INVESTIGATIONS INTO EARTHQUAKE RESISTANCE OF LARGE-PANEL BUILDINGS

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The earthquake resistance of large-panel buildings has not yet been tested by intensive earthquakes. In the Tashkent earthquake in April 26, 1966, large-panel buildings happened to be in the zone of low earthquake intensity (approximately up to 6.5). Therefore for checking up the assumptions adopted in design as well as for obtaining data on the behaviour of large-panel buildings subject to horizontal loads of high intensity of great importance are experiments conducted on full-scale structures or their models and experiments on individual load-bearing elements and their joints subject to various forces which to this or that extent have the specific features of seismic loads (1).

For the last years the Central Research Institute of Building Structures and other institutes of the USSR have carried out an extensive programme of such investigations. The results of some investigations are reported here.

1. Seismic loads are specified in the Soviet Code of Practice according to the expected intensity of earthquake and the dynamic characteristics of the structure. Therefore one of the most important tasks of investigations was to define the periods, forms and logarithmic decrements of the damping of natural oscillations of large-panel buildings. With this aim in view full-scale tests of large-panel buildings were conducted, using various kinds of inducing oscillations, the recording of microseisms included (mean values of periods T_1 are given in Table 1). These tests were carried out on large-panel buildings of various structural schemes 4-12-storey high and on such large-panel buildings 4-18-storey high where one or two storeys had framework (2).

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Tests of buildings of the first group have shown that in evaluating design periods and forms of natural oscillations of large-panel buildings an approximate scheme may be used, considering the structure to be a contilever with a uniformly distributed mass and elastic fixing at the base; however, in defining the rigidity of the enclosing structures of the building it is necessary to take account of deformations due to shear and bending and the latter may be ignored for buildings up to 6-storey high. The tests of buildings of the second group have shown that deformations due to bending are predominant in the lower framed storeys of the buildings and that it is necessary to take account of deformations of the upper-storeys of large-panel buildings. (Fig. 1) At the same time these tests have shown that the rigidity of the more flexible (lower) part of the building is notably affected by such load-bearing structures as flights and partitions. Torsion oscillations have a considerable effect on the oscillations of the building the lower part of which is framed.

2. Records of microseisms give data only for very low stresses caused by small oscillations of the base. In this case the rigidity of the building may be considerably affected by friction in joints and other connections which is excluded with big amplitudes of horizontal oscillations. The records of microseisms do not give the answer to the question of how the rigidity of the building changes with the accumulation of damages (cracks) during intensive movement. Tests, using powerful vibro-machines or explosions, may give more full information. In 1967 in the place of Medeo (near Alma-Ata) two explossions (charges of 1117t and 2825t at an interval of 2.13 sec) were made for construction of a sill control dam. Test full-scale structures were built 800 m away from the place of explosion; they were a 4-storey large-panel building, a similar building with brick walls strengthened antiseismically and framed buildings (Fig. 2). Records of soil oscillations made at the moment of explosion are shown that the maximum displacement and acceleration of the soil at the site of the test buildings were 9 mm and 500 cm/sec^2 respectively. The acceleration of movements of the building foundation was 400 cm/sec^2 (Fig. 3). The period of natural oscillations of the building measures before the explosion was equal to 0.15 sec, the logarithmic decrement of daming being 0.3. With the maximum accelerations at the moment of explosion the building had an oscillation period of 0.25-0.3 sec which corresponded to the predominant periods of soil oscillations.

Inspection of the building after the explosion has shown that the walls of the brick building were heavily damaged (Fig. 4); as for the large-panel building its damages were confined to fine cracks at some joints which, however, did not lead to the considerable reduction of its rigidity. The period of natural oscillations of the large-panel building recorded after the explosion is equal to 0.18 sec. The forms of oscillations of the large-panel building are shown in Fig. 5[I)].

3. Failure tests on full-scale buildings (or up to notable damages) are very expensive and may be carried out only in rare cases. More feasible are tests on models erected on powerful vibro-platforms. Such tests provide ample information on the dynamic characteristics of buildings and their elements, distribution of forces, character and sequence of the failure of the structure. Such experiments make it possible to check up various stages of design.

TsNIISK has carried out a series of tests on models of large-panel buildings from 3 to 12 storey high (Fig. 6). Models made from reinforced concrete panels were erected on a powerful vibro-platform with a table 4x6 m in size (3). To define the most convenient scale of the model both in respect to its manufacture and the accuracy of the test results to be obtained three models differing only in scale (1/2, 1/4 and 1/6 of the full size[II)]) were made for testing one of the series (1-464) of 4-storey buildings. These model tests have shown the same results and therefore the scale of 1/6 was further used for the models of higher buildings (1, 2, 3). The models were fixed on the platform so as to provide an elastic rotation and displacement at the base similar to those at the foundation in soils of medium density. Together with the tests of 4-storey models measurements were made of natural oscillations of buildings which allowed to employ the method of simple similarity (4). Comparison of the test results for the model and the full-scale structure is given in Table 2.

When the accelerations of oscillations of the base in models of 1/4 of the full-scale reached 0.12 g (which corresponded to 0.47 g for full-scale values), cracks appeared at some horizontal joints of the model (Fig. 7) and its periods of natural oscillations increased by 18 per cent. After this tests were stopped. Judging from the test results of another series of models (5), such increase of the period is not dangerous for large-panel buildings, since heavy damages of structures are observed with the decrease of the periods of natural oscillations 2.5-3 times.

The comparison of values of accelerations of the base which caused small cracks at the joints of the model of a large-panel building and the full-scale structure during explosion in Medeo shows their good agreement (in both cases - about 0.5 g).

I) Tests in Medeo were carried out by the Institutes "KazpromstroyNIIproekt", TsNIISK, Research Institutes of Ashkhabad and Dushanbe. These tests are continued at present.

II) The model in 1/2 scale of the full size was tested jointly by the Research Institute of Dushanbe (TISSS) and TsNIISK.

4. The tests of the above models allowed to check up the design methods used for defining the distribution of horizontal seismic force between individual load-bearing structures. It is usually assumed that it is possible to consider individual flat systems, being parts of the spatial enclosure of the building, as not depending on each other and resisting a part of the full seismic load. The latter is defined from the assumption that the floor is not deformed in its plane (6, 7).

To design with more accuracy it is necessary to take account of the actual rigidity of floors and walls according to the scheme shown in Fig. 8. The combined action of walls and floors in a building symmetric in plan[1)] makes it necessary to solve the following system of algebraic equations (8).

$$
\underbrace{\begin{array}{l}
\text{V equations}\left\{\begin{array}{l}
P_{11}\lambda_{1111}+P_{12}\lambda_{1211}+\ldots+P_{1v}\lambda_{1v11}+P_{21}\lambda_{2111}+\ldots+P_{2v}\lambda_{2v11}+\ldots+P_{\ell-1,1}\lambda_{\ell-1,111}+\ldots+P_{\ell-1,v}\lambda_{\ell-1,v11}+\lambda_{11p}=0\\
P_{11}\lambda_{1121}+P_{12}\lambda_{1221}+\ldots+P_{1v}\lambda_{1v21}+P_{21}\lambda_{2121}+\ldots+P_{2v}\lambda_{2v21}+\ldots+P_{\ell-1,1}\lambda_{\ell-1,121}+\ldots+P_{\ell-1,v}\lambda_{\ell-1,v21}+\lambda_{21p}=0\\
\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\\
P_{11}\lambda_{11v1}+P_{12}\lambda_{12v1}+\ldots+P_{1v}\lambda_{1vv1}+P_{21}\lambda_{21v1}+\ldots+P_{2v}\lambda_{2vv1}+\ldots+P_{\ell-1,1}\lambda_{\ell-1,1v1}+\ldots+P_{\ell-1,v}\lambda_{\ell-1,vv1}+\lambda_{v1p}=0
\end{array}\right.\\
\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\\
\text{V equations}\left\{\begin{array}{l}
P_{11}\lambda_{1112}+P_{12}\lambda_{1212}+\ldots+P_{1v}\lambda_{1v12}+P_{21}\lambda_{2112}+\ldots+P_{2v}\lambda_{2v12}+\ldots+P_{\ell-1,1}\lambda_{\ell-1,112}+\ldots+P_{\ell-1,v}\lambda_{\ell-1,v12}+\lambda_{12p}=0\\
P_{11}\lambda_{1122}+P_{12}\lambda_{1222}+\ldots+P_{1v}\lambda_{1v22}+P_{21}\lambda_{2122}+\ldots+P_{2v}\lambda_{2v22}+\ldots+P_{\ell-1,1}\lambda_{\ell-1,122}+\ldots+P_{\ell-1,v}\lambda_{\ell-1,v22}+\lambda_{22p}=0\\
\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\\
P_{11}\lambda_{11v2}+P_{12}\lambda_{12v2}+\ldots+P_{1v}\lambda_{1vv2}+P_{21}\lambda_{21v2}+\ldots+P_{2v}\lambda_{2vv2}+\ldots+P_{\ell-1,1}\lambda_{\ell-1,1v2}+\ldots+P_{\ell-1,v}\lambda_{\ell-1,vv2}+\lambda_{v2p}=0
\end{array}\right.\\
\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\\
\text{V equations}\left\{\begin{array}{l}
P_{11}\lambda_{111\ell-1}+P_{12}\lambda_{121\ell-1}+\ldots+P_{1v}\lambda_{1v1\ell-1}+P_{21}\lambda_{211\ell-1}+\ldots+P_{2v}\lambda_{2v1\ell-1}+\ldots+P_{\ell-1,1}\lambda_{\ell-1,11\ell-1}+\ldots+P_{\ell-1,v}\lambda_{\ell-1,v1\ell-1}+\lambda_{1\ell-1p}=0\\
P_{11}\lambda_{112\ell-1}+P_{12}\lambda_{122\ell-1}+\ldots+P_{1v}\lambda_{1v2\ell-1}+P_{21}\lambda_{212\ell-1}+\ldots+P_{2v}\lambda_{2v2\ell-1}+\ldots+P_{\ell-1,1}\lambda_{\ell-1,12\ell-1}+\ldots+P_{\ell-1,v}\lambda_{\ell-1,v2\ell-1}+\lambda_{2\ell-1p}=0\\
\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\\
P_{11}\lambda_{11v\ell-1}+P_{12}\lambda_{12v\ell-1}+\ldots+P_{1v}\lambda_{1vv\ell-1}+P_{21}\lambda_{21v\ell-1}+\ldots+P_{2v}\lambda_{2vv\ell-1}+\ldots+P_{\ell-1,1}\lambda_{\ell-1,1v\ell-1}+\ldots+P_{\ell-1,v}\lambda_{\ell-1,vv\ell-1}+\lambda_{v\ell-1p}=0
\end{array}\right.
\end{array}}_{(\ell-1)\text{ groups of equations}} \tag{1}
$$

The coefficients of the unknowns P_{km} are defined by the following formulaes:

with the symmetric position of the indices

$$\lambda_{nmmn} = \delta_{nmm} + 2\,\delta_{\ell mm} + \delta^{n}_{mnmn} \tag{2}$$

where ℓ is the number of the middle diaphragm; only with symmetric position of extreme indices

$$\lambda_{nmkn} = \delta_{nkm} + 2\,\delta_{\ell km} \tag{3}$$

only with symmetric position of middle indices

I) The load was also assumed to be symmetrical in respect to axis "ℓ".

$$\lambda_{nmmk} = 2\,\delta_{\ell mm} + \delta^{n}_{mkmn} \tag{4}$$

with asymmetric position of indices

$$\lambda_{nmks} = 2\,\delta_{\ell km} \,. \tag{5}$$

The following symbols are used for expressions (1) - (5)

S_{km} - lateral concentrated load, acting at the place of crossing diaphragm "K" at the level of floor number "m";

P_{km} - force, acting in diaphragm number "K" at the level of floor number "m" ($P_{km} \neq P_{mk}$);

δ_{knm} - displacement of diaphragm number "K" at the level of floor number "n" due to lateral unit force applied at the level of floor number "m" (Fig. 9);

P^{n}_{ni} - force, acting on floor number "n" at diaphragm number "i";

δ^{n}_{nkni} -displacement of floor "n" at diaphragm "g" due to the action of a unit force applied at the level of floor "n" at diaphragm "i" with diaphragm "1" being rigid (Fig. 10).

Indices of coefficients λ of the unknowns P are put in the following order: the first two repeat the indices of P, the third corresponds to the equation number in the group and the fourth - to the number of the group containing the equation.

Since

$$\lambda_{nkmn} = \lambda_{nmkn} \;; \quad \lambda_{nkms} = \lambda_{smkn} \tag{6}$$

the system (1) may be considered to be symmetric to the main diagonal.

Free members of equations are defined by the formula

$$\lambda_{t\tau p} = -\sum_{K=1}^{K=v} \delta_{\ell tK} \left[2 \sum_{s=1}^{s=\ell-1} S_{sK} + S_{\ell K} \right] - \sum_{s=1}^{s=\ell-1} S_{st}\, \delta^{n}_{t\tau ts} \tag{7}$$

where t - floor number, corresponding to the equation number within one group of the system of equations (1);

r - number of the diaphragm, corresponding to the number of equation group of the system (1);

ℓ - number of the middle diaphragm.

Solution (ℓ) allows to obtain lateral forces for which diaphragms should be designed as cantilevered systems except diaphragm " ℓ ". The floor is designed for concentrated forces p^{n}_{ni} which are defined by the formula

$$p^{n}_{ni} = S_{in} - P_{in} \tag{8}$$

Diaphragm " ℓ " as well as the cantilevered system is designed for the action of forces applied at the floor level t which are defined by the formula

$$P_{\ell t} = S_{\ell t} + 2\sum_{s=1}^{s=\ell-1}\left(S_{st} - P_{st}\right) \tag{9}$$

Using the results of design of space structures, it became possible to define horizontal deflections of all nodes in a 3-storey model and compare them with the test results; the comparison showed a good agreement of the test and design data (Fig. 11). The divergency in desgin and experimental values for the 3-storey model does not exceed 10%. Values of transverse forces at the level of each floor which were defined in conformity with the Soviet Codes (7) and by the above-mentioned design method, taking account of space conditions, agree well, although at some sections the divergency reaches 50 per cent. Such a divergency is observed for transverse forces at the levels of upper floors. Since in designing individual panels and nodes for 4-5-storey buildings account is taken of transverse forces at the levels of lower floors as well as bearing in mind that the thickness of the cross-section of the diaphragm is constant along the whole height of the building, it may be recognized that the design methods specified in the existing Codes for the above buildings are acceptable. For higher buildings, when the seismic load largely increases and the thickness of the cross-section of the diaphragms is not always constant along the height of has a different reinforcement in lintels account should be taken of the spatial behaviour of the structure.

At present tests of the model of a part of a 12-storey building made in 1/6 scale of the full size are carried out, aiming to further check the method of designing structures of large-panel buildings (Fig. 12).

5. Special tests of flat cross walls (solid and with one row of openings) have been conducted to take account of the yielding of joints, displacements at the base and other factors which characterize the displacements of individual cross walls and floors in system (1) (Fig. 8). For the above spatial model in 1/4 scale the design analysis of walls with openings was made as for frames, using systems of 3-member canonical equations of the method of forces in a manner described in the work (2, 9) for symmetric walls (and skew-symmetric loads) and using 5-member equations (9) for asymmetric walls.

The models of cross walls were parts of the building as if divided at centres of the adjacent window openings. Horizontal forces were applied at the level of floors through a system of distribution beams. Methods of designing beams were programmed for an electronic computer "Ural-4" and displacements for all the three types of diaphragms under horizontal loading were defined. Characteristics of the yielding of the base and joints were assumed in conformity with the experimental data.

The obtained theoretical and experimental values of deflections coincide sufficiently well (Fig. 14, a). Tests have shown that evaluation of displacements without due regard to the yielding of the base and joints results in a big divergency of experimental and theoretical values of deflections and forces and divergency in frequencies of natural oscillations tions. This is seen from the diagram (Fig. 14, b) of changing transverse forces T_t in lintels (t is the number of the floor over which the lintel is provided); the continuous line shows the change of transverse forces in a cast-in-place wall, while the dotted line - in a precast wall. Fig. 14, b shows that the presence of joints largely increases the values of transverse forces in lintels which will increase with the increase of the yielding of joints as compared to the rigidity of panels proper.

6. Of great importance for the earthquake resistance of large-panel buildings is a reliable behaviour of joints. Several designs of joints to be used for various series of large-panel buildings have been developed in the USSR. The evaluation of the strength of joints by testing on small models is difficult; therefore tests of joints at TsNIISK are, as a rule, carried out on full-scale samples, Samples for testing horizontal and vertical joints used in buildings of series I-464-AC (Fig. 15), I-467-AC (Fig. 16) and ПС (Fig. 17) were made in the form of two jointed parts of panels (Fig. 18). The process of manufacturing samples was exactly the same as that used in practice for jointing panels in the building.

Samples were tested under the static and cyclic shear loading. The latter case involved 1,000 cycles of regular loadings with various values of Q_{max} and coefficient of asymmentry $\rho = 0.2$ with the frequency of repetition of 5 hertz which is close to the freqnency of natural oscillations of a large-panel building up to 5-storey high inclusive. The diagram of applying the shear cyclic load on the samples is given in Fig. 18. After the pulsation of loading was stopped, the samples were unloaded and then subjected to gradually increasing static loading up to failure. Some samples, being tested by a statid shear load, were at the same time subject to compression or tension perpendicularly to the plane of the joint.

The damage of horizontal joints began with the formation of cracks in the mortar joint and in the cast-in-situ concrete placed in the joint during the erection of panels. In vertical joints the first cracks appeared along the contact between the concrete of the panel and the concrete filling the joint. The mean values of shear static loads when the first cracks Q_{TP} appeared in the cast-in-place concrete for horizontal joints were within the range of 60–100 per cent of the value of the failure load Q_p, while for vertical joints of series ПС - 50 per cent of Q_p. Since the repair of joints when the concrete is damaged is very difficult, load Q_{TP} was assumed to be the initial in evaluating the load-bearing capacity of joints.

Pulsation of the load did not, as a rule, result in a notable reduction of the strength of joints (even in that case when samples with small cracks which appeared under static loading were subject to it). Exceptions were horizontal joints with rigid reinforcement (Fig. 15b). In tests of these joints the load Q_{TP} was reduced due to pulsation by more than 1/3.

Tests have shown a clear relationship between the shear load Q_{TP} and the values of compressive (N_c) or tensile (N_p) forces normal to the joint (Fig. 19).

Tests have also shown that the following formula may be used to define the design shear strength of joints (7, 10).

$$Q_{TP} = K b l \sqrt{R_{np}} \left(1 + 2.5 \cdot 10^{-5} R_a n d\right) \qquad (10)$$

K - empirical coefficient for testing joints K = 1.7.

When there are compressive forces normal to the joint, the shear strength increases which can be taken into account by the formula

$$Q_{TP.CЖ} = Q_{TP} + \frac{2.5}{\sqrt{R_{np}}} \qquad (11)$$

When there are tensile forces normal to the joint, the shear strength decreases

$$Q_{TP.P} = Q_{TP}\left[1 - \left(\frac{N_P}{F_a R_a}\right)^2\right] \qquad (12)$$

The following symbols are assumed in formulae (10)-(12).

b and l - thickness and length of the cross-section of the joint in shear (cm);

R_{np} - design compressive strength of the joint (kg/cm^2), prism strength;

R_a and F_a-design strength (kg/cm^2) and the area of the cross-section of the reinforcement in the joint (cm^2) respectively;

d and n - diameter (cm) and number of reinforcement projections in the joint.

7. As the model tests have shown, the load-bearing capacity of panels proper is defined by their resistance to horizontal and vertical forces in their planes which leads to the distortion of panels. Samples were made from concretes of various strengths and vibro-masonry (for vibro-brick and vibro-block panels). Tests have shown that for the concrete panels the value of the horizontal load N, causing the first cracking may be defined by the formula:

$$N = R_{cp} L \frac{d}{K_\tau} \tag{13}$$

where N - horizontal load which causes the first cracks at the centre of the panel (kg) when $\tau = R_{cp}$;

τ - tangent stresses (kg) at the centre of the panel respectively;

R_{cp} - shear strength of the panel material (kg/cm^2). For concrete panels $R_{cp} = 0.6\sqrt{R_{np} \cdot R_p}$;

R_p - tensile strength of the panel material (kg/cm^2);

L and d - length and thickness of the wall panel (cm);

K_τ - coefficient of maximum tangent stresses $K_\tau = \frac{\tau L^2 d}{Nh}$;

h - height of the wall panel (cm).

More detailed data on these tests and design of panels for distortion are given in work (11).

Conclusions

The conducted investigations allow to consider large-panel structures to be sufficiently reliable and economical for use in seismic regions. These investigations allowed to develop detailed methods of designing such structures, their design being made in conformity with Codes of Practice.

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Table 1

Purpose and load-bearing structures of the building I)	Dimensions in plan; m	Number of floors	Period of the main form of natural oscillations (sec), T in the direction transverse	longitudinal
1. Large-panel blocks of flats of series 1-464AC	12x72	4 5	0.17 0.3	0.155 0.24
2. Sanatorium of series 1-467 AC	13.5x70.4	7	0.35	0.28
3. Made from vibro-rolled panels	12x86.2	9	0.42	0.32
4. Large-panel block of flats	12x67.2	12	0.52	0.36
5. Large-panel blocks of flats with framed (reinforced concrete) lower one or two storeys used for shops	12x176 14.7x19.3 12x176 13.6x93.5	4 9 10 18	0.28 0.36 0.64 1.14	0.3 - 0.44 1.05

I) Base soils of medium density

Table 2

Characteristics	Period of the main form of natural oscillations				
	measured (sec)	designed for a model (sec)	per cent of divergency (sec)	reduced to the full-scale II) (sec)	relative values
Model in 1/2 scale	0.065	0.063	3.9	-	-
Model in 1/4 scale	0.059	0.057	3.5	0.193	1.02
Model in 1/6 scale	0.05	0.052	3.4	-	-
Full-scale structure	0.19	-	-	0.19	1

II) Taking account of the adjustment for various values of loads for simulation of the dead weight

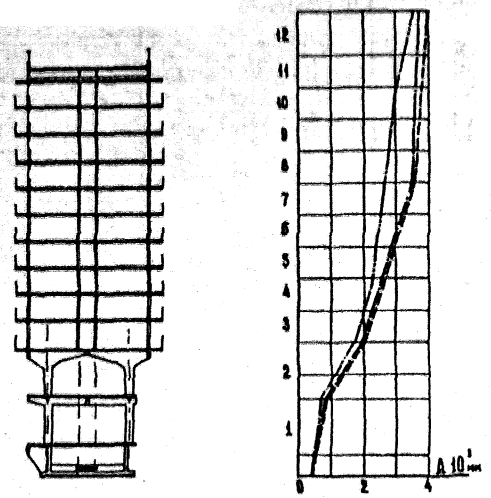


Fig.I.Large-panel building with the framed first storey and forms of its oscillations in the longitudinal direction.

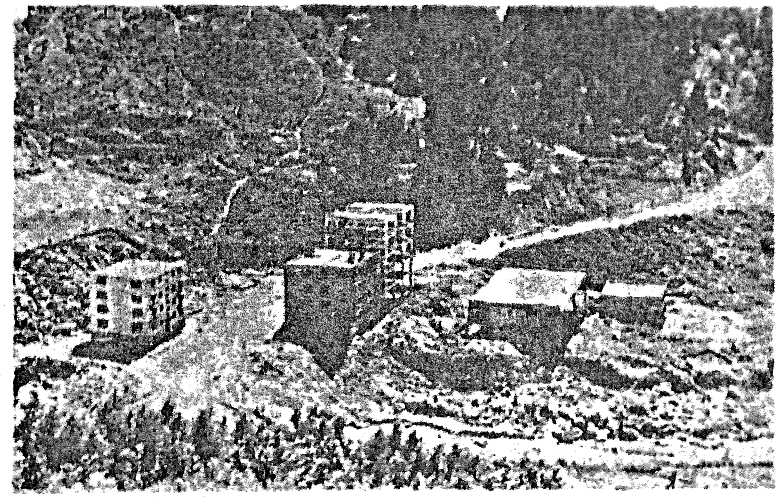


Fig.2.General view of parts of the building in Medeo.

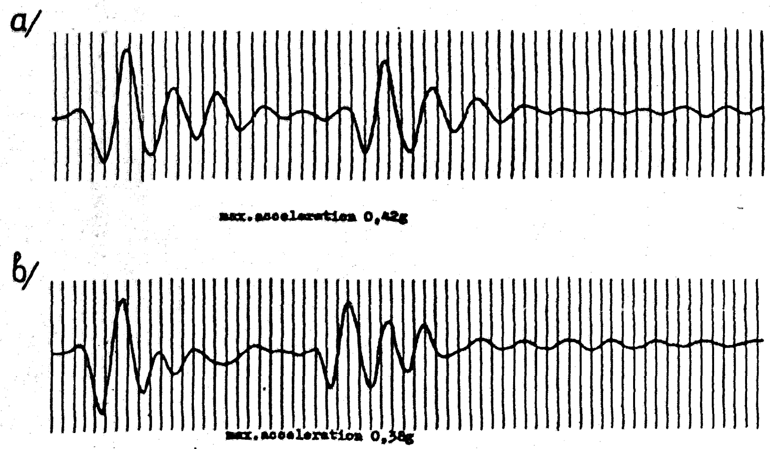


Fig.3. Oscillograms of vibrations of foundations of large panel (a) and brick (b) buildings during the explosion.

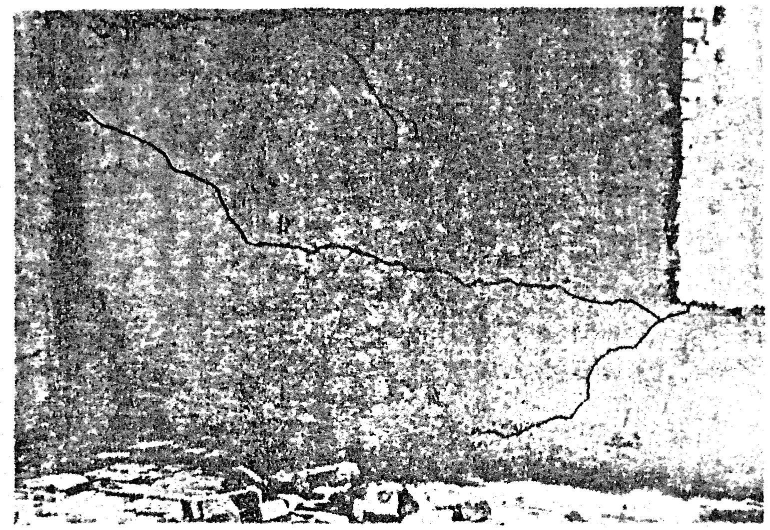


Fig.4.Damages in the wall of the brick building after the explosion.

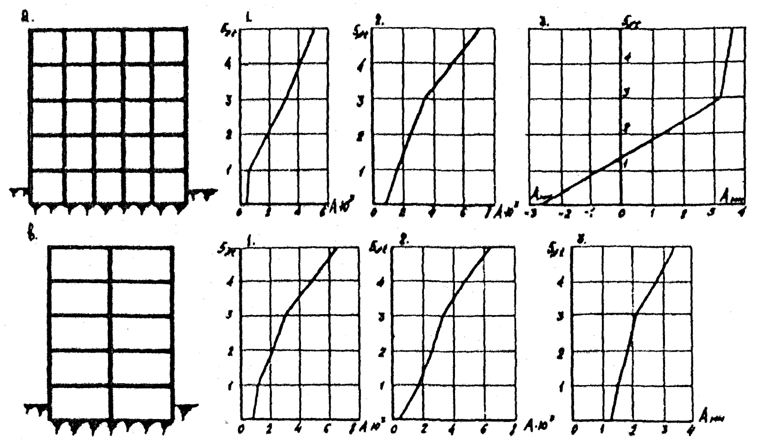


Fig.5.Diagrams of the first form of oscillations of a part of the large-panel building in Medeo in the (a) longitudinal and (b)transverse directions.I.-before the explosion,2-after the explosion;3-during the explosion.

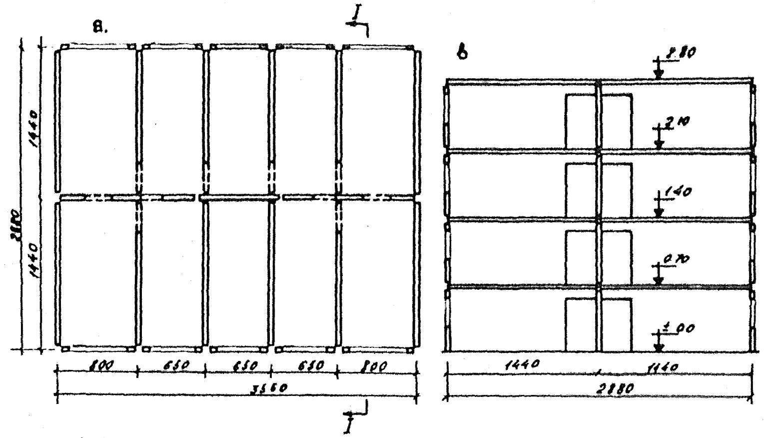


Fig.6.Plan (a) and elevation (b) of the model of the building in I/4 scale of the full size.

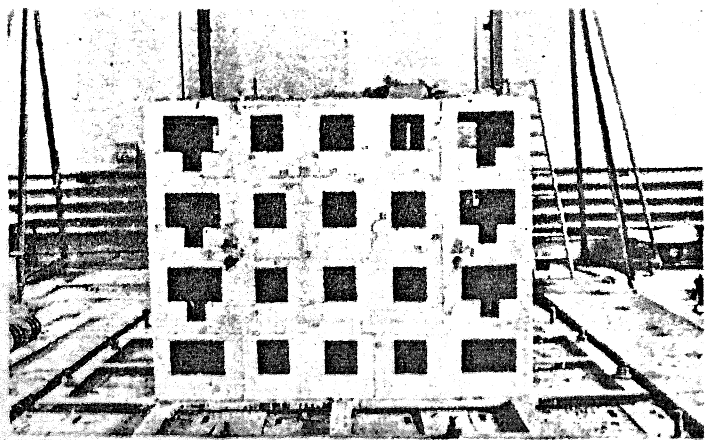


Fig.7.Model of the large-panel building in I/6 scale of the full size after the test.

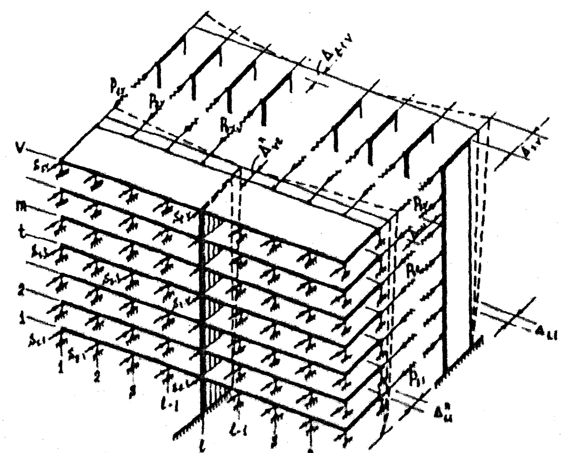


Fig.8. Design scheme of large-panel building.

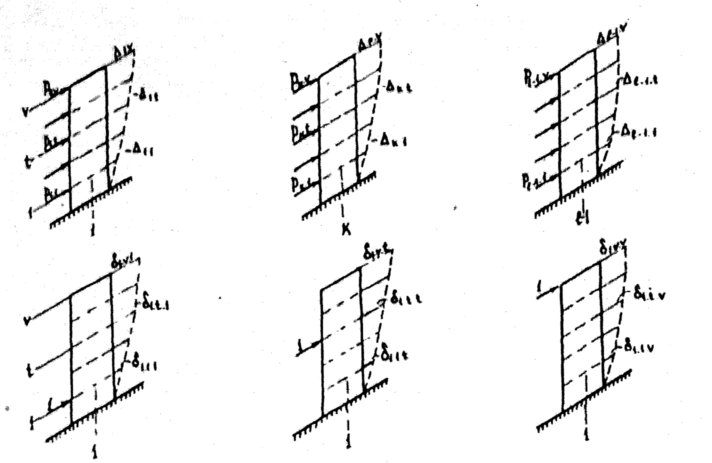


Fig.9.Design scheme for defining displacements of vertical diaphragms of the building (walls).

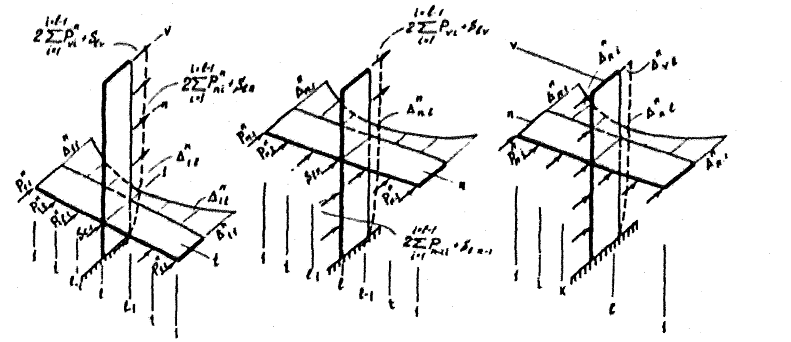


Fig.10.Design scheme for defining displacements of horizontal diaphragms of the building (floors).

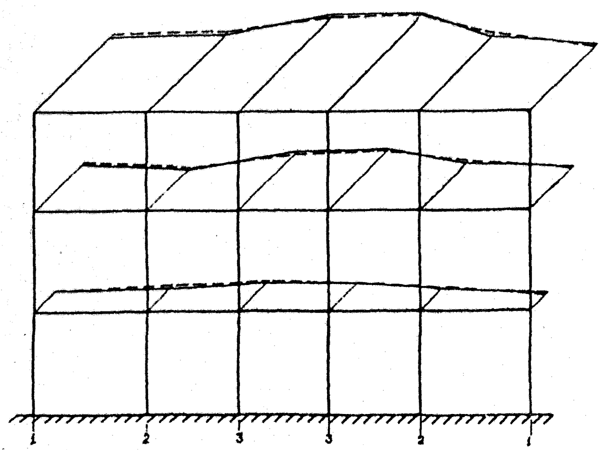
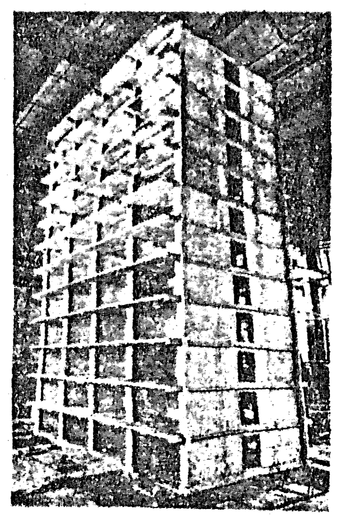


Fig.11. Horizontal deflections of the model of large-panel building obtained in practice (- - - -).



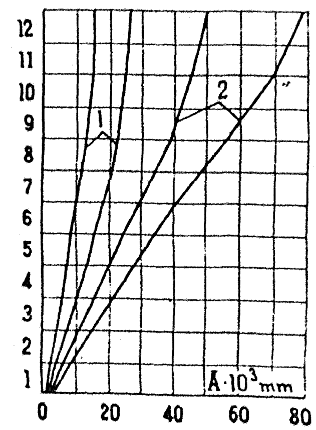


Fig.12.Model of a 12-storey large-panel bilding and diagrams of the form of oscillations in the longitudinal (1) and transverse (2).

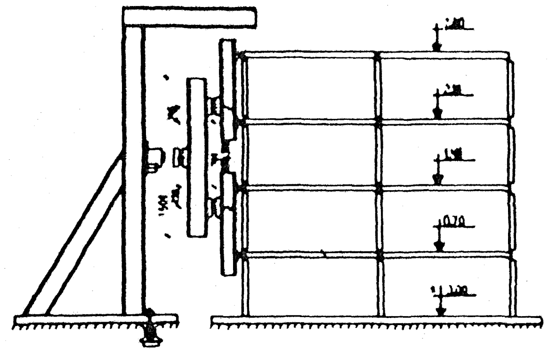


Fig.13.Scheme of testing the model of a vertical diaphragm of the building - wall for horizontal load.

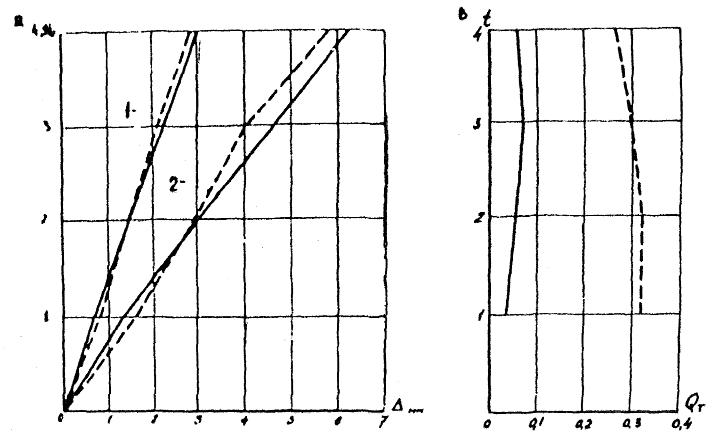


Fig.14. a)Deflections of diaphragms of cross walls obtained in practice (– – – –) and by design (———).1.for a continuous diaphragm,2.for a diaphragm with an aperture. b)Diagrams of variations of the transverse force in lintels with (– – –) and without (———) due account of yielding of joints.

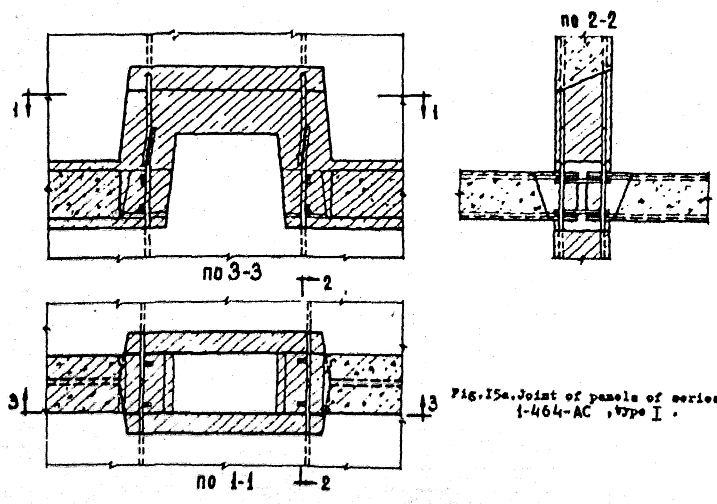


Fig.15a.Joint of panels of series 1-464-AC ,type I .

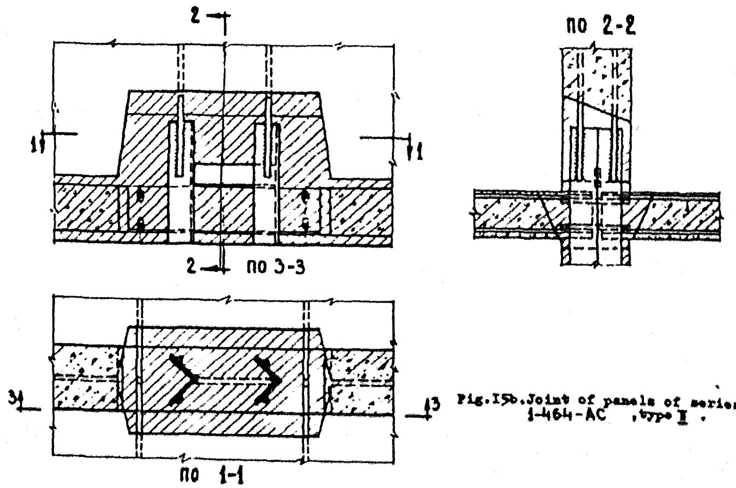


Fig.15b.Joint of panels of series 1-464-AC ,type II .

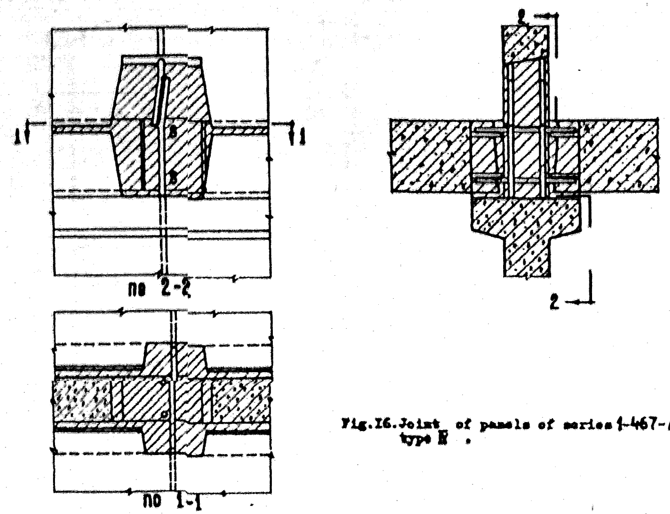


Fig.16.Joint of panels of series 1-467-AC, type II.

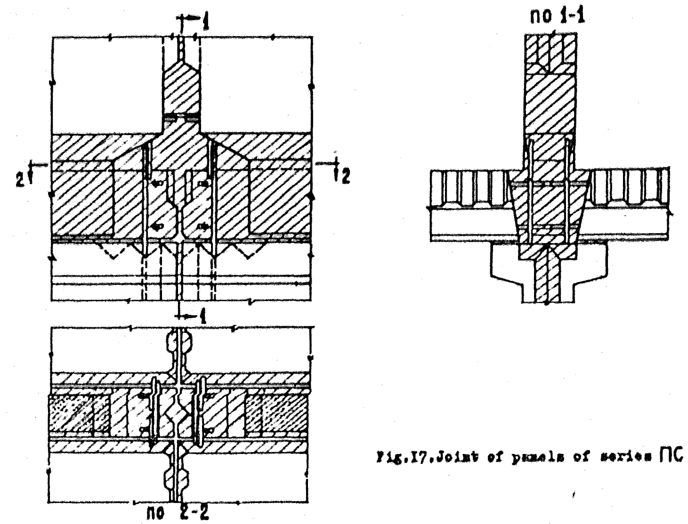


Fig.17.Joint of panels of series ПС, type V.

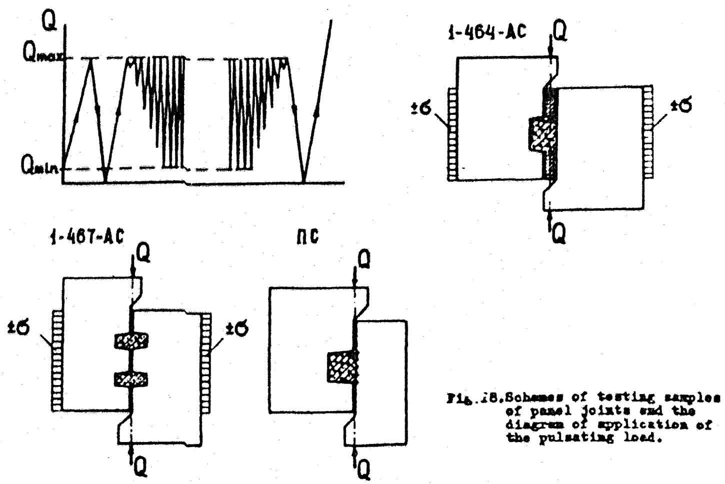


Fig.18.Schemes of testing samples of panel joints and the diagram of application of the pulsating load.

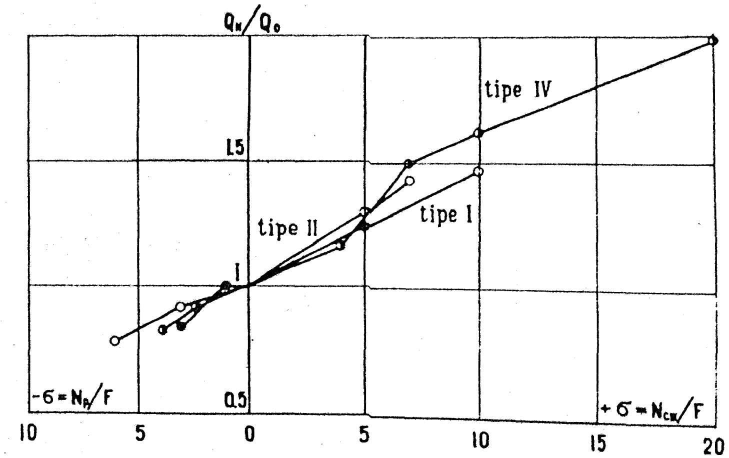


Fig.19.Diagram of the effect of the normal on the load-bearing capacity of the joint in shear.