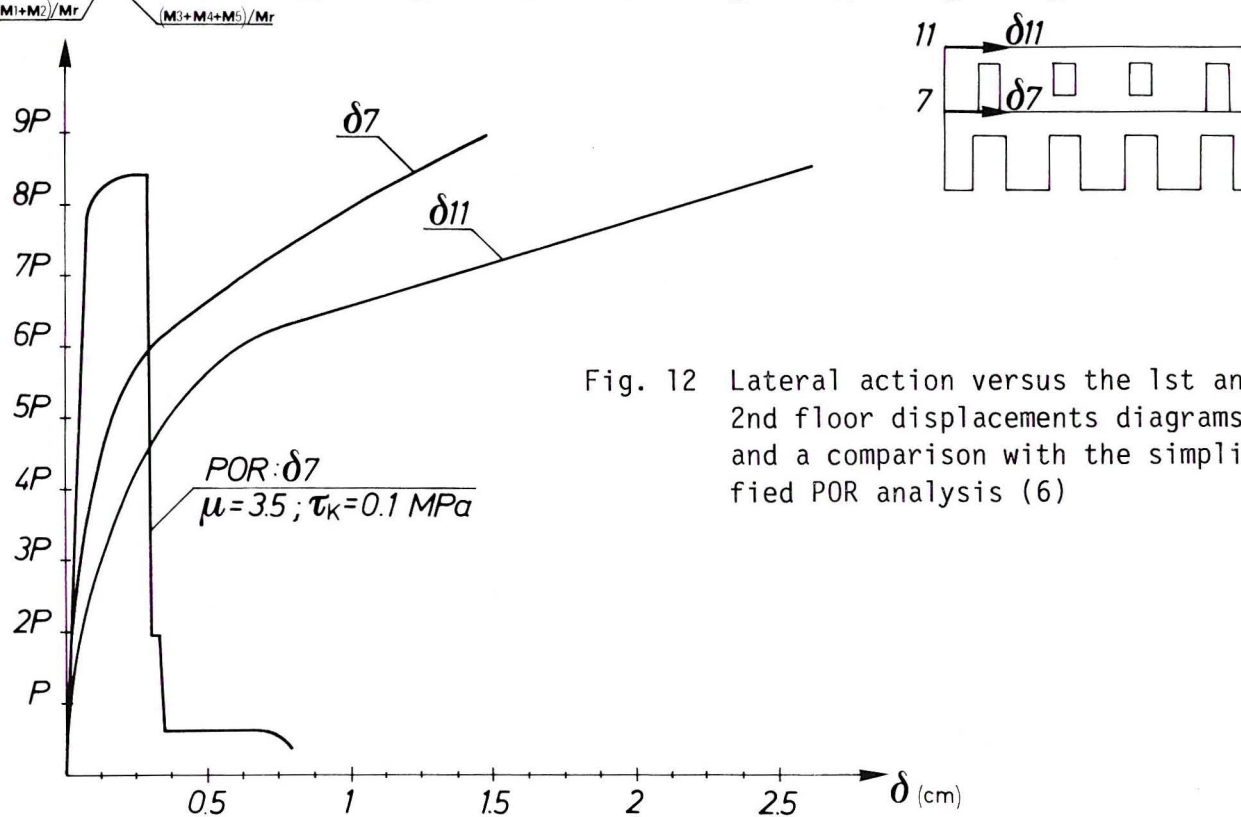


Fig. 12 Lateral action versus the 1st and
2nd floor displacements diagrams
and a comparison with the simpli-
fied POR analysis (6)