THE STUDE OF NON-LINEAR PERFORMANCE OF STRUCTURES IN FRAMELESS EARTQUAKE-PROOF RESIDENTIAL BUILDINGS BY MEANS OF POERFUL VIBRATORS

by

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INTRODUCTION

Comparativly rare cases of strong earthquakes affects on large-panel buildings generally testify on dependability of this type of constructions. Nevertheless artificial vibrations of considerable intensity are desireable for the detailed study of their non-linear dinamic response and for exposing slackenings and safety factors of individual elements and joints.

Laboratory of strength uses for this purpose powerful vibration generators. Characteristics of these generators comparatively with other vibration generators are shown in the table I.

NATURE EXPIRIMENTS AND FUNDAMENTAL RESULTS

The large-panel buildings from 4 to 9 stories carcassless buildings and large models of other types were tested in their resonant zones.

While testing oscillations were forced upon buildings with amplitudes up to $a = (0,3+0,4)\cdot 10^{-3}H$, where H - height of building. Inertial loading on building reached 600-800 tonn and thus was 0,5+2 times more then seismic loading provided by USSR standard.

During testing numerons ocurences of non-linear oscillations took place. Changes of stiffness and another dinamic characteristics of buildings varied depending on the testing conditions.

In several cases visiable damages took place. The results of careasslesв buildings vibration testing have indicated that soil influence on the reaction of whole construction is very essential in the same way as in cases of small oscilations.

However stiffness of building and stiffness of soil change in different manner while loading level increased. When loadings vary within diapason which affects essentricities $e = \frac{M_o}{N} < 0,1b$ (b - length of foundation in the direction of load action) and transversal forces $Q_o < 0,15N$ an average reduce of the stiffness of soil is 25-40% (fig.1). Very smoll

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irreversible deformation of soil take place in this interval, therefore one may suppose that nonlinear deformation of soil most probably has nonlinearelastic rather than nonelastic character.

In the same interval of loadings the stiffness of building is reduced more than that of soil for majority of buildings (fig.2).

Hence the ratio $C_{soil}/C_{building}$ increases with the increase of vibration intensity (fig.3). This is especially true for higher levels of loadings when active crack generation and damages of construction begins. Let us consider, for example, the results of testing of monolitic model of 10-storeyed building on the scale of 1/4 on the natural soil (fig.3,4).

It should be noted that with diminution of intensity of vibrations the stiffness of building is reestablished considerably less than that of soil.

It indicates that the decisive role in forming of nonlinear properties of large-panel and other carcassless buildings constructions plays processes of fragile type (the local damages, crack generations and so on).

The mentioned peculiarites of building and soil stiffnesses changes lead to the change of dynamic parameters of the buildings. The loadings which were reached affects the decrease of frequencies for large-panel buildings on 30%, for large blocks buildings on 65%, for models on 70%.

The degree of damage of buildings at such reduce of frequency corresponds with behavior of analogeous buildings at large intencity vibrations described in the available papers. The own frequencies of buildings change as a rule irreversible.

The nonlinear processes that take place in the building constructions and in the soil during the intensitive vibrations are reflect on the shape and parametrs of resonanse curves.

If the vibration intencity is comparatively small the resonance curves at the increase of the loading level acquires asimmetrical shape (fig.3) - their descending branches draw near skeleton curve, while ascending curves draw near vertical. At the subsequent increase of the level loading the whole resonanse curve for many tested carcassless buildings disposes on the one side of the skeleton curve and reaches it only at the top point.

This fact is especially expressive for buildings which received the demage during testing. That kind of resonanse surves is typical neither for nonlinear elastic sistems nor for elastoplastic sistems with large dissipate of energy. In a number of cases at the large loading levels the family of linear resonance curves is formed with subsequently decreasing stiffness. These facts have not been described in publications since usnally stationary hysteresis diagrams of deformation are considered.

The results of the vibration tests of carcassless houses and their models were bring to light essential peculiarities of nonlinear deformation of the soil of these buildings. It was established that swinging oscillations of building on the soil were accompanied with plastic deformations of the soil in its compessed zone and with the foundation of building taking-off the soil in the extended zone an also with breaking the contact between the foundation and the soil in the compressed zone owing to irreversible compessing of the soil (fig.6). Using the well-known diagrams soil deformation as a function of compression the scheme was developed for calvulating of nonlinearly deformed soil.

The result of static, dynamic and seismic calculatings using this scheme was utterly verified by tests on an 10-storeys model of monolithic building on the scale of 1/4 on the natural ground (fig.7).

This scheme explains experimental regularities in change of soil stiffness mentioned above. It shows that the nonlinear behavior of the soil leads to considerable reduction of soil stiffness (for example, with lo = 0,51, l - wideness of the foundation base the stiffness of soil reduce on 30% at the elastic take - off and on 65% with using Prandtle diagram for soil deformation estimation) and therefore to reducion of own oscilation frequency and inertial loading on the construction.

However during this process the considerable part of foundation takes - off the soil and support area decrease. In increases a danger of buckling of soil. When the peaks of overloading of seismic affects take place this is especially dangerous.

The results of vibration tests indicate that a stiffness of the building itself is noticeably changes at the increase of vibrations intensity.

The basic reasons for large-panel and large block buildings stiffness decrease are follow:

1. The non elastic behavior of the material and elements of building, crack generation and demages.

2. The change of stiffness and discomposure of joints and connections.

3. The change of three dimensional behavior of building constructions.

The non-linear deformations of the large-panel buildings are to a greater degree determined by behavior of the horisontal joints. It is clear from analisis of strength state of large-panel buildings, results of experiments and literary data.

When horisontal joints subjected to bending moment are disclosed large-panel shear wall can be considered as beam with regular cracks.

Authors proposed the method of desigh of large-panel diaphragm which permitted to receive a good coincidence for experimental and design data.

This method take account of nonlinear pliability of horisontal joints, nonelastic behavior of vertical still crossing joint and diminution of stiffness of slackening her of puting joint.

The playbility of vertical joints of perpendicular walls can render very strong influence on the nonlinear response of a building. That was confirme by testing of two masonry buildings.

One of them having the block bond in joints of walls and another having not (fig.7).

However in the majority of cases the vertical joints fat the large-panel buildings the vertical joints of walls of the large panel building retained the strength and stiffness almost up to the destruction of the whole buildings.

CONCLUSIONS

From tests on number of natural buildings, realized by means superpowerful principal regularitis of dinamic parameter changes were determined during intensive vibrations. It was also determined important structural causes non-linear response of building.

Table I

The vibration generators	W-2, authors	W-3, authors	D.Hudson, USA; BGS-A-200 Japan	T.Hisada K.Nanagawa, Japan	Kondo, Japan
Max.force (tonn)	80	200	9	15	35
The moment of unbalances (kgm)	1560	4000	360	350	240
Min.frequency with max.force (CPS)	3,7	3,5	2,5	5,0	6,0
Interval of frequencies (CPS)	0,4-8,0	0,4-10	-	to 7	to 10
Quantity of vibrators	4	6	4	1	1
Weight (tonn)	7,7	13	-	-	-
Summary power (kW)	50	100	4,5	-	22,5

fig.1

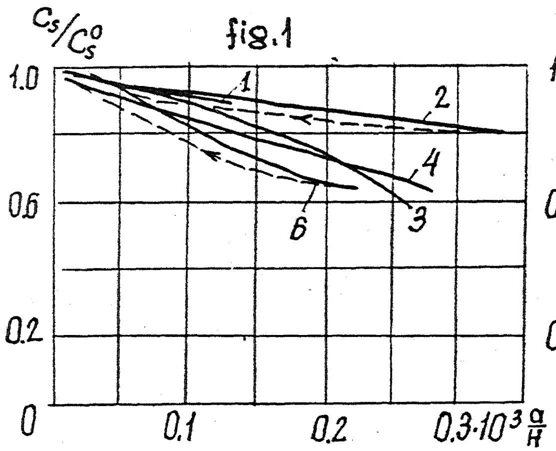
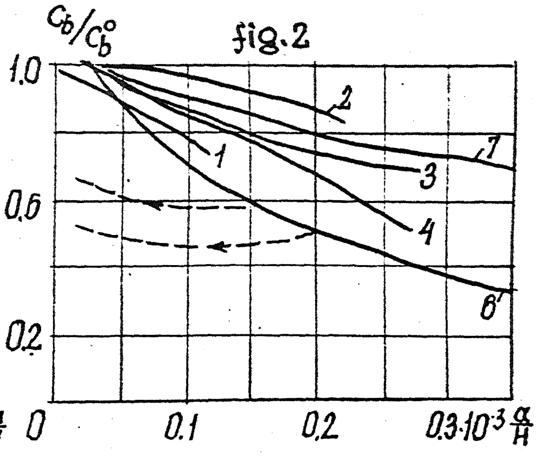


fig.2



1-large panel-4-st. 2.large panel-9-st. 3.large panel-8st. 4.large panel-9st. 5-monolithic model-10st. 6.large block-4st 7.masonry-5st.

fig.3

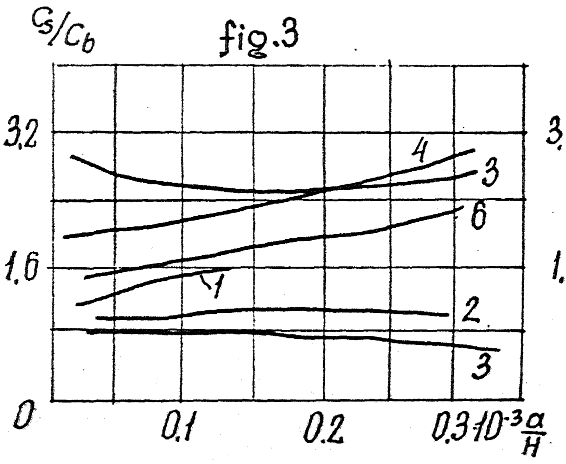


fig.4

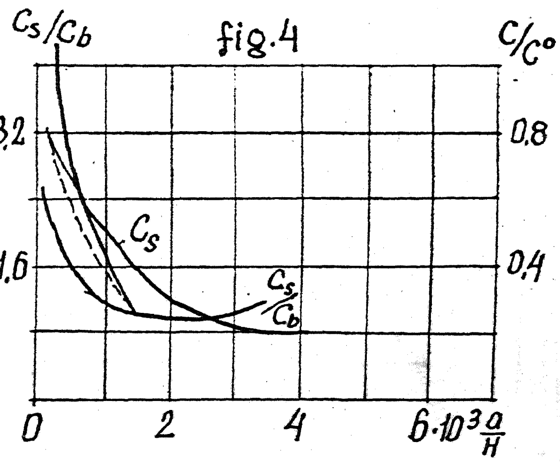
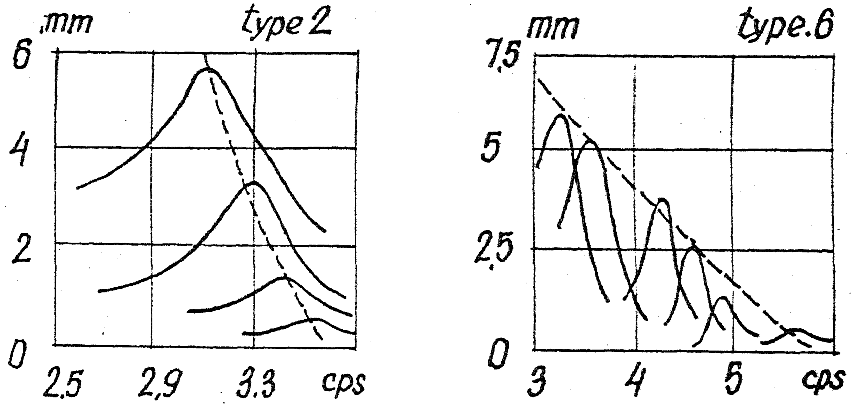


fig.5



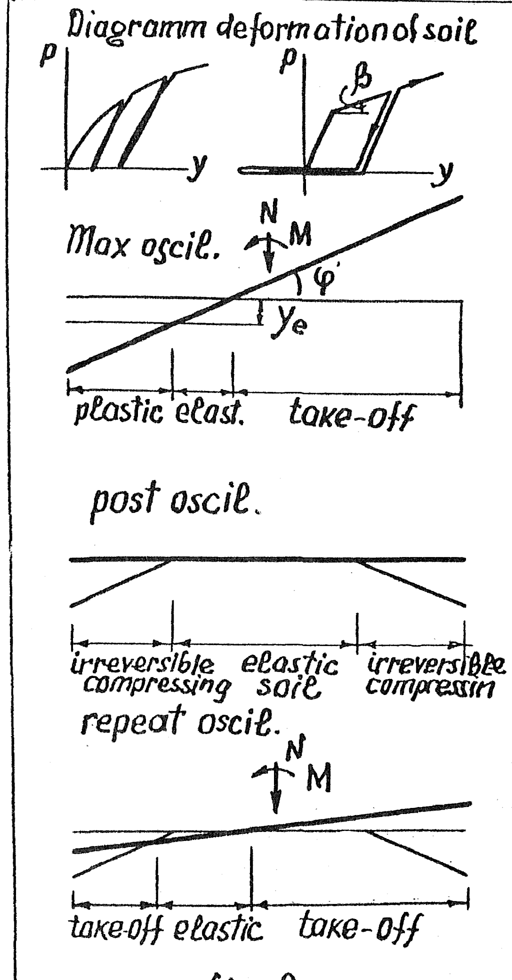


fig.6

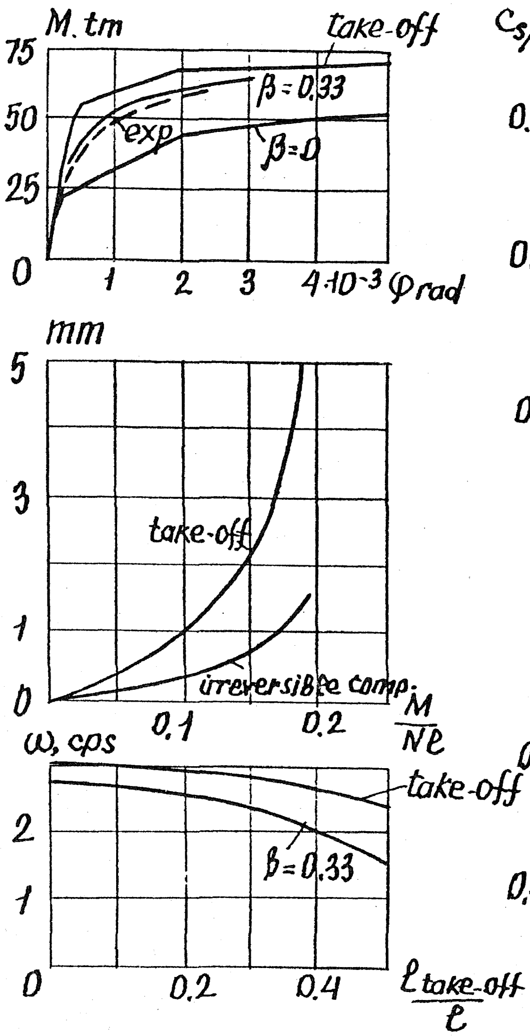


fig.7

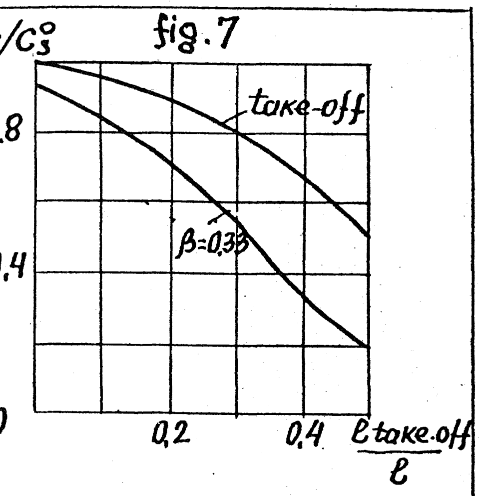


fig.8

1. strength. vert. joint
2. strengthless vert. joint