# STRENGTH RESERVES OF BUILDINGS DEPENDING UPON THE PRONENESS TO NON-RESILIENT DEFORMATION UNDER SEISMIC EFFECTS

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ANNOTATION

The report deals with the analysis of experimental data of vibro-tests on full-scale buildings and large-size models. The concern of the report is earthquake aftereffects related to the proneness of different structural systems (frame, large-panel, in-situ) to non-resilient deformation. There was obtained the dependence between the parameters of response reduction coefficient (stresses) for an elastic system, non-resilient deformation and rigidity reduction due to brittle failures which occur in buildings under seismic effects.

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The analysis of earthquake aftereffects has proved the buildings prone to non-resilient deformation to be more resistant to seismic effects.

Buildings of different structural systems (frame, brickwork, out of blocks, box-units, large-panel, in-situ, etc.) calculated for the same seismic loading and provided with antiseismic measures in accordance with existing norms, possess different strength reserves under earthquake effects due to various proneness to non-resilient deformation.

The influence of non-resilient deformation upon the resistance of structures is especially apparent under intensive seismic effects. The bearing structures of buildings designed without due regard for non-resilient deformation are usually collapsing in a fragile manner, thus, causing a considerable damage.

In the past there were obtained finite formulas relevant to the dependence of structures response reduction (stresses) upon the parameters of non-linear deformation (non-resilient deformation coefficient) [1]. However, they did not take into consideration certain actual conditions of deformation in structural systems of buildings.

The analysis of earthquake aftereffects as well as vibro-tests on full-scale buildings and large-size models showed that the reduction of structures rigidity occurs not only due to non-linear deformation (plastic deformation) but due to local fragile damages in certain zones of structures, structural connections and elements (brittle failure of ties).

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The results of large-panel building tests and the analysis of earthquake aftereffects have proved that the weakening of rigidity in buildings is connected with a non-linear character of wall deformations due to the formation of horizontal cracks along panel joints and a non-linear deformation of lintels. These two factors entail a smooth weakening in the rigidity and with the loads being removed, wall rigidity is considerably restored owing to dead weight of structures. The irreversible loss of rigidity in large-panel buildings occurs due to local damages in lintels, joints and panel displacements and can be considered negligible (approximately 20-25 per cent of the initial rigidity).

While testing an in-situ large-size r.c. model of a frameless multi-storey building the reduction of structure rigidity was developing due to cracks along the technological seams where failure in concrete grip was observed (plastic deformation) and due to inclined cracks in bearing walls which caused the fracture of walls (brittle failure).

In the tested frame buildings the reduction in structural system rigidity occurs due to non-linear deformations of cross bars and cross bar-column connections (plastic deformation) and due to fracture of certain zones of the connections and columns along the inclined cracks (brittle failure). In the terms of proneness to non-linear deformation, frame buildings are intermediate between in-situ and large-panel buildings.

Experimental investigations allowed to establish the actual dependences of deformation parameters and responses (stresses in a structural system of a buildings). It was established that under equal conditions of loading structure response lessens due to casual brittle fractures and cracks in structures.

In order to determine a quantitative dependence of parameters of structural response reduction, non-linear deformation and structure brittle fracture we obtained the following formula:

$$K_{cp} = \sqrt{2 K_{H\Lambda} - \frac{1}{K_{\text{ж}}}} \qquad \text{(Eq. 1)}$$

The derivation of this formula was based on the precondition of energy equation in collapses due to earthquake in an elastic-deforming system and in a system with changeable rigidity (Fig. 1).

The following designations were adopted in the dependence (Eq. 1):

$K_{cp} = \frac{S_1}{S_2}$ - the reduction of stresses in a real structure compared with elastic one

$K_{H\Lambda} = \frac{A_2}{A_3}$ - coefficient of non-linearity

$K_{\text{ж}} = \frac{C_p}{C_o}$ - coefficient of rigidity reduction

$C_o$ - initial rigidity of the system

$C_p$ - rigidity after structural fracture

The parameters obtained from experimental tests on different types of buildings are tabulated in the table below

| Type of building | $K_{нл}$ | $K_{жс}$ | $K_{ср}$ |
|---|---|---|---|
| 1. Fragment of full-scale 9 storey large-panel keramzite concrete building erected in Ordzhonikidze | 4.2 | 0.8 | 2.66 |
| 2. Fragment model of 10 storey large-panel building | 5 | 0.4 | 2.7 |
| 3. Fragment model of 10 storey in-situ building with horizontal technological seams | 6 | 0.12 | 2.1 |
| 4. Fragment model of in-situ building without technological seams (the rated data) | 2.5 | 0.2 | 1 |
| 5. Frame buildings in Frunze | 3.0 | 0.5 | 2 |
| 6. 5 storey frame building in Ashkhabad | 3.0 | 0.5 | 2 |

As it is seen from the table, large-panel buildings are characterized by the greatest strength reserve, in-situ buildings designed without due regard for non-resilient deformation factor are characterized by the least strength reserve.

Reference

Park R., Paulay T. Reinforced concrete structures. Wiley, 1975

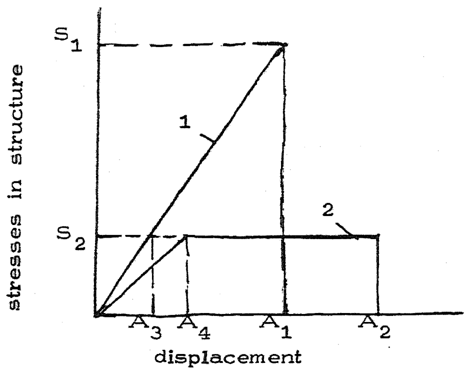


Fig. 1. Diagrams of deformation for an elastic system (1) and a system with changeable rigidity (2)

$S_1$, $S_2$ - maximum stresses in (1) and (2)

$A_1$, $A_2$ - maximum displacements (1) and (2)

$A_3$ - initial displacement

$A_4$ - displacement (2) after rigidity reduction