STIFFNESS DEGRADATION AND ENERGY DISSIPATION IN REINFORCED CONCRETE LAMELLAR STRUCTURES UNDER EARTHQUAKE EXCITATIONS

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S U M M A R Y

Two structural scale models were constructed and tested up to failure by means of a seismic shaking table. This paper present a brief description of the models, test procedure and a selected serie of results with special reference to the stiffness degradation and energy dissipation. The seismic excitations were supplemented with free vibration and static tests for determining the dynamic characteristics and stiffness properties of the structure at selected behaviour stages.

INTRODUCTION

The investigation described in this paper is part of a research program regarding the performance of lamellar framed structures under seismic excitation. The main feature of this type of frame building is the lamellar shape (L, T or +) of column cross sections.

Two structural models were constructed and test by means of an earthquake simulator platform. The first model, A, representative of a five story flat-slab building, was realized at a scale of 1:5. The prototype structure on which the test model was designed consisted of lamellar columns at 5.4o m center on a square grid with interstory heights of 2.85 m. The slab thickness was 25 cm whereas the monolithical beams connecting the precast slab panels were 6o cm width

The second model, designated Model B, represented approximately a 1:6 scale structure of a ten story prototype la-

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mellar building with symmetrical plane dimensions. All beams and floor units were prefabricated in laboratory conditions and transported with care to the model construction place, while columns were casted on the shaking table in special steel formworks. The execution of the models reproduced all the prototype details with some minor modifications.The both models were subjected in a similar manner to a serie of scaled sequences of earthquake motions with successively increasing intensities.

TEST MODELS AND INSTRUMENTATION

The dimensions and main structural features for the test models are shown in Figs.1 and 2. Model A consisted of monolithical columns and precast floor slabs connected through integral beams on both horizontal directions 1 . In the case of Model B each floor was composed of eight precast panels supported on precast beams, the thickness of the floor plate being 2.5 cm. The small scale reinforcement used for constructing the models ranged in yield stress from 21o to 26o N/mm^2. The average compressive strength of the small agregate concrete used in constructing the two models was about 115 % of the prescribed value at the time of testing. To simulate the influence of the live loads and non-structural members, a mass of 725 kg was added to each current story of Model A with the exception of top story where the attached mass was 65o kg. In the case of Model B, the additional masses at current floors and top floor were 44o kg and 41o kg respectively. Each test structure was attached rigidly to the shaking table and was instrumented properly, as was the shaking table itself. A view of Model A on the shaking table showing the additional weights at the floors may be seen in Fig.3, while Fig.4 shows details from the construction of Model B.

The structural response quantities measured included horizontal floor accelerations and displacements, as well as several dynamic strains, primarily in the bottom two floors (Figs.5 and 6).

The base motions used in the dynamic tests were artificial or actual earthquakes selected so as to have their energy concentrated in narrow frequency bands. The principal input motions for Model A were the simulated earthquakes A2 and D2 whereas for Model B the artificial earthquake B2 [4] was also used. Other imputs which were used for special studies included either the same earthquakes but with their time scale changed to shift the response spectrum peak toward the fundamental period of the model, or a particular imput (ROM) incorporating some characteritics of local eartnquakes. All these imputs were applied in a number of successive runs at several different intensities ranging from very slight motions causing only elastic displacements to strong earthquakes which caused inelastic structural deformations and finally the failure of test structures. Both free vibration and static tests were conducted at chosen interval of time to determine the natural frequencies, damping factors and stiffness degradation along seismic test history. The more important findings of these tests are incorporated in this paper.

TEST RESULTS

Shown in Tables 1 and 2 are details of the test program and maximum responses obtained. First mode periods and damping factors of the two models are given in Table 3 and 4 respectively.

In the case of Model A a horizontal static load, P, was applied in increments at the top floor (in one direction only) to evaluate some of the flexibility coefficients of the structure in its successive behaviour stages.

For Model B a similar force was applied at the eighth floor. The horizontal displacements, D, of all floors were measured using dial ganges (Figs. 5 and 6) attached to a reference frame. A row of the lateral flexibility matrix was thus obtained for each model and compared with the results of analytical predictions. Selected deflection shapes are shown in Figs.7 and 8. The force-displacement relationship obtained at each story was plotted as a hysteresis loop. Figs.9 and 1o show typical P-D diagrams.

CONCLUDING REMARKS

The two structural systems with lamellar columns described herein provide relatively large flexibility of plan layout and overall economy as a result of a high degree of prefabrication and feasibility of construction. As many of these buildings have been planned for areas often subjected to earthquakes, an experimental study simulating seismic conditions, was required to improve the analytical procedures for predicting the performance of structures and to facilitate the understanding of their behaviour as evidenced by tests.

As regard the stiffness degradation and energy dissipation, it is useful to point out a number of observations and findings, considering separately the two test models.

Model A. After a number of 1o2 sequences of seismic excitations on a time length of about 55o sec., the stiffness of the structure with respect to lateral loading was only approximately 15 % of the initial stiffness. While the maximum top level force used in static tests represented a small action relatively to lateral forces developed within the structure during dynamic action, the lateral load-deflection curves showed the effect of extensive damage occured in component members. The peak base acceleration during the last excitation runs exceeded 1 g and the observed maximum top displacement was 1/15o of the structure height. The test showed that a large amount of ductility was available and evidenced, as it would be expected, a relative flexible structure. However the stiffening effect of nonstructural elements was not present in the model.

The tests indicated that the model structure was not provided with sufficient reserve of strength capacity to ensure that the principal energy-dissipating mechanism through integral beams was maintained at the large deformations occured in the inelastic range.

For improving the seismic behaviour of these types of structures some recommendations regarding the proportioning of columns and integral beams were made and the need of sufficient shear reinforcement in the flexural critical regions was emphasized. The use of the system in conjunction with shear walls is preferred in seismic zones.

Model B. The reinforced concrete structure withstood the intense base motions without collapse, although a large amount of damage was observed in the component members. After the completion of the seismic program (exceeding 15o sequences) the stiffness determined through static tests was about 3o % of the initial stiffness.

As the intensity of the base motion was increased in succesive tests, the observed periods increased because of further cracking occured in beams, floors and finally in column sections.

The maximum story drift in the damage stage prior to failure was 1/2oo of the story height.

The change of first mode period with change of stiffness was not constant throughout the tests. Although the inelastic behaviour was more satisfactory than that of Model B, the need of adequate measures to prevent large nonstructural damage to strong seismic action was pointed out.

To sustain the principal energy-dissipating system some recommendations have been made for improving the strength of lamellar column, especially the corner columns and their reinforcement detailing in the joints.

ACKNOWLEDGMENTS

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Table 1 - Test Program and Extreme Responses - Model A

Test No.	Number of runs	Maximum base acceleration	Input motion	Programmed duration (s)	Structure condition	Maximum story acceleration* (m/s²)	Maximum story displacement* (mm)
1	9	0.64	D_2	2	Quasielastic**	$A_5 = 2.43$	$D_5 = 2.3$
2	13	1.86				$A_2 = 2.38$	$D_2 = 1.4$
3	9	3.02				$A_1 = 2.17$	$D_1 = 0.5$
4	9	4.65			Development of cracking	$A_5 = 7.14$	$D_5 = 11.4$
5	9	7.05				$A_2 = 4.85$	$D_2 = 5.9$
6	4	4.51	A_2	18		$A_1 = 8.02$	$D_1 = 2.7$
7	4	7.95			Damaged prior to failure	$A_5 = 15.12$	$D_5 = 44.2$
8	3	9.54					
9	3	10.60	ROM	6			$D_4 = 32.4$
10	8	10.60				$A_3 = 12.6$	$D_3 = 24.8$
11	13	13.78				$A_2 = 9.18$	$D_2 = 13$
12	18	12.20				$A_1 = 13.68$	$D_1 = 7.8$

* Nonsimultaneous

** Cracking appeard in the integral beam joint during last runs of test No. 2

Table 2 - Test Program and Extreme Responses - Model B.

Test No.	Number of runs	Maximum base acceleration (m/s²)	Input motion	Programmed duration (s)	Structure condition	Maximum story acceleration* (m/s²)	Maximum story displacement* (mm)
1	15	1.98	D_2	4	Quasielastic	$A_{10} = 2.25$	$D_{10} = 4.67$
2	18	1.39				$A_7 = 1.74$	$D_7 = 3.14$
3	4	1.65	B_2	18		$A_4 = 1.74$	$D_4 = 1.27$
4	16	2.51				$A_1 = 1.56$	$D_1 = 0.44$
5	18	6.80		9	Initiation of cracking	$A_{10} = 6.47$	$D_{10} = 10.11$
6	11	4.62	A_2	17	Development of cracking	$A_{10} = 10.29$	$D_{10} = 21.40$
7	11	7.92				$A_7 = 8.06$	$D_7 = 16.26$
8	9	8.31	ROM	10		$A_1 = 5.98$	$D_1 = 0.86$
9	8	7.26		6	Damaged prior to failure	$A_{10} = 10.78$	$D_{10} = 17.20$
10	14	9.24	El centro	14			
11	12	5.74	ROM	6		$A_7 = 7.80$	$D_7 = 15.11$
12	5	3.96	A_2	17		$A_4 = 6.38$	$D_4 = 12.73$
13	7	5.28		10		$A_1 = 3.68$	$D_2 = 5.02$
14	7	6.47	ROM	6			$D_1 = 2.74$

* Nonsimultaneous.

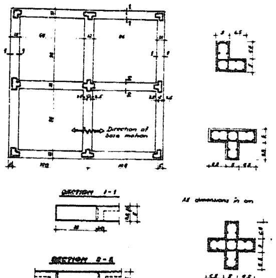


Fig. 1
Test Structure
Description —
Model A

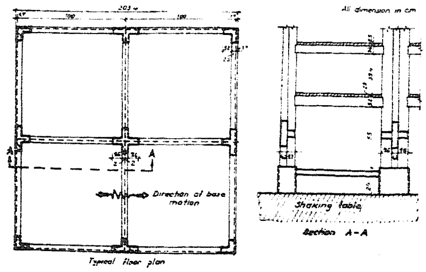


Fig. 2 - Test Structure Description-Model B

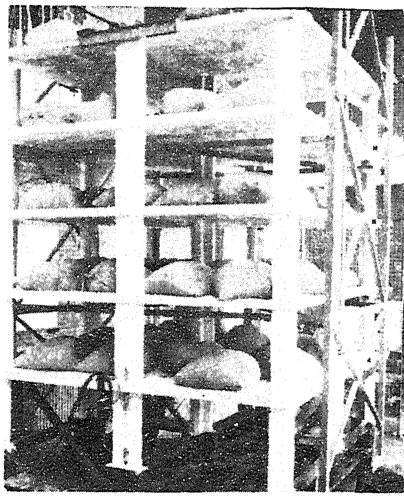


Fig. 3-Test Model B on Shaking Table

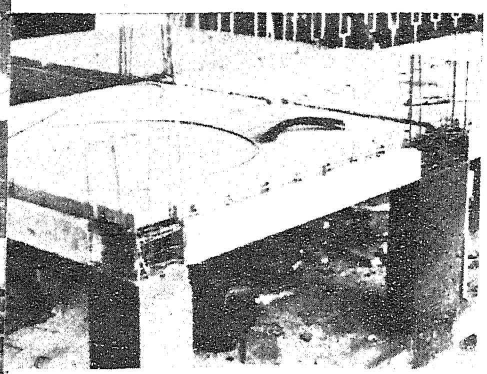


Fig. 4-Details from the Construction of Model B at Floor 7.

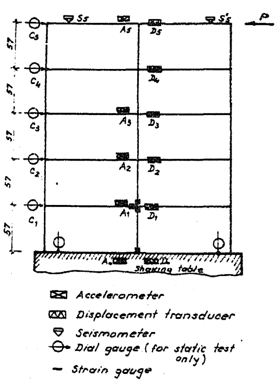


Fig 5
Instrumentation-
Model A

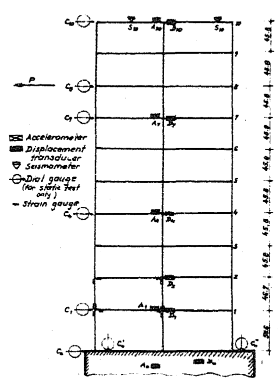
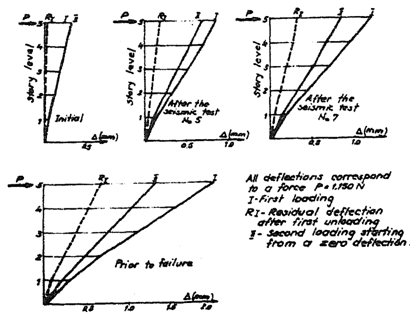


Fig. 6
Instrumentation -
Model B

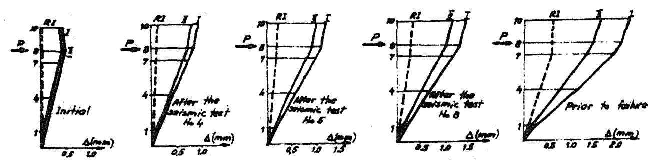


All deflections correspond to a force P = 1,150 N
I - First loading
RI - Residual deflection after first unloading
II - Second loading starting from a zero deflection.

Fig. 7 Successive Deflections of Model A under a Static Force at Top Floor

Table 3 First Mode Period and Damping - Model A

Free vibrat. test No.	Structure condition	Period T (s)	Damping ν (%)
1	Before the seismic excitation tests	0.190	1.05
2	After the seismic test No. 5	0.236	1.35
3	After the seismic test No. 7	0.252	1.45
4	After the seismic test No. 12	0.290	1.75



All deflections correspond to a force P = 2,000 N
I - First loading
RI - Residual deflection after first unloading
II - Second loading starting from a zero deflection

Fig. 8 - Succesive Deflections of Model B under a Static Force at Floor 8

Table 4 First Mode Period and Damping - Model B.

Free vibrat. test No	Structure condition	Period T (s)	Damping ν (%)
1	Before the seismic excitation tests	0.265	0.90
2	After the seismic test No 4	0.285	1.03
3	After the seismic test No 5	0.320	1.66
4	After the seismic test No 8	0.330	1.75
5	After the seismic test No 14	0.390	2.10

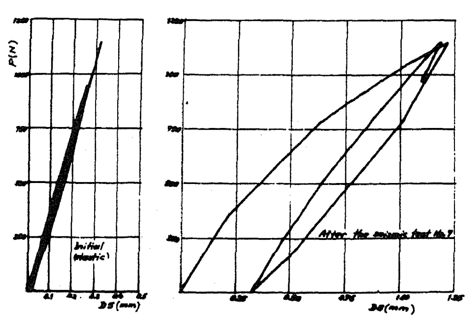


Fig 9 P-D_5 Diagram - Model A

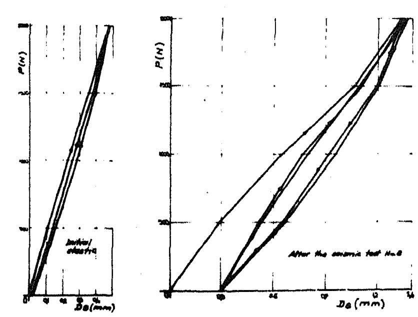


Fig. 10 P-D_8 Diagram - Model B