STUDY OF NON-LINEAR WORK OF CONSTRUCTION OF REINFORCED PRECAST CONCRETE AND MONOLITHIC FRAMEWORK BUILDINGS BY POWERFUL VIBRATION GENERATORS

by

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Non-linear responce of structures to earthquakes has become an object of a vast investigation. This may be observed both in experimental and in theoretical analysis of bearing capasity and earthquake resistance of precast and monolithic structural frames of residential and public buildings.

Such investigations are carried out in the Strength Tests Laboratory by the resonant method of testing full-scale buildings by means of high-power vibrational generators.

The resonant testing method and the type of generator used make it possible to simulate a wide range of, inertial loads applied to the tested structure, from the slight disturbances (microseisms) to the design earthquake loads and even higher, The range of frequencies obtained on the generator's axle make it possible to induce practically all the natural modes and resonant frequencies nesessary for analysis and design of structural frames (rod modes, three-dimensional and torsional modes, including higher modes).

Actual values of the design parameters, as well as their variations may be traced by loading the tested structure with a gradually increasing inertial load; these parameters are as follows: natural modes and corresponding frequencies, generalized rigidity, damping characteristics etc.

In its turn, the analysi of shanges in the parameters makes it possible to evaluate the non-linearity of the structural responce to increasing inertial loadings during the tests.

Table I contains experimental data on tests of nine different structural frames; typical natural frequency spectra of these buildings are presented in fig.I.

The frequency ratio of the first four vibrational modes of structural frames lae between I:2,5:4:5 and I:3:5:7,5; this ratio is about I:3:5:7 for few-story frames.

As the floor slabs are much more rigid than the vertical frames, the horizontal in-plane vibrational bending modes are hardly traced for structural frames. On the contrary, torsional vibratios of such buildings obtain substantial significance due to the low rigidity of the lateral frame, because their frequency is close to the principal vibrational frequency (fig.I), thus easily induced by any disturbance, and, besides, these vibrations cause overloading and even destruction of extreme columns (fig.2).

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The torsional modes are diffisult to separate because the corresponding frequencies lie close to the translatäonal frequencies; therefore two generators were located in counter-phase upon frame N8, which made it possible to determine three torsional modes (I:2,78:4;9), although corresponding frequencies almost equaled the translational frequencies.

The magnituae of torsional vibrations, as well as their influence upon the earthquake resistance of buildings depend on the earthqual e loading irregularity and on the presence of rotational components in the earthquake movements over the complete building. Information obtained on the torsional modes of structural frames does not seam sufficient for account of torsional vibrations in earthquake disturbance, because of the lask of corresponding seismological data.

Damping ratio of natural modes differes substantially for separate buildings. The same phenomena is observed for displacement magnitudes in case the generators are located at the top level (table2). This difference may be explained, to a certain extent, by the difference of investigation methods with respect to various modes (for example, different speeds of passing the resonance, influence of location of generators etc.).This problems requires further investigation.

Non-linear load-displacement relation is usually observed in testing full-scale structures by lagre inertial loads approaching the design level. Besides the non-linearity of the materials of these structures, the actual non-linearity of structural responce is caused primarily by such phenomena as: brittle local destructions (mainly in nodes), gradual of spontaneous changes of rigidity, destruction of separate braces in the braced systems etc.(fig.3), as follows from the analysis of experimental data and from visual control of the state of tested structures.

Resonant tests discovered the following non-linear effects:
- relation between the vibrational parameters (natural frequency, damping etc.) and the amplitudes; for example, for frame N3 (see table N I) the initial frequency is I,65 hertz, the frequency at the design levelï is 0,95 hz, ior frame N4 (fig.4) these quantities are 3,12 and I,54, respectively (the first value is twice the second);
- non-linear relation between disturbañce level (mass of debalance) and amplitude of typical points;
- various residual phenomena (irresversible changes of rigidity, frequency etc.).

Howerer, in testing structural frames, no other typical non-linear phenomena were observed (special type resonant curves with unstable sections, deviations from harmonic ascillations under harmonic load etc.).

The bearing capacity of pure frames with rigid nods appeared to depend on the actual rigidity of the nods, or their actual priancy, which is reserse to the rigidity. This influence was observed earlier only in respect of deformability of frames (mainly in theoretical works). Pliansy of nodes appears to have the same significance for the strength of frames, especially of multistorey buildings.

The usual elastic design does not account for the actual redistribution of bending moments in the structural elements due to the observed nodal pliancy; howerer, in critical cases, such as earthquakes, this redistribution may cause excessive increas of bending moments and consequent hinging of the bottom columns, first at the bottom sections and then at the top sections; this will be followed by excessive translational displacements and by complete destruction of the building.

Thus, the pliancy of nods, which is usually non-linear, leada to changes of the design model of the structure accompanying its deflections under the loading, and to decrease of its bearing capacity. For example, the following values of the largest possible inertial loads were calculated for frame N3:

for absolutely rigid nods - 84,5 t;
for pliable (actual) nods - 55,0 t,

i.e. almost I,5 times lower.

Largest possible inertial loads correspond to the critical point of the "deflection-load" curve, i.e. to the point between the ascending and the descending branches of this curve.

building Nº / number of storeys

1 5st | I tors.I free I free II

— C.P.S.

2 5st | I | II | III | IV
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 → C.P.S.

3 4 st | I tors I (I)* | II (II) tors — C.P.S.

4 2 st | I — C.P.S.

5 9 st | (I) I | II | III — C.P.S.

6 12 st (I) I | (II) II | (III) | (IV) III — C.P.S.

7 11 st | (I) | I tors I (II) | (III) II | (IV) — C.P.S.

8 13 st | I tors I | II tors II | III tors III — C.P.S.

9 5 st | I — C.P.S.

0   1   2   3   4   5   6   7 → C.P.S.

— 2737 —
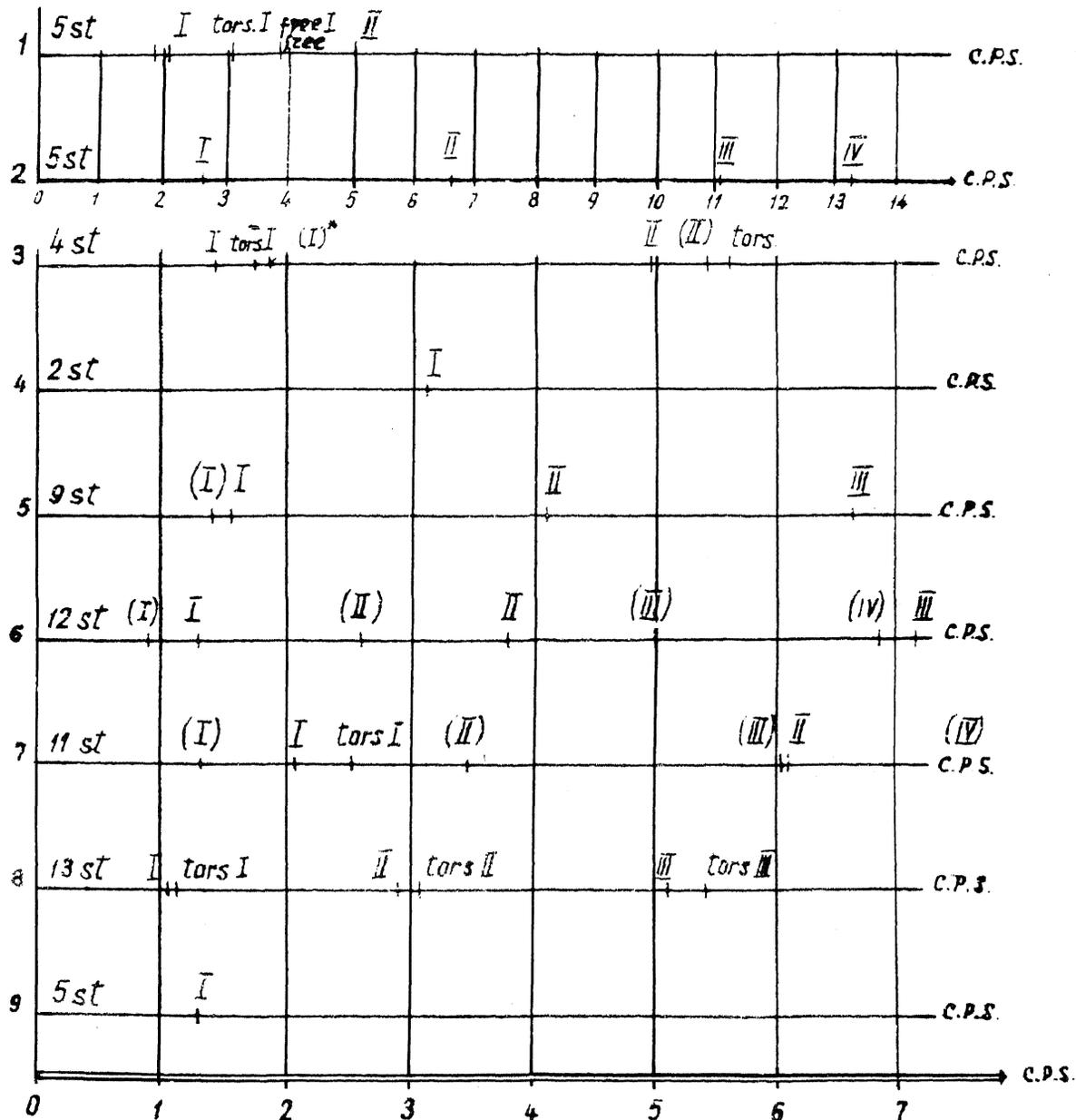
Building N	BREAF DESCRIPTION OF TESTED BUILDINGS	Dimensions of building in "m"	Direction of vibrations	First mode frequency in hz — At a min load	At a max load	At a max load — Max (t)	Maximum design load	Ratio of initial frequency and frequency of max load	Amplitude of deflection of the top level of building at max load	Acceleration at the top level of (fraction of "g") at maximum load	Degree of design earthquake region	Generator model	Damping for modes I	II	III	IY	A2/A1*	A3/A1	A4/A1
1	5-storeyed precast reinforced concret frame. Kishinev	30×12 H=20	Lateral / trans/lation	1,92 / 2,08	1,59 / 2,08	2,8 / 8,15	0,05 / 0,146	1,02 / 1,00	0,23 / 0,52	0,003 / 0,009	7	B-1	-	-	-	-	1,42 / -	-	-
2	5-storeyed precast and monolithic reinforced frame. Dushanbee	95×15,8 H=14	lateral / transl.	2,5 / 2,7	2,39 / 2,16	19,94 / 17,83	0,094 / 0,088	1,07 / 1,26	0,95 / 0,92	0,021 / 0,017	9	B-1	0,44 / 0,22	0,22	0,22	0,52	1,44 / 0,93	1,53 / -	2,00 / -
3	4-storeyed experimental precast-and-monolithic frame. Ashkhabad	15,6×11,9 H=11,7	lateral / transl.	1,65 / 1,46	0,98 / 1,43	42,0 / 8,38	0,893 / 0,306	1,73 / 1,02	26,0 / 5,66	0,094 / 0,046	9	B-2	0,14 / 0,09	- / 0,08	-	-	1,0 / 1,16	-	-
4	2-storeyed precast-and-monolithic frame. Public centre. Dushanbea	18×8 H=7,6	latе-ral	3,12	1,34	62,7	1,6	2,03	64,0	0,61	9	B-2	-	-	-	-	-	-	-
5	9-storeyed precast-and-monolithic frame. Resid.building. Tashkent	12×42 H=27	lateral / transl.	1,54 / 1,39	1,20 / 1,18	159,0 / 158,9	0,25 / 0,25	1,28 / 1,18	10,9 / 11,3	0,064 / 0,056	9	B-2	0,17 / 0,13	-	-	-	0,785 / 0,8	0,85 / 0,76	-
6	12-storeyed precast and monolithic frame. Residential building. Erevan	12×42 H=37,2	lateral / transl.	1,31 / 0,89	1,02 / 0,51	139,4 / -	0,504 / -	1,29 / 1,75	8,0 / 2,0	0,097 / 0,002	7	B-2	0,13 / -	0,14 / 0,16	0,18	-	1,0 / 0,313	- / 1,25	1,25
7	11-stor. monolithic stone frame. Residential building. Erevan	16,9×34 H=34,5	lateral / transl.	2,05 / 1,92	1,77 / 1,19	962,8 / -	0,55 / -	1,16 / 1,11	8,0 / 8,9	0,101 / 0,051	7	B-2	0,098 / 0,109	0,091 / 0,10	-	-	0,575 / 1,29	- / 0,53	-
8	13-stor. precast-and-monolithic frame. Residential building. Erevan	14,4×30 H=40	late-ral	1,04	0,89	135,2	0,8	1,17	11,3	0,036	7	B-2	-	-	-	-	-	-	-
9	5-storeyed monolithic frame. Hospital. Djermuk	13×42,5 H=17,7	lateral / transl.	1,30 / 1,97	0,90 / 0,97	86,3 / 131,2	0,68 / 1,04	1,44 / 1,41	24,4 / 26,4	0,070 / 0,100	8	B-2	-	-	-	-	-	-	-

$A_1, A_2 \ldots$ — amplitudes of the top levels for the corresponding modes.