ULTIMATE STRESSES IN LARGE-PANEL BUILDINGS EXPOSED TO SEISMIC LOADS

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ANNOTATION

The report deals with possible types of damages and the schemes of collapse due to earthquakes in large-panel buildings - the most widely spread ones and in perspective types of mass residential buildings in the USSR. During past earthquakes with up to 8 point force, large-panel buildings suffered from minimum damages compared with other types. To reveal the nature and the extent of damages under intensive oscillations is important for correct evaluation of buildings reliability and economic efficiency. Some aspects of the problem are under consideration in this paper with due regard for the data obtained during the investigation of earthquake consequences in Gazli in 1976 and from the vibro-testing of a full-scale fragment of a 9-storey building.

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Uptil recently there were few data concerning the bearing capacity and nature of damages in large-panel buildings under seismic effects. The results of investigations carried out by T.Zhunusov, Yu.Gamburg, T.Hisada, D.Diaconu, the authors of the paper and others related to low-rise buildings, as a rule, revealed considerable reserves in bearing capacity of panel buildings and the importance of joints from the view point of their influence upon the nature of damages and the collapse of buildings. The experience gained during earthquakes with seismic force up to 8 points in Petropavlovsk-Kamchatsk, Dzhambul, Daghestan and in the Carpathian Mountains [1] proved the above statement as the damages were small in these buildings and were observed in joints between panels and in lintels, in the main.

At present, large-panel buildings are becoming increasingly common in mass standard housing in seismic regions of the USSR, their height reaches up to 9 storeys in regions with the rated seismicity being 9 points and 14-16 storeys - in 7 point seismicity regions. In view of the fact, in order to evaluate properly their reliability and economic efficiency it is necessary to know exactly the types and the extent of possible damages, the nature of non-linear deformations, the limits of the bearing capacity and the schemes of collapse.

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Certain useful data were received during the investigations of earthquake consequences in Gazli (the Uzbek SSR) and from the vibro-testing of a fragment of 9-storey building in Ordzhonikidze (the North Caucasus).

As a result of two strong earthquakes in Gazli, two and four storey large-panel buildings without seismic resistant devices suffered from considerable damages though these damages were not so great as in brickwork and framed buildings. The investigation of these buildings proved that the extent of their damage was determined by the structure of joint connections [2]. The main kind of damage was failure of joints which were not provided with sufficient amount of well anchored metal ties. There were observed 10-15 cm shifts of panels, the slipping of floor slabs from walls, the striking of elements against each other. Damages in panels were insignificant in the form of vertical cracks, as a rule. The nature of cracks cannot be explained by shift or by eccentric compression (bending) in the panel plane. However, while calculating buildings as elastic systems with adapted earthquake accelerogram the stresses in panels exceeded their bearing capacity under eccentric compression. It is indicative of non-linear response of large-panel buildings in Gazli to earthquakes in 1976.

The nature of non-linear deformation in large-panel buildings [3] with antiseismic devices is revealed on the whole and will be discussed below. It is connected mainly with the lessening of diaphragms bending rigidity of bearing walls due to the horizontal joints opening and the crack forming in lintels and panels under intensive seismic effects. Considerable shifts of walls in low-rise buildings in Gazli could cause the other nature of non-linear deformation connected with dry friction along horizontal joints. The investigation of this question is carried out on the basis of earthquake accelerogram calculations of buildings in Gazli with due regard for displacements and frictions in walls. The calculation scheme is based on the following:

1. in low walls shear deformations prevail;
2. due to weakness or absence of ties between panels they are not taken into account in the calculations;
3. in horizontal joints panels are connected by dry friction;
4. vertical oscillations are negligible, it allows to take into account the influence of vertical component of acceleration of the soil base only by the value of vertical load.

In view of the fact the calculated model on n -storey building in Gazli was presented as a cantilever rod with $n+1$ masses (fig. 1) displacing in strictly horizontal direction, whose movement is defined by the following equation:

$$\sum_{j=1}^{i} m_j \ddot{y}_j + b_i \dot{y}_i + R_i(y_i - y_{i+1}) = \sum_{j=1}^{i} m_j \ddot{y}_0 \qquad \text{(Eq.1)}$$

where m_j - masses of storeys

b_i - coefficient of viscous resistance

$\ddot{y}_0 = \ddot{S}_y$ - horizontal acceleration of the base specified by the accelerogram

$R_i(y_i - y_{i+1})$ - restoring force equal in the elastic stage $(y_i - y_{i+1})\, C_i$,

C_i - shear rigidity of a storey.

As the force of dry friction during the movement along the horizontal plane can be presented by

$$F = f N \operatorname{sign} \dot{y}$$

where f - friction coefficient
N - vertical load

then, the diagrams of deformation system with friction is analogous to bi-linear hysteresic diagram but with variable quantity of elastic force (fig. 2) being

$$R_i(y_i - y_{i+1}) = \begin{cases} (y_i - y_{i+1}) \cdot C_i & \text{if } R_i \leq f N_i \\ f N_i = f(g + \ddot{S}_x) \sum m_j, \end{cases} \qquad \text{(Eq. 2)}$$

where $\ddot{S}_x$ - vertical acceleration of the soil base, specified by the accelerogram.

The integration of equations (Eq. 1) was done by Runge-Kutta method modified for the case when the right parts are with discontinuous derivatives [4]

The calculated accelerogram

As soil movement accelerograms in Gazli were not recorded there was used the accelerogram of the second more intensive earthquake recorded in epicentric zone at the distance of 30 km from Gazli; where earthquake intensity made up 10 points, MSK-64. In order to bring the intensity of earthquake effects to conformity with macroseismic data obtained in Gazli, horizontal accelerations of this accelerogram were reduced by two times and the vertical ones - by 5 times.

As a result, the maximum horizontal acceleration made up 0.315 g at the period of 0.184 sec, and the vertical one - 0.2 g at the period of 0.063 sec.

The results of the calculation show that at the adopted intensity of effects the stresses in elements could reach the value of friction forces. However, irreversible wall shifts must have been negligible: up to 1.5 mm in two storey buildings and 2 mm in four storey buildings (fig. 3). Due to negligible shifts the degree of non-linearity is small and the considered nature of non-linearity does not correspond to the facts. Thus, the stresses in panels according to the calculations were to exceed the breaking loads under eccentric compression (table), while the damages of such kind were found only in horizontal joints but not in the panels. In this way, the results of calculations indirectly confirm the fact that in buildings in Gazli the non-linearity of deformation and the nature of damages during the first earthquake were connected to a considerable extent, with the appearance of openings in horizontal joints and with the failures in vertical joints and lintels. These failures dismembered the whole structural system into separate elements which changed the character of structural supports and loads distribution. Aftershock earthquakes (more than 100 was registered during the month) resulted in the gradual progress in the above failures which aggravated the dismembering of the system. During the second earthquake the structural systems were already dismebered, it resulted in a sharp increase in the non-uniformity of loads on separate elements deformations beyond the wall planes, as for shifts of certain panels they increased by several times. At the same time such considerable changes in the structural system could entail considerable decrease in seismic loads, that is how relatively little damages in the panels can be explained.

In large-panel buildings with antiseismic devices and with reliable vertical joints, the spatial rigidity within every floor is substantially strengthened and it makes the structural dismembering less probable under seismic effects. The behaviour of horizontal joints and lintels has even more influence upon the non-linearity of deformation. This process was studied in detail during resonance tests of the 9 storey fragment.

The structure under test is a fragment of 9 storey large-panel residential building, 10.8 m long (three spacings of cross walls), all elements are of keramzite concrete (fig. 4).

Methodics of tests. Vibro-tests were carried out in 1976 with the application of a vibro-machine "B-2" located on the roof of a fragment, in 1979 the tests were continued with a more powerful vibro-machine "B-3" During tests the eccentricity of the vibro-machines reached up to 1360 kg-m ("B-2") and 3000 kg-m ("B-3") which allowed to create the force at the shaft of vibrators up to 750 kN and 1600 kN accordingly with frequencies being 3.7 cps.

The tests were carried out by steps determined by the eccentricity mass. At every step the resonance occurred with the frequency of excitation being increased and decreased.

The results of tests. At the highest level of loading, the amplitude of fragment displacement reached 31.5 mm ("B-2") and 46 mm ("B-3"). The shearing force at the ground floor reached 2000 kN and 2450 kN, it exceeds the design seismic load calculated for the fragment by 2.9 and 3.5 times. Besides, there occurred considerable damages in structures which did not lead, however, to the exhaust of the fragment bearing capacity. The damages consisted of openings in horizontal joints, gaps between the foundation and the base, cracks in support areas of lintels and cracks in certain vertical joints of lower storeys. Particularly, the opening of horizontal joints was up to 1 mm, the displacement along the horizontal joints of the ground floor - up to 0.4 mm and in the basement - up to 1.3 mm. These damages led to the essential non-linearity of deformations: during "B-2" tests the resonance frequency was reduced from 2.86 cps up to 1.61 cps, the fragments rigidity - by 2.2 times, the base rigidity - by 3 times. "B-3" tests resulted in the further rigidity drop, the resonance frequency was decreased by 1.4 cps. There was developed a non-linear rated model of a large-panel building taking into consideration the abovementioned types of damages. With the help of this model a generalized **diagram** of the fragment deformation was obtained which was completely confirmed by the results of the tests (fig. 5). The calculations showed that the exhaust of the fragment bearing capacity was expected due to the crushing of wall compressed zones, the value of load being 2600-2800 kN (summarized force approximated 2000kN), the amplitude being about 80mm. In this way, the load reserve coefficient makes up 3.8-4, the plastic coefficient is about 20. The obtained diagram of the fragment deformation allowed to estimate the possible behaviour pf the structures under actual earthquake conditions, the calculations were made on earthquake accelerograms which are the nearest to the proper frequencies of the fragment in the terms of the spectral composition (Ferndale, Hollister, Eureka, El Centro, San Fernando, Karpatskey, Gazli). Two ultimate types of hysteretic properties were considered - with degrading rigidity (fig. 6A) and elastic-plastic with constant rigidity (fig. 6B). The calculations showed that during the tests there was created a higher level of a tress-deformative state than it is possible under real 8-point earthquakes (fig. 7). The state of the fragment under tests as well as the calculations on real earthquake accelerograms testify that the ultimate state of the fragment designed for the 7-point rated seismicity can occur only under real 9-point earthquakes.

References

1. S.V.Polyakov. Aftereffects of intensive earthquakes. Moscow, Stroyizdat, 1978.

2. G.N.Ashkinadze, L.B.Gendelman. The state of large-panel buildings in Gazli after earthquakes on 8 April and 17 May 1976. A collection of articles "Seismoresistant construction", No 11, Moscow, TsINIS, 1976.

3. G.N.Ashkinadze. Non-linear deformation of structures of large-panel buildings under oscillations. "Residential construction", No 7, 1977.

4. Structural behaviour of residential buildings made out of large-size elements, No 4, Moscow, Stroyizdat, 1974.

5. Ibanez P., Spencer R. Experience with a field computerized vibration analysis system. ANCO Engineers, Santa Monica, California, 1979.

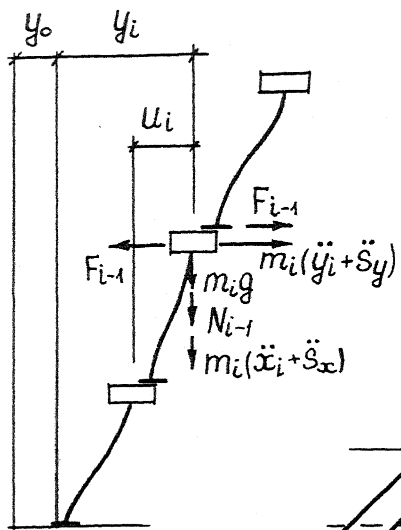

Fig. 1. The rated model of buildings in Gazli

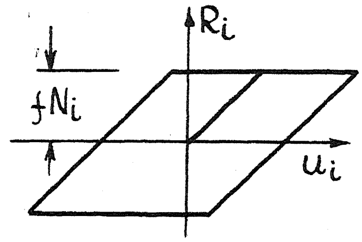

Fig. 2. Diagram of storey deformation

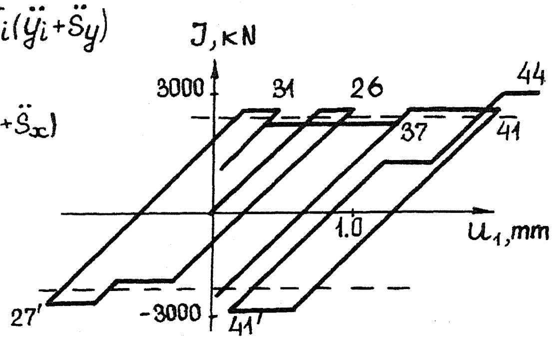

Fig. 3. Dependence "inertia force-storey deformation" during earthquake for the second floor of 2-storey building

Table. The rated stresses and wall strength

Wall	Buildings	N floor	N Compressive force kN	M Moment from rated loads, kNm	Ultimate length of compressed zone sustaining force N	Ratio of ultimate moment to rated one [M]/M	[Q]/Q Panel	[Q]/Q Joint
Blind panel, 5.7 m long	4-storey buildings	4	120	87			22.0	1.87
		3	225	235	0.16	2.7	10.1	1.88
		2	330	435	0.2	2.11	7.65	2.1
		1	435	660	0.25	1.81	6.6	2.5
	2-storey	2	120	74			25.2	2.07
		1	225	190	0.16	3.3	15.2	2.46

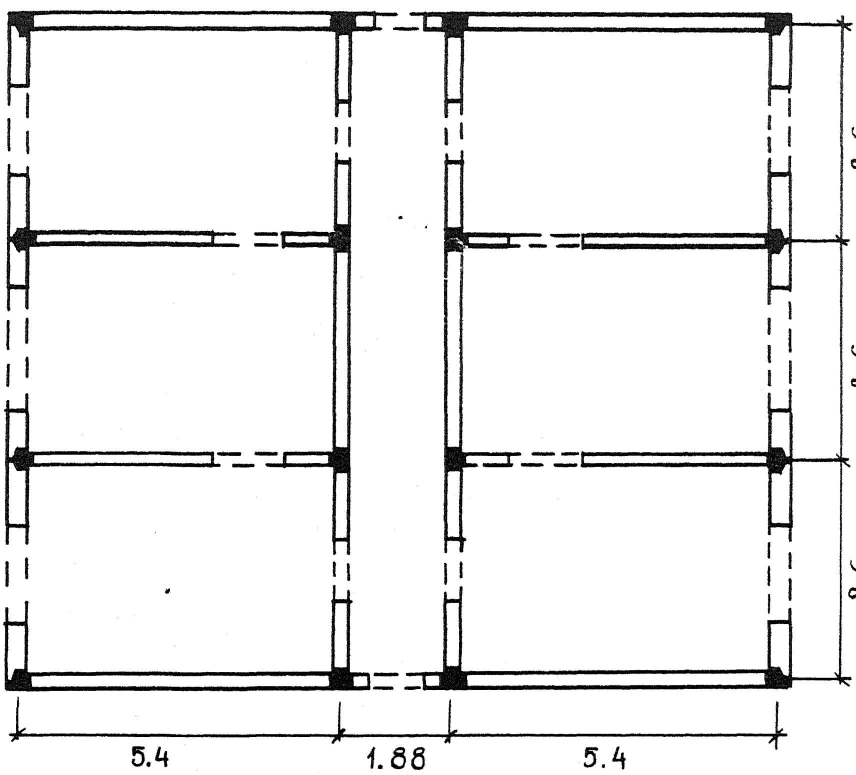

Fig. 4. Plan of fragment of 9-storey building

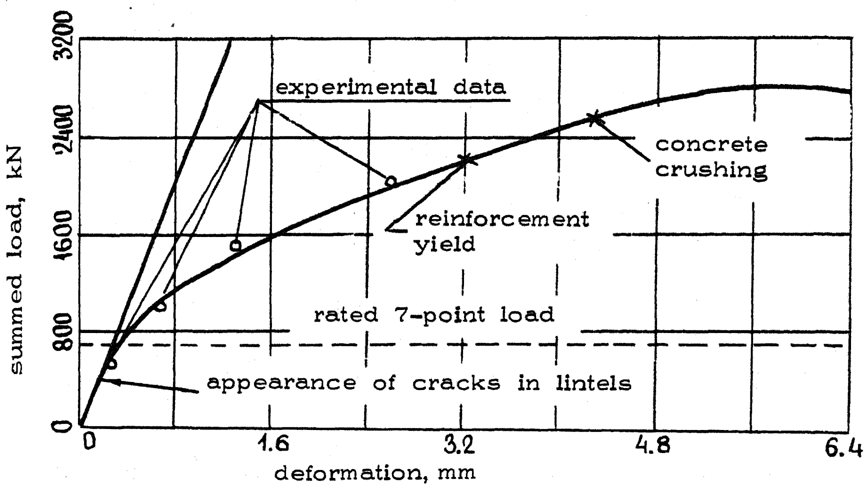

Fig. 5. Deformation of upper structure of fragment

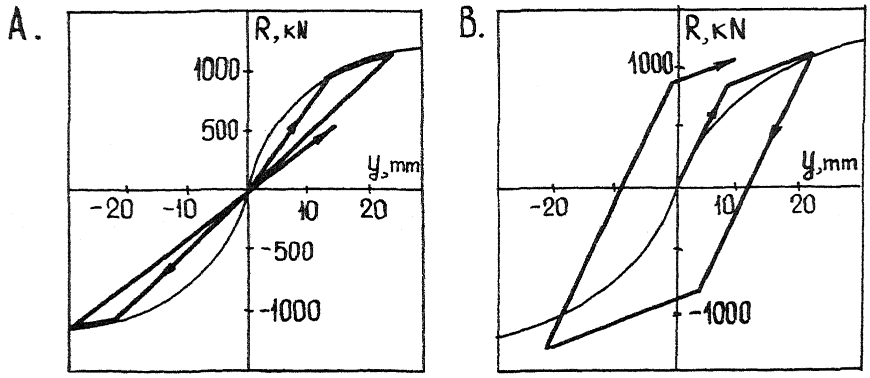

Fig. 6. Two rated hysteresic dependences "restoring force - displacement"

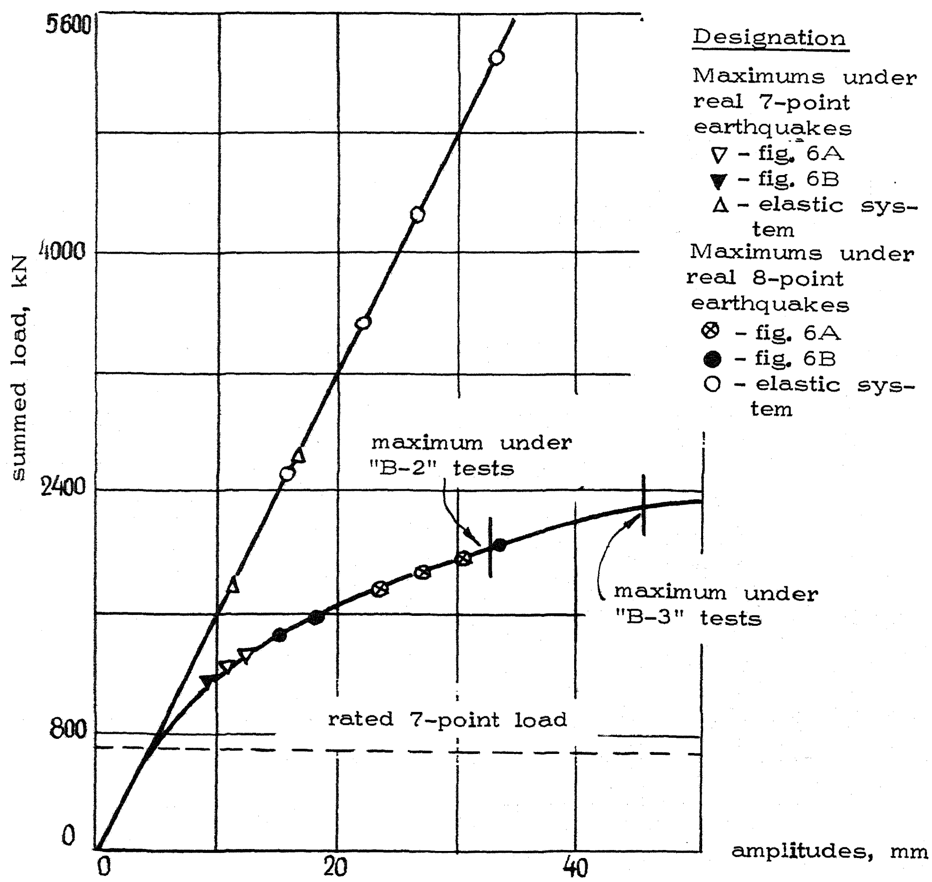

Fig. 7. Relationship between loads and displacements under seismic effects and tests