EVALUATION OF THE RESPONSE OF THE IMPERIAL COUNTY SERVICES BUILDING

M. E. Kreger (I)
Presenting Author: M. E. Kreger

SUMMARY

This paper describes the Imperial County Services Building in El Centro, California, and the damage sustained by the structural system during the 15 October 1979 Imperial Valley earthquake. Recorded acceleration response of the structure and calculated base-shear and moment responses are presented and interpreted. The procedure used for determining how and when columns failed at ground level is described.

INTRODUCTION

The Imperial County Services Building of El Centro, California (Fig. 1) was severely damaged during the 15 October 1979 Imperial Valley earthquake. The most significant damage sustained by the structure during the earthquake was the material failure of four east-end columns at ground level (Fig. 2). The event was unique because response of the structure as columns failed (during the earthquake) was recorded by a network of 13 strong-motion accelerographs.

Observed damage in the structure and recorded acceleration data were evaluated with the aid of calculated base-shear and base-moment response. Fourier amplitude spectra, base-motion response spectra, and calculated base-shear strengths. Additional understanding of the nonlinear dynamic behavior of a structural system used in the Imperial County Services Building (ICSB) was gained through simulated earthquake tests conducted on a small-scale reinforced concrete structure (geometric scale of approximately 1/12). Details of the small-scale test structure and results of the investigation are contained in Ref. 1.

This paper describes the structural systems used in the ICSB, presents recorded acceleration response data and calculated base-shear and base-moment responses, and describes the analyses used to understand how and when four first-story columns at the east end of the ICSB failed.

DESCRIPTION OF THE IMPERIAL COUNTY SERVICES BUILDING

The ICSB (Fig. 1) was a six-story, reinforced concrete building, 136 ft 10 in. (41.71 m) by 85 ft 4 in. (26.01 m) in plan. The perspective drawing of the structure illustrates the open nature of the first story. Nonstructural elements were omitted from the first story of Fig. 1 to make the location and number of structural walls evident.

Floor plans of the structure are shown in Fig. 3. The structure was five bays long in the east-west (longitudinal) direction and three bays

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I) Assistant Professor of Civil Engineering, The University of Texas at Austin, Texas, U.S.A.

long in the north-south (transverse) direction; regular bay lengths of 25 ft (7.62 m) were used. Story heights were not regular over the height of the structure. Story heights measured 14 ft 6 in. (4.42 m) for the first story, 13 ft 6 in. (4.11 m) for the second through fifth stories, and 13 ft 2 in. (4.01 m) for the sixth story. Story heights were measured from top-of-slab to top-of-slab. The distance from the top of the ground slab to the top of a typical pile cap was 2 ft 2 in. (0.66 m).

The structure was designed in conformance with the 1967 version of the Uniform Building Code. Typical floors were designed for live loads of 80 psf (3800 Pa). The roof was designed for a live load of 20 psf (960 Pa). Maximum design soil bearing pressures were 1000 psf (48000 Pa) for dead load only, and 1200 psf (57000 Pa) for dead and live load combined. Nominal 28-day concrete strengths were 4000 psi (27.6 MPa) for beams, walls, joists and slabs above grade; 5000 psi (34.5 PMPa) for columns and prestressed piles; 3000 psi (20.7 MPa) for pile caps, grade beams, and Raymond step-taper piles. Grade 40 steel was used throughout the structure.

Vertical-Load-Carrying System

Gravity loads imparted to the structure through slab-joist systems were transferred to the foundation through four frames oriented in the east-west direction. Slab-joist systems spanned in the north-south direction between frames. Two types of frames, designated as interior and exterior frames were used in the structure. First story columns of both frame types were anchored in pile caps which were cast over groups of six, nine, or twelve piles that penetrated 40 to 45 ft (12.2 to 13.7 m) below the base of respective pile caps. Adjacent pile caps were linked together by tie beams.

Lateral-Force-Resisting Systems

Two different lateral-force-resisting systems were used in the two principal directions of the structure. Moment-resisting frames were used in the longitudinal (east-west) direction and a complex wall system was used in the transverse (north-south) direction.

Interior and exterior frames were treated as ductile moment-resisting and moment-resisting frames, respectively. Because the engineer elected to use only the two interior frames for lateral force resistance, the design base shear coefficient was determined to be 0.04.

The complex of walls used to resist lateral forces in the transverse (N-S) direction comprised four first story walls located in interior bays along lines B, D, E and F (Fig. 3) and two walls rising above the first story at the east and west ends of the structure (lines A and H). Figure 3 indicates that east and west end walls were located 30 ft 11 in. (9.42 m) and 5 ft 11 in. (1.80 m) from the nearest first story walls. The resulting wall configuration contained a discontinuity at the first level in the form of a stagger in the plane of vertical stiffness. A thicker slab (5 in. instead of the typical 3 in. slab) was provided at the first level to act apparently as a diaphragm for transferring forces from end walls to first story walls. The design base shear coefficient in the transverse (N-S) direction was 0.085.

RESPONSE OF THE STRUCTURE

Structural response is presented here in the form of selected acceleration response histories and calculated base-shear and base-moment response waveforms. Structural behavior inferred from response waveforms and Fourier amplitude spectra is described here.

Acceleration waveforms for north-south response recorded at the east end of the structure and east-west response recorded at the center of the structure are presented in Fig. 4. North-south base-shear and base-moment response and east-west base-shear and base-moment response were estimated using measured acceleration responses and their interprolations, mass at each floor level, and mass heights as input. Base-shear and moment responses for the two principal directions of the structure are presented in Fig. 5. The base of the structure was considered to be at ground level.

Maximum top-level and base accelerations were 0.58 and 0.29 g in the transverse (N-S) direction and 0.45 and 0.33 g in the longitudinal (E-W) direction. The maximum base-shear coefficients were estimated to be 0.31 and 0.24 in the N-S and E-W directions which corresponded with 3.6 and 6.0 times the design base-shear coefficients for the two principal directions. Initial frequencies (Ref. 2) and effective frequencies following the interval of strong shaking were 2.24 and 1.65 Hz for the N-S direction, and 1.55 and 0.65 Hz for the E-W direction. Changes in response frequencies corresponded with approximately 50 and 80 percent reductions in stiffness in the N-S and E-W directions.

Response in the two principal directions were distinctly different, as demonstrated by the differences in top level acceleration responses and base-shear and moment responses. Initial and final frequencies differed as noted above. High amplitude response in the N-S direction subsided at approximately 10.5 sec. but continued in the E-W direction up to 14 sec.

Higher modes of response were apparent in both directions of the structure. Apparent second mode response was observed in the E-W direction up to approximately 6.8 sec. Response then shifted to primarily a fundamental mode with some contributions by higher modes. It is interesting that vertical distributions of acceleration rarely resembled inverted triangular distributions used to proportion the E-W frames. North-south acceleration response also contained some higher-mode response during strong shaking.

EVALUATION OF RESPONSE

The north-south wall at the east end of the structure was discontinued at the first level to be continued to the foundation at the first interior column line located 30 ft 11 in. (9.50 m) away (Fig. 3). The stiffness discontinuity at the east end would result in overturning moment, due to N-S lateral forces, to be resisted almost entirely by axial forces in east end columns. The small scale test was conducted to investigate the nonlinear dynamic response of the complex wall system. Results of the investigation were incorporated in the evaluation procedure used to determine how and when column failures occurred. Highlights of the evaluation process are presented here.

Consideration of north-south and east-west forces acting independently on the structure indicated that column failures were not likely to have occurred due to forces acting solely in one direction. It was appropriate to consider the forces resulting in east end columns due to N-S and E-W loads acting simultaneously on the structure.

Pairs of forces deemed most significant (E-W moment and axial load) were determined for east end columns from E-W base-shear and N-S base-moment response for the east half of the structure. A method for selecting instances when forces acting on east end columns may have been critical was needed because no direct and conclusive evidence of failure was offered by measured acceleration response histories. North-south response waveforms did show a sudden decrease in response frequency at 9.2 sec. followed by a reduction in response amplitudes after 10.1 sec., both of which are common indicators of significant change in stiffness and/or strength. However, response of a 1.55 Hz linear oscillator to the N-S base acceleration record simulated this behavior (Fig. 6) indicating the sudden change in response frequency and amplitude was probably caused by the base motion. A set of 17 pairs of forces was selected for evaluation based on the relative magnitude of an index (the combined loading index) that consisted of the absolute sum of normalized E-W base-shear and N-S base-moment response (Fig. 7).

Relative potential of selected pairs of forces to cause failure was evaluated by (1) the combined loading index, and (2) the horizontal position of each force pair in relation to an interaction diagram plot (Fig. 8). The set of forces which demonstrated the greatest potential for the crushing type of failure observed in the structure corresponded with a time of approximately 10.1 sec. (point 15 in Fig. 8). Failure of one corner column was concluded to have occurred at that time. Correspondence of the N-S base-moment response and response of the 1.55 Hz linear oscillator indicated that failure of another corner column was not likely to have occurred after 10.1 sec. (Fig. 6). Failure of the other corner column was then concluded to have occurred at 9.6 sec. because column forces at that time demonstrated the next highest potential for failure and the forces occurred within the same cycle of strong N-S building response.

Interior east end columns are believed to have failed at approximately 11.2 sec. because of the increased vertical load due to failure of corner columns and the coincidence of the following: high E-W base-shear response, an observed spike at 11.27 sec. in N-S east end acceleration records (Fig. 4), and the end of correspondence of N-S base-moment response and the response of the 1.55 Hz oscillator at 11.2 sec.

CONCLUDING REMARKS

How and when the structure failed is of importance not only for explaining why the building sustained such severe damage, but more importantly for demonstrating that the behavior of a complex structural system in an earthquake cannot necessarily be perceived using design methods within the scope of an accepted code of practice. It is unlikely that the behavior of the east end columns could have been foreseen based on design base-shear coefficients of 0.04 and 0.085 (E-W and N-S directions). Although a design based on these coefficients would very likely have produced a satisfactory structure for an ordinary system (west end columns did not fail), the use of these coefficients in the design of a system which is considerably different from those considered in the development of the design methods is likely to result in structural problems as demonstrated in this study.

ACKNOWLEDGMENT

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REFERENCES

1. Kreger, M. E., and M. A. Sozen, "A Study of the Causes of Column Failures in the Imperial County Services Building During the 15 October 1979 Imperial Valley Earthquake," Civil Engineering Studies, Structural Research Series No. 509, University of Illinois, Urbana, August 1983.

2. Pardoen, G. C., "Imperial County Services Building, Ambient Vibration Test Results," Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand, 79-14, December 1979.

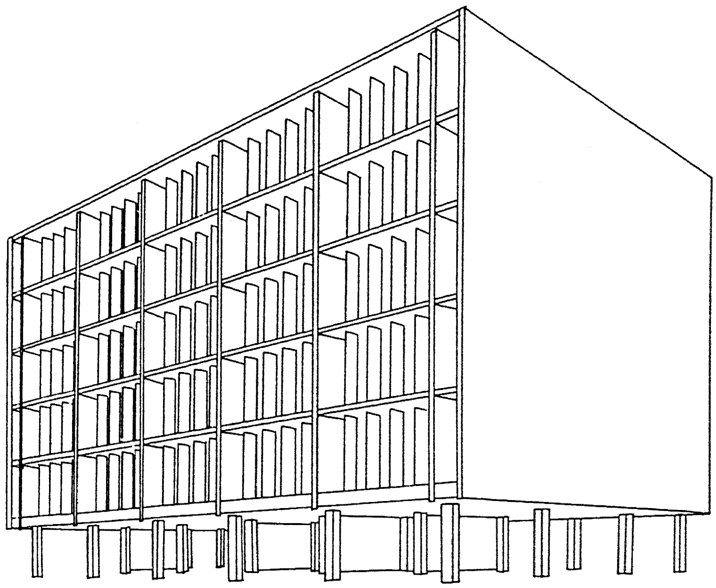


Fig. 1 Perspective drawing of the Imperial County Services Building

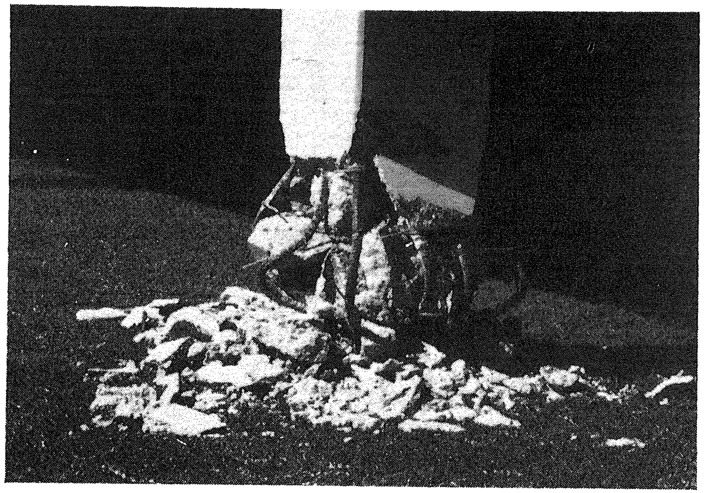


Fig. 2 Photograph of damaged east end column

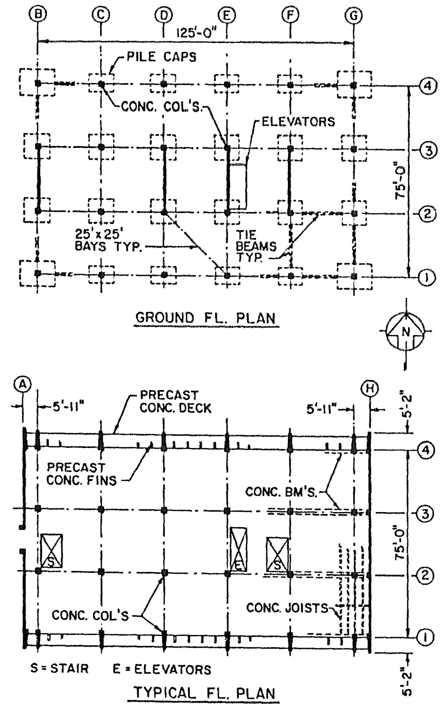
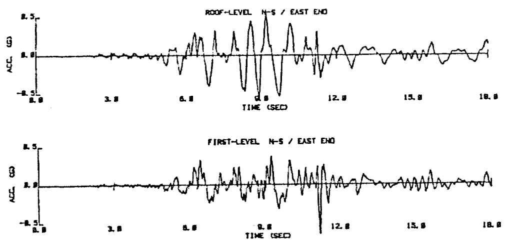
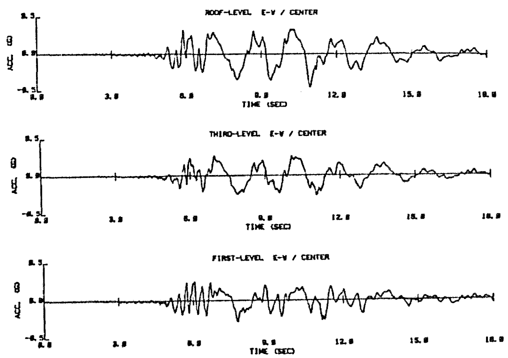


Fig. 3 Floor plans for the Imperial County Services Building

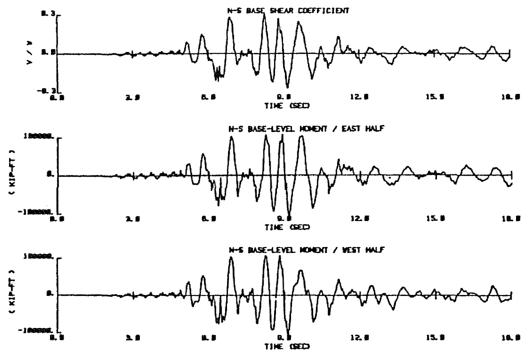


(a) N-S acceleration response

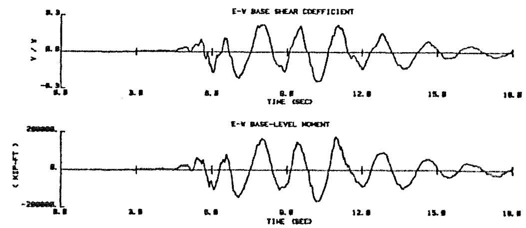


(b) E-W acceleration response

Fig. 4 Acceleration response histories for the Imperial County Services Building



(a) N-S base-shear and moment response



(b) E-W base-shear and moment response

Fig. 5 Base-shear and moment response histories for the Imperial County Services Building

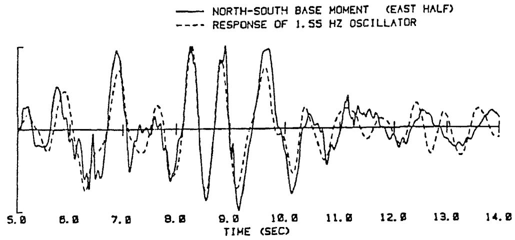


Fig. 6 Superposition of N-S base-moment response and response of a 1.55 Hz linear oscillator

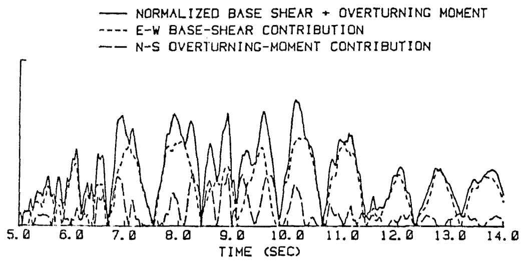


Fig. 7 Combined loading index indicating potentially critical combinations of N-S and E-W loads

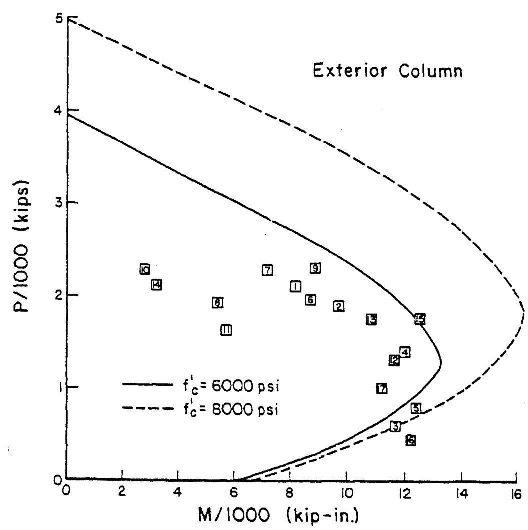


Fig. 8 Estimated column forces and load-moment interaction diagrams for east end columns