EARTHQUAKE RESPONSE OF A FOUR-STORY RC BUILDING

D. A. Foutch (I)
M. E. Barenberg (II)
Presenting Author: D. A. Foutch

SUMMARY

The Freitas Building, a four-story RC shear wall building, was shaken by earthquake motions that produced forces in the structure that greatly exceeded those prescribed in the building code. However, the building experienced almost no damage. To determine why the building performed so well, the observed behavior of the building which included nine accelerograms was compared to the computed behavior of several elastic and inelastic models subjected to static and dynamic loads. Some of the models included foundation flexibility. The calculated results compared favorably with the observed behavior and indicated that the building was actually designed to resist forces greater than those in the building code.

INTRODUCTION

The art and science of the aseismic design of buildings have been greatly advanced over the last 25 years, in part, because of lessons learned from actual earthquakes. Most of these lessons have been learned from buildings that have been heavily damaged because buildings that sustained little damage have not received much study. Thus, some important lessons may have been lost.

The purpose of this study was to investigate a building that performed very well during the Santa Barbara Earthquake of August 13, 1978. The earthquake had a Richter magnitude of 5.1-5.7 and an MM intensity of about VII in downtown Santa Barbara (Ref. 1). Although this was not a great earthquake in seismological terms, it did generate peak ground accelerations of about 0.45 g and damage in excess of $7 million.

The building chosen for this study is the Freitas Building which is located in downtown Santa Barbara about 6 km from the epicenter of the earthquake. This building was chosen for several reasons. Prior to the earthquake, nine accelerometers were placed in the building by The Office of Strong Motion Studies, California Division of Mines and Geology (Ref. 2). In addition, they also provided a complete set of structural drawings for the building. Thus, much data was available on the Freitas Building. Also, even though recorded ground accelerations in excess of 0.20 g and roof accelerations in excess of 0.50 g indicated that the structure was subjected to much larger loads than the code level forces, almost no damage was observed in the building. Thus, the Freitas Building seemed to be an ideal subject for study.

(I) Assoc. Prof., Civil Engineering, University of Illinois at Urbana-Champaign, Illinois, USA

(II) Research Assistant, Civil Engineering, University of Illinois at Urbana-Champaign, Illinois, USA

Building Description

The Freitas Building is a four-story reinforced concrete and steel building that measures 109 ft. 5 in. by 137 ft. 6 in. in plan and stands 53 ft. above grade. A basement was provided under the western portion of the building and an open walkway was provided at the first story level at the east end of the building. Typical plan and section views are shown in Fig. 1. Lateral loads are resisted by four full-height reinforced concrete shear walls located at the perimeter on each side of the building. Vertical loads are carried by a composite (metal deck with concrete slab) floor system supported on simple-framed steel beams and columns. Spread footings are provided under each column and shear wall and bell caissons are also provided at the ends of each shear wall.

The locations of the accelerometers are shown in Fig. 1. These locations were nearly ideal for this study since torsional motion, if it was present, could probably be determined by comparing the measured responses at the centers and edges of the second floor and the roof. Also, by placing instruments on the second floor as opposed to the third floor, the possibility of detecting damage to the first story, which carried the largest shear and overturning moment, was enhanced.

Preliminary Investigation

The preliminary investigation was directed at determining whether torsional motions were excited in the building. Fourier spectra, cross-spectra and phase spectra were computed for the absolute and relative accelerations measured by instruments 6, 7, 1 and 5. No torsional response was detected. To determine if this was reasonable, a detailed elastic three-dimensional (two translation and one rotation per floor) frame analysis using the ADINA program (Ref. 3) was performed. The frequencies and mode shapes were calculated and a step-by-step integration of the equations of motion was done using the recorded base accelerations as input. Again, the rotations of the floors were extremely small compared to the translations. This was obviously because the centers of mass and stiffness at each floor nearly coincided. Thus, it was concluded that one of the shear walls could be isolated and studied independently to evaluate its performance.

Investigation of the North Shear Wall

The shear wall at the north edge of the building was chosen for detailed investigation because it resisted earthquake forces in the E-W direction which were considerably higher than the forces in the N-S direction. Also, the north wall was isolated from all of the other walls, unlike the south wall which was coupled to the west wall.

Elevation and section views of the north wall and foundation system are shown in Fig. 2. The specified minimum concrete strength was 3000 psi and the assumed strength for this study was 3500 psi. The results discussed below were not significantly dependent on the choice of concrete strength. The rolled shapes placed in the wall at either end were of A36 steel and the reinforcing bars were grade 60 steel.

The first step was to determine if the actual earthquake forces that the north wall experienced were significantly higher than the design forces. The recorded accelerations on the basement, second floor and roof are shown in Fig. 3. The peak acceleration at the base was 0.23 g and at the roof, 0.55 g, although these occurred at different times. The mass associated with each floor was determined from the structural and architectural drawings.

The accelerations at the third and fourth floors were estimated by assuming a straight line distribution between the measured absolute accelerations at the second floor and roof. The earthquake forces at each floor were then estimated by multiplying the accelerations times the respective mass at each floor. From these calculations the total base shear and overturning moment could be estimated at any instant of time. The peak base shear occurred at 2.38 sec. and was about 0.33 W, where W is the total weight of the building. Using the 1967 edition of the SEAOC code (Ref. 4) the design base shear would be 0.063 W for a building period of 0.5 sec. (measured) and 0.068 W for a period of 0.4 sec. Even with a load factor of 1.4 applied, these the values would only be 0.088 W and 0.095 W respectively. This clearly indicates that the building experienced earthquake forces which greatly exceeded the design forces. In spite of these large forces, the building experienced no observable nonstructural damage and only "very few diagonal cracks in the shear walls" (Ref. 1). This remarkable behavior was assumed to be attributable to one, or a combination, of three reasons: (1) the actual strength of the wall was much greater than the design-level forces would require; (2) the nonstructural walls carried a significant share of the earthquake forces without being damaged; and (3) rocking of the wall at the foundation level caused the damage to be less than might be expected. Several approaches were used to try to differentiate between these effects.

The base shear and overturning moment for the north wall were computed from the measured accelerations and estimated story masses as described above. These were found to be 955 kips and 34,500 ft-kips, respectively, at a time of 2.38 sec. This was also the time at which the maximum relative roof and second floor displacements occurred.

The average nominal shear stress for incipient cracking in the first story would be about $2\sqrt{f'_c}$ which would correspond to a shear force of 440 kips. If we include the effects of vertical load in the wall the cracking shear might increase to 470 kips (Ref. 5). The ultimate shear capacity and yield moment were estimated to be 1000 kips and 30,600 ft-kips, respectively. Since only a very small amount of diagonal cracking occurred during the earthquake, it might be reasonable to assume that the shear carried by the wall was in the range of 500-1000 kips. Thus, from the above discussion, it is clear that the capacity of the wall greatly exceeded the requirements for the assumed design forces. In addition, if one simply compares the estimated forces from the earthquake to the calculated strength of the wall, the amount of damage observed was probably consistent with these values.

The dynamic properties of the building were also investigated by studying the measured data and comparing the results with calculated properties from various simple models. To get the relative motion of the second floor and roof with respect to the base, the acceleration, velocity and displacement time histories of the base were subtracted from the roof and second floor records. Analysis of the relative motion data revealed that the fundamental frequency decreased from about 2.0 Hz prior to the onset of the strongest shaking to a minimum of 1.65 Hz during the strongest shaking. The maximum relative displacement for the roof was 1.6 in., and the ratio of the relative roof displacement to the relative second floor displacement during the strong shaking in the fundamental mode was 0.28.

A plane stress finite element model of the wall was constructed, and this was converted to a two-column "shear frame" model which yielded equal floor displacements as the finite element model under inverse triangular loading. The fundamental natural frequency of this model was slightly in excess of 2.0 Hz. The stiffness of the wall was reduced in the lower two stories where M/M_{cr} exceeded 1 under the maximum earthquake forces. This reduced the frequency to 1.85 Hz which was still above 1.65 Hz. Further reduction in frequency would result from a further reduction in stiffness of the wall or from including foundation compliance into the model.

Translational and rotational springs were added to the base of the wall and adjusted until the natural frequency was 1.65 Hz and the mode shape matched the measured one. The recorded base motion was used as input to the model and a step-by-step integration of the equations of motion was performed. This was done several times with different levels of damping until the maximum relative roof displacement of 1.6 in. was obtained. The values of EI for the columns of the fixed base model were then reduced until the natural frequency was 1.65 Hz and the analysis repeated. The relative displacement time histories for the roof computed from the recorded data and from the calculated responses of the two models are shown in Fig. 4 and the mode shapes are shown in Fig. 5. The fixed-base model required 15% damping and the flexible-base model required 20% damping to match the peak displacement. Both values seem very high considering the small amount of damage present. Both models matched the recorded roof response very well, but the mode shape for the flexible-base model was a much better match for the building's actual mode shape in the lower stories.

The final step in the investigation was to use a more realistic, inelastic, model to determine if the load-deflection characteristics of the wall were compatible with the previous results. The wall was modeled as a rectangular reinforced concrete beam. A moment-curvature relation was determined for a representative section in each story and for sections where a large change in reinforcement ratio occurred. Each $M-\phi$ curve included the effects of dead load carried by the wall. Horizontal loads were applied proportional to the mass times the amplitude of the fundamental mode shape at the floors. For a given loading, the curvatures were calculated and integrated twice to obtain the deflections. By progressively increasing the load, a base shear vs. deflection curve was calculated for the wall. This curve was modified to include the flexible support conditions and, again, to include the effects of shear deformation. These curves are shown in Fig. 6. The base shear calculated for initial flexural yielding was about 820 kips. The base shear for the fixed-base models for a top story deflection of 1.6 in. with and without shear deformation were 960 kips and 910 kips, respectively, but was only 800 kips for the flexible-base model. The deflected shapes of the wall models when the roof displacement was 1.6 in. are shown in Fig. 7 along with the recorded maximum relative displacements. By including shear deformation in the fixed-base model the deflected shape more closely matches the mode shape. The bottom stories would be even softer than indicated since cracking was not considered in the calculation of the shear deformation.

The overall strength and stiffness of the wall would be higher than those predicted by all of the above calculations due to strain rate effects, and the steel frame and nonstructural walls would be expected to carry at least a small amount of the shear and overturning moment. Also, the actual yield stresses of the steel would be higher than the nominal values. Considering these three points along with the above analysis indicates that the overturning moment actually carried by the wall probably did not cause yielding. Even if the yield deformations were slightly exceeded at the base, the damage expected would still be consistent with that observed.

Concluding Remarks

Given the unpredictable nature of reinforced concrete, all of the above calculations, particularly those relating load to deflection, must be viewed with a great deal of skepticism. This being the case, one must conclude that none of the above models may be identified as the most correct representation of the real wall. This highlights the difficulty of trying to conclusively explain observed building responses when insufficient data is available. Even though the results above may not identify the behavior of the wall in exact detail, it seems clear that the major reason for the seemingly good performance of the building during the earthquake was that the wall design provided much greater strength than the building code required.

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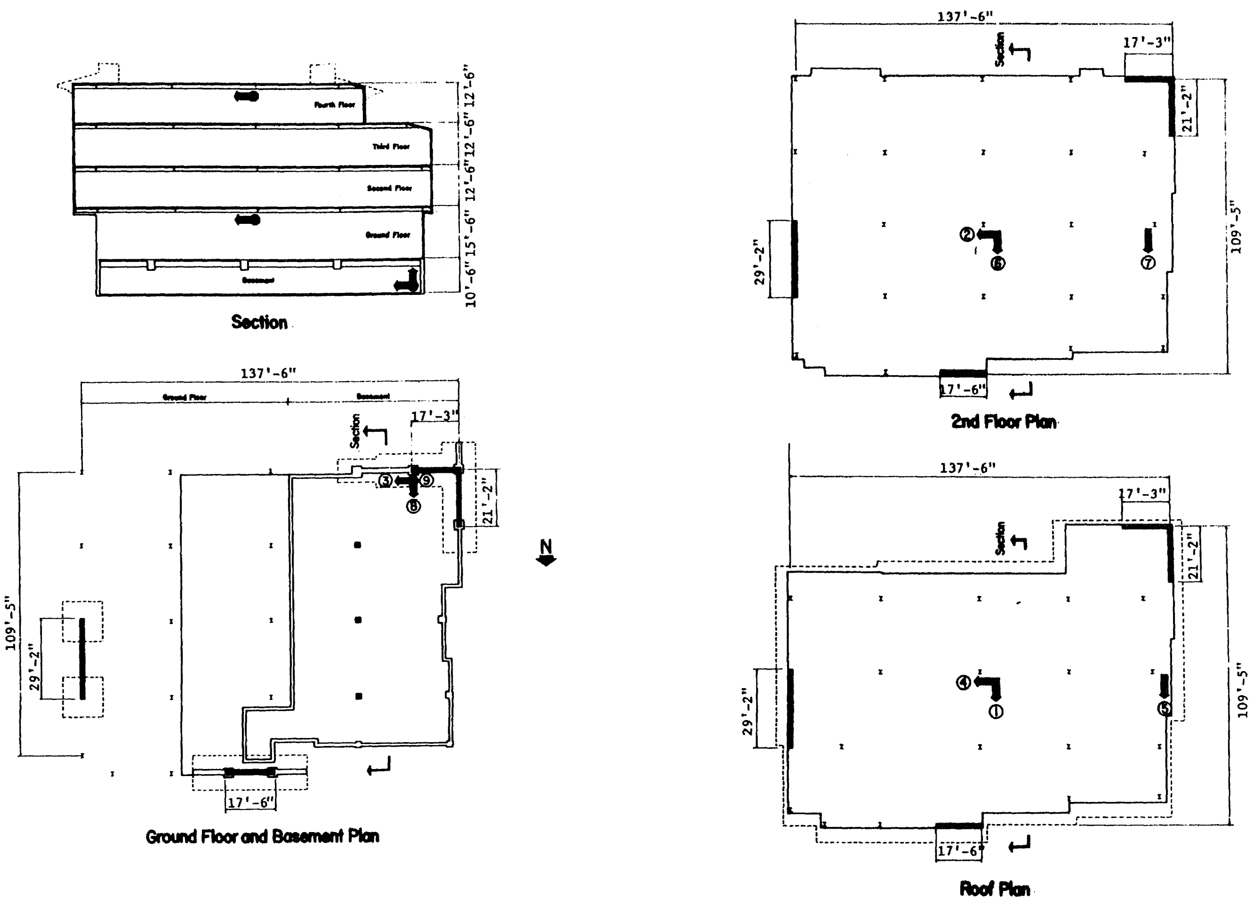

Fig. 1 Plan and Section Views of the Freitas Building (from Ref. 2)

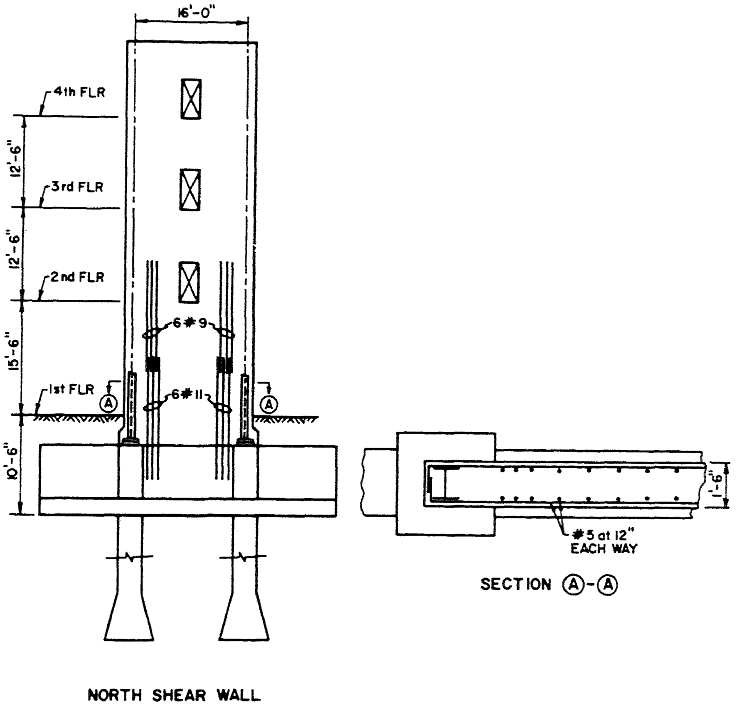

Fig. 2

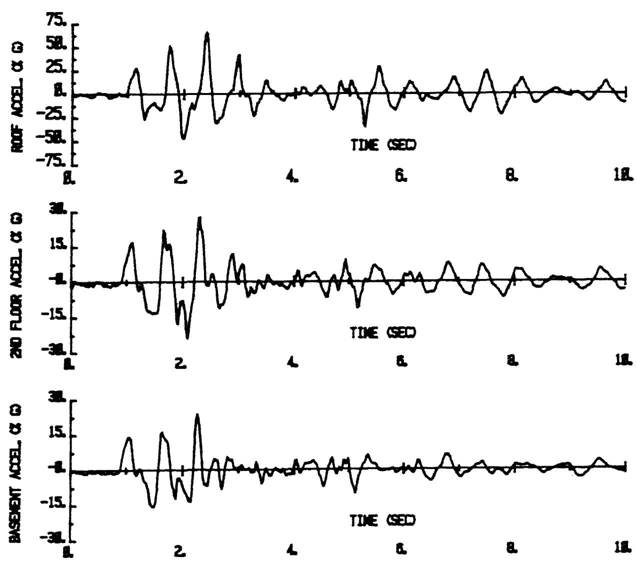

Fig. 3 Recorded Accelograms

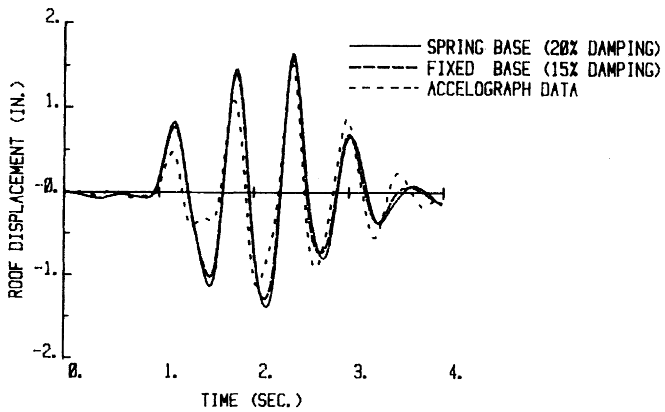

Fig. 4 Relative Roof Displacements

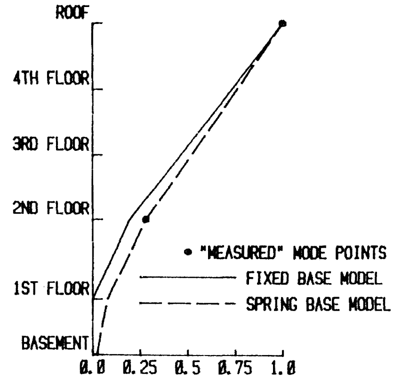

Fig. 5 Fundemental Mode Shapes

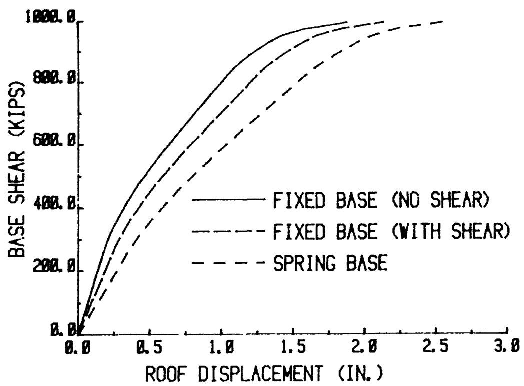

Fig. 6 Load vs. Displacement for Roof

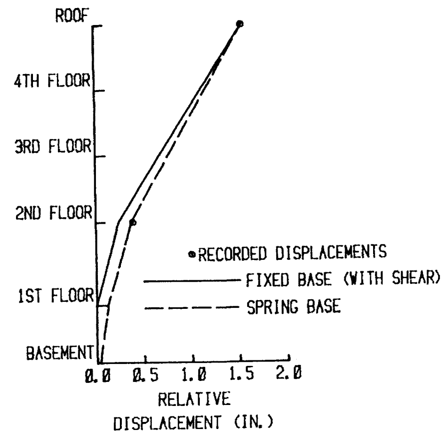

Fig. 7