INTRODUCTION

This paper summarizes a program of vibration investigations of two identical 4-story reinforced concrete test structures (Figure 1), which were constructed in 1965 at the Nevada Test Site. These investigations were conducted by URS/John A. Blume & Associates, Engineers (URS/Blume), for the Nevada Operations Office of the U.S. Energy Research and Development Administration (ERDA, formerly the Atomic Energy Commission) as part of a structural response program associated with the detonation of underground nuclear explosions. The structures were built to obtain experimental data on the dynamic response characteristics of high-rise concrete buildings, ultimately leading to the development of improved techniques for predicting damage and response to ground motion. Effects Prediction Guidelines for Structures Subjected to Ground Motion (9) summarizes URS/Blume's work in the ERDA program.

The test structures are 12 feet by 20 feet in plan and consist of four 9-foot stories. Four rectangularly tied corner columns are 16 inches by 14 inches. Span-drel beams are 14 inches by 12 inches in the 12-foot direction and 16 inches by 15 inches in the 20-foot direction. The floor slabs are 6 inches thick, reinforced for two-way action. The design of these structures was consistent with the 1963 edition of the American Concrete Institute building code; design for lateral loads was based on horizontal static forces recommended by the 1961 version of the Uniform Building Code (UBC) for Seismic Zone 3. Provisions for ductility and reserve energy absorption capacity (1, 2) were also incorporated into the design of these structures. In anticipation of the possible additional weight of testing equipment and nonstructural partitions, the actual dead load plus 100 psf live load was used in computing the weight of each story for the design lateral force calculations. Thus, when loaded with dead load plus live load, the structures satisfied UBC Seismic Zone 3 requirements; but, when only dead load was present (the most common configuration), these structures had nearly twice the capacity of the 1961 UBC Seismic Zone 3, or approximately the capacity required by the 1976 UBC Seismic Zone 4. The design and construction of the structures is discussed in detail in Reference 3.

TEST EQUIPMENT AND PROCEDURES

In the course of the 10-year program, various methods of dynamic excitation were used to test the 4-story structures. The most frequent source of dynamic exci-
tation was the ground motion generated by the detonation of underground nuclear explo-
sions at the test site. The ground zeros or epicenters of these simulated earth-
quakes were located from less than 2 miles to more than 30 miles from the test
structures, producing ground motion signals at the structures with a variety of am-
plitudes and frequencies. The maximum roof displacements observed during under-
ground nuclear testing were between 2.0 and 2.5 cm. References 3 and 5 give addi-
tional data.

Another method of dynamic excitation was the pull-release procedure, also
known as a pull test. This method imposes a static horizontal deflection on the
structure by pulling with a predetermined force on a steel cable attached to the
building and releasing this force suddenly, causing the structure to experience free
vibration. Although all modes are initially excited, the fundamental mode dominates
the response after the first few cycles of motion. In the course of these tests,
forces up to 9000 pounds were applied at various floor levels of the test structures,
causing maximum dynamic roof displacements up to 0.5 cm. The principal advantages
of this procedure are ease of field implementation and ease of data reduction to de-
termine the period and damping ratio of the recorded response motion. The pull-re-
lease procedure was used during Test Series A through N.

The 4-story test structures were also tested using counterrotating-mass and
reciprocating-mass vibration generators. These mechanical devices can produce ap-
proximately steady-state harmonic motion and enable in-depth study of the response
of structures over a wide range of frequencies and amplitudes. Through the use of
these devices, it was possible to isolate and excite four structural modes of vibra-
tion and determine the dynamic response characteristics of each. The counterrotat-
ing-mass device was used in addition to pull tests during Test Series C, D, F, H, L,
and N, and the reciprocating-mass device was used during Test Series A.

For very quick and approximate determination of the fundamental mode period
and damping ratio, the man-induced vibration technique was used. In this method, a
man sways his body back and forth in approximate resonance with the fundamental mode.
Because of the large dynamic amplification factor of these lightly damped structures,
motion at and above the human perceptibility level can be produced.

In the course of the vibration tests of these structures, a great variety of
instrumentation and recording procedures were used to measure the horizontal and
vertical motion of the floors. Velocity and acceleration time histories of motion
were recorded both on paper strip charts and analog tapes. Many analog records have
been digitized and transferred to computerized magnetic tapes for further analysis.

METHODS OF RESPONSE DATA ANALYSIS

For most vibration tests, response data were analyzed using manual techniques.
In the early years of the testing program, hand analysis of the strip charts was the
only reliable method for determining the response characteristics of the structures
(i.e., period, damping ratios, mode shape, and maximum amplitude of response). With
the improvement of digitization and computerized analysis techniques, the manual
methods have played a less significant role in analysis of response data.

In recent years, a time domain analysis technique developed by Raggett (8) has
been used extensively to compute response characteristics from several 4-story test
structure vibration tests. This method matches the recorded time history of an isolated modal response with a mathematically generated time history of the response of a single-degree-of-freedom, viscously damped system having an assumed period, damping ratio, and modal participation factor. These response parameters are varied (within certain predetermined limits) by an iterative search procedure until the generated time history matches the recorded time history with a minimum squared error. The period, damping ratio, and participation factor resulting in the minimum squared error are output as the response properties of the isolated mode. Judgment must be exercised in using this method because the iterative search procedure may produce occasional anomalous results.

RESULTS

In the course of the testing program, a great deal of high-quality response data has been collected and analyzed. Unfortunately, in this short paper, it is not possible to discuss each test extensively. Six UHS/Blume reports (3, 4, 5, 6, 7, 9) may be consulted for further details. The structures were tested with and without several kinds of nonstructural partitions. The following discussion of results reflects these variations.

Results from Tests without Partitions. A significant result of the bare-frame vibration tests conducted on the 4-story test structures is the demonstration of the amplitude- and time-dependent nature of the response characteristics of reinforced concrete structures. Figure 2 is a plot of fundamental period of vibration versus roof displacement for the transverse direction of the south structure. This figure shows that the period tended to increase with amplitude at levels of motion well below the design level. This apparent degradation of stiffness at motion levels normally considered to be within the linear, elastic range of behavior demonstrates that response periods measured at low-level, ambient motions may be considerably less than those exhibited during a major earthquake, windstorm, or other kind of dynamic excitation.

These results may bring about some uncertainty concerning the most appropriate period for lateral force-resistant design. Whereas a lower value would probably result in greater design forces (according to the UBC design procedure), a higher value may be more realistic for significant dynamic responses. The period to be used to calculate design forces should therefore be based on the informed judgment of the designer.

A closer examination of Figure 2 shows that, in some cases, several years elapsed between tests and that, after some of these intervals, for example, Test Series N and O, the structure displayed a decrease in period or a recovery of lateral stiffness. There is no adequate theoretical explanation for this phenomenon, but it may be a result of the filling of hairline cracks with blowing desert sand or a recementation ("healing") of minute fractures within the concrete members. More research needs to be done to determine the factors that cause such a recovery.

Figure 3 is a plot of fundamental mode damping ratios versus roof displacement for the transverse direction of the south test structure. The curves in this figure demonstrate a low-amplitude adjustment of energy absorption capacity that produces a relatively constant fundamental mode damping ratio at higher amplitudes. Also apparent is the increase or decrease in the damping ratio that occurred between tests.
The magnitude of the change is probably related to the vibration exposure and time between tests; however, the nature of this dependence has not been determined. These results also show the fallacy in using ambient-level damping ratios to predict structural response at significant motion amplitudes.

The frequency-response curves shown in Figure 4 were obtained during Test Series D using the counterrotating-mass vibration generator. The velocity and period for the fundamental mode (i.e., the spike at 2 Hz) compares well with the data in Figure 2 for Test Series C and D.

Figure 5 shows results obtained in the longitudinal direction of the south structure during Test Series 0. This series, conducted in April 1974 using a reciprocating-mass vibration generator, covered a broader range of amplitudes than the tests discussed above and included the dynamic properties of the fundamental mode and higher modes of vibration. The magnitudes of the input forces used during these tests were selected so that approximately 75% of reinforcing yield stress would be experienced in the most highly stressed members. Thus, these tests were designed to cause motion amplitudes greater than any previously experienced but less than those causing structural damage.

The period of vibration is plotted versus root-mean-square roof velocity in Figure 5 for the four modes of vibration isolated during Test Series 0. The results for the first mode demonstrate the previously discussed low-amplitude stiffness adjustment, leading to a relatively constant period at high amplitudes. During Test Series 0, the first mode period reached a value of 0.53 second, which is approximately 45% greater than the period measured during Test Series A in 1966. (Test Series A involved considerably lower amplitudes than Test Series 0.) This corresponds to a 53% reduction in stiffness over an 8-year testing period that included amplitudes from the ambient level to approximately the yield level.

Figure 6 is a plot of the damping data obtained from Test Series 0. Like the period results, the damping ratios for all modes demonstrate amplitude dependence to some degree. It is significant that damping ratios for all modes are within a relatively narrow range of values between approximately 1.1% and 2.1% of critical. This suggests that, for this structure, the use of the same damping ratio for all modes is justified for design purposes.

A significant result of the entire 4-story test structure testing program, and particularly Test Series 0, is the comparison of measured dynamic properties with similar values computed from mathematical structural models. Figure 7 compares the periods and mode shapes measured in Test Series 0 with computed values, shown in parentheses. The calculated values are based on a mathematical model with fixed bottom-story columns and gross moment of inertia for the beams and columns. In general, the computed and recorded results are reasonably close when mathematical modeling assumptions, computation procedures, and instrumentation limitations are considered.

Results from Tests with Partitions. In addition to the bare-frame tests, the 4-story structures were equipped with various types of nonstructural partitions, including plywood, gypsum wallboard, concrete block, and hollow clay tile infill walls. In general, the test results show that the nonstructural partitions add to the stiffness and damping of the fundamental mode response and that these effects change with age and exposure to lateral motion. These changes are probably due to a combination
of degradation of the basic partition stiffness and loosening of the structure-partition connection.

Figures 8 and 9 are typical of the results obtained from tests using nonstructural partitions. Figure 8 shows a reduction in fundamental mode period that occurred when gypsum wallboard partitions were installed in the longitudinal direction of the north structure between Test Series A and B, and Figure 9 shows an increase in the damping ratio. After Test Series B, both period and damping increased with further testing and higher amplitudes. The period reached a maximum value between 0.45 and 0.46 second during Test Series H and then decreased for Test Series J and K. In contrast to the periods, the damping ratios increased dramatically from less than 1% to approximately 4.5% after the installation of gypsum wallboard partitions. After this increase, the damping ratios remained relatively constant for a number of tests with wallboard partitions. These changes in dynamic characteristics reflect the combination of partition plus structure stiffness and energy absorption variations.

Results obtained from tests using masonry block infill walls include an interesting phenomenon. After several vibration tests, the mortar joint between the block wall and the structure began to deteriorate, and a gap was formed. This gap created a dual structural response system. In this system with small displacements, the infill walls did not participate because voids between the structure’s frame and the masonry walls isolated the walls. In the system with larger displacements, there was contact between wall and frame, and therefore a stiffer structure and a shorter period. This nonlinear behavior resulted from the geometric configuration rather than from a change in material properties. Further details of this behavior and a mathematical model of the structure, including the masonry walls and gap, are given in Reference 6.

The combination of these results indicates that nonstructural partitions may have a significant effect on the dynamic response of structures. Some account of partition stiffness and energy absorption should be included in the design process.

CONCLUSION

The data presented in this paper demonstrate the time- and amplitude-dependent nature of the dynamic response characteristics of two 4-story reinforced concrete structures. This dependence should be considered in lateral force-resistant design and dynamic response prediction of concrete structures. Comparison of these test results with theoretical periods of vibration and mode shapes showed that conventional structural modeling techniques could be used to accurately represent response in the elastic range.

Test results obtained with nonstructural partitions reflect the effect these elements may have on response properties. The stiffness and energy absorption characteristics of nonstructural partitions should be considered in design and analysis of structures.

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equipment and operated the vibration generator used for Test Series 0. In addition to funding the project, ERDA also provided valuable administrative assistance.

Appendix I.--References

FIGURE 1 4-STORY CONCRETE TEST STRUCTURES
FIGURE 2 DISPLACEMENT VS FUNDAMENTAL PERIOD, NO PARTITIONS, TRANSVERSE DIRECTION, SOUTH STRUCTURE

FIGURE 3 DISPLACEMENT VS FUNDAMENTAL MODE, NO PARTITIONS, TRANSVERSE DIRECTION, SOUTH STRUCTURE

FIGURE 4 FREQUENCY-RESPONSE CURVES OF ROOF FOR TEST SERIES D, TRANSVERSE DIRECTION, SOUTH STRUCTURE
FIGURE 5 VELOCITY VS PERIOD FOR TEST SERIES 0, LONGITUDINAL DIRECTION, SOUTH STRUCTURE

FIGURE 6 VELOCITY VS DAMPING FOR TEST SERIES 0, LONGITUDINAL DIRECTION, SOUTH STRUCTURE

FIGURE 7 NORMALIZED NODE SHAPES FOR TEST SERIES 0, LONGITUDINAL DIRECTION, SOUTH STRUCTURE
FIGURE 8 EFFECTS OF GYPSUM WALLBOARD PARTITIONS ON FUNDAMENTAL PERIOD, LONGITUDINAL DIRECTION, NORTH STRUCTURE

FIGURE 9 EFFECTS OF GYPSUM WALLBOARD PARTITIONS ON FUNDAMENTAL MODE DAMPING, LONGITUDINAL DIRECTION, NORTH STRUCTURE