VIBRATION TESTING OF AN EPOXY-REPAIRED FULL-SCALE REINFORCED CONCRETE STRUCTURE

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SUMMARY

A full-scale, four-story structure with a reinforced concrete frame was deliberately damaged by forced vibration, repaired by epoxy injection, and again deliberately damaged. At low-amplitude motions, the repaired structure was slightly less stiff than the original structure. However, at large deflections associated with severe damage, the stiffness of the repaired structure did not degrade as much as that of the original structure, and cracking in the beam-column connections was less severe in the repaired structure after the test than in the initial structure. Epoxy injection is considered an adequate method for repairing earthquake-damaged structures.

INTRODUCTION

Epoxy-injection techniques were used extensively to repair cracking of highway bridges, buildings, and other reinforced concrete structures damaged by the 1964 Alaska, 1969 Santa Rosa, California, and 1971 San Fernando, California, earthquakes, among others. In this method, a high-strength epoxy is injected into the cracked concrete, filling the voids and rebonding the fractured members. Experiments at the University of California, Berkeley (UCB), have shown this method to be quite effective for repairing laboratory specimens. However, the results of dynamic shaking-table tests conducted at UCB on two identical two-story reinforced concrete frame scale models suggested that epoxy repairs might not restore a damaged structure to its original stiffness. No information was then available concerning the effectiveness of epoxy repairs for full-scale structures subjected to high-amplitude, destructive-level vibration. In order to evaluate the effectiveness of epoxy repairs for full-scale structures, an epoxy-repaired full-scale reinforced concrete structure was tested in 1979. This paper summarizes the findings of that test program as reported in Ref. 1.

THE FOUR-STORY REINFORCED CONCRETE STRUCTURE

In 1965 and 1966, two identical, full-scale, four-story reinforced concrete structures (see Figures 1 and 2) designed specifically for field investigation associated with a structural-response program conducted by URS/John A. Blume & Associates, Engineers (URS/Blume) for the U.S. Department of Energy

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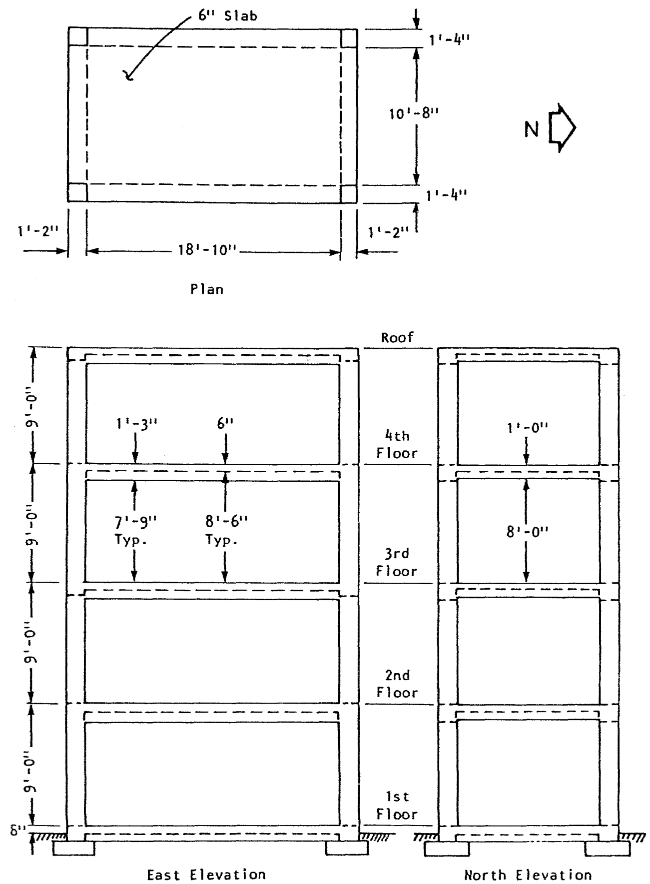

FIGURE 1 FOUR-STORY CONCRETE TEST STRUCTURE

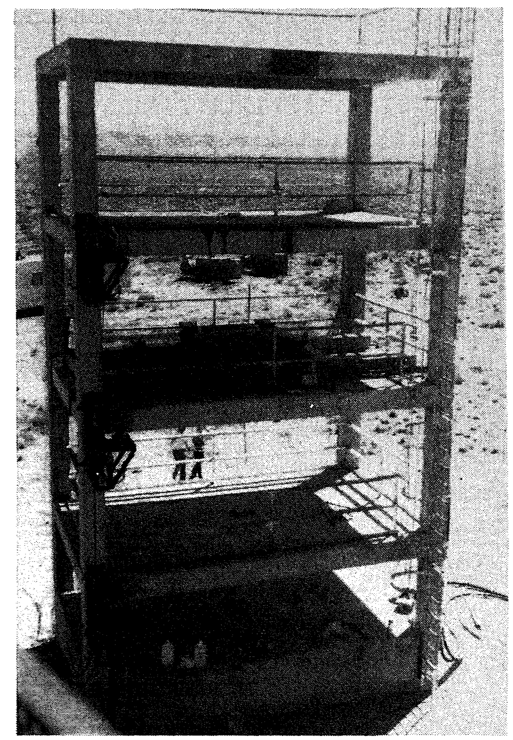

FIGURE 2 TEST STRUCTURE WITH VIBRATION GENERATOR ON THIRD FLOOR

(DOE) were constructed at DOE's Nevada Test Site. Each test structure is 12 ft by 20 ft center to center in plan and has four 9-ft stories. There are four rectangularly tied corner columns, 16 in. by 14 in. Spandrel beams are 16 in. by 15 in. in the 20-ft direction and 14 in. by 12 in. in the 12-ft direction. The floor slabs are 6 in. thick and are reinforced for two-way action.

Between 1966 and 1973, the two structures were subjected to ground motion caused by more than 50 underground nuclear explosions. They were also subjected to numerous nondestructive vibration tests during that period, including pull-release tests, vibration-generator tests, and human-induced-vibration tests.

In 1974, one of the structures was deliberately forced into the range of inelastic response by a reciprocating-mass vibration generator. Structural damage was extensive, consisting of X-cracking and spalling at beam-column connections. The type and extent of the damage were similar to what might be expected from a major earthquake. The results of that test were published in 1976 (Ref. 2).

RETESTING OF THE EPOXY-REPAIRED STRUCTURE

In 1975, the damaged structure was repaired by the epoxy-injection technique. The repaired structure provided an excellent specimen for determining the effectiveness of epoxy-injection techniques in recovering the original physical and engineering properties of structures. In 1979, the epoxy-repaired structure was again deliberately forced into the range of inelastic response.

Vibration Generator

The reciprocating-mass vibration generator built for the tests was capable of producing a maximum force of 12,000 lb and had a frequency range of 1 to 50 Hz. The vibration generator was designed and assembled by Sandia National Laboratories, Albuquerque, New Mexico.

The inertial force transmitted to the test structure was generated by a large oscillating mass driven by a hydraulic piston. A mass of approximately 15,000 lb was used in the 1974 tests and most of the 1979 tests, but it was increased to about 24,000 lb for the 1979 destructive test. The mass moved on four V-groove casters, with a maximum displacement of 3.9 in. from zero to peak. The vibration generator and associated hardware are described in more detail by Smallwood and Hunter (Ref. 3).

Instrumentation

Eleven L-7 velocity meters were used to measure structural response during the tests. The input force applied to the building was recorded simultaneously with these 11 channels of response data on a 12-channel recording system. These response data were recorded on magnetic tape in analog form and were later digitized for use in analysis. A detailed description of the L-7 seismograph system is given by Navarro and Wuollet (Ref. 4).

Five movie cameras were installed for the destructive test to provide permanent documentation of the structural response and damage. Four movie cameras were mounted on the test structure (at a beam-column connection at the top of each story), and one was located about 100 ft away from the structure to record the overall motion of the structure.

<u>Test Procedure</u>

The test sequence and procedures were generally the same as those followed for the 1974 tests. Although the testing was conducted principally to measure the dynamic-response behavior of the repaired structure in the inelastic range, low-amplitude testing in the quasi-elastic range was also conducted before and after the destructive test. With the vibration generator mounted on the roof, the structure was forced into low-amplitude motion in the 12-ft direction at frequencies corresponding to the first, second, and third modal frequencies for that direction. The vibration generator was rotated 90° on the roof, and the tests were repeated for the 20-ft direction. For the destructive test, the vibration generator was moved to the third floor and oriented along the 20-ft direction (see Figure 2). Following the test, the vibration generator was left in the same location and the structure was again forced into low-amplitude motion at the lower modal frequencies.

OBSERVATIONS AND RESULTS

The cracking at the beam-column connections appeared to be much less severe in the 1979 destructive test than in the 1974 test. Figure 3 shows damage from the 1974 test at the northeast corner of the third floor, and Figure 4 shows damage from the 1979 test at the same location. The less severe cracking in the epoxy-repaired structure may be the result of a better bond between the reinforcing bars and the concrete in the repaired structure than the bond that existed in the original structure.

The results of this study show that for low-amplitude motions the two structures had essentially the same stiffnesses at very low roof displacements, but the stiffness of the epoxy-repaired structure fell dramatically with respect to the original structural stiffness as the roof displacement increased (see Figure 5). This result was expected, however, because not all cracks could be repaired and because the epoxy that was used is less stiff than concrete. A plot of the destructive-test data shows that as the amplitude of the structure's response increased, the difference in stiffness between the epoxy-repaired structure and the original structure decreased. At a roof displacement of about 7 cm, the stiffness of the original structure underwent significant degradation; however, the epoxy-repaired structure continued to degrade in a very gradual fashion. This is further evidence that the epoxy-repaired structure had better beam-column connections than the original structure.

Several mathematical models were developed to represent the dynamic properties of the test structure in the longitudinal direction. One model was based on stiffness guidelines provided in Section 10.10.1 of the ACI 318-77 Commentary (Ref. 5) for frames that are free to sway. The stiffness of this model is indicated on Figure 5. When the stiffness of the epoxy-repaired structure corresponded to that of the model, the drift was approximately equal to the *Uniform Building Code* (*UBC*) drift limits of 0.5% of the story height.

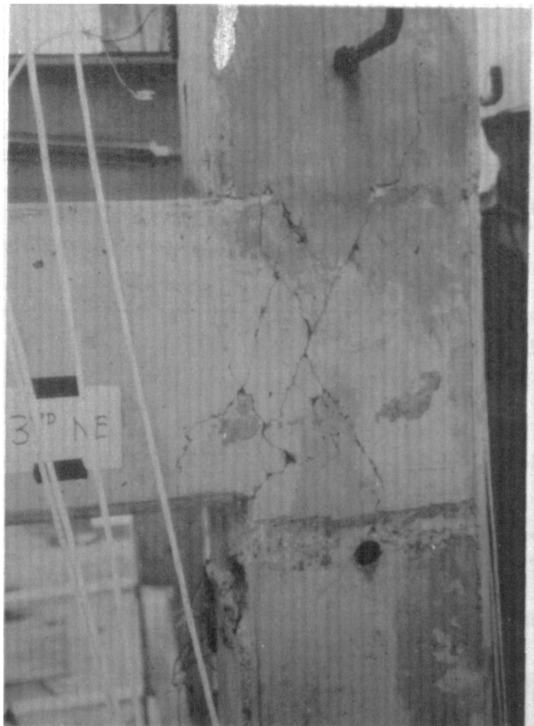

FIGURE 3 DAMAGE FROM 1974 TEST AT NORTHEAST CORNER OF THIRD FLOOR OF ORIGINAL STRUCTURE

FIGURE 4 DAMAGE FROM 1979 TEST AT NORTHEAST CORNER OF THIRD FLOOR OF EPOXY-REPAIRED STRUCTURE

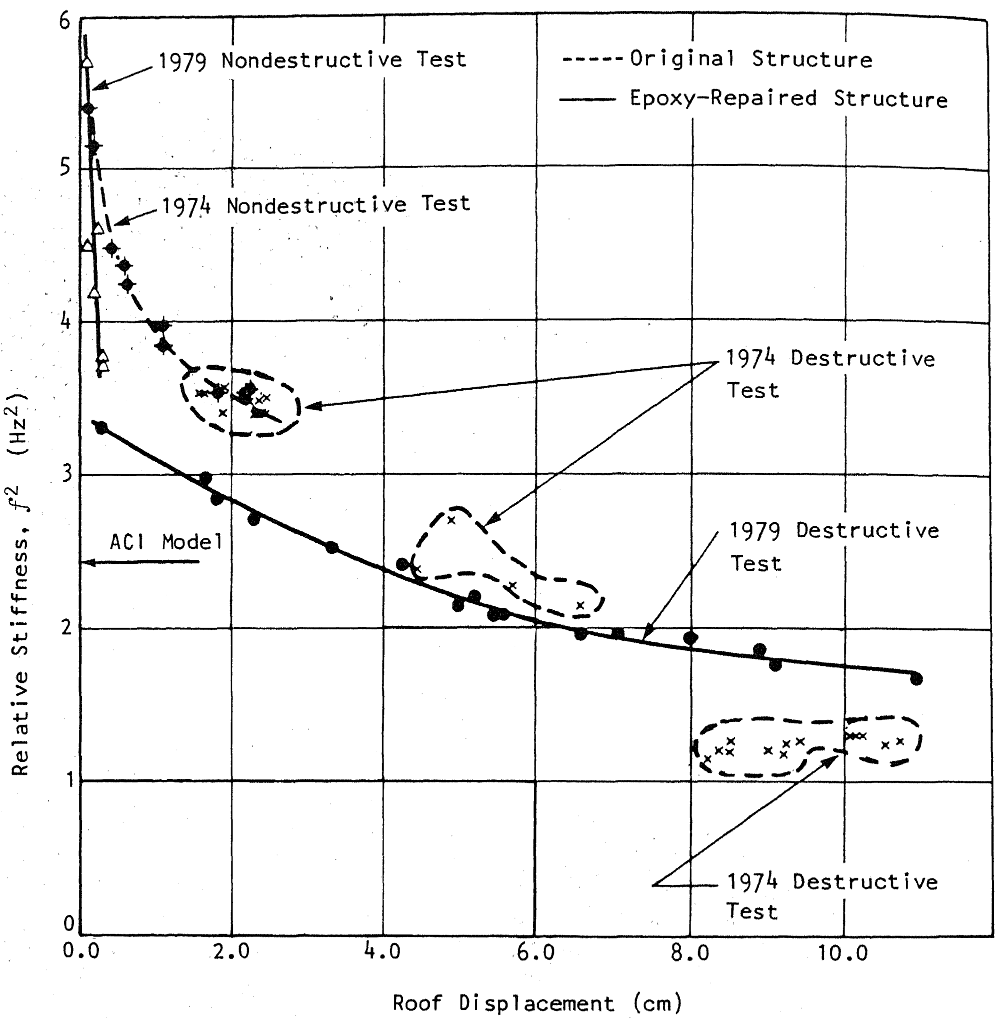

FIGURE 5 STIFFNESS VERSUS ROOF DISPLACEMENT FOR NONDESTRUCTIVE AND DESTRUCTIVE TESTS OF ORIGINAL AND EPOXY-REPAIRED STRUCTURES, FIRST MODE

However, when the original structure was at the same stiffness, the drift was about 50% greater than the *UBC* drift limits.

CONCLUSIONS

On the basis of these test results, the epoxy-injection technique appears to be a satisfactory method for repairing earthquake-damaged structures. However, it is important to note that when subjected to high temperatures, as in a building fire, the epoxy compounds will lose strength and may even burn. Furthermore, there is no current information on the effect of long-term aging on epoxy compounds.

ACKNOWLEDGMENTS

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