TESTS AND CALCULATION OF THE SEISMIC BEHAVIOUR OF CONCRETE STRUCTURES

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1. Introduction

From the safety point of view, it is important to know the actual behaviour and the safety level of the various nuclear power plant buildings, when they are submitted to earthquake loads.

This paper deals with the frame type buildings, which are generally the most sensible to earthquakes. Its objectives are to describe the main phenomena governing the behaviour of such structures, when the earthquake level increases up to the structure collapse, to point out what type of calculation model shall be used to obtain good results and to give an estimation of the safety factors corresponding to the usual design practice.

Extended experimental research on the behaviour of reinforced concrete beams and frames submitted to monotonic or cyclic loading has been done /1/,/2/,/3/.

These tests are very useful to build constitutive laws models /4/, but, as they do not reproduce the earthquake loads, they do not simulate directly what happens to the structure during an earthquake.

For that reason, since 1966, several authors have performed dynamics tests using vibration generators /5/ or shaking-tables /6/,/7/,/8/. As an example of that type of test and of the corresponding results, we describe here with more details the tests made at the Saclay Center, on a shaking-table called VESUVE, on simple beams and frames.

2. Saclay Test Program

2.1 Description of the Structures

For the first serie of tests a reinforced concrete column clamped at its basis and with a mass of two tons fixed on top was designed to study the behaviour with simple flexion without shear. The model for the test is a reduction of a typical column of an actual nuclear power plant building by a factor 3. The height is 1.44 m and the section is a square 17x17 cm. There are 12 reinforcing steel 8 mm diameter and the stirrups are spaced each 5 cm.

For the second serie of tests it was decided to verify if results for a beam could be extended to a frame structure. For that purpose a frame, with two columns 1.55 m high and an horizontal beam 1.70 m long loaded with a total mass of 15 Tons was designed.

Those frames models are also an approximative reduction of a typical frame from an actual building by a factor 3. The design of each model was made according to the requirements of the 1968 uniform building code available in France (CC BA 68 Rules) /9/.

The mechanical characteristics of the reinforcement and the concrete used in the computation of the response of the test structure, were determined from their measured stress-strain relationships. The Young's modulus are 200 000 MPa for steel and 32 000 MPa for concrete. The ultimate strengh are 560 MPa for steel and 42,5 MPa for concrete.

2.2 Description of the Test Program

First some static tests were performed on four colums and two frames loaded with their masses. The force was applied by an horizontal hydraulic actuator at 1 m above the basis for the columns and in front of the horizontal beam for the frames. Two kinds of tests were conducted : monotonic loadings giving the moment curvature relationship and alternate loading giving a cyclic behaviour law.

For the dynamic tests on the shaking table VESUVE, two reference times histories with different characteristics were selected : the TAFT 1952 NS component and the SAN FRANCISCO 1957 NS component. For the frame test a third time history was choosen : the AEDF 73 which is an artificial time-history giving the same response spectrum as TAFT with a twice longer duration. SAN FRANCISCO duration is shorter than TAFT duration. To account for the model scale, the accelerograms used in the tests were contracted in time by a factor 3.

The eight series of tests performed with column are described on table I. The five series of tests performed with frames are described on table II. Acceleration at the top and the basis, relative displacement and strain on the reinforcement were recorded during each test for several points of the column (figure 1) and of the frame (figure 2).

2.3 Tests Results

The main result of the cyclic test is that, even at a low level, the concrete tensile cracks result in a significant stiffness decrease. After the first cycle, below the load level corresponding to steel yielding, the behaviour at the same level remains quasi-elastic with the reduced stiffness /10/,/11/.

The dynamic test confirmed the static tests results : the fundamental frequency of the structure decreases as the level of the load increases. This is clearly shown by the top displacement time histories plotted at low and high level for each structures (figures 3 and 4). For each earthquake, the natural frequency and the damping factor of the structure were determined from the movement after the end of the earthquake when the structure behaves like a free damped oscillator /12/,/13/. Figures 5 and 6 show the decrease of the frequency and the increase of the damping as a function of the maximum relative deflexion of the columns during the earthquake, the relative deflexion being the ratio between the top displacement and the column height. The points on the curves giving the initial natural frequency and damping factor were determined before the first test by excitation at a very low level with an electromagnetic vibration generator. Natural frequency thus obtained agrees very well with the calculated one in the hypothesis of a pure elastic behaviour.

The set of tests did not show a significant difference of response between a structure directly submitted to a high level earthquake and a structure submitted to the same earthquake after a serie of earthquakes with increasing levels. The maximum relative deflexions of structures as a function of the earthquakes level characterized by the maximum velocity of the table during the earthquake are plotted on figures 7 and 8.

3. Comparison of the Test Results with Standard Design Method.

The usual calculation method can be divided into the following steps :

- Calculation of the natural frequencies with elastic behaviour of concrete.

- Estimation of the maximum loads on each beam from the response spectrum with a conventional damping factor (around 7 %) /14/.

- Non linear static analysis of the beam behaviour submitted to the calculated loads, taking into account the cracking effects, to obtain the maximum tensile stresses in the reinforcements and the maximum compressive stresses in the concrete.

- Comparison of those stresses to reference limits prescribed in national codes (CCBA 68 in France). The limits differ for normal and extreme conditions.

Such an analysis gibes the maximum level of earthquake the structure can withstand in normal and extreme conditions. The corresponding points are plotted on figures 7 and 8 (elastic calculations). It is interesting to notice the important value of the safety factors (structure collapse earthquake level divided by extreme conditions limiting level) : 2.5 and 4 for the beam, 6.5 and 9 for the frame. The variation between the beam and the frame can be explained by the initial natural frequencies differences (5 Hz versus 3 Hz) and the gravity load stresses différences (0,7 MPa versus 4 MPa). The displacements are underestimated especially for the beam. In that case, the usual method is overconversative for the loads and unconversative for the displacements.

4. Non Linear Behaviour Model

An acceptable non linear behaviour model incorporating all the important effects observed during the tests can be summarized as follows :

The moment-curvature relationship for a beam section under given permanent normal stresses can be satisfactorily approximated by a trilinear function : the first linear part corresponds to the linear elestic range for all materials, the second part to progressive concrete cracking in tension, the third part to steel yielding. When the concrete compressive ultimate strength is reached, there is a fourth zone where the moment decreases when the curvature increases. That moment-curvature relationship can be calculated from the concrete and steel constitive laws and the beam and reinforcement geometries : the static monotonic tests have proved that the calculated curve fits well the experimental curve when the correct material properties are used (including aging effects for concrete).

An acceptable cycling behaviour law, deduced from the cycling tests, is described on figure 9. The main difference with a plastic model is the stiffness reduction for part 2 of the moment-curvature curve which is necessary to account for the fundamental frequency evolution measured in the tests.

That model has been introduced in the general beams and frames TEDEL computer code /16/, which takes also into account the non linear geometrical effects, and the dynamic responses of the tested structures have been computed for different levels of the TAFT and SAN FRANCISCO accelerograms. Some calculated maximum displacements are plotted on Fig.7 and 8. The comparison of experimental and calculated results is good and proves the ability of the type of model described to simulate correctly the behaviour of concrete beams and frame, up to collapse. That gives the possibility to verify the safety of buildings designed by usual methods, to make parametric analysis of the safety factors inherent to the current design methods as a function of the buildings type, size and fundamental frequency, and to check the calidity of simpler methods used to account for the non linearity effect as for example the Newmark method.

The non linear geometrical effects as the inverted-pendulum effect, do not play an important role in the tests presented. But for tall structures, it is necessary to take them into account, as they may reduce significantly the collapse level of the structure.

For some structures, it is probably necessary to include the normal force variation influence on the moment-curvature law.

5. Conclusion

The main effects of an earthquake on frame-type structures have been described, as they were observed during our tests at Saclay, or during similar tests /6/,/7/,/8/. It has been shown also that the behaviour of such structures can be correctly approximated up to the ruin by behaviour models suitably defined. Specially in the intermediate range, where there are cracks in concrete, but where the steel remains elastic, a correct model shall differ from an ordinary elasto-plastic model.

The tests have also shown that structures calculated with the conventional design methods have resistance safety factors which may vary between 2.5 and 9. Such factors are good from a safety point of view ; however, their dispersion shows that different structures designed by such methods may be unequally safe. To modify that situation, it is necessary to use methods taking into account the stiffness reduction due to tensile concrete cracking, and the ductile effects.

This paper mainly deals with the monodirectional behaviour of beams. Some recent studies /17/,/18/ consider the bidirectional flexural behaviour. It still remains a lot of works in that field to obtain methods applicable to industrial structures.

References

/1/ WIGHT, J.K., SOZEN, M.A., "Shear strengh decay in reinforced concrete columns subjected to large deflexion reversals", University of Illinois, UILU Eng. 73-2017 - URBANA 1977.

/2/ SPASIC, B., "Comportamiento de los elementos estructural de concreto armado sometidos a fuerza axial y momentos alternados", Bolletin del Instituto de Materiales y Modelos Estructurales (IMME), Ano 12 n°49, MADRID 1975.

/3/ OKADA, T., SEKI, M., "A simulation of Earthquake response of reinforced concrete buildings", Preprints, 6th world conference on earthquake engineering, New Dehli, January 10-14, 1977, Paper 9-5.

/4/ BERTERO, V.V., POPOV, E.P., MA, S.M., "Model of cyclic inelastic flexural behaviour of reinforced concrete members", Transactions 4th Intl. Conf. on Structural Mechanics in Reactor Technology, San Francisco, U.S.A., August 1977, Paper K3/14.

/5/ CHEN, C.K., CZARNECKI, R.M., SCHOLL, R.E., "Vibration tests of a 4 story reinforced concrete test structure", JAB 99-119, URS/John A. Blume & Associates Engineers, San Francisco 1976.

/6/ HIDALGO, P., CLOUGH, R.W., "Earthquake simulator study of a reinforced concrete frame" University of California, EERC 74-13, BERKELEY 1974.

/7/ DARIO, A.J., SOZEN, M.A., "Behaviour of 10 story reinforced concrete walls subjected to earthquake motion", University of Illinois, UILU Eng. 76-2017, URBANA, 1976.

/8/ MIHAI, C., DIACONA, D. et al., "On the static and seismic behaviour of a structural model of an industrial storied hall", Preprints 6th world conference on earthquake engineering, New Dehli, January 10-14, 1977, paper 9-7.

/9/ "Règles techniques de conception et de calcul des ouvrages et constructions en béton armé", CC BA 68 - PARIS 1968.

/10/ SOUBRET, R., "Essais statiques et dynamiques sur poteaux", CEBTP St-Rémy-les-Chevreuse 1976.

/11/ SOUBRET, R., "Essais statiques et dynamiques sur portiques", CEBTP, St-Rémy-les-Chevreuse, 1978.

/12/ GAUVAIN, J., JEANDIDIER, C., QUEVAL, J.C., "Essais sismiques de poteaux en béton armé" (Rapport EMT/78/177) CEA-DEDR - Saclay 1978.

/13/ GAUVAIN, J., JEANDIDIER, C., QUEVAL, J.C., "Etude sismique de portiques : essais". (N.T. EMT/SMTS/VIBR 79/13) CEA-DEDR - Saclay 1979.

/14/ JEANPIERRE F., LIVOLANT, M., "Analyse de la tenue des structures de réacteurs nucléaires aux séismes - Méthodes de calcul - Méthodes expérimentales" - Colloque sur les méthodes d'étude et simulation des chocs - LYON 1974 - CEA/DEDR.

/15/ HOFFMANN, A., ROCHE R., LIVOLANT, M., GAUVAIN, J., "Système CEASEMT - Poutres et coques - Quelques considérations simples sur les modèles globaux de plasticité" (Rapport EMT/77/22) CEA/DEDR - Saclay 1977.

/16/ HOFFMANN, A., JEANPIERRE, F., AXISA, F., "Programme TEDEL - Tuyauteries - charpentes 3D - Elastique - Plastique - Dynamique - Flambage - Fluage - Thermoplasticité - grands déplacements - Fluides"- ASME (Rapport EMT/77/64) CEA/DEDR - Saclay 1977.

/17/ TAKIZAWA, H., AOYAMA, H., "Biaxial effects in modelling earthquake response of R/C structures" - Int. Jour. Earthquake Eng. Struct. Dyn. Vol 4 - p 523-552 (1976) TOKYO.

/18/ OKADA, T., SEKI, M., ASAI, S. - "Response of concrete columns to bidirectional horizontal force and constant axial force", Bull. of Earthquake resistant structure Research Center, N°10, Dec. 1976, TOKYO.

TABLE I

Earthquake	Beam Ref.	Number of earthquake	Max. velocity of the table	Observations
TAFT	JT 9	8	0.075 m/s ↗ 0.50 m/s	Increasing level
	JT 10	4	0.50 m/s	High level
	JT 14	40	0.13 m/s	Fatigue effect
	JT 15	8	0.075 m/s ↗ 0.50 m/s	Increasing level
	JT 6	10	0.075 m/s ↗ 0.50 m/s	Biaxial effect
SAN FRANCISCO	JT 11	13	0.06 m/s ↗ 0.55 m/s	Increasing level
	JT 12	8	0.55 m/s	High level
	JT 13	13	0.06 m/s ↗ 0.55 m/s	Increasing level

TABLE II

Earthquake	Frame Ref.	Number of earthquake	Max. velocity of the table	Observations
TAFT	JT 19	8	0.049 m/s ↗ 0.37 m/s	Increasing level
	JT 20	6	0.37 m/s	high level
SAN FRANCISCO	JT 21	23	0.080 m/s ↗ 0.59 m/s	Increasing level
	JT 22	9	0.59 m/s	high level
AEDF	JT 23	7	0.063 m/s ↗ 0.36 m/s	Increasing level

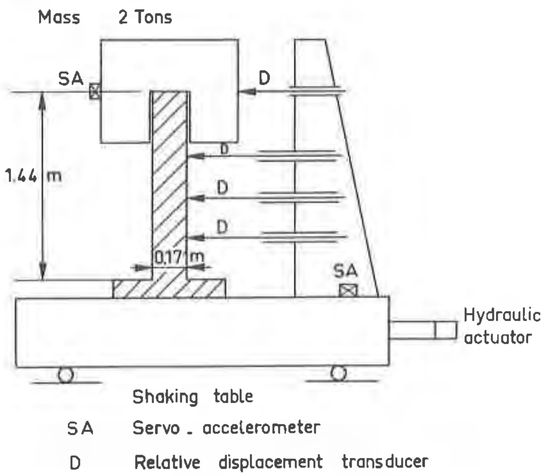

Fig. 1 Dynamic beam test installation diagram.

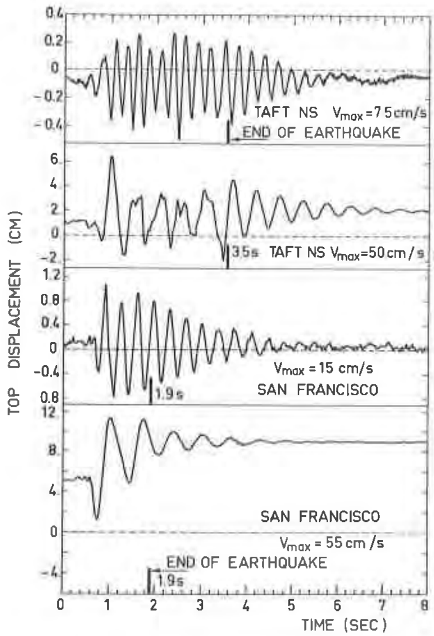

Fig. 3 Time-history of beam top displacement at low and high levels with TAFT NS and SAN FRANCISCO.

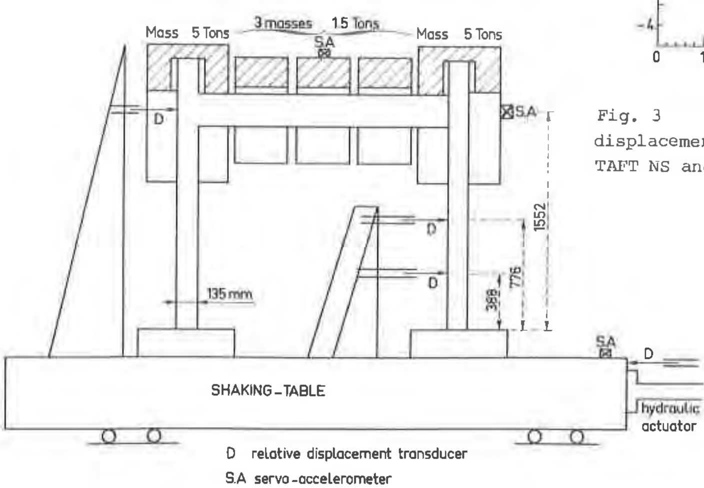

Fig. 2 Dynamic frame test installation diagram.

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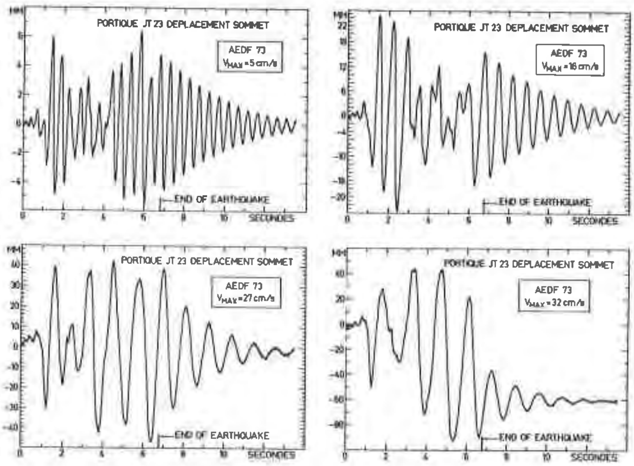

Fig. 4 Time-history of frame top displacement for increasing levels with AEDF 73.

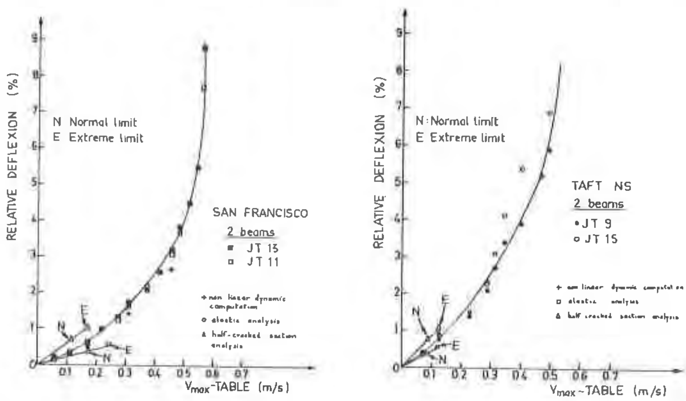

Fig. 5 Frequency and damping of the beam as a function of the maximum relative deflexion.

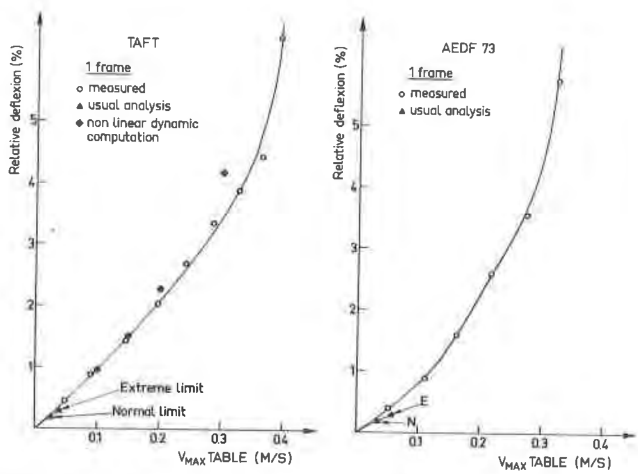

Fig. 6 Frequency and damping of the frame as a function of the maximum relative deflexion.

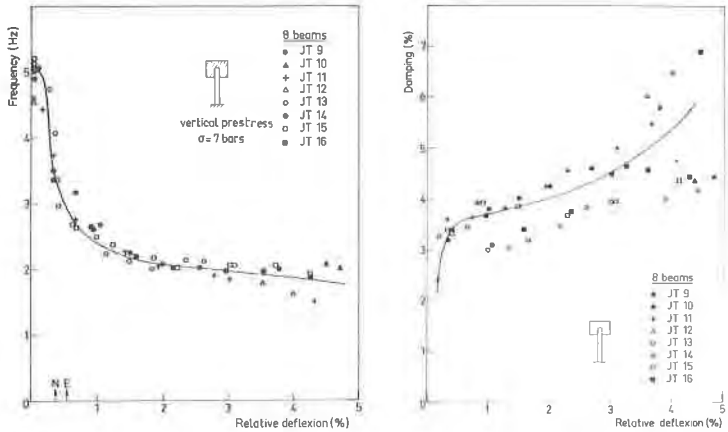

Fig. 7 Maximal relative deflexion of the beam as a function of the maximal earthquake velocity.

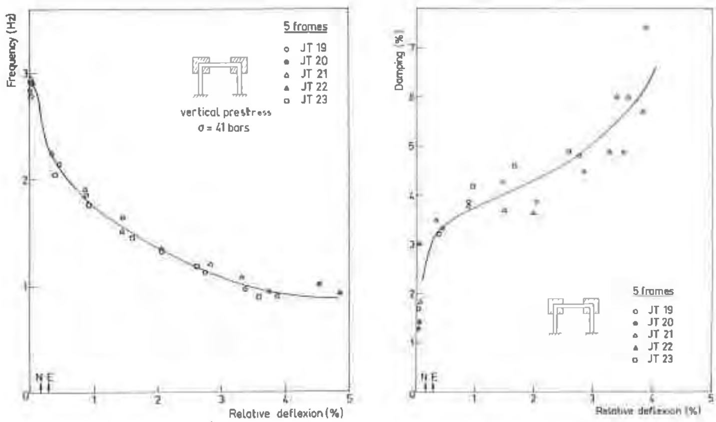

Fig. 8 Maximal relative deflexion of the frame as a function of the maximal earthquake velocity.

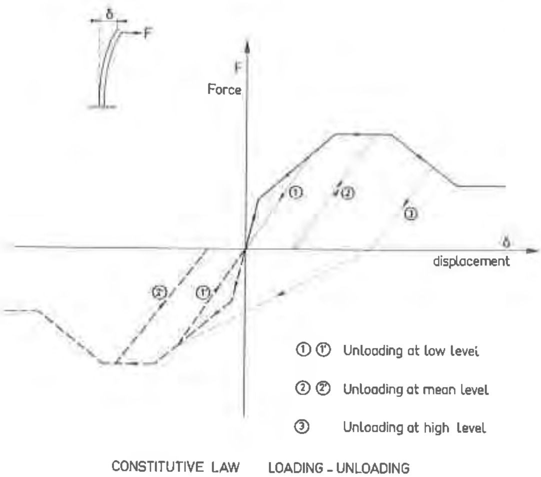

Fig. 9 Force-displacement diagram.