NONELASTIC HYSTERETIC BEHAVIOUR OF RC COLUMNS AND INFILLING WALLS

by
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SUMMARY

Investigation of the response of structures to seismic excitation requires the determination of the nonelastic behaviour of the principal structural and nonstructural elements: columns, beams and infilling walls.

Reinforced concrete columns with different cross-section, reinforcement and stirrups are tested for axial load combined with reversive bending and shearing.

Influence of the stirrups space on the bearing capacity and history of distortion is investigated by testing of short columns.

The stiffness degradation, mode of failure, ductility factor and absorption of energy in terms of the vertical and horizontal load, are determined.

The bearing capacity of the columns subjected to compression, shearing and bending in terms of longitudinal and cross reinforcement is analytically represented.

The results obtained can be used for investigation of the response of RC frame buildings with infilling masonry walls in the nonelastic stage.

I. Introduction

The investigation of the structures taking into account the plastic deformations reveals additional reserves of resistance and leads to a considerable decrease of earthquake forces. The elucidation of the earthquake resistance of the structures, taking into account the plastic deformations, requires knowledge of the real deformative properties of the structures (limit load, limit deformation and degree of development of the plastic deformations) beyond the elasticity stage in order to utilize these data in theoretical investigations.

Some results from theoretical and experimental investigations carried out on columns, frames with and without infilling masonry subjected to vertical and horizontal loading are given in the present paper. The investigated elements are models of parts of real structures subjected to the same loading as in real structures.

II. MODELS AND METHODS OF INVESTIGATION

Models of principal structural elements from package-lift slabs buildings and monolithic reinforced frame buildings with infilling masonry have been investigated.

Reinforced concrete columns with cross-section 12/12 cm and 12/20 cm reinforced by 4⌀12 bars (yield stress σ_y^s = 3,400 kg/cm^2) at distances of 5, 10 and 15 cm were tested for

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compression loading ($6_{v} = 0.5\ R^{c}$) in combination with reversive shearing and bending (Fig. 1).

The same columns combined with two slabs form the second type of tested models. Some of the columns are prefabricated and welded to the slabs, and some are monolithically cast in place with the slabs. The frames from Fig. 1 are tested for: static monotonous reversive horizontal loading with and without vertical loading in the columns, response of the models to sinusoidal dynamic loading in elastic and nonelastic stage.

The monolithic and prefabricated frame with infilling masonry are tested at the same static and dynamic programme (Fig. 1).

III. SOME RESULTS FROM THE INVESTIGATIONS

1. The experimental setup for testing the columns makes it possible to create in the models bending moments, shear and compression forces, i.e. conditions corresponding to the real state of the columns in frame structures. From the hysteresis curves obtained it can be seen that the work of these elements can be represented in three stages: elastic - until the appearance of cracks, elasto-plastic - from the appearance of cracks until yielding in the reinforcement and plastic - development of large deformations.

The bending in which cracks appear in the concrete is determined by

$$M_c = W\left(R_t + \frac{N}{F'}\right) \qquad (1)$$

where

W, F' - resistance moment, area of the working section,

N - compression force,

R - tensile strength during bending.

The bending moment upon existence of a longitudinal force, in the case of simultaneous occurrence of yielding in the tension and compression reinforcement, can be estimated using (1) if the distance between the stirrups **exceeds 8 cm.** When the distance between the stirrups is less than 8 cm, the formula for the bending moment will be

$$M_y = W\left[\frac{b}{H}\ \frac{F_s R_s^{7}}{F} - \frac{0.5N}{F}\left(1 - \frac{N}{F R^{c}}\right) - \frac{R_s^{h} D F_s^{h}}{F S}\right. \qquad (2)$$

where

F_s, F_s^{h} - area of the longitudinal reinforcement and the stirrup,

F - area of cross-section,

S - distance between hoops,

R^{c} - the prismatic strength of the concrete,

$R_s^{7}\ R_s^{h}$ - yield strength of the steel and hoops,

D - length of the hoops.

The comparison of the theoretica and experimental data is given in Fig. 2. The results are used for the study of the earthquake resistance of a six-storey frame structure.

2. The experimental studies of frames with and without infilling masonry subjected to dynamic sinusoidal loading can be used to obtain the change of the natural period in function of the amplitude of vibration (Fig. 3). The graphs illustrate

that in the elastic **stages** the natural period does not depend on the amplitude of vibration and conversely, with accumulation of destruction in the models (upper graphs) the period-amplitude relation acquires a markedly nonlinear character. From our and other studies 4 it can be seen that the natural period of vibration changes in the process of destruction of the models. For prefabricated frame with masonry T_o=0.0295 s for uncracked element, T_c=0.0385 s for cracks in the masonry and T_y=0.059 for a model with large distortions, i.e. the period increases about twice.

The relative absorption of the vibration energy found according to the experimental resonance curves is δ =0.18 for a model without cracks, δ =0.63 for a model with cracks in the masonry and δ =0.79 for a model with large distortions:

$$\delta = \frac{\pi}{2} \frac{w_2^2 - w_1^2}{w_R^2} \sqrt{\frac{A^2}{A_{max} - A^2}} \sqrt{\frac{1-2D^2}{(1-D^2)^2}} \qquad (3)$$

where

w_R - resonance frequency,

w_2, w_1 - frequencies from the resonance curve for $A=0.707\ A_{max}$,

A_{max} - resonance amplitude,

D - damping (assumed to be a small value and the last term of the formula is $\approx$ 1.0).

The energy absorption coefficients in the elements $\psi = \frac{\Delta W}{W}$ where ΔW is the energy absorbed in the structure for one cycle, and W is the total elastic energy for one cycle, changes during the different stages of destruction (Fig. 5), three characteristic zones being differentiated. Zone I corresponds to small deformations of the **model**. Upon further increase of the displacement (deformation), it originally increases (zone II) and then gradually decreases (zone III). The horizontal line corresponds to damping according to the norms. The behaviour of the structures beyond the elasticity limits is considered as an "unstable" branch resulting in the development of large plastic deformations and destructions which limit the scope of utilization of the structure. For structures working under earthquake excitation the considerable irreversible deformations are a necessary condition for their preservation from complete destruction. The decrease of the forces in the structure is the result of the change in its stiffness and static **scheme**. In the descending branch of the diagram of energy absorption the structure may still be "stable".

The criterion for the computational ultimate stage is assumed to be the energy dissipation coefficient ε - the ratio of the energy lost for deformation of the structure to the energy for deformation during the elastic stage.

For the descending branch the relation is

$$\frac{\Delta W}{W_y} = \frac{\Delta}{\Delta_y} \left[\exp\left(-\frac{\Delta}{\Delta_y}\right) - \frac{a}{b}\right] \qquad (4)$$

The criterion for the ultimate stage is

$$\varepsilon = (1a/b) \frac{\Delta}{\Delta_y} \Big/ \left[\left(\frac{\Delta}{\Delta_y}\right)^2_{max} \exp\left(-\frac{\Delta}{\Delta_y}\right)_{max}\right] \qquad (5)$$

Some values of ε for $\frac{\Delta}{\Delta_y} = \frac{\Delta}{\Delta_y}{}_{max}$ and $\frac{\Delta}{\Delta_y} = \frac{\Delta}{\Delta_y}{}_d$ are given on Fig. 4.

CONCLUSIONS

The data given in the present work permit the determination of the reserve of the baring capacity beyond the elasticity limits. They give an opportunity to assess to what extent this reserve is used to guarantee the earthquake resistance of the structures and the extent to which the ultimate stage has occurred using criterion (5).

The results of the investigations are used for determining the response of buildings with infilling masonry to real earthquakes, taking into account nonelastic deformation.

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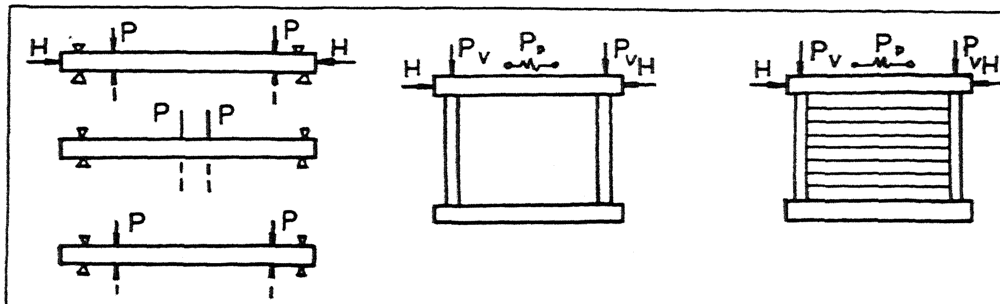

fig 1

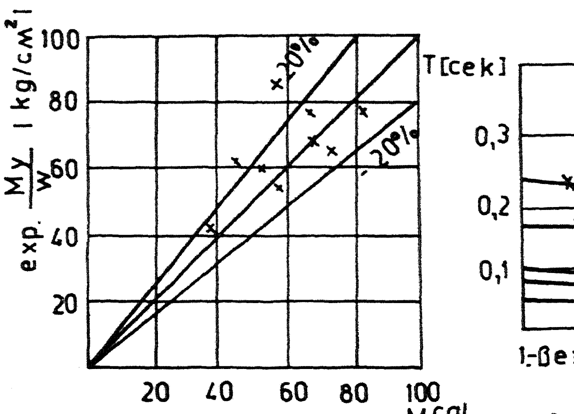

fig. 2

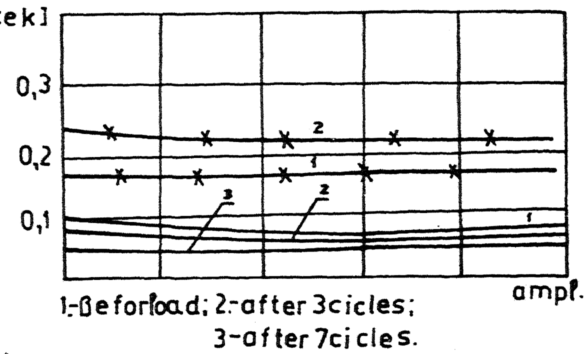

fig.3

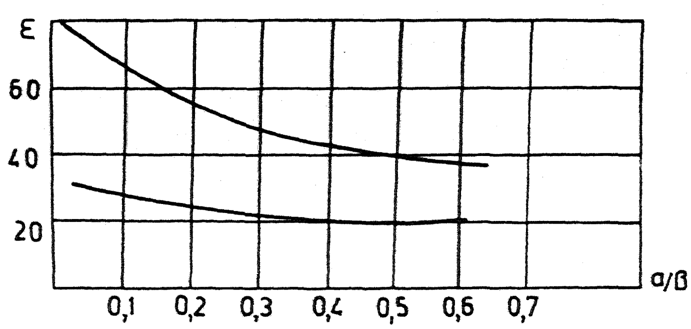

fig.4 monol.frame +masonry

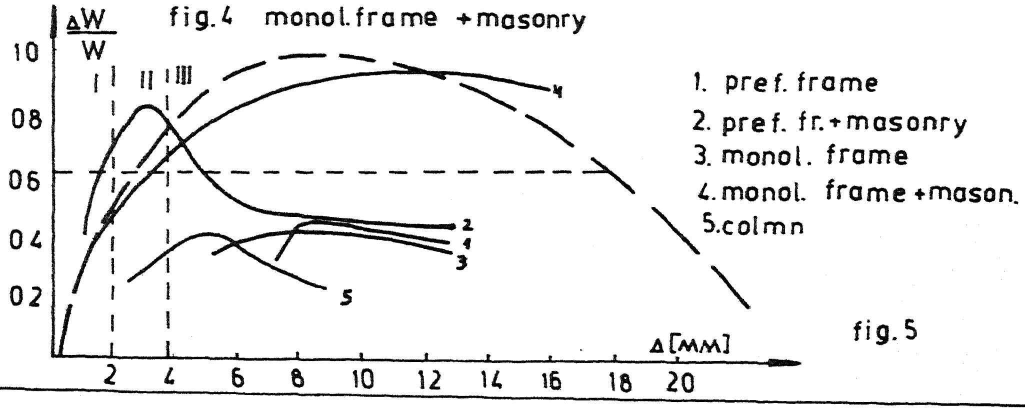

fig. 5