VIBRATION TESTS OF A 4-STORY CONCRETE STRUCTURE

by

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SYNOPSIS

A series of forced vibration tests on a full-scale 4-story reinforced concrete test structure was performed to investigate its dynamic response before, after, and during the time it underwent structural damage. Nondestructive tests were conducted first, exciting four translational modes at force levels within the elastic limit, during which the structure suffered no structural damage. Next, a destructive test excited only the lowest translational mode at high-amplitude destructive levels, during which the structure exhibited inelastic response and suffered major structural damage. Post-destructive tests used force levels similar to the nondestructive tests.

INTRODUCTION

URS/John A. Blume & Associates, Engineers (URS/Blume), conducts structural response investigations for the U.S. Energy Research and Development Administration (ERDA) in connection with its safety program for underground nuclear explosive testing. The development of methods for predicting building response to and damage from underground nuclear explosions (UNEs)--summarized in a URS/Blume publication[1]--has been a major goal of this effort.

As part of this program, two identical full-scale 4-story reinforced concrete structures (Figure 1) were built in 1965-1966 at the Nevada Test Site to investigate their dynamic response behavior. The design of these structures conformed to the American Concrete Institute Building Code Requirements of 1963, and the design for lateral loads was approximately two times the requirements of the 1961 *Uniform Building Code* for Seismic Zone 3. Provisions for ductility and reserve energy absorption capacity[2] were also incorporated. Since their construction, the structures have been subjected to a continuing program of vibration testing, and sufficient data had been collected[3,4] on the elastic response of these structures. In 1974 it was decided to conduct a high-amplitude vibration test that would cause the south structure (free of partitions) to deform beyond its elastic limit and cause major structural damage. This paper summarizes results of the 1974 testing program.[5]

TEST EQUIPMENT, PROCEDURES, AND DATA ANALYSIS

A reciprocating mass vibration generator specifically for use in these high-amplitude tests was designed and assembled by Sandia Laboratories of Albuquerque, New Mexico. This device had a frequency range up to 50 Hz and was capable of producing a maximum force of 5,500 kg at frequencies greater than 1.6 Hz.

A network of seismometers was installed on the structure to measure the horizontal and vertical motion of the floors. Velocity and accelera-

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tion time histories of motion were recorded both on paper strip charts and analog tapes, which have been digitized and transferred to magnetic tapes for further analysis.

This test effort included nondestructive testing, destructive testing, and post-destructive testing. Nondestructive testing was conducted first; the vibration generator was located on the roof, and the building was shaken in both transverse and longitudinal directions at motion amplitudes higher than those obtained from previous vibration testing (between 1966 and 1973) but less than those that would cause structural damage. (Some additional nondestructive tests were also conducted in the longitudinal direction, with the vibration generator located at the third floor.) The destructive test was performed only in the longitudinal direction, with the vibration generator placed at the third floor of the structure. This arrangement for the destructive test was based on a parameter study that identified the optimum method for shaking, considering the structure characteristics and limitations of the vibration generator. The building was excited in the fundamental mode at a high-amplitude destructive level, during which the structure exhibited inelastic response and suffered major structural damage. Post-destructive testing was similar to the destructive test, but lower forces were used and higher modes were excited during this test series.

Data presented in this paper are based on frequency dwell tests, which consisted of dwelling at each mode frequency and linearly increasing the force with time until a predetermined maximum was reached. The procedure used to obtain the dynamic response properties of the test structure from the digital record is based on a curve-fitting method in the time domain, as developed by Raggett.[7] Output from this computer analysis includes the period, damping ratio, and root-mean-square (RMS; i.e., 0.707 times the peak amplitude of sinusoidal motion) amplitude of the data segment analyzed, resulting in the minimum squared error.

TEST RESULTS

Structural Damage. The test structure was visually inspected before and after each test series. No visible damage was noted after the nondestructive tests. The structure sustained substantial damage during the destructive test. The most severe damage was found at beam-column joints at the second and third floors, where the exterior concrete had spalled, exposing reinforcing steel within the joints. Nonsymmetrical damage to the structure was also discovered. Although the damage was extensive and similar to that expected during a major destructive earthquake, the structure was never in danger of collapse during this vibration test.

Results from Nondestructive and Post-Destructive Tests. Response properties before and after the high-amplitude, destructive-level vibration test (i.e., for the undamaged and damaged structure in the longitudinal direction) are presented in Figures 2 and 3. In Figure 2, the period of vibration is plotted against the corresponding RMS velocity of the roof. The increase of the period of the undamaged structure with increasing amplitude of response, as shown in Figure 2, indicates the structure has stiffness properties of a nonlinear softening spring system. The change in fundamental period of the undamaged structure was more pronounced at lower amplitudes, and eventually the period approached a nearly constant value. Figure 2 also shows that the period of the damaged structure increased as the force levels were increased. A comparison of nondestructive and post-destructive test data in Figure 2 demonstrates that the periods had lengthened substantially as a result of the destructive test, indicating reduced structure stiffness.

In Figure 3, the damping values of the undamaged structure, which consistently increased with an increase in the level of excitation at low amplitudes, tended to be nearly constant at higher amplitudes. Damping in the fundamental mode was found to vary between 1.3% and 1.9%. Damping in the second mode appeared to be higher than in the first, third, and fourth modes and ranged between 1.9% and 2.1%. Figure 3 also demonstrates that the damping ratios of the damaged structure are approximately 3.8%, 2.8%, and 1.6% for the first, second, and fourth modes, respectively. The tendency for damping to increase with amplitudes was not very pronounced except for the fourth mode. A comparison of damping values of the undamaged and damaged structure indicates that the damaged structure had greater damping values than the undamaged structure at all modes measured.

During the nondestructive test, the deflected shapes for several modes were measured and found to be fairly constant regardless of the amplitude of excitation. The same was true for these measurements from the post-destructive test. Average values of the normalized deflected shape for each mode measured are presented in Table 1. Table 1 shows that, during the nondestructive tests, the normalized deflected shapes of the undamaged structure with the vibration generator located at the roof were found to differ from those with the vibration generator located at the third floor, although this test series was conducted at the same order of amplitude. This finding suggests that the location of the vibration generator has a significant influence on the deflected shape. Also included in Table 1 are theoretical values, which were based on a mathematical model with fixed bottom-story columns and gross moment of inertia for the beams and columns.[3] The computed results agree well with recorded data with the vibration generator located at the roof. Table 1 also shows that the deflected shapes for the damaged structure differed substantially from those of the undamaged structure and indicated some discontinuity, especially in the fundamental mode of vibration.

Results from the Destructive Test. Figures 4 and 5 show RMS roof velocity amplitude versus period of vibration and damping, respectively. Each point in the figures is based on the analysis of about 10 sec of response. These data samples were taken at approximately one- to one-half-minute intervals and were numbered chronologically. Figure 6 shows plots of the period of vibration and damping versus the chronological numbered data points and also shows the approximate boundaries between the elastic and inelastic ranges and the damaged and undamaged structures.

Figures 4 and 6 suggest the following behavior (see dotted lines). At all amplitudes of response below the elastic limit, the structure had approximately the same stiffness and period of vibration (about 0.54 sec). As the input force was increased beyond the yield capacity of the structure, the structure experienced nonlinear hysteretic behavior, which was characterized by reduced stiffness and an increased period of vibration. As the input force was further increased, major damage to the structure occurred. The structure showed a significant change of period (from 0.7 sec to 0.9 sec) between Data Points 16 and 17. After Data Point 17, the periods of vibration again remained nearly constant (0.9 sec).

Figures 5 and 6 (see solid lines) show that the damping in the elastic range was found to remain constant at approximately 2.1%. Beyond the elastic limit, the structure experienced a marked change in damping to a nearly constant level of approximately 2.7%. As the input force was further increased and reached its maximum level and the structure suffered major cracking, a significant increase in damping occurred and damping was found to vary erratically between 3.2% and 4.1%.

The deflected shape during the destructive test was found to vary with respect to the amplitudes of vibration. However, it was found that the structure might experience three distinct shapes of vibration: Shape A in the elastic limit; Shape B in the inelastic limit, but with minor structural damage; and Shape C, with major structural damage. Average values of the deflected shape for each of these three groups are plotted in Figure 7, which clearly indicates that the shape of vibration changed as the structure experienced different stages of forced vibration. Note that an unexpected result of the fourth floor ordinate was observed during the destructive test (a similar result was also found during the non- and post-destructive tests shown in Table 1). This finding suggests that it is not possible to excite the entire structure in the first mode of vibration with the vibration generator located at the third floor.

CONCLUSION

Results of nondestructive and post-destructive tests on this concrete structure indicate that its response at small amplitudes is that of a slightly nonlinear softening system. The response properties of the damaged structure differ from those of the undamaged structure, with a longer period, higher damping, and some mode shape discontinuity. Results of the destructive test indicate that the structure experienced a period of linear response within the elastic limit (but above the design capacity of the structure), followed by a period of nonlinear response beyond the yield capacity. As the input force was further increased, major damage to the structure occurred, resulting in a permanent change in its damping and stiffness characteristics.

REFERENCES

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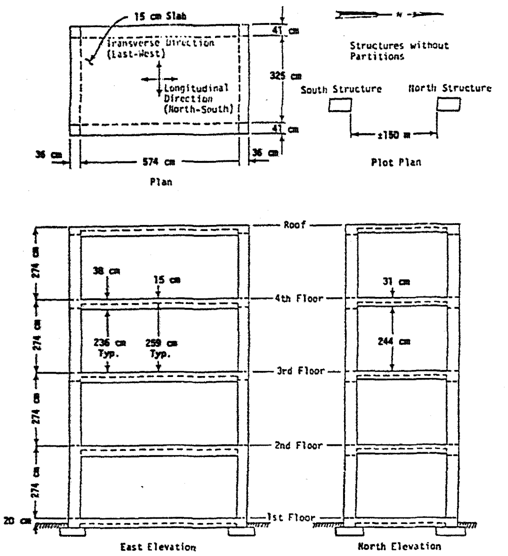

FIGURE 1 4-STORY CONCRETE TEST STRUCTURES

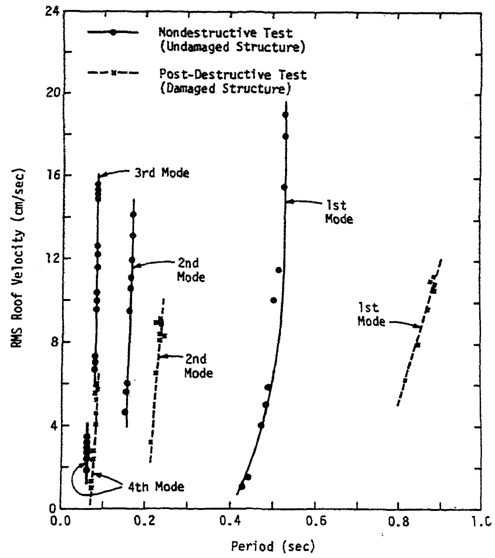

FIGURE 2 VELOCITY VERSUS PERIOD FOR NON- AND POST-DESTRUCTIVE TESTS

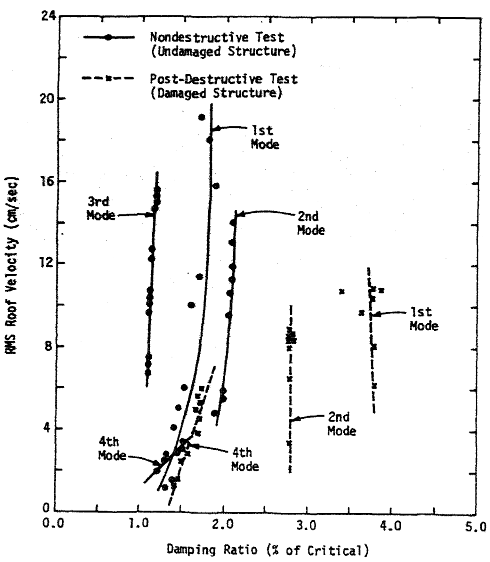

FIGURE 3 VELOCITY VERSUS DAMPING FOR NON- AND POST-DESTRUCTIVE TESTS

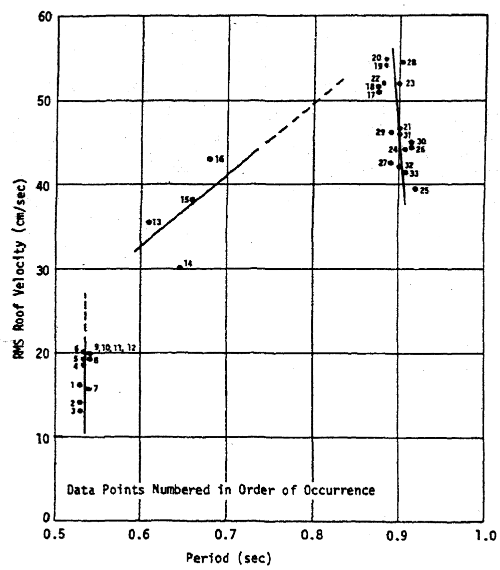

FIGURE 4 VELOCITY VERSUS PERIODS FOR DESTRUCTIVE TEST

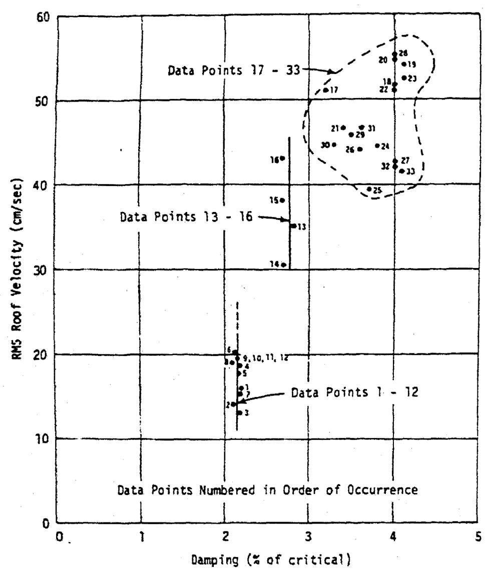

FIGURE 5 VELOCITY VERSUS DAMPING FOR DESTRUCTIVE TEST

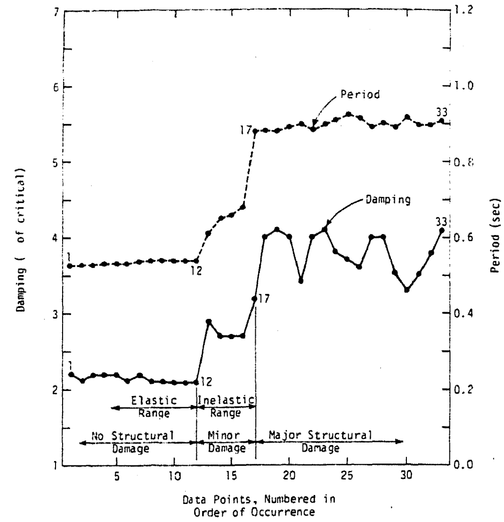

FIGURE 6 VARIATIONS IN DAMPING AND PERIOD IN ORDER OF OCCURRENCE FOR DESTRUCTIVE TEST

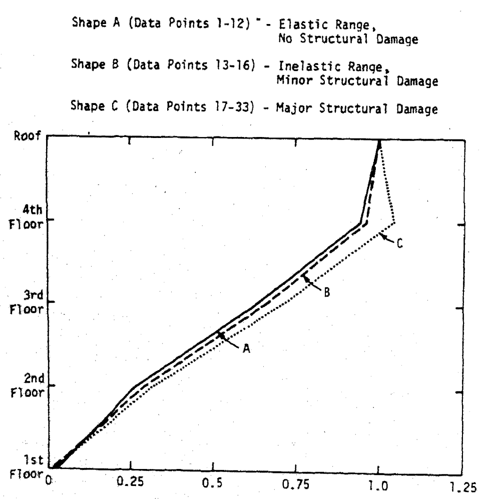

FIGURE 7 NORMALIZED DEFLECTED SHAPES FOR DESTRUCTIVE TEST

Test	V.G.[a] Located @	Floor	Normalized Deflected Shapes 1st Mode	2nd Mode	3rd Mode	4th Mode
Nondestructive (undamaged structure)	Roof	Roof	1.00	1.00	1.00	N.R.[b]
		4th	0.71	-0.28	-1.19	N.R.
		3rd	0.52	-1.08	-0.28	N.R.
		2nd	0.23	-0.80	1.88	N.R.
		1st	0.004	-0.011	0.026	N.R.
	3rd Floor	Roof	1.00	1.00	N.R.	1.00
		4th	0.95	-0.64	N.R.	-2.33
		3rd	0.63	-0.68	N.R.	3.11
		2nd	0.26	-0.49	N.R.	-4.20
		1st	0.005	-0.011	N.R.	-0.175
	(Theoretical values)	Roof	1.00	1.00	1.00	1.00
		4th	0.84	-0.16	-1.44	-2.46
		3rd	0.56	-1.04	-0.05	3.08
		2nd	0.28	-0.75	1.56	-2.75
		1st	0	0	0	0
Post-Destructive (damaged)	3rd Floor	Roof	1.00	1.00	N.R.	1.00
		4th	1.23	-0.21	N.R.	-2.44
		3rd	0.84	-1.45	N.R.	2.89
		2nd	0.35	-1.10	N.R.	-3.00
		1st	0.003	-0.008	N.R.	-0.106

[a]V.G. = Vibration Generator, [b]N.R. = Not Recorded

TABLE 1 NORMALIZED DEFLECTED SHAPES FOR NON- AND POST-DESTRUCTIVE TESTS