

EXPERIMENTAL AND ANALYTICAL STUDIES OF THE IMPERIAL COUNTY SERVICES BUILDING

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SUMMARY

In this paper ambient vibration tests, which were completed in the first half of 1979 and provided characteristic natural frequencies, mode shapes and structural damping, are summarized. Additionally the responses recorded in the building by strong motion instrumentation during the October 15, 1979 Imperial Valley M 6.4 earthquake are outlined and are correlated with the results of both elastic and inelastic mathematic model studies.

Although good agreement between the experimental and observed evidence on the one hand, and the computer derived responses on the other, are indicated in the case of elastic analyses, to achieve this in the inelastic range it was found necessary to select individual member strengths greater than those justified by standard design consideration. This lends weight to the contention that current member modeling techniques do not allow adequately for the ultimate strengths of structural components under dynamic loading.

INTRODUCTION

The Imperial County Services Building is of particular interest as the severe damage which it sustained in the October 15, 1979 Imperial Valley M 6.4 earthquake coincided with a comprehensive set of strong motion records being obtained from instruments mounted both in the building itself and on the ground close to it [1]. See Figure 1. Additionally, vibration testing undertaken prior to the earthquake had enabled the pre-shaking dynamic properties to be established reliably. In these circumstances post earthquake analyses were made on the basis of a much better data base than is normally possible.

The building was of reinforced concrete, six stories high and five bays long by three bays wide on a 25 ft. grid. See Figure 2. Each of the east and west ends consisted of an exterior shear wall extending from the second to the sixth level, leaving open ends in the first story. The end shear walls were outside the line of the exterior lower floor columns. Additionally a non-symmetrical layout of transverse shear walls existed in the central bay of the lower floor columns. These columns failed in varying degrees of severity, the ones on the eastern outer line shattering and shortening sufficiently to necessitate the subsequent demolition of the entire building.

AMBIENT VIBRATION TEST RESULTS

Ambient vibration tests were conducted on three separate days in 1979 by placing seismometers in strategic locations throughout the building in both the N-S and E-W directions [2]. With the exception of the torsion tests the seismometers were usually located at a common point in plan view but at different floors throughout the building. In all cases at least one seismometer remained at roof level. The torsion tests were conducted by placing the four seismometers at the roof level with each seismometer located in a particular corner of the building.

Apart from one ambient vibration test that was conducted by "tapping" into the accelerometers of the California Division of Mines and Geology, velocity data was primarily acquired and recorded for the ambient tests.

The principal objective of the ambient vibration tests was to determine estimates of fundamental frequencies and the associated mode shapes as well as an estimate of structural damping associated with low level excitation. With the implementation of the Fast Fourier Transform on a hardwired microcomputer such as the Zonic DMS 5003, the calculation of time and frequency domain functions related to these fundamental vibration characteristics was relatively straightforward.

Using the Fourier Transform procedure the vibration characteristics of the Imperial County Services Building were determined and are summarized in Table 1.

Direction	Frequency	Method	Damping (% Critical)	Mode Shape Amplitudes			
	Hz			Roof	6th	4th	2nd
E-W	1.54	PS	5.50 → 6.42	1.00	.62	.24	.15
		AC	4.38 → 5.54				
N-S	2.25	PS	11.96 → 17.32	1.00	.77	.60	.37
		AC	5.00 → 11.71				
Torsion	2.83	PS	8.24 → 9.15	1.00	.93	.67	.28
		AC	6.48 → 8.83				
PS = Power Spectrum			AC = Auto Correlation				

TABLE 1

The results indicate that it would be misleading to attribute a single damping value for a particular mode of vibration. In fact one purpose of computing the damping value by the power spectrum and auto correlation function techniques was to indicate the range of potential values and the variability according to the technique used. The discrepancy in damping values is particularly accentuated in the N-S configuration. Although the power spectrum curves displayed in Figure 3 suggest a single degree of freedom system of frequency near 2.20 Hz, the broadband response gives almost unrealistically high damping values when using the half power point method.

Although other ambient vibration tests were conducted the amplitudes at the other story levels are not assessed because, in the absence of a clearly defined input spectrum for each of the seismometers, it is not justified to put too much credence in the mode shape data obtained by ratioing the appropriate Fourier components. Specifically the mode shape amplitudes could be misleading unless one could be assured that all seismometer locations were subjected to a reasonable approximation of white noise or band limited white noise. The transfer function was of most help for determining the in-phase and out-of-phase relationships of the transducers rather than as a vehicle for computing the amplitude ratios.

EARTHQUAKE RESPONSE

This particular 6-story building had been instrumented by the California Division of Mines and Geology in accordance with its building instrumentation criteria for the California Strong Motion Instrumentation Program [3]. In all, the building was instrumented with 13 accelerometers as well as having a triaxial "free field" accelerograph located approximately 100 meters east of the building.

The 13-channel CRA-1 system in the building consisted of 9 FBA-1 single-axis force-balance accelerometers located at various locations throughout the

upper stories of the structure, one east-west oriented HS-0 horizontal starter at roof level, one FBA-3 triaxial force-balance accelerometer package, one FBA-1 accelerometer, one 13-channel central recording unit, and a VS-1 vertical starter at ground level. The FBA accelerometers had a natural frequency of approximately 50 Hz and were connected by low-voltage data cable to the central recording unit. The battery powered recording unit was triggered by horizontal or vertical motion .01 g, or more and recorded on 7-inch (178-mm) light-sensitive film. The system was designed to record acceleration with frequency components in the 0 to 50 Hz range and with maximum amplitudes of 1 g (980 cm/sec²). Real time was provided by a WWVB receiver and time-tick generator system; however, this recorder was not connected with the SMA-1 accelerometer located east of the building.

The FBA accelerometer location (Figure 2) was selected in order to provide information on overall building response as well as ground input motion. The primary purpose of the three north-south oriented accelerometers at the roof and second floor levels (accelerometers 1,2,3,7,8 and 9) was to obtain and isolate north-south translational, torsional, and in-plane floor bending response. In conjunction with the north-south oriented accelerometers at ground level (accelerometers 10 and 11), these accelerometers provided translational and torsional response, mode shape, and ground to second floor inter-story motion information. Similarly, the accelerometers at the ground floor, second floor, fourth floor, and roof levels in the more flexible east-west (frame) direction (accelerometers 4,5,6 and 13) provided east-west translational response, mode shape, and inter-story motion information. The two N-S oriented accelerometers at ground level (accelerometers 10 and 11) were intended to identify collectively the extent to which differential horizontal ground motion had occurred, and the vertical accelerometer at ground level (accelerometer 12) provided information on vertical motion at this location. There were no vertically oriented accelerometers above ground level.

By analyzing the appropriate traces of the strong motion records, one can obtain an approximate horizontal plane movement time history such as is depicted in Figure 4. This figure represents the second floor to ground relative displacement time history during the 6-12 second duration of the earthquake. The time history curve in Figure 4 was obtained by plotting the E-W motion (the difference of traces 6 and 13) and the N-S motion (the difference of traces 8 and 11) at each time interval of the digitized earthquake response records [4]. These figures are of particular interest when one considers the relative motion of the ground level to second floor and its effect on column failure.

Consider, for example, the effect of this interstory drift on the flexural behavior of the columns. Using standard structural engineering code based calculations it can be shown that the ductile frames experienced E-W deflections of approximately 7 times that of the code allowables and yet these columns performed without flexural collapse. Frame flexural failure of the 4 easternmost columns must be ruled out since a flexural failure due to the interstory drift would have caused most, if not all, columns to failure. The lack of apparent frame flexural failure suggests that there was sufficient frame ductility despite the fact that comparisons of the moment developed by such interstory drift with column interaction diagrams indicated that the forces were much greater than that needed to produce collapse.

ELASTIC ANALYSIS

The elastic analysis considered the ICSB as a three-dimensional shear wall and framed structure subjected to either N-S or E-W loading (never concurrently) for the following loading conditions:

- A) The 1967 Uniform Building Code's lateral forces for both the N-S and E-W directions.
- B) The N-S component of the 18 May 1940 Imperial Valley earthquake assumed as input for both the N-S and E-W directions.
- C) The N-S and E-W components of the 15 October 1979 Imperial Valley Earthquake recorded by the free field accelerograph located adjacent to the ICSB.

In defining the analytical model of the ICSB for the ETABS [5] code, the complete building was assumed to be composed of structural elements which could be separated into a series of rectangular frames of arbitrary plan. Isolated shear walls, such as those which extend from the ground level up to the second floor level of the ICSB, were considered to be frames consisting of a continuous column line (having the associated shear wall properties) and a dummy column line in order to define the principal axis of the shear wall. Each frame of the ICSB was treated as an independent substructure in which the complete structure stiffness matrix was then formed under the assumption that all frames connected at each floor level by a diaphragm which is rigid on its own plane. The analytical model developed for the ETABS computer code was the one which most nearly represented the experimentally derived frequencies. Specifically the fundamental frequencies and mode shapes derived experimentally and those predicted by the analytical model depicted in Figure 5 after "fine tuning" are compared.

Direction Frequency	ANALYTICAL		EXPERIMENTAL	
	N-S 2.27 Hz	E-W 1.49 Hz	N-S 2.25 Hz	E-W 1.54 Hz
Mode Shape				
Amplitudes				
Roof	1.00	1.00	1.00	1.00
6th	.94	.95	.77	.62
4th	.79	.70	.60	.24
2nd	.58	.28	.37	.15

TABLE 2

The elastic analyses' results provided a general insight into the building's macroscopic response due to the various load conditions as well as denoting those structural components which can be expected to experience inelastic behavior due to severe seismic loading. In general the internal force resultants of the beams, columns, and shear walls are of primary interest with the deformation quantities being of secondary interest. No attempt was made to tabularize the force resultants for all structural members arising from all load conditions, but attention was devoted to the significantly loaded members and, in view of the catastrophic failure of the 4 columns along column line G (see Figure 5), particularly to the ground level columns.

Detailed numerical results have been presented in Reference 6 for seven representative load conditions in which a numerical as well as commentary summary are provided. While it is recognized that the strong motion of the 1979 Imperial Valley earthquake precludes entirely elastic behavior of the ICSB, it is evident that elastic analyses play a useful role in post-earthquake examinations by clarifying structural action. In particular the elastic analyses clearly pin-pointed the east end column weaknesses which are subsequently confirmed by the damage sustained.

INELASTIC ANALYSIS

Inelastic time-history analyses were undertaken [7] with a yield surface which takes into account the combined effects of biaxial bending and axial load specified to represent the failed columns. The yielding elements were characterized by a bi-linear moment-curvature relationship with zero post-elastic stiffness. The frequency and location of post-elastic excursions were noted as were the reactions obtained in other ground floor columns.

Detailed analyses were undertaken for one exterior and one interior damaged column. Particular attention was given to the response between 6 sec. and 10 sec. after the earthquake commenced as no yielding occurred in the first 6 sec. The axial loads, X moments and Y moments, together with the frequency and duration of the yield excursions were determined.

The corner column model exhibited yielding, associated with tensile load, at 6.8 sec. Neither transverse nor longitudinal frame action appeared to be responsible exclusively for the yielding. The maximum displacement ductility demands were found to be of the order of 2.4 as is shown in Figure 6.

Analyses of the interior frame column post-elastic behavior indicated significantly more yield excursions corresponding to the high moment associated with longitudinal frame movement. The combination of this motion and the somewhat smaller transverse movement produced axial failure of the members with little contribution from axial loads. The smaller amount of reinforcing steel, compared with the corner column, allowed a lower moment capacity in the interior one which in turn correlates with more frequent yield excursions. The maximum displacement-ductility demands for this member were established to be approximately 5.6 in both north-south and east-west directions of motion.

Notwithstanding the modeling of the other ground floor columns to remain elastic, the magnitude of the moments determined under longitudinal frame actions suggested that each of these members would have yielded at some stage of the applied movements if more realistic representation had been used. This correlates satisfactorily with the post-earthquake reports of cracking and spalling at the base of the ground floor columns.

CONCLUSIONS

The inelastic analyses enabled the relative importance of the various individual force actions to be evaluated. The large shear wall above the second floor at the east end of the building was subject to overturning with consequence high axial loads being applied to the corner columns. The uplift effect was shown to be particularly severe as it corresponded to reduced member moment capacities which prompted the longitudinal or transverse moments to yield the columns. These analyses confirmed the opinion that, although qualitative predictions of earthquake response may be made with a reasonable degree of confidence using present modeling techniques, these methods do not regularly provide accurate quantitative assessments. These computed responses of the ICSB, which is destined to become one of the most thoroughly investigated structures in earthquake engineering history, help explain the catastrophic failure of the building during the 15 October 1979 earthquake.

Reference

1. Earthquake Engineering Research Institute Reconnaissance Report, "Imperial County, California, Earthquake-October 15, 1979," Gregg E. Brandow, Coordinator, David J. Leeds, Editor, Feb. 1980
2. Pardoen, G.C., "Imperial County Services Building-Ambient Vibration Test

Results," Report 79-14, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand, December, 1979.

3. Rojahn, C., Ragsdale, J.T., "Strong Motion Records from the Imperial County Services Building, El Centro," EERI Reconnaissance Report, Feb. 1980, pp. 172-184.
4. Pardo, G.C., Hart, G.C., Bunce, B.T., "Earthquake and Ambient Response of El Centro County Services Building", ASCE/EMD Dynamic Response of Structures: Experimentation, Observation, Prediction, Control, 1981, pp 164-173.
5. Wilson, E.L., Hollings, J.P., Dovey, H.H., "Three Dimensional Analysis of Building Systems (Extended Version)," EERC-75-13, University of Berkeley, Berkeley, California, April 1975.
6. Pardo, G.C., Carr, A.J., Moss, P.J., "Imperial County Services Building-Elastic and Inelastic Analysis Results," Report 79-15, Dept. of Civil Engrg, University of Canterbury, Christchurch, New Zealand, Dec. 1979.
7. Shepherd, R., & Plunkett, A.W., "Damage Analyses of Imperial County Services Building", Proceedings, ASCE, Vol. 109, ST7, July 1983, pp 1711-1726.

Figure 1
Imperial County
Services Building

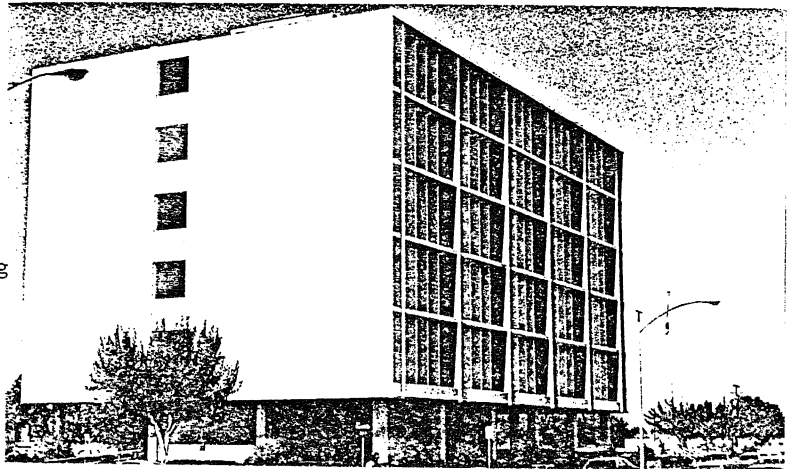


Figure 2
Strong Motion
Instrumentation

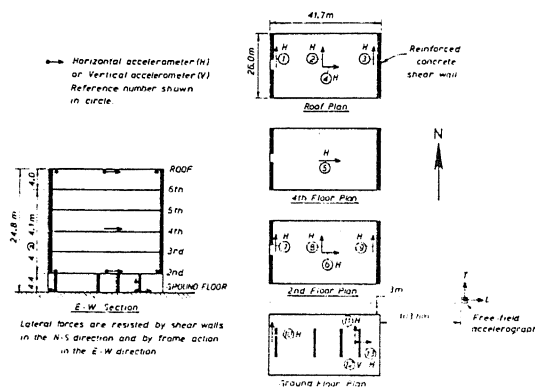


Figure 3
Typical Results
of Ambient Vibration
Tests

