Seismic Response of a 1:6 Reinforced Concrete Scale-Model Structure with Flexible Floor Diaphragms

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A comprehensive investigation to study the effect of flexible floor diaphragms on the inelastic seismic response of reinforced concrete buildings is presented. A combined experimental and analytical approach is utilized. The experimental program consists of shaking-table tests of single-story 1:6 scale-model structures, as well as quasi-static component tests of beam-supported floor panels, flexible beam-columns, and stiff shearwall-beam frames under high-intensity cyclic loadings. An analytical modeling scheme is developed to include the inelastic effects of flexible floor diaphragms in the analysis of reinforced concrete buildings. A macro-model approach is used, where the effects of both in-plane flexure and shear are included. Presented here are an extended description and the results of an experimental study of the shaking-table response of a scaled-model structure, and its pertinent component tests. The correlation between the analytical predictions obtained using the developed model and the experimental responses is examined.

Keywords: beams (supported); concrete slabs; diaphragms; earthquake-resistant structures; floors; models; reinforced concrete; tests.

In building structures subjected to lateral loads, the floor slabs which usually have the task of transferring gravity loads to the vertical structural systems also set as a transfer medium for inertial forces to the lateral load-resisting system in a diaphragm action. For the simplicity of the analysis procedure and due to lack of understanding of the in-plane behavior of reinforced concrete floor systems, floor slabs are frequently treated as perfectly rigid elements in current design practice. However, experience and research have clearly demonstrated the importance of the influence of flexible diaphragms on the seismic response of many types of buildings. This influence is more pronounced for long rectangular low-rise buildings, particularly where a dual bracing system consisting of flexible moment-resisting frames and stiff shearwalls, L- and Y-shaped plan buildings, and setback building structures, especially when cracking and yielding are expected during earthquakes.

A realistic evaluation of the stiffness, strength, and ductility of the building depends not only on the inelastic characteristics of its vertical load-resisting elements (beam-column frames and shearwalls) but also on the seismic behavior of the reinforced concrete floor slabs and their interaction with the rest of the structural elements. Only after such an evaluation can the designer make a rational decision regarding the selection of desirable objectives. Should the floor diaphragms ever become inelastic? Are there any advantages in having a building with ductile floor diaphragms? Is the detailing of ductile floors significantly different from that of rigid diaphragms? Thus, an understanding of the in-plane behavior of reinforced concrete floor slabs and their influence on the dynamic characteristics and response of structures (i.e., fundamental period and distribution of the lateral forces in the vertical elements) is necessary. These effects are usually ignored in an analysis when a rigid floor diaphragm is assumed.

OBJECTIVES AND SCOPE

The present investigation is a continuation of comprehensive research started at Lehigh University in 1977 and expanded into a cooperative research program with the State University of New York at Buffalo. The objectives of this study are focused on the effects of inelastic floor behavior on the seismic response of reinforced concrete buildings, and on the development of simplified design guidelines and recommendations for improving seismic codes to incorporate the influence of flexible floor diaphragms. To attain these objectives, several component tests of 1:6 scaled models of slab systems, flexible and stiff vertical lateral load-resisting systems, and full assemblies (i.e., single-story structures) were tested under quasi-static cyclic loading and...
Special load cells were designed, fabricated, and calibrated to measure shear force, bending moment, and axial loads at the base of the frames below the footings. The average rotation at the base of the end walls and midframe columns were monitored using displacement transducers mounted at a short distance from the base, one at each side of the vertical members. Internal wall ages including rosettes were embedded inside the walls. In slabs to optimize the measurement of shear, bending, and axial strain distributions in these members. The dynamic response of the model structure, as well as the shaking-table excitations, were monitored by a microcomputer-controlled data-acquisition system. One hundred and five channels of data were recorded at a frequency of 400 samples per channel per sec.

The shaking-table testing program consisted of two stages. The initial series of tests included a number of low-intensity scaled earthquakes (Taft 1952, El-Centro 1940, and San Francisco 1957, with a maximum acceleration of 0.06 g). The time axis of the recorded earthquakes were scaled down by a factor of 1/10 to satisfy the similarity requirement. The main objectives of the low-intensity tests were: 1) to check the operation of the instrumentation, data-acquisition system, and shaking table; 2) to check the performance of the scaled model (e.g., symmetry and magnitude of the response amplifications achieved); and 3) to check the experimental results against the analytical results. Accomplishing these objectives resulted in modifying the initial member stiffness in the analytical model to reflect the present state of the test model on the shaking table.

The second stage of testing consisted of a high-intensity base motion that would result in failure of the structure. The analytical model was used as an important tool in selecting this base motion. The modified Taft 1952 accelerogram scaled to a peak intensity of 1.1 g was selected. This scale selection was done with due consideration for (1) the effect of the interface concrete block on amplification of the input motion; 2) interaction of the model structure with the shaking table during inelastic response; and 3) capacity limit of the shaking table. Dynamic characteristics of the model structure (nuclear frequency, damping, and stiffness properties) were monitored continuously by studying the response of the scaled model under low-intensity white noise excitations (with a peak acceleration of 0.06 g).

**ANALYTICAL MODELING OF INELASTIC BEHAVIOR**

An enhanced computer code for the inelastic analysis of reinforced concrete buildings, IDARC, was extended to include inelastic diaphragm elements to model buildings with flexible floors. IDARC is a computer program for two-dimensional analysis of 3D building systems in which a set of frames parallel to the loading direction is interconnected by transverse elements to permit flexural-torsional coupling. The details of the development of the analytical schemes may be found in Park, Reinhorn, and Kunnath.**

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**Fig. 7—Typical discretized section of structure end component modeling**

In the revised modeling scheme of IDARC (see Reference 15), a reinforced concrete building is idealized as a series of plane frames linked together by inelastic floor slabs and transverse beams. Each frame must lie in the same vertical plane. Consequently, a building is modeled using the following element types: floor slabs, beam-columns, shearwalls, edge columns (boundary elements of walls), and transverse beams. A discretized section of a building using all of the preceding element types is shown in Fig. 7. All components of the building, except transverse beams, are modeled as trilinear, degrading inelastic elements with concentrated plasticity at member ends, and a distributed flexibility rule to account for the spread of plasticity. A linear variation of flexibility is assumed in deriving the flexibility matrix.

**Floor-slab panel model**

A typical floor slab is modeled using two degrees of freedom per node: one lateral and one rotational. Two inelastic springs are used to model shear and flexure independently. The springs are then connected in series to account for the effects of shear cracking. A generalized technique for evaluating the flexural capacity of floor slabs is also developed. The analytical envelope is modified based on observed experimental data to fit a trilinear curve, which enables the subsequent hysteretic component modeling. The program is also capable of modeling the floor diaphragm as either a rigid or an elastic system for the purpose of comparative studies.
TESTING SETUP AND PROCEDURE

Component tests

The single bay frames (with and without shear walls) were tested in similar setups, which consisted of a mechanical jack with a 20 kips (90 kN) load cell supported against a reaction beam, applying a lateral load on the specimens at the center of the plane of the cross section of beam and slab. For the shear-wall-beam frame specimen, the concrete footing was fastened directly to the test bed by 3 in. (7.6 cm) diameter anchors. For the beam-column frame specimen, a pair of load cells was used between the concrete footing and the test bed to measure the reaction forces at the base of each column. The setup for the slab specimen was designed to simulate the loading condition at the middle slab panel of the single-story model structure. The specimen was rigidly supported along one transverse beam, while a triangular steel frame attached to the opposite edge was used to apply the in-plane load to the specimen. The length of the loading steel frame was designed to produce the desired ratio of bending moment to shear force predicted by the analytical model at the critical slab section. Additional gravity loads for the three-component specimens were applied by hanging steel blocks underneath the slabs. The amount of additional weight for each component test depended on its tributary area in the single-story model structure.

All the component tests were subjected to initial small force-controlled cycles gradually approaching the yielding load. Subsequently, the loading was shifted to displacement-controlled cycles, where the peak displacement at each cycle was increased by one-half of yield displacement until the specimen failed. The overall displacements of the specimens were obtained using linear variable displacement transducers (LVDTs), while clip gages were used to measure section rotations at beam-column or beam-wall joints. Also, a limited number of surface and internal strain gages was used to monitor shear and axial strains of the concrete members. Additional details on the component testing are documented in Yu and Lu.6

Shaking-table test

The single-story scaled model was placed on the test bed on 10 load cells, located beneath the model. To simulate gravity load on the model dictated by the scale model, the test specimen was placed on the floor slabs. Rubber pads were used under the bricks to minimize altering the in-plane stiffness of the floor diaphragm.

The structure was instrumented primarily to generate information on the global and also local behavior of the test specimens. Accelerometers and displacement transducers were used to measure the absolute accelerations and displacements at the floor levels, as well as at the base of the model at each frame parallel to the base motion. A limited number of accelerometers was used to monitor the response of the structure in the longitudinal direction perpendicular to the base motion.
reinforcement in the linking beam between the walls. Further details of the design of test specimen are given in other data and Reference 16. The similarity requirements for the reduced scale model dictate that the model reinforcement and microconcrete mechanical characteristics be the same as typical prototype materials. Due to the small dimensions of the specimen, the maximum aggregate size was restricted to 1/4 in. (6.3 mm). The microconcrete mix suggestions given by Kim, El-Attar, and White2 were used for the initial trials. Following several iterations, a microconcrete mix proportion of 1:0.8:3.92:2.1 of Type III portland cement, water, model sand (particle size passing through No. 8 sieve), and model aggregate (particle sizes ranging from No. 8 sieve to No. 3 sieve) was selected. A superplasticizer was used to increase the slump of the mixture. The model sand-to-model aggregate ratio had to be increased from the suggested val-

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Fig. 1—Overall geometry of 1:6 scale-model structure

with simulated earthquake loading on a shaking table.

The main objective of this paper is to describe the experimental program and report the results of the shaking-table test of a single-story four-span microcon-crete model structure with stiff end frames and the relevant component tests. In addition, a brief description of an inelastic analytical model developed for this study is presented, and the correlation of the analytical predictions of the shaking-table test is discussed.

RESEARCH SIGNIFICANCE

Experimental and analytical work on the behavior of reinforced concrete floor diaphragms under inelastic cyclic loading has been rare. The lack of information on the in-plane characteristics and the influence on the seismic response of reinforced concrete buildings has been the primary reason for not including diaphragm behavior in current design procedures. The experimental and analytical study presented here can be a valuable source of information for understanding the seismic response of reinforced concrete buildings with inelastic floor diaphragms. The shaking-table testing of a 1:6 reinforced concrete scale-model structure provides evidence of the effectiveness of scale-modeling techniques used to obtain information on the integral behavior of a complex system of diaphragms, frames, and shearwalls. Furthermore, the comparison of the experimental results with the analytical predictions show that it is feasible to predict the response of a complex inelastic system when the behavior of its components is known. Experimental results validate the analytical model's use in future studies to determine the importance of diaphragm floor diaphragms.

DESCRIPTION OF THE MODEL STRUCTURE

Limited by the geometry and capacity of the shaking table at SUNY/Buffalo, a 1:6 scale-model structure was designed based on a single-story four-span prototype building that satisfied minimum requirements of ACI 318-83 and the New York City Building Code. The structure consisted of four square panels supported by four peripheral shearwalls (one pair at each end) and three parallel one-bay frames. A concrete strength of 4000 psi, Grade 40 reinforcement for slabs, and Grade 50 reinforcement for beams, columns, and walls were used.

Overall geometry and dimensions of the model test structure, the composite specimens (lumbar frame, interior frame, and slab panel), and the reinforcement details of a typical slab panel are shown in Fig. 1 through 5, respectively. Special rectangular footings are designed to provide adequate space for anchorage of wall and column reinforcements as well as to function as a relatively rigid link (in combination with the load-measuring devices) to the shaking table. This provides accurate measurement of the frame reaction forces without distorting the simulated base motion applied.

As an analytical model was developed (ID=AC2) and used to design the test structure so that diaphragm yielding would occur prior to yielding of the stiff end frames within the capacity limitations of the shaking table. This resulted in increasing wall reinforcement to about 1 percent of gross sectional area and doubling the
for floor slabs except for the inclusion of axial effects and the incorporation of edge columns at the ends of the wall. Walls may, however, be modeled with or without edge columns.

Transverse beam elements
To incorporate the effects of transverse elements to account for their restraining action due to the axial movements of vertical elements, especially edge columns in shearwalls, and permit flexural-torsional coupling with main elements, each transverse T-beam is modeled using elastic springs with one vertical and one rotational (toroidal) degree of freedom, as shown in Fig. 7. Transverse elements are basically of two types: beams that connect to shearwalls and beams connected to the main beams (i.e., in the direction of loading).

Hysteretic modeling of components
The hysteretic model used for the analysis incorporates three parameters in conjunction with a non-symmetric trilinear curve to establish the rules under which inelastic loading reversals take place. Details of the general meaning and effect of the parameters can be found in Reference 19. The three main characteristics represented in the model are stiffness degradation, strength deterioration, and bond-slip or punching. These hysteretic parameters can be combined in various ways to achieve a range of hysteretic behavior patterns typical to reinforced concrete sections.

The developed analytical model was used for planning experimental studies of scaled models. The predictions helped to design the scaled models in terms of expected peak responses and damage behavior. Subsequently, the test results were used to calibrate and improve the computational model.

EXPERIMENTAL RESULTS

Component test results
The overall behavior of the component tests under quasi-static cyclic loading are presented in Fig. 8(a) through (c), where the load-displacement plots of end frame, middle frame, and slab panel specimens are shown. The behavior of all the components was symmetrical with respect to the loading direction. The wall-beam specimen (end frame) yielded at a lateral displacement of 0.18 in. (4.57 mm) (0.5 percent of the height of the frame) when the load reached 12.5 kips (55.6 kN) at the eighth load cycle (see Fig. 8(a)). The yielding occurred primarily at the base of the walls. Maximum load resisted by the specimen was about 13.7 kips (90.9 kN), obtained at a displacement of 0.28 in. (7.11 mm) in one direction, while maximum load resisted in the reverse direction was 13.4 kips (59.6 kN), which occurred at an applied lateral displacement of 0.59 in. (12.7 mm). Further cycling of the specimen re-
sulted in decrease of the load-carrying capacity of the specimen. A clear pinching behavior was observed at high-intensity loading cycles, which was due to opening and closing of the wide cracks at the base of the walls. A maximum displacement of 1.1 in. (27.9 mm) was reached at the 22nd cycle [load of 8.64 kips (38.4 kN)] just prior to specimen failure. This displacement was 3 percent of frame height, which resulted in a high ductility ratio of 6.1. Note that specimen yielding is defined at the observed yield of the reinforcing bar at the critical section.

Similar loading procedures were used on the beam-column component (middle frame) and the slab component. The middle frame specimen yielded at a displacement of 0.72 in. (18.3 mm) (2 percent of the frame height), where the lateral load was 1.1 kips (4.9 kN) at the 72nd cycle of loading. Maximum load capacity of 1.2 kips (5.3 kN) for the frame was obtained when a peak displacement of 1.23 in. (31.2 mm) was applied. Further cycling of the specimen gradually degraded the load-carrying capacity of the specimen. The specimen failed at the 28th cycle, where a maximum displacement of 3.4 in. (86.4 mm) (ductility ratio of 4.7) was reached. Unlike the wall-beam specimen, no pinching behavior was observed for beam-column specimen [see Fig. 8(b)].

The load-displacement plot of the slab panel component (see Fig. 8(c)) indicates that the slab yielding started at a displacement of 0.24 in. (6.1 mm) (0.5 percent of the panel length), where the in-plane load was 2.2 kips (12.6 kN). In-plane cracks were observed adjacent to the beam at the fixed end of slab panel, and also at a distance of 15 in. (38.1 cm) from the fixed support. Maximum in-plane load of 2.98 kips (13.2 kN) occurred at a displacement of 0.73 in. (18.5 mm). The load dropped to 2.78 kips (12.4 kN) at the following cycle (29th cycle), where the maximum displacement [0.84 in. (21.3 mm)] occurred just prior to failure. This was 1.8 percent of the slab panel length [48 in. (121.9 cm)], which resulted in a ductility ratio of 3.5. Limited pinching behavior was observed during the final load cycles. Detailed results are presented in Reference 18.

Shaking-table test results

Experimental results of the 1:6 scale-model structure are summarized in Fig. 9 through 11 for the second stage of shaking-table testing, where a high-intensity base excitation was used. The time histories of the acceleration at the base of the structure (input excitation for the structure), as well as the acceleration at the top of the structure (diaphragm response at the middle and end frames), are shown in Fig. 9(a) through (c). The maximum base acceleration was 1.38 g, which occurred at 3.53 sec. However, the first major peak (1.24 g), responsible for inelastic diaphragm behavior, occurred at 1.44 sec. Comparisons of peak top and base accelerations indicate that, due to the inelastic response of the model structure, the amplifications obtained ranged between 1.17 to 1.34. Top accelerations for the end frames were similar (within 4 percent), which indicates

Fig. 9—Model structure base (input) and top accelerations (slab response)
that structure response was symmetrical; thus, only the time history plot for one frame is shown in Fig. 9(b).

Floor diaphragm yielding occurred at the interior panels, where wide in-plane cracking of the slab was observed at midspan of the model structure, just next to the middle frame beam.

Total structure base shear history and its distribution between the end frames and interior frames are shown in Fig. 10 (a) through (c). For ease of comparison, three points (A, B, and C) are labeled at different stages of loading. At Point A (0.61 sec.), the structure remains elastic, while at Point B (1.52 sec), the floor diaphragm has experienced the first cycle of inelastic loading, and Point C (6.09 sec.) represents the model structure after four cycles of inelastic loadings. Maximum total base shear measured at 25.1 kips (113.0 kN) was caused by the first cycle of high-intensity base excitation, which was responsible for the initial in-plane cracking of the slab panels at Point B (see Fig. 10(a)). Maximum shear resisted by the end frames was 23.17 kips (103.1 kN), which also occurred at Point B (see Fig. 10(c)). This load, causing flexural cracking at the base of the wall, was just below the yielding value estimated by component tests (25.0 kips (111.2 kN)); thus, the end frames did not experience considerable damage.

It was observed that a number of inelastic cycles of diaphragm loading between 1.51 and 6.09 sec (Points B and C in Fig. 10) has changed the base shear distribution from end frames toward the interior frames. The base shear contribution of interior frame was only 4.6 percent of the total shear during the elastic phase of the excitation (Point A). This share was almost doubled (8.0 percent) at Point B as the middle slab panel was subjected to the first inelastic load cycle. At Point C, although the total structure base shear was only 12.96 kips (57.6 kN), interior frames resisted up to 20.4 percent of the total shear, where maximum shear value of 2.64 kips (11.7 kN) was obtained.

Top displacement time histories at the end frame and middle frame of the model structure are compared in Fig. 11. Maximum displacement of 0.47 in. (1.19 cm) was obtained at 6.24 sec, which was 6.2 times the maximum end frame displacement. This is due to in-plane yielding of the slab panels, which also resulted in some residual displacements at middle frame (see Fig. 11). Note that the measured period of the structure in-
increased from 0.083 sec (before the test) to 0.123 sec (after the test), which was an indication of the stiffness reduction of the model structure caused by floor diaphragm yielding.

ANALYTICAL RESULTS AND CORRELATION WITH TEST RESULTS

The developed analytical model was used for inelastic analysis of the single-story model structure tested on the shaking table during the second stage of the testing program. The base acceleration record obtained from the test (Fig. 96a) was used. The initial member stiffnesses and the initial damping of the model structure (obtained as 4 percent of critical) were obtained from the first stage of testing. Most of these values were similar to the values obtained from component tests.

The results of the component tests were used to obtain the inelastic parameters required for the hysteretic modeling and the corresponding force-deformation envelopes.

Analytical results are presented and compared with experimental values in Fig. 12 through 14, where model base shear, middle frame base shear, and middle frame top displacement time histories are plotted for the initial 5 sec. The correlation of the results is good, with some discrepancies that may be attributed to the loading rate effect, which was not accounted for in the analysis and the approximations used in selecting the inelastic parameters. For example, the peak base shear obtained experimentally is about 15 percent higher than the analytically predicted value. This can be due to increased yielding capacity of the model structure caused by the high rate of loading obtained from the shaking table test. Consequently, due to earlier yielding of ele-

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that were used to determine the inelastic parameters, and also to include the effects of loading rate.

CONCLUSIONS

A summary of the description and overall results of shaking-table testing of a single-story four-span 1:6 reinforced concrete scale-model structure with flexible floor diaphragms (supported by end walls and interior moment-resisting frames) has been presented, with quasi-static cyclic tests of relevant components. It is found that the scaled models can be appropriately used to investigate the nonlinear dynamic response of buildings with flexible floor diaphragms.

An analytical program is developed to include the inelastic behavior of floor diaphragms in the seismic evaluation of reinforced concrete buildings. The results of the component tests were used to predict the dynamic response of the model structure on the shaking table. The developed computational tool was used for planning the experimental program. The model structure was subjected to high-intensity base motion, which successfully produced inelastic diaphragm behavior resulting in yielding of the structure. The correlation of the analytical predictions with the experimental responses is discussed.

Floor diaphragm yielding occurred at the interior slab panels at the midspan of the model structure adjacent to the middle frame, where wide in-plane cracking of the slab was observed. Inelastic cyclic loading of the floor diaphragms during early seconds of the base motion resulted in a considerable amount of shear force distribution among the interior frames, while the overall structure stiffness was reduced substantially. The shape of the base shear resisted by the interior frames more than quadrupled while the period of the model structure increased by 50 percent. The increased ductility demand on the interior frames caused by shear force distribution for the tested structure was not significant. However, it is expected that for some structures, these demands can exceed the customarily provided design capacity.

Further calibration studies are currently under way that use the results of the experimental program to refine the analytical model. Simulation studies are also being carried out to obtain practical design guidelines for improvement of seismic codes to incorporate the effects of flexible floor diaphragms.

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REFERENCES

1. Karadag, H. F.; Nakashima, M.; Huang, T.; and Lu, L. W., "Static and Dynamic Analysis of Buildings Considering the Effect of


13. AIC Committee 311, "Building Code Requirements for Reinforced Concrete (ACI 318-83)," American Concrete Institute, Detroit, 1987, 113 pp.


