EXPERIMENTAL STUDY OF A FLAT-PLATE BUILDING MODEL

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SUMMARY

An experimental and analytical study of the seismic behavior of a two-story reinforced concrete flat-plate framed model is described. The model was constructed at approximately one-third scale with details following recommendations in United States for structures in regions of low to moderate seismic risk. Primary tests include low, moderate, and high intensity earthquake simulations on a shaking table. This paper documents the experiments and discusses some of the observed responses.

INTRODUCTION

Occasional failures of reinforced concrete flat-plate framing systems during severe ground shaking have led to widespread rejection of the flat plate as a viable system in regions of high seismicity. Economics and good performance under gravity loads have led to equally widespread acceptance of the system in regions of lower seismic risk. Recently, growing awareness of the potential for strong ground shaking in eastern and midwestern regions of the United States has given rise to concerns about performance of the flat plate during low to moderate-intensity shaking. In response to these concerns, an experimental research program has been undertaken to study behavior of a flat-plate frame under low, moderate, and high-intensity base motions. A one-third scale model of a two-story, three-bay flat plate with edge beams has been constructed and tested on the University of California Earthquake Simulator. Component tests of isolated plate-column connections provide insight into local behavior of the complete structure. This paper describes the test structure and some of the test results.

DESIGN OF PROTOTYPE STRUCTURE

The prototype structure is a two-story flat-plate frame having three bays along one principle axis and multiple bays along the other. Columns are arranged regularly at 6.1 m (20 ft) on centers. An edge beam spans the perimeter of each floor.

The frame is proportioned for gravity loads according to strength design provisions of Ref. 1. Live load is assumed at 2.87 kPa (60 psf).

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Under full dead and live loads (with no load factors), the nominal shear stress on a section half a slab thickness from the column is $0.083\sqrt{f'_c}$ in MPa units ($1.5\sqrt{f'_c}$ in psi units) for interior slab-column connections. Nominal shear stresses attributable to shear and moment transfer at exterior connections are such that edge beams are required. Nominal axial stress at the base of interior columns is $0.1f'_c$.

Seismic design assumes a maximum probable event represented by intensity VII on the Modified Mercalli scale (Zone 2 of the UBC, Ref. 2). The equivalent static lateral force method of the UBC is used for determining seismic effects. For lateral-load analysis, the flat-plate frame is modelled by gross-section properties with equivalent beams having flexural inertia of half the slab width. Using these assumptions and load combinations recommended by either ACI or UBC, it is found that seismic design forces do not govern the proportions.

DIMENSIONS AND DETAILS OF TEST STRUCTURE

The test model was constructed at a scale of 1 to 3.33, and only two bays of the multiple-bay direction were constructed. The scaled test model is depicted in Fig. 1. Base columns were cast monolithically with stiff "footings", which in turn were supported on stiff transducers. Thus, fixity at the column base level was not achieved completely, and might be considered qualitatively similar to the degree of fixity afforded by a real soil foundation. Stiffness of the combined footing-transducer system exceeds the stiffness of an equal length of column.

Care was taken to simulate materials and details of full-scale construction. Concrete was cast in three lifts, one for footings and one each per floor. Mean concrete strengths at time of test were 37 MPa (5300 psi) in compression and 4.8 MPa (700 psi) in modulus of rupture tests. Longitudinal column and edge-beam steel was No. 2 deformed bar having mean yield stress of 470 MPa (68 ksi). Slab reinforcement was 4.52-mm (0.178-in.) diameter deformed bar having mean yield stress of 440 MPa (63 ksi). Transverse column and edge-beam reinforcement was plain 3.0-mm (0.12-in.) diameter wire with mean yield stress of 620 MPa (90 ksi).

All details follow closely recommendations given in Ref. 3 for structures in regions of low-to-moderate seismic risk. Details of slab reinforcement at an interior joint are in Fig. 2. An extra mat of reinforcement is provided near the column region to enhance behavior. Longitudinal steel ratio in columns is 0.013. At column ends, all bars are restrained in the corner of a tie having spacing of 51 mm (2.0 in.). Stirrup spacing in end regions of edge beams is 25 mm (1.0 in).

Supplementary lead weights were supported on tops of slabs. The weights were positioned to simulate moments and shears due to slab dead loads occuring in the prototype structure. Connections ensured that the weights did not slip during testing and did not enhance stiffness or strength of the model. Total weight of the model including lead ballast was 210 kN (47 kips).

TESTS AND INSTRUMENTATION

Base motions simulated the motion recorded in El Centro during the 1940 Imperial Valley Earthquake. In some tests, a single horizontal base motion (modelling the NS component) was imparted parallel to the three-bay direction of the structure (Fig. 1). In other tests, the horizontal component was augmented by the corresponding vertical component. The time scale of base motions was compressed by a factor of 1.71. The horizontal acceleration record obtained during the "design" test is plotted in Fig. 3. Test intensities were increased incrementally, with horizontal peak base accelerations ranging between 0.015 and 0.83 g.

Before the first simulation and subsequent to each, a free vibration test was conducted by initially displacing the model with a constant force and then releasing suddenly.

Instrumentation provided continuous response records of relative floor displacements and absolute floor acceleration. Base shears could be obtained by the product of floor accelerations and masses, and checked by the sum of shears measured by transducers located below footings (Fig. 1). Strain gauges were at selected locations on slab and column reinforcement.

INITIAL DYNAMIC PROPERTIES

Initial dynamic properties are obtained from free-vibration responses before the earthquake simulations. The initial period was 0.21 sec. For the full-scale building this corresponds to a period of 0.38 sec, which is substantially larger than the value given by 0.1N, where N = number of stories, as given by the UBC (2). This is indicative of the flexibility inherent in flat-plate framing. Using half the slab width as an equivalent beam (Ref. 4) the period is computed to be 0.19 sec for gross-section stiffness. The slightly longer measured period can be attributed to initial minor cracking. Initial equivalent viscous damping obtained by the log decrement is approximately 1.5 percent of critical. This value is typical for uncracked or lightly-cracked reinforced concrete structures.

RESPONSES TO SIMULATED EARTHQUAKES

Five earthquake simulations were conducted with peak horizontal accelerations below 0.1g so that responses to low-level events could be observed. Maximum drift levels reached only 0.1 percent of structure height. Vibration periods and damping factors obtained in free-vibration tests were the same as those obtained before testing, indicating that damage was negligible. Only slight cracking was observed.

The design-level test had a peak horizontal acceleration of 0.19g, with no vertical input (Fig. 3). Top-floor relative displacement and base-shear records (Fig. 4) reveal that displacement and base-shear responses were predominantly first mode. Maximum top displacement was only 0.3 percent of structure height, a value well within accepted limits for drift control. The base shear coefficient obtained was 0.29 (ie, 29 percent of structure weight).

Damage induced by the design-level test was light. Cracking was apparent in slabs around the columns and in columns at the footing level. Maximum crack widths did not exceed 0.3 mm, and there was no indication of shear distress. Yield was not detected by any strain gages. During a subsequent free-vibration test, the vibration period was 0.22 sec and the damping factor had increased noticeably to 0.025. The effective period during the peak response, taken as time between three successive zero crossings, was 0.26 sec, indicating an effective stiffness during peak response of approximately 65 percent of the initial stiffness.

Responses to a subsequent test for which peak horizontal base acceleration was 0.61 g are plotted in Fig. 5. For this test, maximum drift level was 3.4 percent of height, and peak base-shear coefficient was 0.8. During the peak response, the period was 0.54 sec, indicating an effective secant stiffness of only 15 percent of the initial gross-section stiffness. Viscous damping obtained during a free-vibration test was 5 percent of critical. Flexural cracking, with widths indicative of yield, was observed on both top and bottom surfaces of slabs and ran the entire width of the slab. Fan-shaped cracks were apparent around all interior columns, and wide torsion cracks were apparent in edge beams. In addition, crushing was observed at the base level of columns.

A final simulation having peak horizontal and vertical accelerations of 0.83 and 0.20 g, respectively, resulted in top drift of 5.5 percent, and base-shear coefficient of 0.84. The pattern of cracking indicated that punching failure was imminent at one of the first-floor interior columns. Severe torsional damage to the edge beams resulted in loss of cover concrete near the columns. Column base crushing was apparent, but bar buckling did not occur.

LOAD-DEFORMATION RESPONSE OF STRUCTURE

The measured envelope relationship between base shear and top displacement is in Fig. 6. The relationship does not indicate a sharp cracking or yielding point. This can be expected because cracking and yielding occur initially in the slab at the columns and spread gradually in the transverse direction.

The initial stiffness can be approximated using the gross section with half the slab width as an equivalent beam, and with lateral loads distributed uniformly over height. By assuming loads are uniform over height, computed stiffness is approximately ten percent higher than if loads vary linearly. The experiments indicate the true variation was typically between these extremes. The initial calculated slope compares reasonably with the measured slope (Fig. 6) It is consistent with observed damage in that it indicates deviation (hence, cracking) beyond displacements of approximately 0.3 to 0.4 percent of structure height.

The maximum base-shear coefficient obtained near collapse was 0.84, which is significantly in excess of design base shears stipulated by design codes (2). Significant deviations between design base shear and actual base shear are frequently attributable to member overdesigns, material overstrengths, and internal force redistribution which is not considered in the design analysis model. This situation is exacerbated for the test structure because requirements for gravity load design exceed those for seismic design.

Observed damage indicates a collapse mechanism involving plastic hinges in slabs at all columns and in columns at the base. Flexural capacities of slab-column connections have been determined experimentally from reversed-loading tests on slab-column subassemblies. Assuming a uniform lateral-load distribution, computed base shear capacity is 156 kN. Measured capacity exceeds this value by approximately ten percent. The overstrength might be attributable to enhancement of material strengths due to strain-rate effects, or redistribution of internal forces in the continuous model which was not possible in the simpler slab-column subassemblies.

Using measured load-deformation behavior of the isolated slab-column subassemblies, it is possible to reconstruct the load-deflection behavior of the complete structure. As a simple approximation, a model was devised which assumed inflection points at midlengths of slabs and columns, and which assumed lateral displacements of floor levels varied linearly over height (as observed during the tests). Under the uniform lateral-load distribution, the calculated relationship between base shear and top displacement is as depicted in Fig. 6. Agreement with the measured envelope is acceptable.

SUMMARY AND CONCLUSIONS

A two-story, reinforced concrete flat-plate frame with edge beams was designed and detailed according to current building code provisions for regions of low to moderate seismic risk. A one-third scale model of the frame was subjected to simulated earthquakes of various intensities. Based on data and discussions presented in the main body of this paper, the following are concluded:

(1) Gravity load is likely to control proportions of low-rise flat-plate frames. As a consequence, seismic forces may be significantly in excess of those indicated by current code provisions.

(2) Load-deflection behavior of the complete flat-plate frame to failure could be interpreted with reasonable accuracy using measured behavior of isolated plate-column subassemblies.

(3) Under lateral loads, the flat plate is relatively more flexible than a conventional beam-column structure, but sufficiently stiff for low-rise structures during low to moderate seismic excitations.

(4) The test structure possessed sufficient strength and stiffness

to stand alone during moderate-intensity shaking, and sufficient ductility to act in parallel with a more rigid structural system for strong base motions. It must be qualified that this conclusion may not be general, and structures with higher nominal slab shear stresses, less-continuous reinforcement details, less-regular framing system, and multi-directional base excitation may not fare so well. Damage in the edge beams indicated that collapse would have been likely during moderate shaking had the edge beam (or other form of "shear" reinforcement) been eliminated at the edge of the plate.

REFERENCES

(1) "Building Code Requirements for Reinforced Concrete (ACI 318-77)," American Concrete Institute, Detroit, 1977.

(2) "Uniform Building Code," International Conference of Building Officials, Whittier, California, 1982.

(3) ACI Committee 318, "Proposed Revisions to Building Code Requirements for Reinforced Concrete (ACI 318-77) and Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-77)," Concrete International V. 4, No. 12, December 1982, pp. 38-127.

(4) Pecknold, D. A., "Slab Effective Width for Equivalent Frame Analysis," ACI Journal, April, 1975, pp. 135-137.

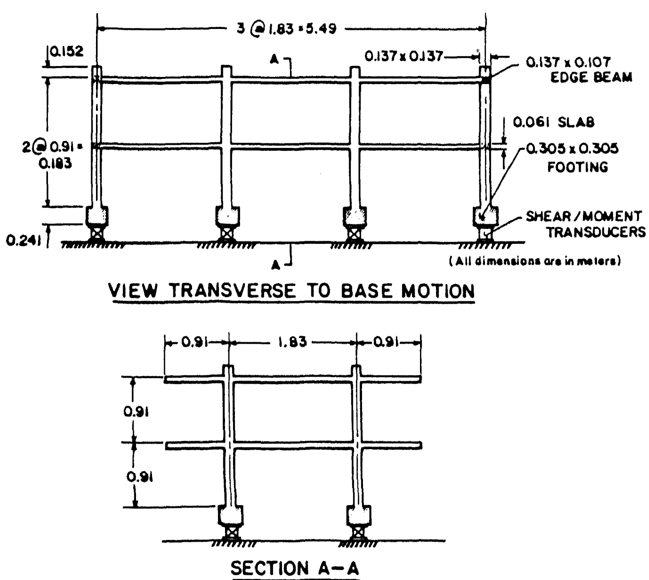
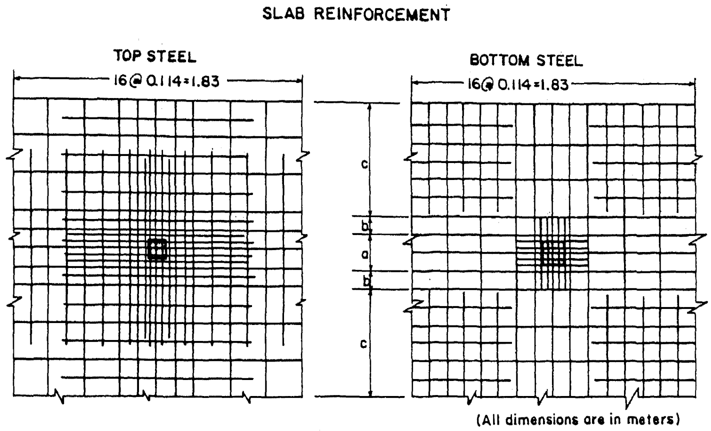


Fig. 1. Test Structure Configuration.



Mark	Width	TOP STEEL REINFORCEMENT			BOTTOM STEEL REINFORCEMENT		
		Spacing	Bar Lengths	d_b	Spacing	Bar Lengths	d_b
a	0.229m	0.038	2 @ 1.12 2 @ 1.22 2 @ 3.632	.0045	0.038	4 @ 0.457 3 @ 3.632	.0045
b	0.114m	0.057	1 @ 1.22	.0045	NO BARS IN THIS SPACE		
c	0.686m	0.114	3 @ 1.22 4 @ 3.63	.0045	0.114	3 @ 1.372 4 @ 3.632	.0045

Fig. 2. Interior Slab Reinforcement Details.

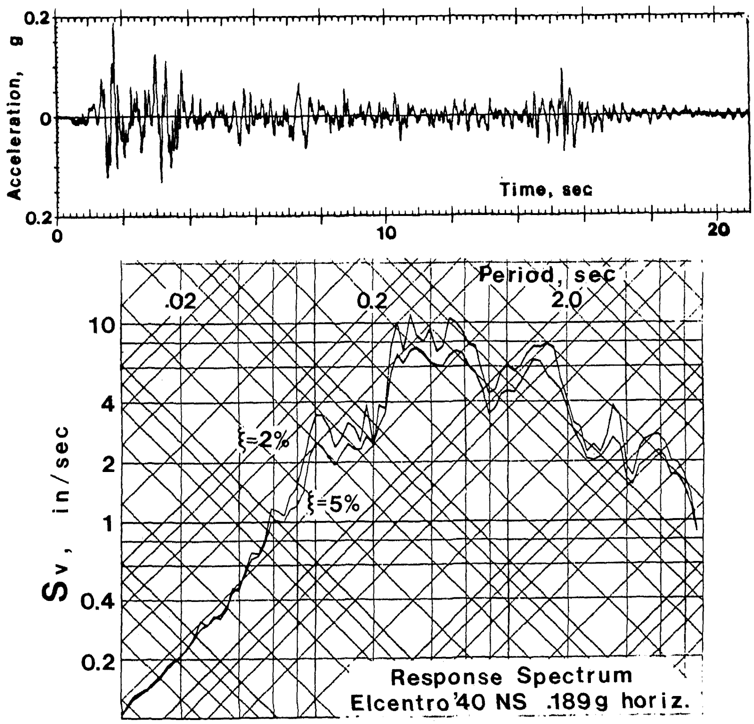


Fig. 3. Base Acceleration and Response Spectrum for "Design" Test.

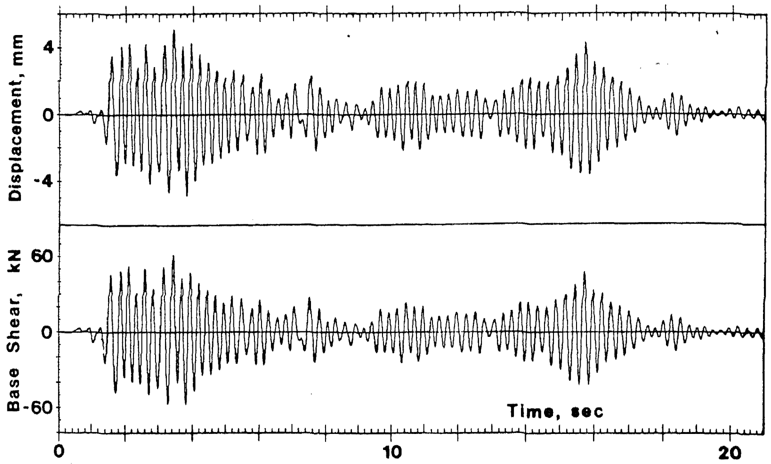


Fig. 4. Top-Floor Displacement and Base-Shear Records for "Design" Test.

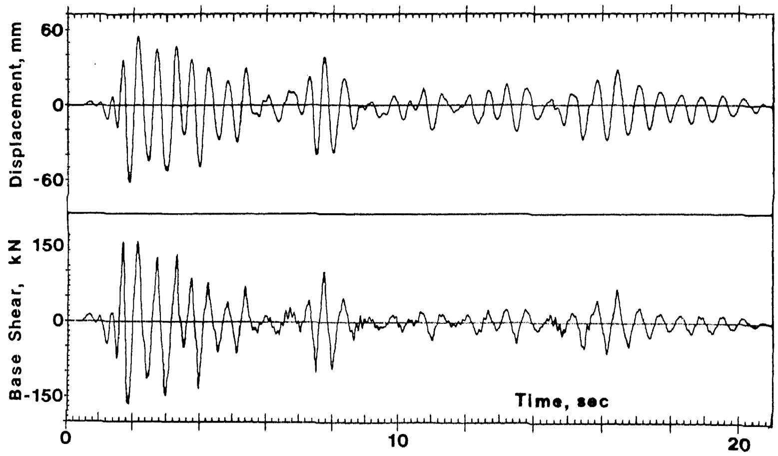


Fig. 5. Top-Floor Displacement and Base-Shear Records for 0.6g Test.

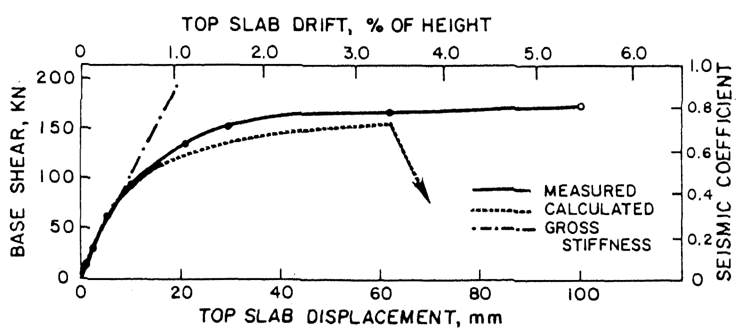


Fig. 6. Envelope of Top-Floor Displacement and Base Shear.