RESPONSE OF LARGE-PANEL BUILDINGS FOR EARTHQUAKE EXCITATION IN NONELASTIC STAGE

by

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SUMMARY

The response of a six-storey large-panel building is investigated taking into account nonelastic deformations in wall-panels connections. The reduced San Fernando 1971 earthquake S16°E component is used.

The equations of motion are solved using a step-by-step integration procedure with changing the stiffness in each successive interval.

Nonelastic strength and deformation characteristics of the wall-panel connections at reversive loading are determined by theoretical and experimental investigations.

Skeleton curves and hysteresis loops, ductility factor and absorption of energy in terms of the vertical and horizontal load history are determined.

The influence of the nonelastic deformations on the redistribution of forces among the vertical diaphragms is investigated.

I. INTRODUCTION

Industrialisation of construction has recently lead to a continuously growing use of large-panels and prefabricated constructions. Using such constructions in seismic zones sets a number of problems.

The construction system of large-panel buildings is formed by vertcal diaphragms in transversial and long direction formed by connection of separate wall-and floor-panels. The resistance ability of the system thus formed to seismic activity depends mainly on the connections between the separate panels and less on the resistance capacity of the panels themselves, which have sufficient strength.Despite the tension concentration in the connections their resistance capacity has to be equal or greater than that of the panels. A number of investigations /2,3,4/are known on the bearing capa city of certain connections in large-panel buildings, but what is needed are investigations on the cyclic load with a development of the nonelastic deformations. The study of the response of large-panel buildings to seismic actins in nonelastic stage is an extremely complicated problem and

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requires a number of assumptions and prerequisites for the formation of a mathematical model as well as a number of data concerning strength and deformation characteristics of the panels and connections.

The purpose of these investigations is to study the response of large-panel buildings to strong earthquakes taking into account nonelastic deformations in the connections.

II. METHOD OF ANALYSIS

The analysis of large-panel buildings as shell structure is extremely complicated due to the presence of many kinds of vertical diaphragms with openings different in size and position. Therefore we will regard this type of constructions as a discrete system with masses concentrated on the levels of the storeys.

In the most general case the spatial response of such a system can be investigated in nonelastic stage for a spatial threecomponent seismic action by using the following system of differential equations:

$$(1)\qquad \begin{bmatrix} [m] \end{bmatrix} \begin{Bmatrix} \{\ddot{T}\} \\ \cdots \\ \{\ddot{R}\} \end{Bmatrix} + \begin{bmatrix} [C_{TR}] \end{bmatrix} \begin{Bmatrix} \{\dot{T}\} \\ \cdots \\ \{\dot{R}\} \end{Bmatrix} + \begin{bmatrix} [K_{TR}] \end{bmatrix} \begin{Bmatrix} \{T\} \\ \cdots \\ \{R\} \end{Bmatrix} = - \begin{bmatrix} [m] \end{bmatrix} \begin{Bmatrix} a_T(t)\{I\} \\ a_R(t)\{I\} \end{Bmatrix}$$

or if w e devide the resistance capacity of the system into two parts - resistance capycity of deformations in the panels and resistance capacity of deformations in the connec tions between the panels then the third member from (1) could be presented in two parts:

$$(2)\qquad \begin{bmatrix} [\overline{K}_{TR}] \end{bmatrix} \begin{Bmatrix} \{T\} \\ \cdots \\ \{R\} \end{Bmatrix} = \begin{bmatrix} [K_{TR}] \end{bmatrix} \begin{Bmatrix} \{T\} \\ \cdots \\ \{R\} \end{Bmatrix} - \begin{bmatrix} [K'_{TR}] \end{bmatrix} \begin{Bmatrix} \{X_T\} \\ \cdots \\ \{X_R\} \end{Bmatrix}$$

where: $\{T\},\{\dot{T}\},\{\ddot{T}\}$ - vectors of displacement, velocity and acceleration along the three axes x,y,z

$\{R\},\{\dot{R}\},\{\ddot{R}\}$ - vectors of rotation, angular velocity and acceleration towards the three axes

$\{X_T\},\{X_R\}$ - vectors of displacement from deformations by shear and axial force and bending moment of the connections

$[m]$ - submatrices of masses of a diagonal matrix

$[K_{TR}]$ - submatrices of panel stiffness

$[K'_{TR}]$ - submatrices of nonelastic stiffness in joints

$[C_{TR}]$ - submatrices of the damping by translation and rotation

$a_T(t), a_R(t)$ - normalized function of seismic acceleration of translation and rotation.

The spatial investigation of the response of multistorey buldings in nonelastic stage by systems (1) and (2) is exceptionally difficult and the following assumptions and simplifications are made: floor slabs are infinitely rigid in their plane and flexible out of the plane; the modes of vibration are independent; the structural response is investigated separately for each axis x and y; the strength and the stiffness of the structure are equal for all storeys in elastic stage only; the stiffness of the connection s is function of the stress conditions and relative displacements; the damping is a viscous and is taken 5% of the critical.

The strength and deformation properties of wall-panel constructions are determined by specific investigations described in (8) and part III. **In these investigations the elements are loaded in such a way as they work in the structure under seismic actions.**

The stiffness matrices of vertical diaphragms are determined by using some simplifications (5,6,7).

The dynamic analysis is carried out by step-by-step procedure assuming that for each short time increment the structure has constant stiffness. The stiffness is changed from one interval to the next and thus the nonlinear response is obtained.

Under these conditions the response of the large-panel building is investigated taking into account nonelastic deformations in wall-panel connections for San Fernando 1971 reduced earthquake S 16°E component.

The forces in the entire structure are distributed among the separate vertical bearing diaphragms proportionally to their stiffness in nonelastic stage.

Response analysis taking into account nonelastic deformation both in floor-panels and joints will be considered later.

The response with soil-structure interaction and nonelastic deformations in a building can be investigated by the following simplified system equations:

$$(3) \quad \begin{vmatrix} [M_s] & [0] \\ [0] & [M_f] \end{vmatrix} \begin{Bmatrix} \{\ddot{x}_s\} \\ \{\ddot{x}_f\} \end{Bmatrix} + \begin{vmatrix} [C_s] & [C_{sf}] \\ [C_{sf}]^T & [C_{ff}] \end{vmatrix} \begin{Bmatrix} \{\dot{x}_s\} \\ \{\dot{x}_f\} \end{Bmatrix} + \begin{vmatrix} [\bar{K}_s] & [K_{sf}] \\ [K_{sf}]^T & [K_{ff}] \end{vmatrix} \begin{Bmatrix} \{x_s\} \\ \{x_f\} \end{Bmatrix} = \begin{Bmatrix} \{0\} \\ \{R_f\} \end{Bmatrix} - \ddot{x}_g(t)[M_s]\{1\}$$

where $\{R_f\} = [G]\ddot{x}_f$; $[G]$ - matrix of the ground rigidities

$[M_s], [C_s], [\bar{K}_s]$ - submatrices of the masses, of damping and the rigidities of the structure

$[M_f], [C_f], [\bar{K}_f]$ same but of soil

III. STRENGTH AND DEFORMATION CHARACTERISTICS OF WALL PANEL CONNECTIONS

The strength and deformation characteristics required for the investigations mentioned above are determined analysis of S.Kosev's data (3) and testing of wall panel joints subjected to reversive shear combined with compression (Fig.1)

During earthquake excitation some of the connections in vertical diaphragms are subjected to shear and compression, others to shear and tension, and third to shear only. This is the reason for the bearing and deformation capacities to be determined at the conditions mentioned.

The shear strength of the connections is determined in function of the strength of concrete and reinforcement and size of the connections.

The shear strength of the joint can be determined with :

$$Q_{sh} = m\alpha\, 0{,}9\sqrt{R_c}\left(1{,}7\, b\ell + 61 F_s \beta \cos\alpha\right)$$

(4)

where $\beta = 1{,}0$ for $\mu \leq 3\%$ reinforcement $\beta = \frac{15}{12+\mu}$ for $3\% < \mu < 4{,}5\%$; factor for plastic deformations $-\alpha = 1{,}35$

The shear strength of the connection with compression is

(5) $$Q_{sh,c} = Q_{sh} + \frac{3}{1+0{,}5\mu} \frac{N_c}{0{,}9\sqrt{R_c}} + f N_c$$

$f = 0{,}40$ – coefficient of friction. N_c – compression force acting on the total length of connection.

The shear strength of the connection with tension:

(6) $$Q_{sh,t} = Q_{sh}\left[1 - \left(\frac{N_t}{F_s R_s}\right)^2\right]; \qquad N_c \text{ and } N_t = \pm \sigma_v L b$$

where ℓ and b – length and width of the joint

L – total length of the connection (fig 1)

R – strength of the concrete determined by cubes

The deformations and the rigidities of the connections in elastic and nonelastic stage are determined using (4),(5),and (6). The theoretical investigations are compared to the experimental ones, carried out on models of the type shown on Fig.1. The hysterises curves of one of the models are shown on Fig.3. The bearing and deformation capacities are strongly influenced by the forces of friction.

The influence of the number of cycles on the stiffness of the connections can be taken into consideration through the factor:

(7) $$\gamma = K_o\left(1 - \frac{\alpha_n \sum_{i=1}^{n} \Delta_i - \Delta_{i-1}}{n\, \Delta_{max}}\right)$$

where α_n is factor of the number of cycles.

One of the basic moments in the analysis of the large-panel buildings is the problem of the normal pressure stresses reduction in the connections in result of the negative vertical component of the earthquake.

IV. RESULTS FROM THE INVESTIGATIONS

On hand of the investigations the following conclusions are made:

1. The application of equation (2) gave the possibility to study the influence of the yielding of the connections in the vertical diaphragms on the response of the building. On Fig.4 and 5 are determined the shearing forces and the displacement in the vertical diaphragms at several prerequisites: a/ without deformation in the connections; b/ with deformations in the connections; c/ deformations in the joints without friction in the connections.
2. The resistance capacity of the connections is considerably increased by taking into account friction in the lower storeys. Therefore negligence of friction effect leads to appearance of plastic deformations in the connections,Fig.5
3. The friction force in the connections is a nonlinear function of the normal loading due to the fact that at development of deformation the friction is changed by friction with rolling, realized by the sand in the concrete. This is the reason for the reduced value of the coefficient of friction to be accepted f=0,40. At higher values of the vertical loading and at signchanging cyclic loading the average value of this coefficient could be further reduced.
4. Due to the presence of diaphragms symetrically positioned in a plane with approximately equal stiffness, the redestribution of forces among them does not appear as in some other structures.
5. The experimental investigations on fullscale buildings showed that the intervals of the free vibrations on the two axes x and y differ insignificantly despite of the great differences of stiffness. The reason is the spatial work of the structure.
6. The character of the horizontal floor displacements /Fig. 5/ show that they are caused not only by shearing but from bending as well.
7. One of the most essential problems when investigating the response of large-panel buildings in nonelastic stage is the determination of the maximum nonelastic deformations, which can be admitted only in regard ofthe eccentricity of the vertical load, which is created in the wall panels.

V. CONCLUSION

Due to the specific construction systems of large-panel buildings the analysis of their response in nonelastic stage is pretty complicated and requires the assuption of many prerequisites at the formation of the mathematical model. The experimental investigations on the connections used in practice enabled us to give an account of the actual work of these connections in nonelastic stage and to use the results when investigating the response. More precise solutions and varying prerequisites as well as accounting for the deformation of the floor slabs is necessary.

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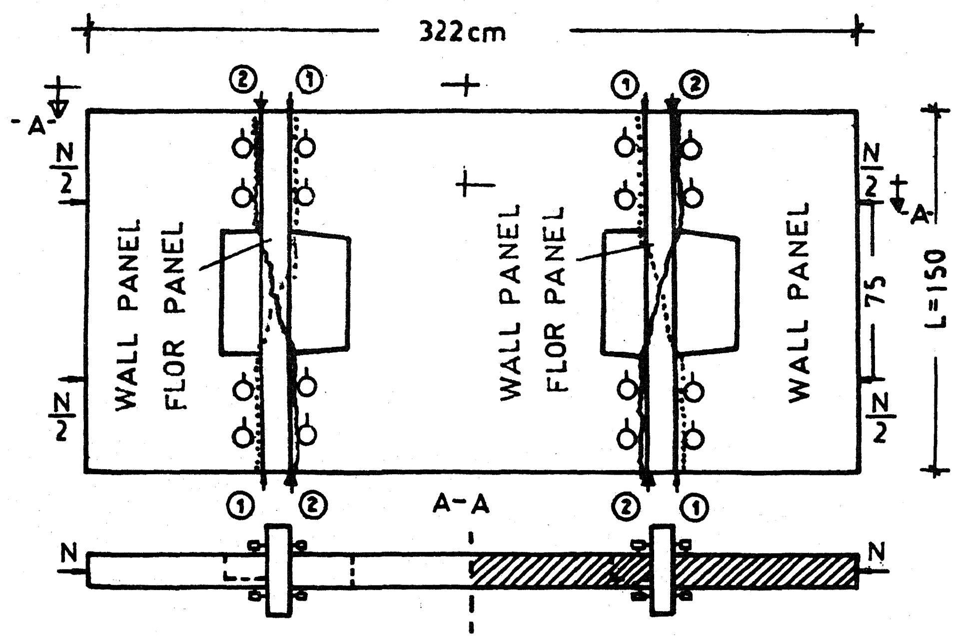
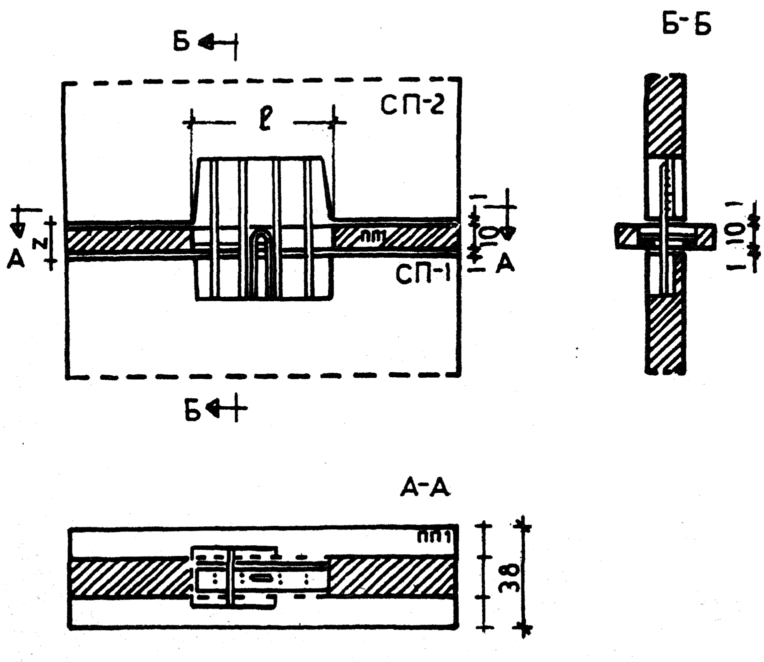


322cm
WALL PANEL
FLOR PANEL
WALL PANEL
FLOR PANEL
WALL PANEL
L=150
75
A-A
N

fig.1



Б
СП-2
СП-1
ℓ
A
Б-Б
A-A
38

fig.2

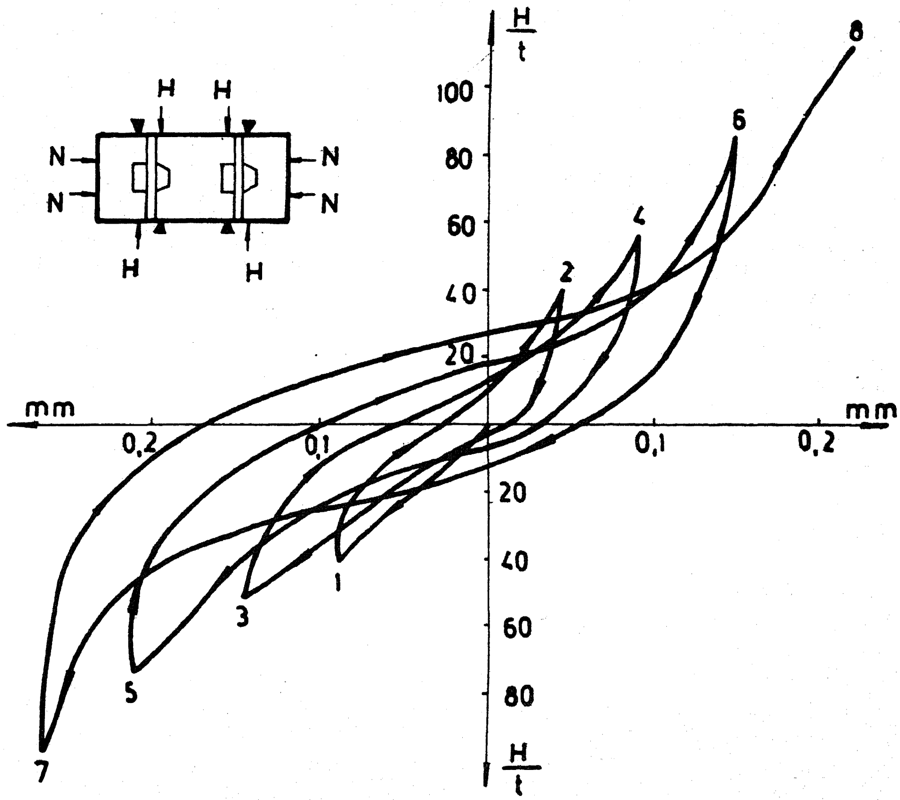


fig . 3

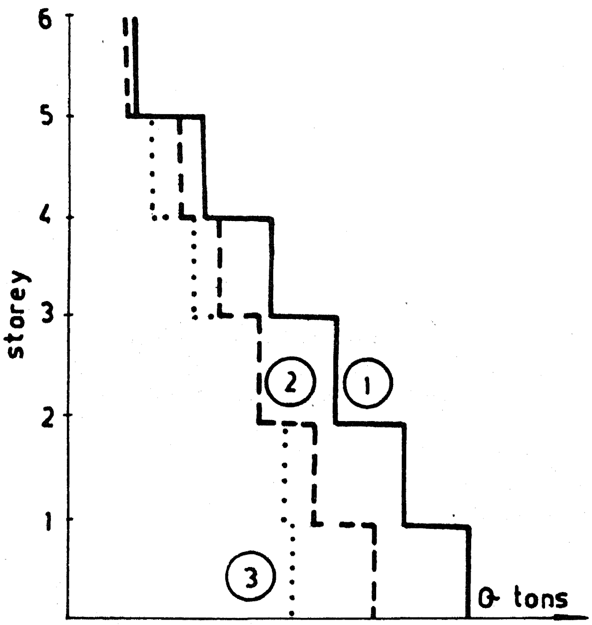


fig. 4

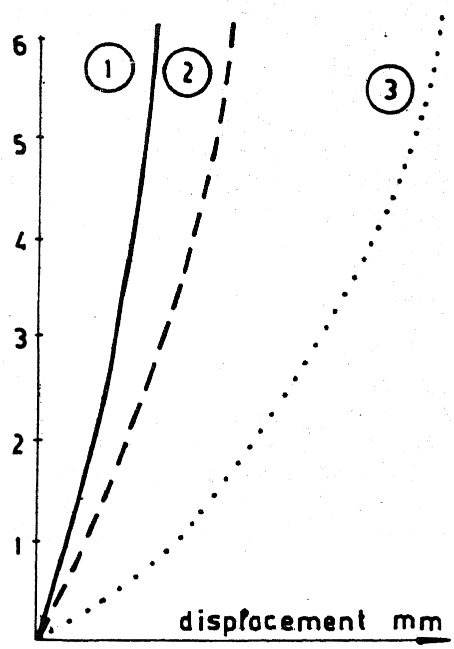


fig. 5

DISCUSSION

T.P. Tassios (Greece)

The authors are complimented, it is the kind of work very much needed long ago. Nevertheless, the following comments might be raised, since the basic experimental data used are not given in the paper. To this purpose, reference is made here to our own results on the dynamic behaviour of R.C. panel joint Dr. thesis, S. Tsoukantas, (National Technical University of Athens).

1. Every dynamic analysis for high level loading should clearly mention the limit of its applicability against the risk of cummulative damage. A criterion of MINER'S type could be applied if low fatigue curves are known, like our curve Fig. A.

2. There is a threefold variability of the shear stiffness of precast panel's joints, due to: a) The level of loading, as e.g. in our Fig. B., b) The number of cycles effectuated under constant limit shear stress, like in our Fig. C, and c) a remarkable scattering of individual values, among "similar" joints. In view of this very large, not always systematic, scattering deterministic method of analysis it may be proved misleading in some cases.

3. On the basis of our research on full-scale RC panel joints, it seems that the damping value (5%) taken into account is rather high (see also paper 11-241 in this Congress).

In view of the above mentioned questions, the authors are kindly requested to offer details of their experimental findings under dynamic loading, together with some further clarification of their notations, as well as their opinion on the possibilities of their analytical method to give realistic results, inspite the highly stochastic nature of the phenomena involved.

Author's Closure

With regard to the question of Mr. Tassios, we wish to state that first we would like to point out that because of the limitations in the number of the pages we were prevented to present all the available data, but only a part of them. The problem of the response of large panel buildings in nonelastic stage is much more complicated than other cases, for example R.C. frame buildings. That is the reason for making some assumptions.

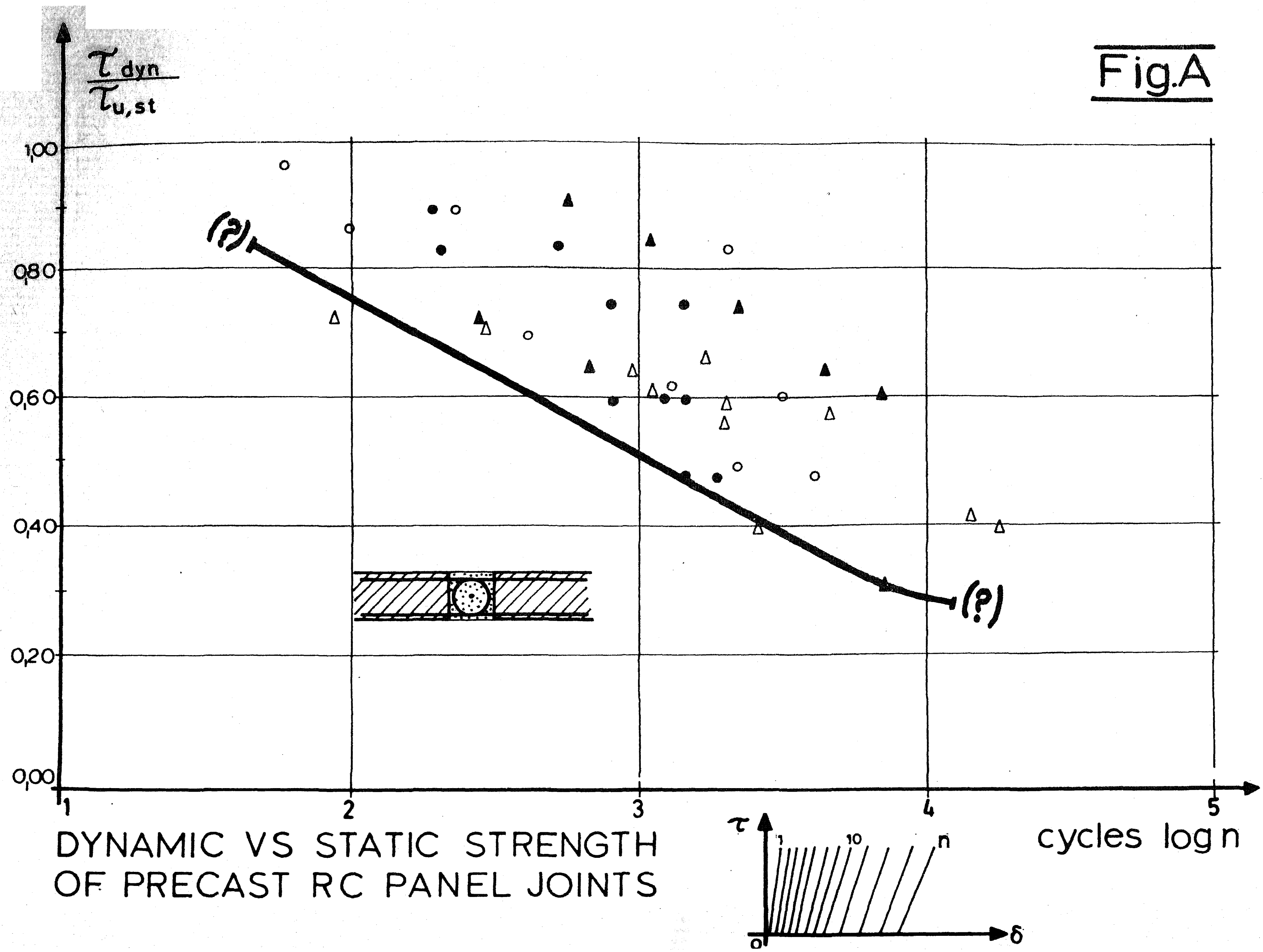
The problem for determining the limit deformations criteria in large panel buildings is quite complicated and depends on numerous factors (type of the construction, kind of the joints, number of the storeys, etc.). For the present no criteria for limit values in nonelastic stage are established.

It is true that shear stiffness of precast panels and of special joints depends on numerous factors, some of which are mentioned in Mr. Tassios's second question. The influence of others is mentioned in our paper 3-345. Besides these factors, the influence of the vertical component of the earthquake and especially of its negative value causing decrease in the vertical load and change in the friction should also be accounted.

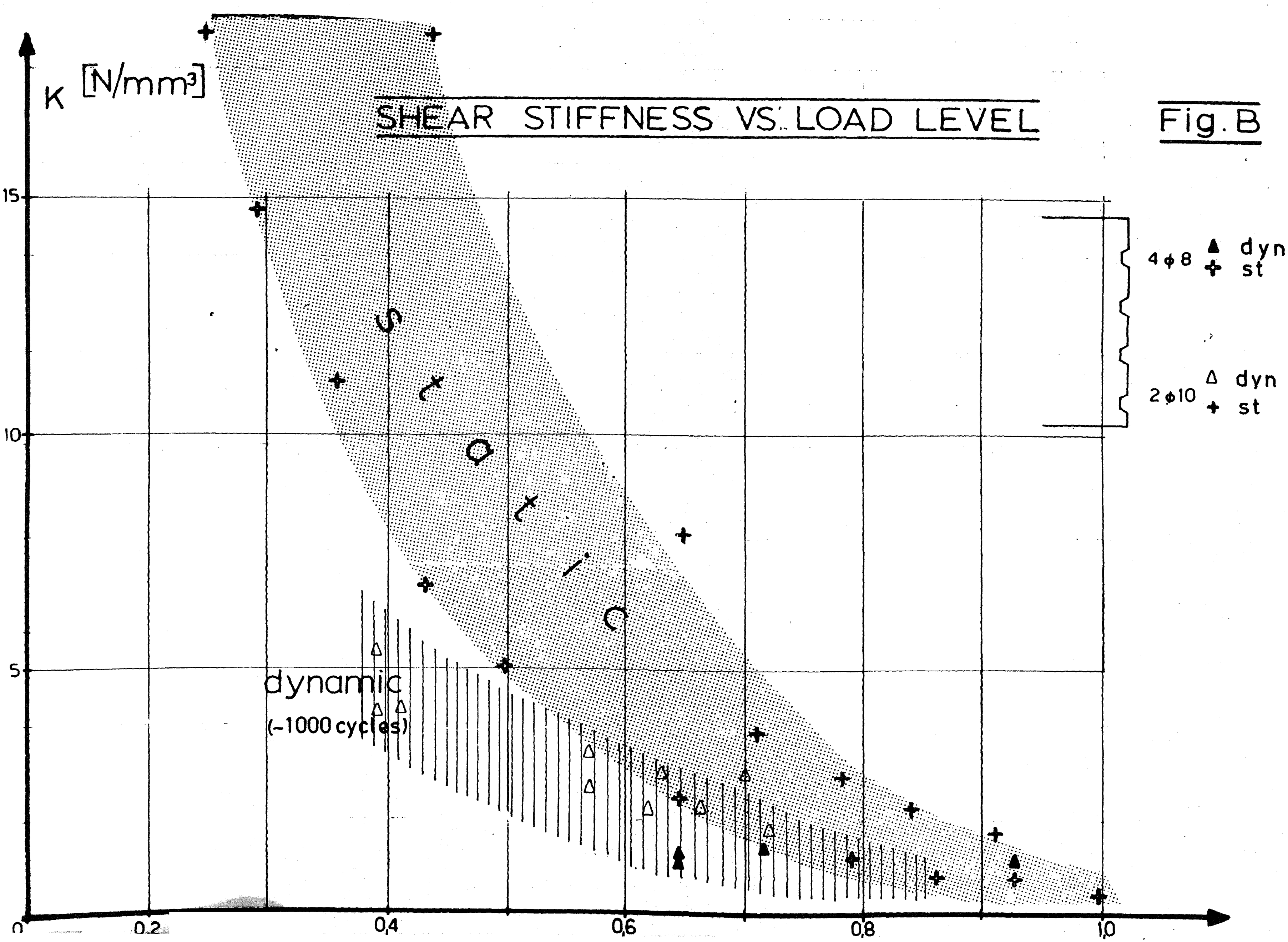
Our investigations on real large panel buildings show lower damping value also - to 1.71%. The assumed by us damping - 5% is conventional, all the more that we also multiply the accelerograms by the coefficients 1.00, 0.50 and 0.30 in order to draw some conclusions for weaker earthquakes. The determination of the factor by which an accelerogram should be multiplied is considerably more complicated than to make the assumptions of 2 or 5% damping.

We share Mr. Tassios' opinion that the earthquake has "highly stochastic nature" because of which we have used some assumptions not only about the character of the earthquake but for the mathematical model of the construction too. This model could be improved by accounting the influence of some additional factors like the deformations of the vertical joints etc.

Fig.A

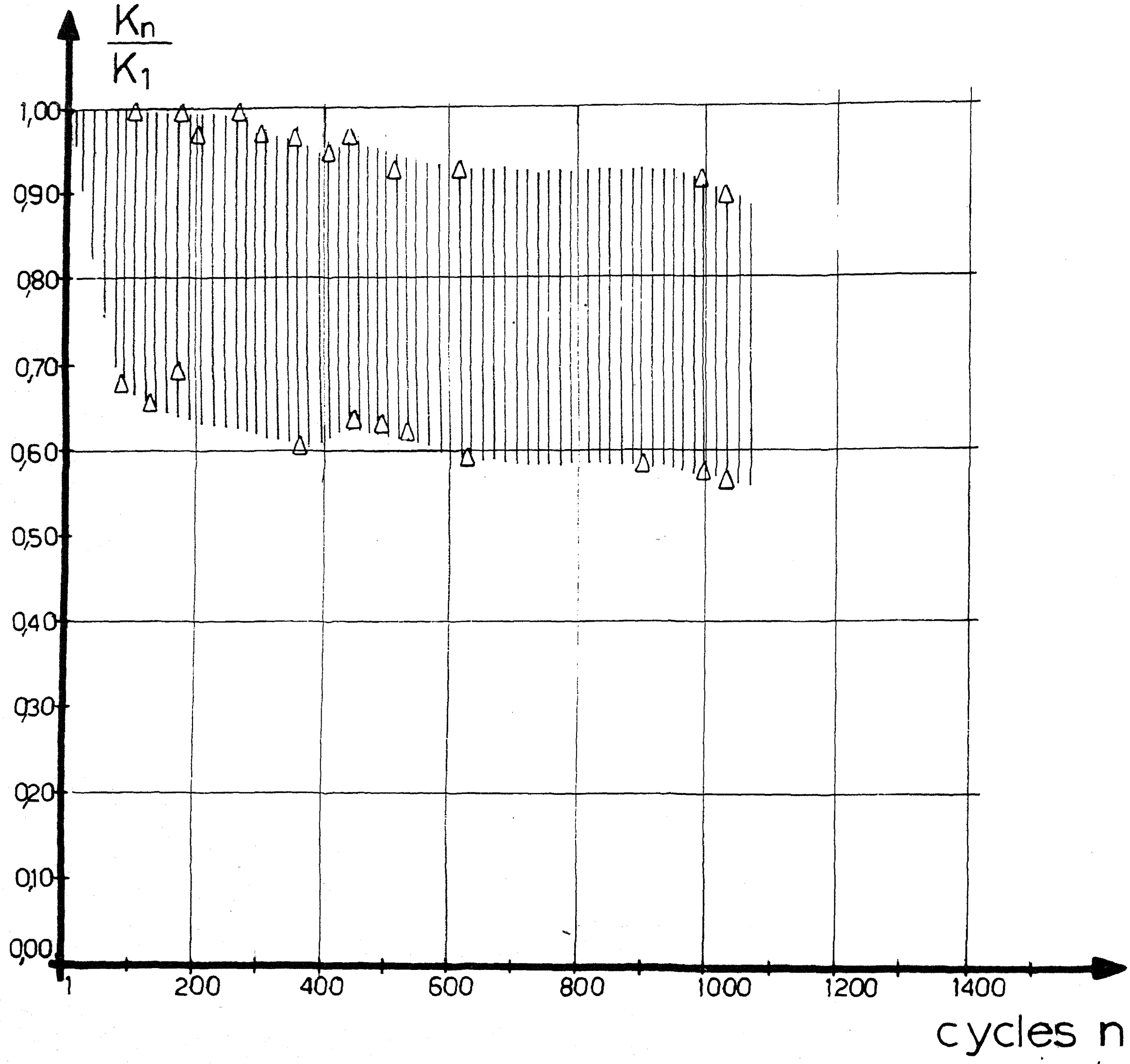


DYNAMIC VS STATIC STRENGTH
OF PRECAST RC PANEL JOINTS



SHEAR STIFFNESS VS. LOAD LEVEL
K [N/mm³]
15
10
5
0
0.2
0.4
0.6
0.8
1.0
static
dynamic
(~1000 cycles)
4 ϕ 8 ▲ dyn
✣ st
2 ϕ10 Δ dyn
+ st

Fig. B



SCATTERING OF NORMALIZED SHEAR STIFFNESSES OF PRECAST R.C. PANELS' JOINTS, UNDER DYNAMIC LOADING