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ANALYTICAL AND EXPERIMENTAL INVESTIGATION OF STRUCTURAL RESPONSE - THE IMPERIAL COUNTY SERVICES BUILDING

FINAL REPORT - PFR78-22863

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Results are presented of a study undertaken to explain the failure of the Imperial County Services Bulding in El Centro, California, during the October 1979 earthquake. Ambient and forced vibration tests were conducted and elastic and inelastic response analyses were performed; the results of these investigations appear in the appendices. An analytical finite element model was developed that correlates the building's response due to low level forced vibration. This model was extended to include the nonlinear effects of strong motion. Publications and presentations on the Imperial County Services Building's seismic response are listed.

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Analytical and Experimental Investigation of Structural Response - The Imperial County Services Building

Final Report - PFR78-22863

National Science Foundation

Gerard C. Pardoen

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ANALYTICAL AND EXPERIMENTAL INVESTIGATION OF STRUCTURAL RESPONSE

THE IMPERIAL COUNTY SERVICES BUILDING

Background

The success or failure of a structural design depends, to a great extent, upon the formulation of the appropriate conceptual and mathematical representation of the structure's characteristics. Inasmuch as the structure's analytical model dictates a significant part of the overall design, it is imperative, particularly in a high seismic area, that the model accurately represent the full-scale structure. One means of validating analytical procedures is to perform experimental studies of full-scale structures and compare these results with those of the analytical model.

This research effort was devoted to an in-depth experimental and analytical study of the Imperial County Services Building in El Centro, California. The Imperial County Services Building offered an unusual challenge due to its architectural features, its location in a highly seismic region, and the breadth of the strong motion instrumentation in the near vicinity. The investigation of this building entailed specific experimental and analytical tasks. The experimental tasks involved the ambient and forced vibration testing of the building and the subsequent data reduction effort whereas the analytical tasks involved an accurate modeling of the building for dynamic response.

An ambient vibration survey consists of measuring the microtremors in the structure caused by wind, people, cars, machinery, and other culturally induced excitations. Inasmuch as some of the most significant wind conditions in El Centro occur in February and March, the ambient vibration survey was conducted in February 1979. A subsequent ambient vibration survey was conducted in May 1979.

A forced vibration test represents another important approach to the dynamic testing of structures. During these tests the structure is excited in a steady-state vibration with one or more eccentric mass vibration generators in which the shakers can be oriented so as to produce either of the two lateral or one torsional mode. The forced vibration tests of the Imperial County Services Building were conducted during the latter part of July and early part of August 1979. Two eccentric mass shakers from UCLA's Earthquake and Wind Laboratory were mounted on the penthouse's foundation in such a manner that, with proper synchonization, the two lateral and torsional motions were excited.

The analytical modeling of the buiding entailed the development of a finite element model that represented the building's dynamic response due to the low level ambient and forced vibration motion produced by the wind and eccentric mass shakers. Inasmuch as these levels of excitation and response were assumed to be linear, then a series linear elastic dynamic analyses were performed. In order to assess the effects of strong motion, the finite element model was modified to accommodate the nonlinear, elastic-plastic behavior associated with earthquake motion.

Although the correlation of an analytical model with the low level forced vibration experimental results is not new, the Imperial County Services Building did offer some amenities that were unique and attractive. Consider the following characteristics of this site:

1. This particular 6-story building had been instrumented by the California Department of Mines and Geology in accordance with its building instrumentation criteria for the California Strong Motion Instrumentation Program. The instrumentation in this reinforced concrete frame and shear wall building consisted of a triaxial package of accelerometers at ground level, four single axis horizontal accelerometers at the second floor level, one at the fourth floor, and four at the roof level. In all, the building is instrumented with 13 accelerometers. Lateral forces were resisted by shear walls in the N-S direction and by frame action in the E-W direction. There was a shear wall discontinuity at the second floor.

2. Approximately 340 feet east of the building was a "free field" accelerograph maintained by USGS. The location of this accelerograph was particularly significant when one considers the potential characterization of the ground motion in soil-structure interaction.

3. El Centro is in a highly seismic region. In November 1976, a swarm of more than 400 earthquakes occurred near Calipatria in the Imperial Valley. A total of 18 strong motion records were recovered from 7 accelerograph stations located within 32 Km of the epicenter. These instruments are owned by the U. S. Geological Survey (USGS), the California Division of Mines and Geology (CDMG), and the Earthquake Engineering Research Laboratory of the California Institute of Technnology (CIT). This cooperative effort includes the development of a specialized strong motion network in the Imperial Valley to fulfill such specific research needs as source-mechanism and ground-motion attenuation studies. The relatively dense instrumentation coverage in this region of recurring small - to moderate-size events provided an ideal situation in which those data necessary to implement studies can be accumulated. It should be noted that the Imperial County Services Building's instruments had been triggered no less than a half dozen times since November 1976.

4. The candidate building had the unusual architectural feature of having the shear walls discontinuous at the second floor level. This feature offered some analytical modeling challenges in correlating the analytical and experimental results.

5. The candidate building was instrumented with an anemometer and its associated hard copy recording devices.

While these items indicate a justification for conducting an analytical and experimental study of the Imperial County Services Building, it is of interest to review the projects research objectives as enunciated in the 1978 proposal to the National Science Foundation:

> "The principal research project objective will be to further develop the analytical procedures necessary for characterizing the earthquake resposone of structures based upon both experimental and analytical studies. The Imperial County Services Building offers an unusual challenge due to its architectural features, its location in a highly seismic region, and the breadth of the strong motion instrumentation in the near vicinity. Some of the activities which are related to accomplishing the research include -
>
> 1. Develop a structural dynamic information base for a building that has as much potential as probably any other instrumented building in the U. S. of being subjected to an earthquake. This information base includes the response due to ambient (primarily wind induced), forced (low level ex-

citation), and earthquake (strong motion excitation) induced vibrations as well as the results of an analytical model.

2. Develop the fundamental vibration characteristics of frequency, an estimate of damping, and mode shapes for the two lateral and one torsional mode due to the low level forced and ambient vibration excitation.

3. Assess the difference in measured vibration characteristics determined from the ambient and forced vibration experiments and attempt to quantify the application of the results from these low level excitations to the strong motion characteristics.

4. Develop an analytical finite element model that correlates the building's response due to low level forced vibration.

5. Extend the current linear elastic finite element models to include the nonlinear effects of strong motion on the three dimensional model."

As is well known within the earthquake engineering community the Imperial County Services Building has been and continues to be the source of many investigations following the October 15, 1979 Imperial Valley earthquake. This event was notable since, for the first time, a building instrumented with strong motion recorders suffered major structural failure and the response can be interpreted in the light of well defined dynamic response data. The damage was so extensive that the building was razed in 1980 and a completely new facility has since been constructed.

Results

The results of the research project have been dissemenated in a variety of technical reports, technical papers, and oral presentations to various audiences. Table I presents a summary of the disseminated research results whereas Appendices A through F contain the technical reports and papers.

IMPERIAL COUNTY SERVICES BUILDING

Technical Publications and Presentations

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

2. "Imperial County Services Building - Elastic and Inelastic Response Analyses", with A. J. Carr, P. J. Moss, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand, 79-15, December 1979.

3. Seismic Behavior of the Imperial County Services Building in El Centro, California during the Imperial Valley Earthquake", with G. C. Hart, ASCE National Convention, Hollywood, Florida, October 1980.

4. "Earthquake and Ambient Responses of El Centro County Services Building", with G. C. Hart, B. T. Bunce, ASCE/EMD, Dynamic Response of Structures: Experimentation, Observation, Prediction, Control, 1981, pp 164-174.

5. "Ambient Vibration Test Results of the Imperial County Services Building", accepted for publication in Bulletin of the Seismological Society of America.

6. "Elastic Analysis of the Imperial County Services Building", with P.J.Moss, A. J. Carr, accepted for publication in Bulletin of the Seismological Society of America.

7. "Significance of the October 1979 Imperial Valley Earthquake", presentation at annual meeting of New Zealand Institute of Engineers, Earthquake Engineering Branch, Dunedin, New Zealand, November, 1979.

8. "Elastic and Inelastic Results of the Imperial County Services Building", presentation at annual meeting of Earthquake Engineering Research Institute, Santa Barbara, California, January 1980.

Table I

Conclusions

As proposed, the research investigation conducted ambient and forced vibration studies as well as analytical investigations of the Imperial County Services Building. The low level response characteristics provided a means of comparison for various elastic computer models which, in turn, were useful in interpreting the post-earthquake condition of the building.

The research project presented predictions of the structural response of the Imperial County Services Building due to several static and dynamic load excitations using elastic analysis techniques. Seven representative load conditions were considered 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 elastic behavior of the ICSB, the fact remains that elastic analyses clearly play a useful role in post-earthquake "post mortems" by clarifying structural action. In particular the elastic analyses clearly pinpointed the east end column weaknesses which were subsequently confirmed by the damage sustained. These computed response results of the ICSB, which is destined to become one of the most thoroughly investigated structure in earthquake engineering history, help explain the catastrophic failure of the building during the October 15, 1979 earthquake.

APPENDIX A

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RESEARCH REPORT

IMPERIAL COUNTY SERVICES BUILDING
Ambient Vibration Test Results

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79-14

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(iii) Input/Output Boards-these boards allow for the transfer of data to and from devices such as the DMS, Tektronix terminal and, potentially, an acoustic coupler linked to a mainframe computer.

The version of the DMS which includes local FFT processing capabilities also requires Random Access Memory (RAM) boards to store the computed data.

3. Tektronix Terminal. This device serves as the "control center" for the DMS. It permits the user to display data in graphic and/or numeric form. The display screen itself is a storage-type screen. The terminal also includes an alphanumeric keyboard for the entry of system commands.

4. Hard Copy Unit. Provides permanent paper record of data displayed in the terminal screen. The hard copy processes requires a few seconds, and is useful for documentation purposes.

The software portion of the DMS is composed of two major subsystems:

(i) The basic local software which permits communication with timesharing and extended local software which allows Fourier analysis without timesharing.

(ii) The timesharing software is a family of programs, called the Mechanical Design Library (MDL), owned and maintained by Structural Dynamics Research Corporation. Comshare, Inc., offers the international timesharing network which contains those programs accessed by the DMS.

Presently the DMS is being linked to the PDP 11/34 minicomputer

List of Figures

Ambient Vibration Test Results

Background

The success or failure of a structural design depends, to a great extent, upon the formulation of the appropriate conceptual and mathematical representation of the structure's characteristics. The design success or failure is particularly accentuated in an active earthquake zone. Success can be measured in terms of a cost effective, efficient, aesthetically pleasing design that meets or exceeds a demanding client's need while maintaining the structural integrity under severe seismic loading. Failure, on the other hand, may include overdesign, with its inherent increased costs, due to ignorance or uncertainty in order to meet severe seismic design loads or catastrophic structure failure resulting in injuries or fatalities. Inasmuch as a structure's analytical model indicates a significant part of the overall design, it is imperative, particularly in a high seismic area, that the model accurately represent the full-scale structure. One means of validating analytical procedures is to perform experimental studies of full-scale structures and compare these results with those of the analytical model.

Presently a research effort is being devoted to an in-depth experimental and analytical study of the Imperial County Services Building in El Centro, California (see Figure 1). This research project is being sponsored by the National Science Foundation under its Research Initiation in Earthquake Hazards Mitigation Program. The experimental component of the research project has been devoted to low level structural excitations (ambient and forced vibration); however, the potential for experimental results (and the structure's inherent nonlinear behaviour) due to strong motion exists since the candidate structure is in a highly seismic area

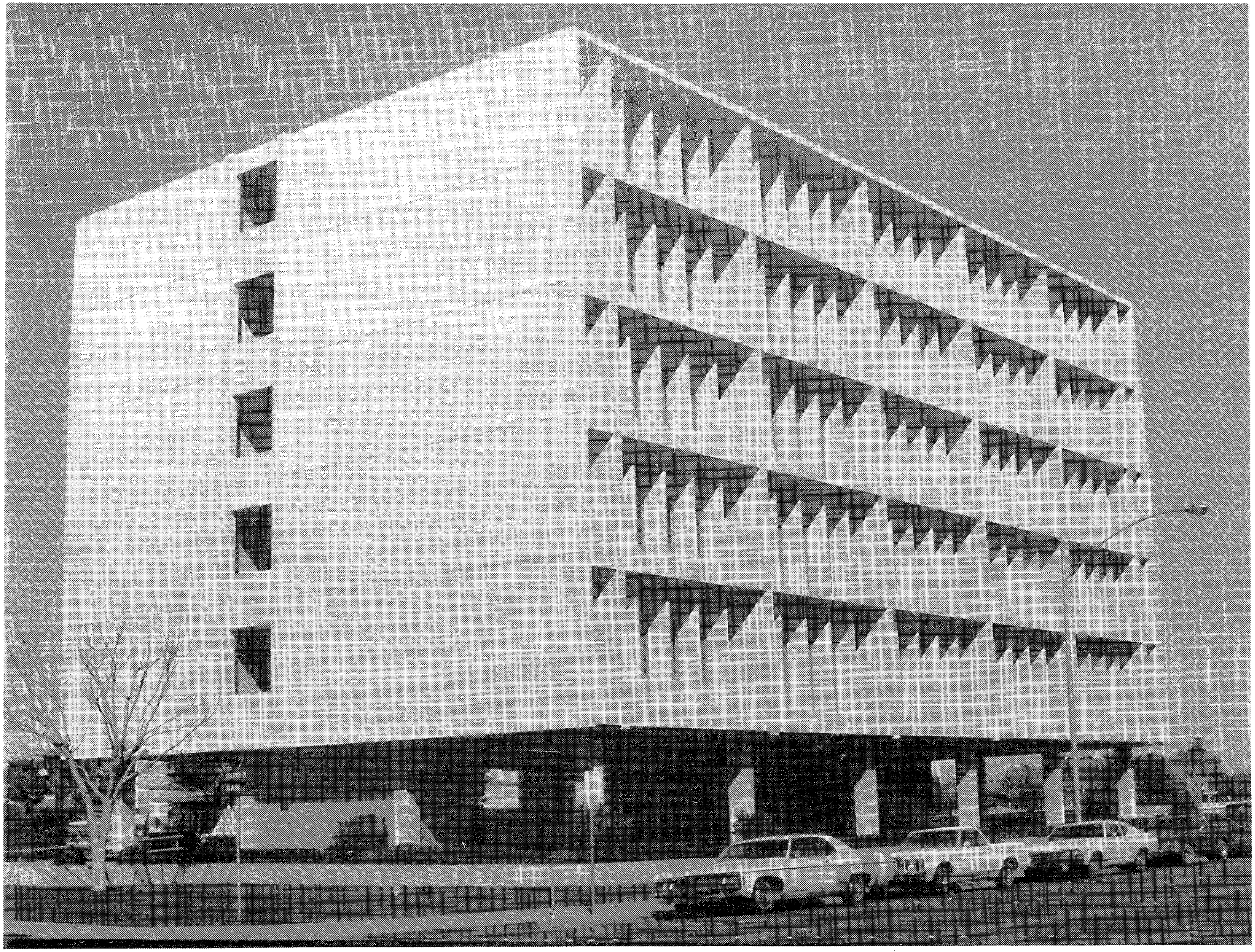


Figure 1 - Imperial County Services Building - View Looking Northeast

and is instrumented under the California Strong Motion Instrumentation Program. On 15 October 1979 this earthquake potential become a reality when a powerful earthquake jolted Southern California and northern Mexico causing extensive damage to the Imperial County Services Building. The earthquake, which measured 6.4 on the Richter scale, was centred on the Imperial Fault near the US - Mexican border. The analytical component of the research project is being devoted to developing a valid mathematical model to accurately represent the low level forced vibration while investigating methods and techniques needed to represent the nonlinear structural behaviour due to the strong motion records of 1934, 1940, and ncw 1979.

This report, one of several related to the Imperial county Services Building, is devoted to the presentation of the data resulting from the ambient vibration tests conducted during February and May 1979. In light of the 15 October 1979 earthquake these results are being disseminated to the engineering community as rapidly as possible. However the complete investigation of the ambient vibration survey results is still an ongoing project so one should view this report as a preliminary one.

Significance of the Imperical County Service Building

Although the correlation of an analytical model with the low level forced vibration experimental results is not new, the Imperial County Services Building does offer some amenities that are unique and attractive. Consider the following characteristics of this site:

1. This particular 6-storey building has been instrumented by the California Department of Mines and Geology in accordance with its building instrumentation criteria for the California Strong Motion Instrumentation Program. The instrumentation in this reinforced concrete frame and shear wall building (see figure 2 and 3) consists

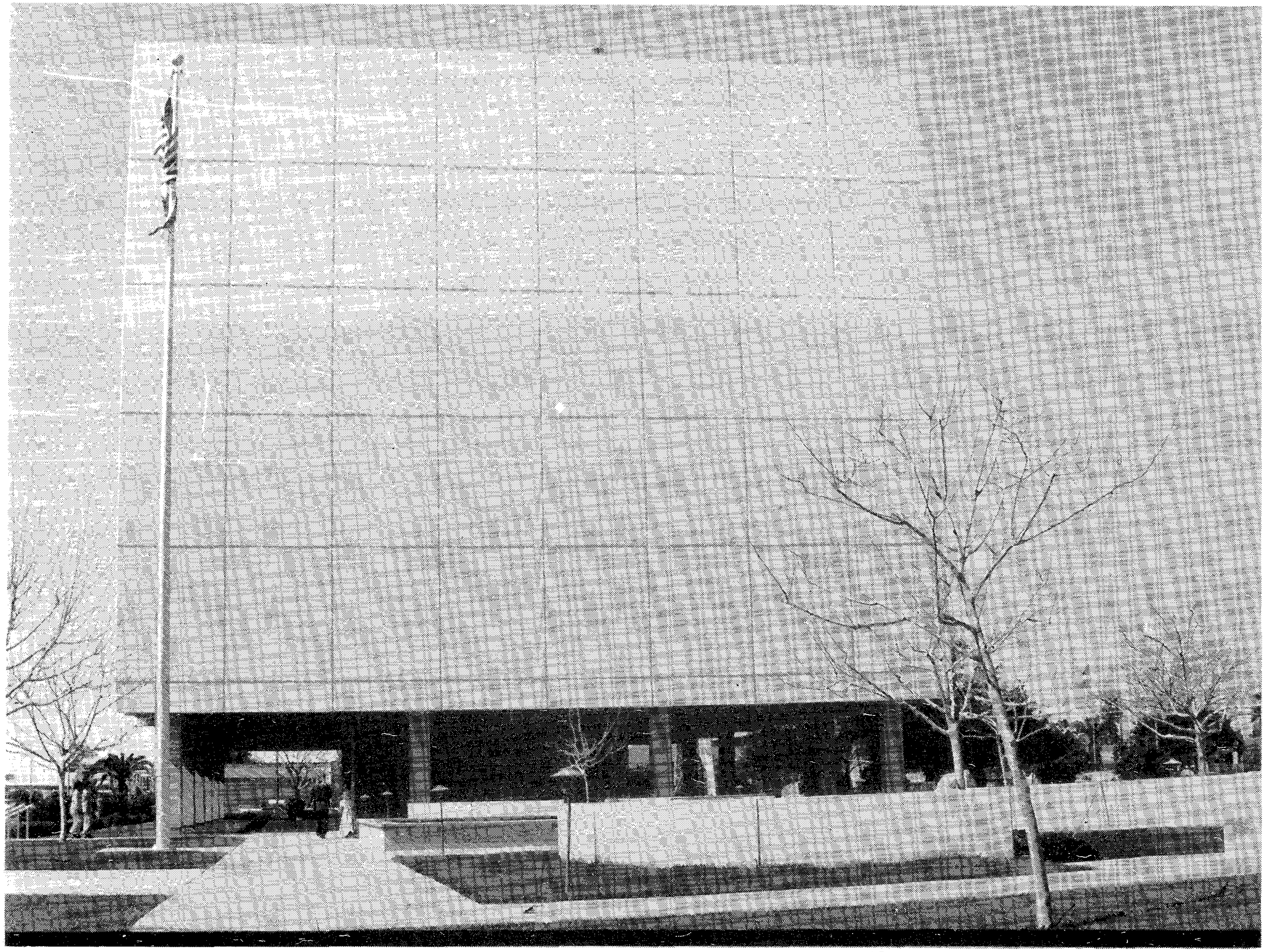


Figure 2 - Imperial County Services Building - East Shear Wall

Figure 3 - Imperial County Services Building - West Shear Wall

of a triaxial package of accelerometers at ground level, four single axis horizontal accelerometers at the second floor level, one at the fourth floor, and four at the roof level. In all the building is instrumented with 13 accelerometers (see Figure 4). Lateral forces are resisted by shear walls in the N-S direction and by frame action in the E-W direction. There is a shear wall discontinuity at the second floor.

2. Approximately 104 meters east of the building is a "free field" accelerograph maintained by USGS. The location of this accelerograph is particularly significant when one considers the potential characterization of the ground motion in soil-structure interaction.

3. El Centro is in a highly seismic region (see Figure 5). The earthquakes of 1934 and 1940 severe as benchmarks for scientists and engineers throughout the world. More recently , in November 1976, a swarm of more than 400 earthquakes occurred near Calipatria in the Imperial Valley. A total of 18 strong motion records were recovered from 7 accelerograph stations located within 32 Km of the epicenter. These instruments are owned by the U.S. Geological Survey (USGS), the California Division of Mines and Geology (CDMG), and the Earthquake Engineering Research Laboratory of the California Institute of Tecnology (CIT). This cooperative effort includes the development of a specialized strong motion network in the Imperial Valley to fulfill such specific research needs as source-mechanism and ground-motion attenuation studies. The relatively dense instrumentation coverage in this region of recurring small- to moderate-size events provides an ideal situation in which those data necessary to implement studies can be accumulated. It should be noted that the Imperial County Services Building's instruments have

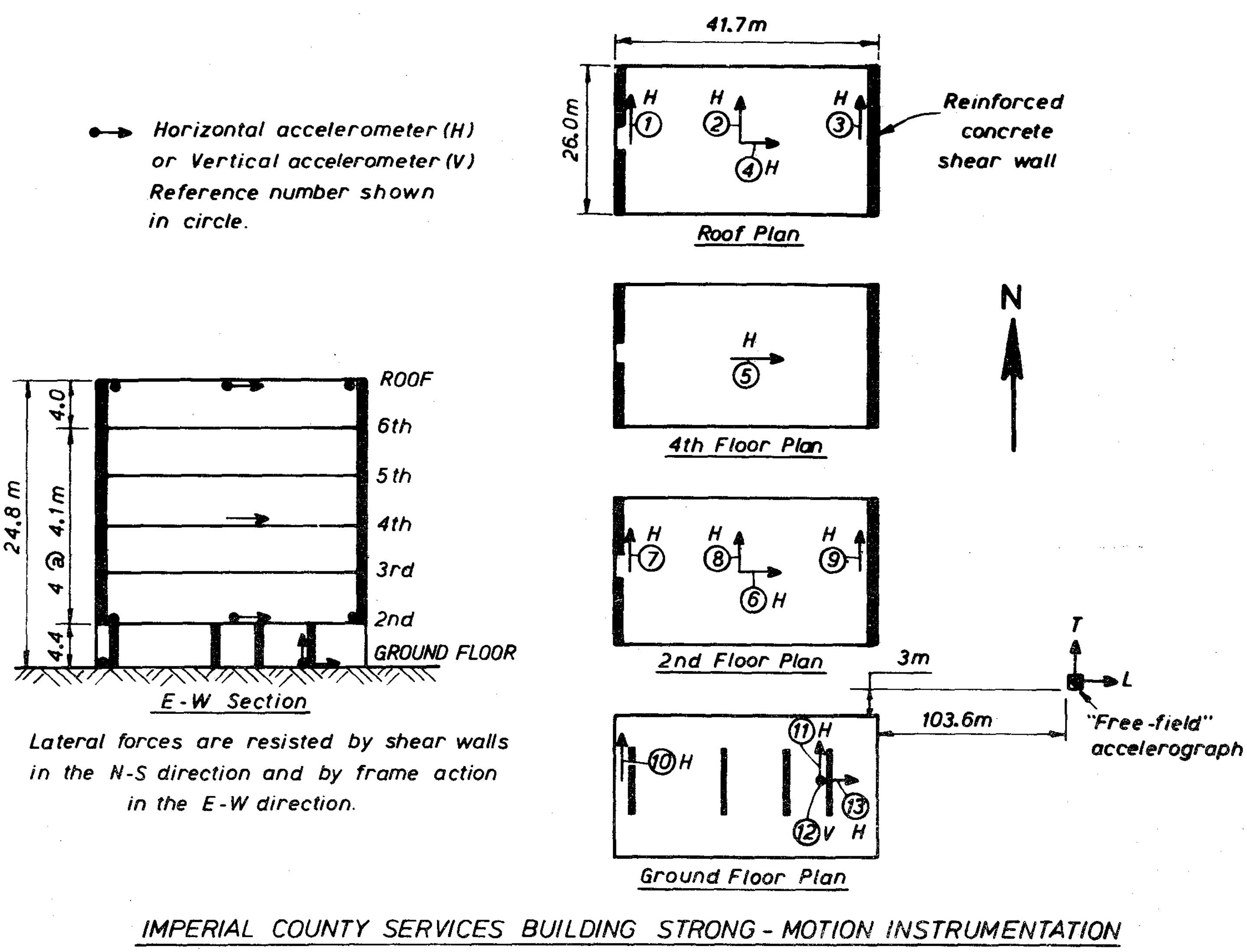
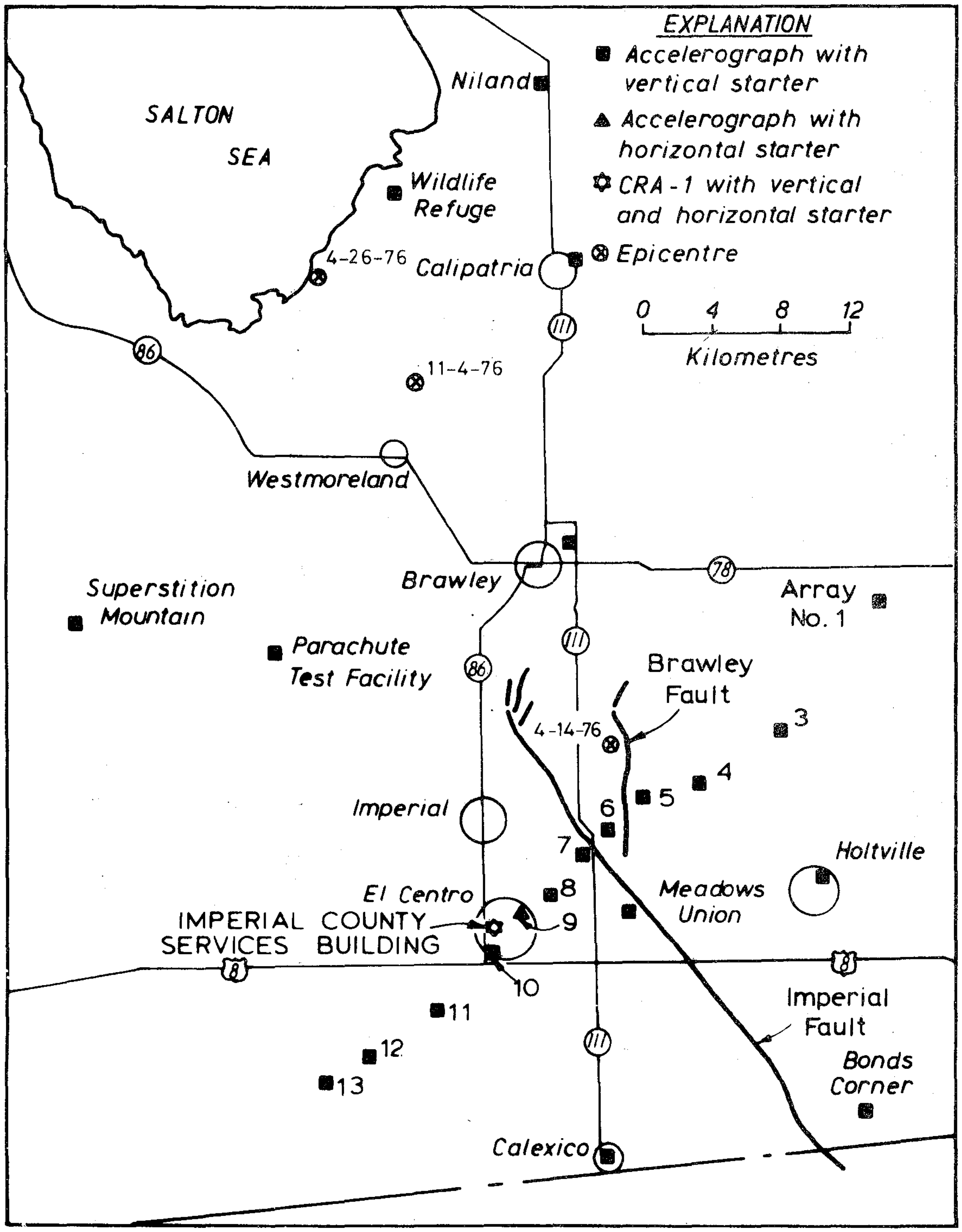


Figure 4 - Imperial County Services Building - Strong-Motion Instrumentation



STRONG-MOTION STATIONS IN THE IMPERIAL VALLEY AREA.

Figure 5 - Strong Motion Stations In the Imperial Valley Area

been triggered no less than a half dozen times since November, 1976, with the last strong motion prior to the 15 October 1979 earthquake occurring on May 5, 1978.

4. The candidate building has the unusual architectural feature of having the shear walls discontinuous at the second floor level. This feature should offer some analytical modeling challenges in correlating the analytical and experimental results.

Test Apparatus

The apparatus associated with the ambient vibration tests consists of data acquisition and data reduction equipment.

The data acquisition equipment includes:

Kinemetric SS-1 Ranger Seismometers (4)- The Ranger, an excellent short-period seismometer, is a highly sensitive, portable vibration sensor for structural dynamic applications. Mechanically, the Ranger is a "moving coil type" (velocity) transducer adaptable for either vertical or horizontal operation. Because of its high sensitivity, rugged construction and compact size, it is ideally suited for ambient and low level forced vibration of buildings.

Kinemetric SC-1 Signal Conditioner - The SC-1 is a wide band, low noise amplifier system, designed with filters for use in low level structural vibration and microseismic measurements. Four input channels, each having its own attenuator and adjustable low-pass filter, provide isolated circuitry for a normal, integrated, and/or differentiated output signal (i.e., velocity, displacement and/or acceleration output using a velocity sensor). All outputs are simultaneously or independendly available for recording. The power supply is by internal rechargeable batteries with an internal battery charger. The unit is contained in a portable, weather-tight, aluminum case, with a removable cover, and weighs approximately 20 pounds.

Hewlett Packard 3960 Instrumentation Tape Recorder - This rugged four-channel, three speed instrumentation device, which uses ¼-inch magnetic tape, is capable of FM recording and reproducing over a band-width from DC to 5 kHz, or direct recording and reproducing over a frequency range between 100 Hz to 64 kHz. The FM mode was used for the ambient vibration tests.

The data reduction equipment includes:

Zonic Technical Laboratories DMS 5003 - The Data Memory System, which is used to manipulate the recorded ambient or forced vibration data, is composed of five major subsystems:

1. A multi-channel digital data acquisition system, called the Data Memory System (DMS), which serves as the "front end" to the system.

2. A microcomputer which has been preprogrammed to perform various DMS functions.

3. A Tektronix 4006 graphics display terminal.

4. A Tektronix 4631 hard copy unit.

5. A dual cassette tape drive system for storing and retrieving user programs, raw data, and processed data.

A more complete description of the data reduction system and its capability is provided in Appendix A.

Test Procedure

The ambient vibration tests were conducted on three separate days in 1979: 26 February, 27 February and 17 May. With the exception of the late evening of 26 February, all three days were characterized by light prevailing winds. Apart from the 5 minute calibration runs in the N-S and E-W directions at the beginning and end of the test sequence, all ambient vibration tests were conducted for approximately 18 minutes. The approximate 18 minute time frame was chosen so that there would be sufficient data to perform the spectral analyses of 100 "snapshots" of time data of 10.24 seconds each.

The ambient vibration tests were conducted by placing the seismometers in strategic locations throughout the building on both the N-S and E-W directions. With the exception of the torsion tests the seismometers were usually located at a common point in a plan view but at different elevations, or 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. The seismometers were oriented in a "pinwheel" fashion for torsion tests, i.e. the seismometer in the NE corner was pointed north, the seismometer in the NW corner was pointed west, the seismometer in the SW corner was pointed south, and the seismometer in the SE corner was pointed east.

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 acquired and recorded for the ambient tests. By using the manufacturer's calibrated values for the velocity transducers, it was a relatively routine matter to obtain absolute magnitudes of velocity rather than just relative values.

Data Reduction - Fourier Transform Method

The principal objective of the ambient vibration tests is to determine estimates of fundamental frequencies and the associated mode shapes as well as an estimate of structural damping. With the implementation of the Fast Fourier Transform (FFT) on hardwired microcomputers the effort in calculating time and frequency domain functions related to these fundamental vibration characteristics has been tremendously decreased. Modern FFT analyzers, such as the Zonic DMS 5003, work on the principle of a fast and efficient calculation[1] of the so-called Discrete Fourier Transform (DFT).

While no attempt will be made to discuss the background of the DFT and FFT, some review of Fourier Series, the Fourier Transform and the Discrete Fourier Transform is advantageous. Specifically the discussion will identify those significant calculations which are used to determine the structure's vibrational characteristics such as the power spectrum, auto correlation, cross spectrum, cross correlation, frequency response, and coherence function. More detailed discussions on the Fourier Transform and the applications of digital signal processing can be determined from various references such as 2-5.

FOURIER SERIES

It is well known that time functions are often interpreted conveniently by the analysis of the frequency content. This approach is derived from the work of the French Mathematician Jean Baptiste Fourier who discovered that periodic time functions can be decomposed into an infinite sum of properly weighted sine and cosine functions of the proper frequencies. The mathematical statement of this discovery is

$$x(t) = a_o + \sum_{n=1}^{\infty} a_n \cos \frac{2\pi nt}{T} + b_n \sin \frac{2\pi nt}{T} \qquad (1)$$

where T is the period of x(t), that is, $x(t) = x(t+T)$

When the coefficients a_n and b_n are calculated using the equations derived by Fourier, the amplitude of each sine and cosine wave in the series is known. Equivalently, when the coefficients a_n and b_n are known, the magnitude and phase at each frequency in x(t) is determined, where is the amplitude at the frequency $f_n = (n/T)$, and $\tan^{-1}(b_n/a_n)$ is the corresponding phase.

FOURIER TRANSFORM

While the Fourier series is a useful tool for determining the frequency content of a time varying signal, it does require that the signal be periodic. To overcome this short-coming, Fourier evaluated this series as he let the period of the waveform approach infinity. The function which resulted is known as the Fourier transform and the Fourier transform pair is defined as

$$X(f) = \int_{\infty}^{\infty} x(t)\, e^{-i2\pi ft}\, dt \qquad \text{(Forward Transform)} \qquad (2)$$

$$x(t) = \int_{\infty}^{\infty} X(f)\, e^{i2\pi ft} df \qquad \text{(Inverse Transform)} \qquad (3)$$

$X(f)$, the Fourier transform of $x(t)$, contains the amplitude and phase information at every frequency present in $x(t)$ without demanding the $x(t)$ be periodic.

DISCRETE FOURIER TRANSFORM

In order to calculate the Fourier transform using a digital computer it is useful to examine the results of digitally computing the forward Fourier transform given in equation (2). In order to implement the Fourier transform digitally, one must convert the continuous input signal into a series of discrete data samples. This is accomplished by sampling (measuring) the input waveform, $x(t)$, at certain intervals of time. Assume that the samples are spaced uniformly in time, separated by an interval Δt. In order to perform the integral (2) the samples must separated by an infinitesimal amount of time (i.e., $\Delta t \rightarrow dt$). Due to possible. Thus a discrete form of the Fourier transform results.

$$X(f) = \Delta t \sum_{n=\infty}^{n=+\infty} x(n\Delta t)\, e^{-i2\pi fn\Delta t} \qquad (4)$$

where $x(n\Delta t)$ are the measured values of the input function. Equation (4) states that, even though one deals with a sampled version of $x(t)$, it is still possible to calculate a valid Fourier transform. However, the Fourier transform as calculated by (4) no longer contains accurate magnitude and phase information at all of the frequencies contained in $X(f)$. Rather, $X(f)$ accurately describes the spectrum of $x(t)$ up to some maximum frequency (F_{max}) which is dependent upon the sampling interval, Δt.

POWER SPECTRUM, AUTO CORRELATION

The power spectral density function is a very useful parameter that basically determines the power distribution as a function of frequency for a signal. It gives a measure of energy distribution of the signal. It is defined for a single record of length T by:

$$G_{xx}(f) = \frac{2}{T} X^*(f) \cdot X(f) \qquad f>0 \qquad (5)$$

where

$G_{xx}(f)$ = power spectral density function

$X(f)$ = Fourier transform of $x(t)$

$X^*(f)$ = complex conjugate of $X(f)$

Note that $G_{xx}(t)$ is defined for positive values of frequency only. The auto correlation function is defined for a single record of length T by:

$$I_{xx}(\tau) = \frac{1}{T} \int_{-T/2}^{T/2} x(t)x(t+\tau)\, dt \qquad (6)$$

It can be readily shown that the power spectral density function is the determine the autocorrelation function is to calculate the power spectral density function and take the inverse Fourier Transform.

CROSS SEPCTRUM, CROSS CORRELATION

The cross spectral density function is very similar to the power spectral density function except that it is calculated from two different signals from a system. It is defined as:

$$G_{yx}(f) = \frac{2}{T} X^*(f) \cdot Y(f) \qquad f>0 \tag{7}$$

where

G_{yx} = cross spectrum between $y(t)$ and $x(t)$

$Y(f)$ = Fourier transform of $y(t)$

$X^*(f)$ = complex conjugate of $X(f)$, the Fourier transform of $x(t)$

The cross correlation function is simply the Inverse Fourier transform of the cross spectral density function.

FREQUENCY RESPONSE

The frequency response can be defined as the ratio of the Fourier transform of the output of a system divided by the Fourier transform of the input. If the following system is considered:

x (t)		y(t)
	System	
Input		Output

then the frequency response is

$$H(f) = \frac{Y(f)}{X(f)} \tag{8}$$

where $X(f)$, $Y(f)$ are the Fourier transforms of $x(t)$, $y(t)$, respectively.

The frequency response for a system will be unique if the system is linear. If it is nonlinear the frequency response will be a function of input to the system must exist. It makes no difference if the input is periodic, random or transient, but it must have frequency content in the range of interest. If it does, then the frequency response can be determined by measuring the input and output response of the system, Fourier transforming the input and output, and then taking the ratio of the two transformed values.

However, an important point concerning the determination of the frequency response is that frequently both the measured input and output signal from the system will have a noise component. Therefore, the Fourier transform of the input and output will include the transform of the noise components which will cause an error in the estimation of the frequency response. In order to reduce this error, the frequency response can be obtained from the following equation:

$$H(f) = \frac{\Sigma Y(f) \cdot X^*(f)}{\Sigma X(f) \cdot X^*(f)} = \frac{\Sigma G_{yx}(f)}{\Sigma G_{xx}(f)} \tag{9}$$

The terms G_{yx} and G_{xx} are the cross spectrum and power spectrum functions, respectively. These values can be summed for independent records of data and the cross spectrum sum divided by the power spectrum sum to determine the frequency response. This technique is particularly useful when considering the random data that is obtained from the ambient vibration tests.

COHERENCE

The coherence function is defined as

$$T_{xy}^{2} = \frac{\overline{G_{yx}(f)}^{\,2}}{\overline{G_{xx}(f)}\ \overline{G_{yy}(f)}} \tag{10}$$

where

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$\underline{T_{xy}}^2$ = coherence function

$\underline{G_{xy}}$ = average cross spectrum between input and output

$\underline{G_{xx}}$ = average power spectrum of input

$\underline{G_{yy}}$ = average power spectrum of the output

The quantity $|G_{xy}(f)|$ is the absolute value of $G_{xy}(f)$.

If the coherence function is equal to one at a given frequency then the output at that frequency is entirely due to the input. If it is less than one, then the output signal also has a noise component or it is due to some non-linear behavior of the system and can be used to determine how many averages must be made to determine the frequency response of the system. The coherence function measures how coherent the output signal is with the input signal.

Note that the above equation states that the quantities used to calculate the coherence function must be averaged values. Coherence functions taken from single records of the above quantities are meaningless since the coherence function will always be calculated as one in that case.

Data Reduction - Random Decrement Technique

In addition to the Fourier Transform methods for determining a structure's vibrational characteristics, an alternative technique, the random decrement method, was implemented. Previously the random decrement technique[6] has been used successfully for failure detection and damping measurements of structures in single station, single mode response cases whereas its present application is to determine estimates of frequency and damping from a building's ambient vibration response. More recent development[7] has extended the technique to multiple station response.

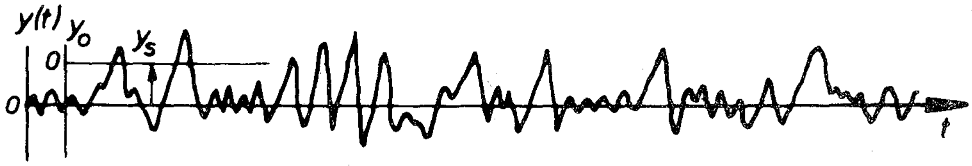
In general, the experimental identification of structural modes of vibration is conducted by measuring the input to the structure under test and the resulting responses due to the input. Some vibration testing techniques, in order to simplify the identification procedure, use the free response of structures. In such cases, although the input excitation need not be measured, some initial excitation is applied to the structure and free responses are measured immediately after the excitation force is removed. As a practical matter the free response of a multistory building may be difficult to achieve.

There are many practical situations where controlled excitation or initial excitation cannot be used. For example, if the structure to be tested is in operation, applying any kind of external force may cause undesirable interruption. Another example is the case of wind and microtremor excitation of a building in which the complete knowledge of the input excitation is not known. In such cases, the use of the random decrement technique (a special averaging procedure that is used to determine the steps and/or impulse response from the random responses) in order to obtain the free vibration response may be attractive.

A typical random response of transducer, such as a Ranger Seismometer, is shown in Figure 6. The random response curve is so complicated and variable that little information can be gleaned from the time history itself - usually various analysis techniques must be performed on this time history in order to condense the meaningful information. One well known technique showm in Figure 6 is the spectral density which may be obtained directly from an ensemble average of the absolute amplitude squared of the Fourier transform of N segments of the time history. The resulting signature has a peak for each structural mode; and for well-separated peaks, the damping ratio of the mode may be obtained by measuring the width of the peak at half the peak value. This so-called bandwidth of the half-power point is equal to $2 \S f_n$. Also the integral of the power spectral density is equal to the mean square value.

DAMPING MEASUREMENTS

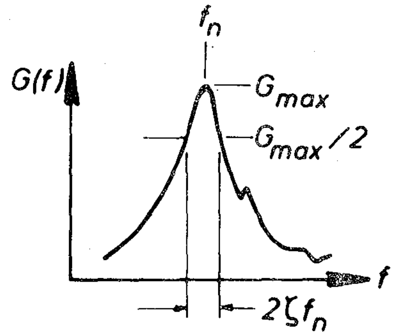
GIVEN :- Random Response (Input Unknown)



FIND :- Damping Ratio (ζ)

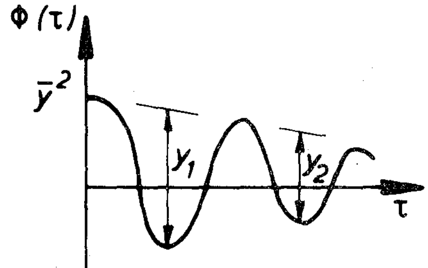
SPECTRAL DENSITY

$$G(f) = \left[\int_0^T y(t) e^{-i 2\pi f t} dt \right]^2$$



AUTOCORRELATION

$$\Phi(\tau) = \lim_{T \to \infty} \frac{1}{T} \int_0^T y(t)\, y(t+\tau)\, dt$$



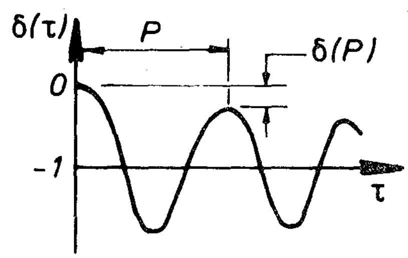
$$\zeta \approx \frac{1}{2\pi} \left(1 - \frac{y_2}{y_1} \right)$$

RANDOM DECREMENT

$$\delta(\tau) = \frac{1}{y_s N} \sum_{n=1}^{N} y_o(t_n + \tau)$$

with condition

$t_n = t$ when $y_o = 0$



$$\zeta \approx \frac{1}{2\pi}\, \delta(P)$$

Figure 6 - Techniques For Calculating Damping From Random Response

the frequencies of the structural modes, the energy in the modes and the approximate damping of isolated modes. However, the main problem of its use in detecting frequency and damping is that it is very dependent on the input as shown by the following equation

$$G_y(f) = |H(f)|^2 G_x(f)$$

in which $H(f)$ is the transfer function of the structure and $G_x(f)$ is the spectral density of the input forces. It may be seen that the amplitude and form of the output spectral density $G_y(f)$ are dependent on the amplitude and form of $G_x(f)$ which in most building vibration cases is unknown. Hence, $G_y(f)$ is only truly representative of the structure if $G_x(f)$ is a constant or white noise.

Another dynamic signature shown on Figure 6 is the autocorrelation for isolated modes, the signature has the same form as the vibration decay curve of a structure with an initial displacement and may be interpreted as such to obtain period and damping of the mode. The autocorrelation is less sensitive than spectral density to variations in the spectral form of the input. The autocorrelation function may be used for measuring damping of isolated modes for multi-mode applications.

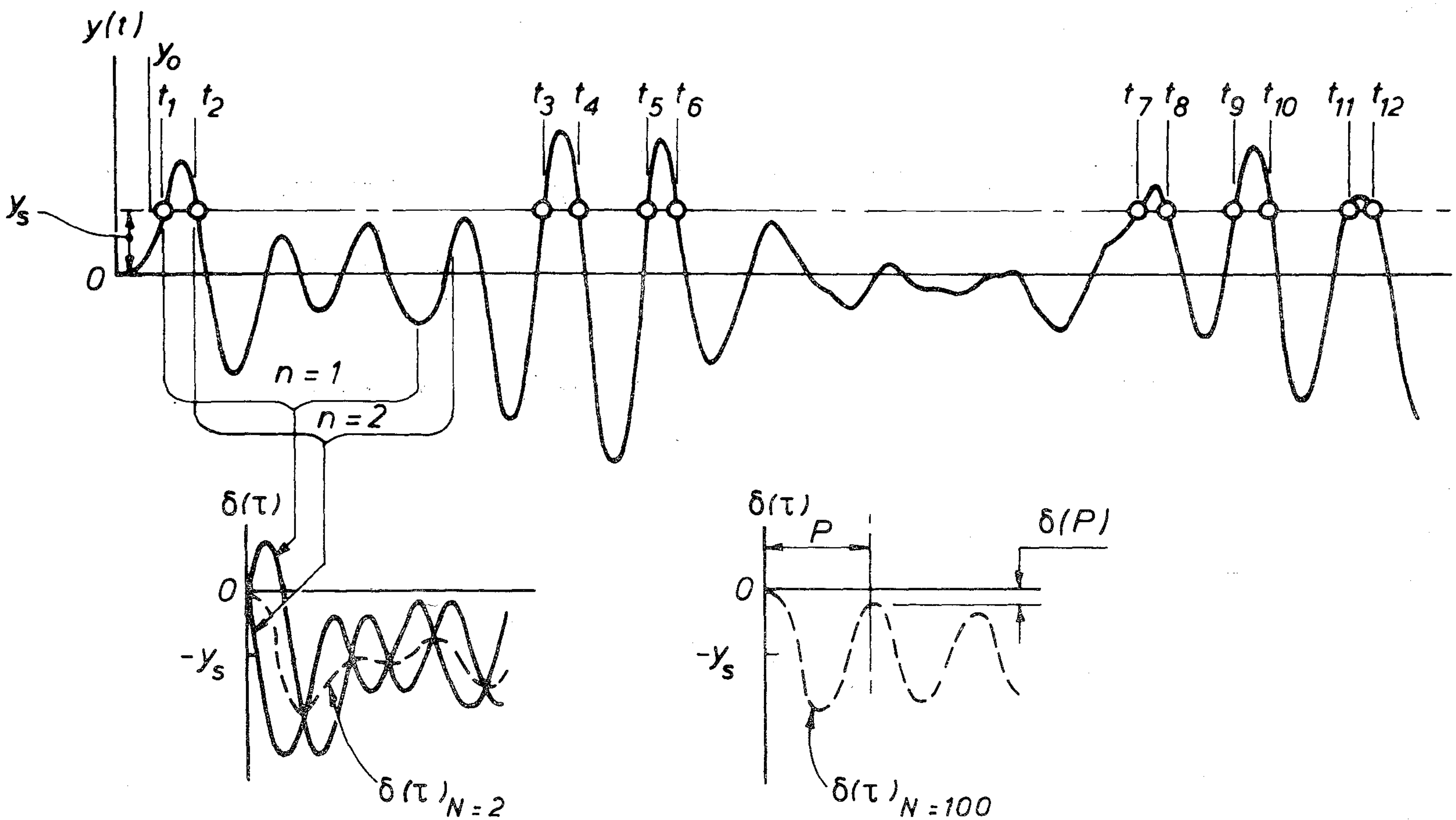
The random decrement signature shown on Figure 6 has an appearance similar to autocorrelation, but it has many properties which make it more useful. The first is that the signature has a constant amplitude, Y_s, which represents a calibrated displacement of the structure. This is important because it fixes the level of the signature and makes it independent of changes in intensity of the input. Also, if the structure has nonlinear damping with amplitude, the fixing of amplitude stabilizes the form of the signature Another property is that the signature has the same dimensions as the original time history since the multiplications are performed. Consequently, in multimode applications troublesome cross products of modes are avoided; and in applications where the input spectral density is not flat, the signature distortion is considerably less.

Although the equation on Figure 6 describes the process, a better feel for the extraction of the signature is obtained by graphically performing the process as shown on Figure 7. First, the selection level, Y_s , is set. Each time the curve passes through $Y_o(t) = 0$, a segment of the curve is placed in summation. The first two segments are shown on the figure, one with and initial condition of a plus slope and one with an initial condition of a minus slope. The average of these two is the signature $\delta(\tau)$ for $N = 2$. As more samples are taken, the signature converges to a form as shown for $N = 100$. For a single-degree-of-freedom system the value $t = P$ would be the period of oscillation.

The hypothesis of the random decrement signature for linear systems is shown in Figure 8. This figure shows the process as the linear superposition of a step, an impulse and random response for each segment of the time history selected. In other words, the step represents the homogeneous solution to an initial displacement, the impulse represents the homogeneous solution to an initial velocity, and the random response represents the particular solution to random inputs which occur during the sample segment. It may be seen that all of the step responses are the same, whereas the impulse responses have initial slopes with alternate plus and minus values of varying magnitued. The random responses tend to average to zero. From this signature one can obtain an estimate of frequency and damping of the structure from its random response without needing to know (or assume) the input spectrum.

Test Results - Frequency

Using the aforementioned Fourier Transform and Random Decrement procedures the frequencies of the Anyslrial County Services Building were determined and are summarized in Table 1.



RANDOM DECREMENT SIGNATURE FOR N=2 AND N=100

Figure 7 - Random Decrement Signature For N=2 And N=100

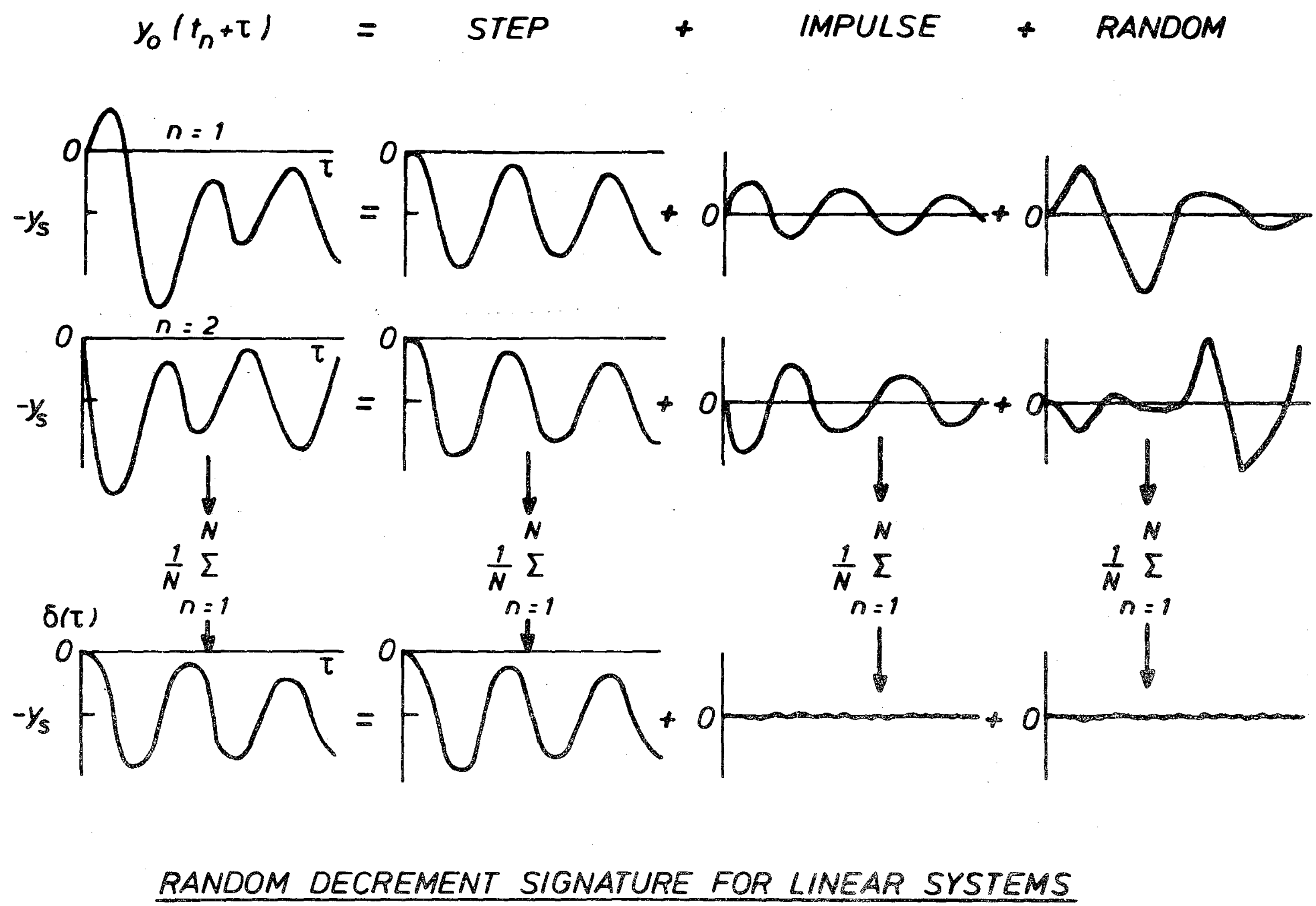


Figure 8 - Random Decrement Signature For Linear Systems

Configuration	Seismometer Direction	Seismometer Locations	Method	Frequency
1	East	B-7,B-6,B-4,B-2	Power Spectrum Auto Correlation Random Decrement	1.54 Hz 1.56 Hz *
2	North	D-7,D-6,D-4,D-2	Power Spectrum Auto Correlation Random Decrement	2.24 Hz 2.25 Hz *
3	North	B-7,B-6,B-4,B-2	Power Spectrum Auto Correlation Random Decrement	2.81 Hz 2.85 Hz *

Table 1 - Frequencies From Ambient Vibration Tests

The power spectrum results reflecting the three basic test configurations are depicted in Figures 9, 10 and 11. Each power spectrum curve represents the average power spectrum resulting from 100 "snapshots" of time data. The letter - number code denoting the seismometers locations refers to both a plan view and story level position - the letters A to H refer to specific plan view locations whereas the numbers refer to the story level (7 = roof, 6 = 6th floor, etc).

A few points are worth noting. First, if the building were truly responding as a single degree of freedom subjected to white noise then the

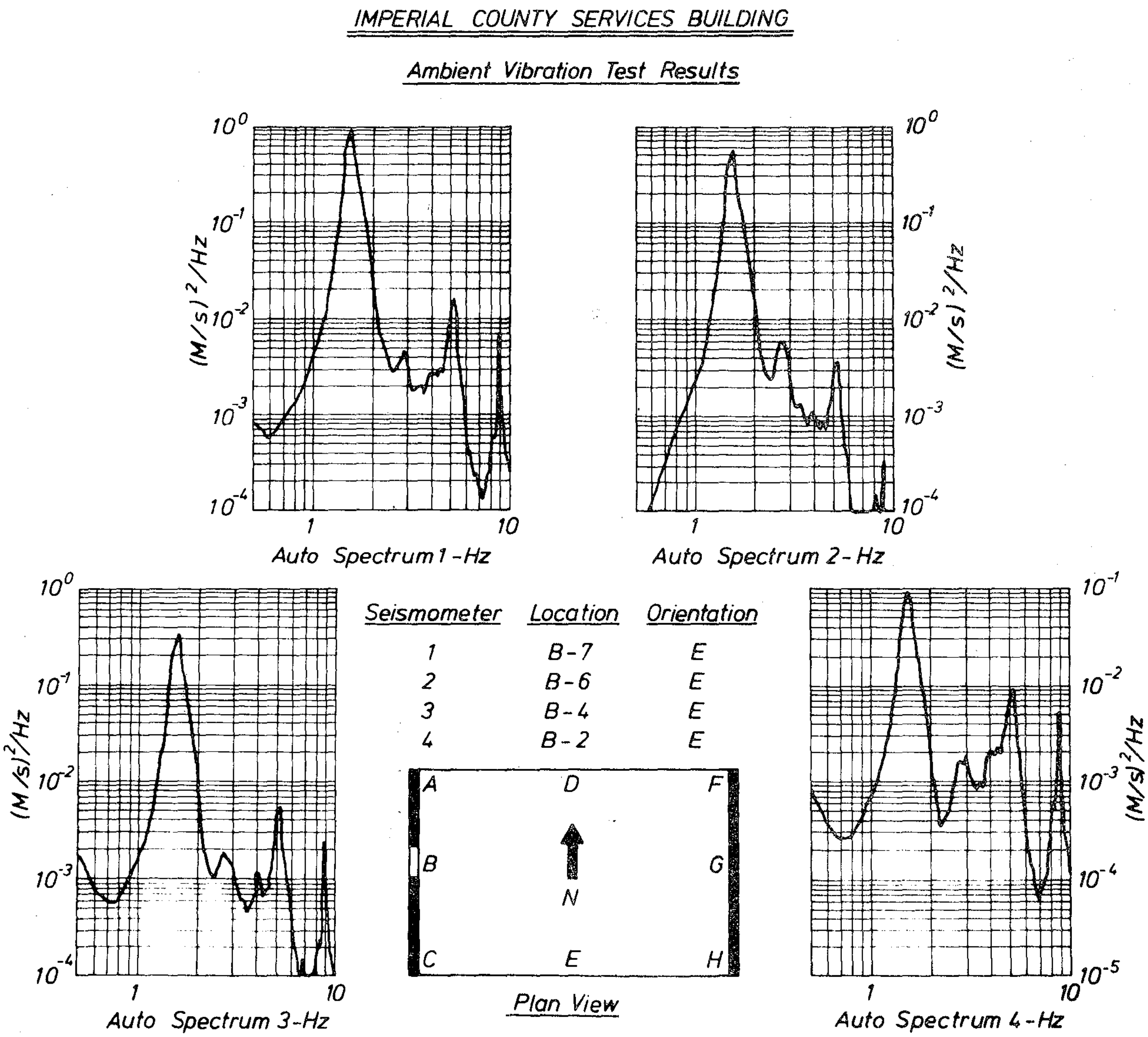


Figure 9 - E-W Power Spectrum Responses At Location B

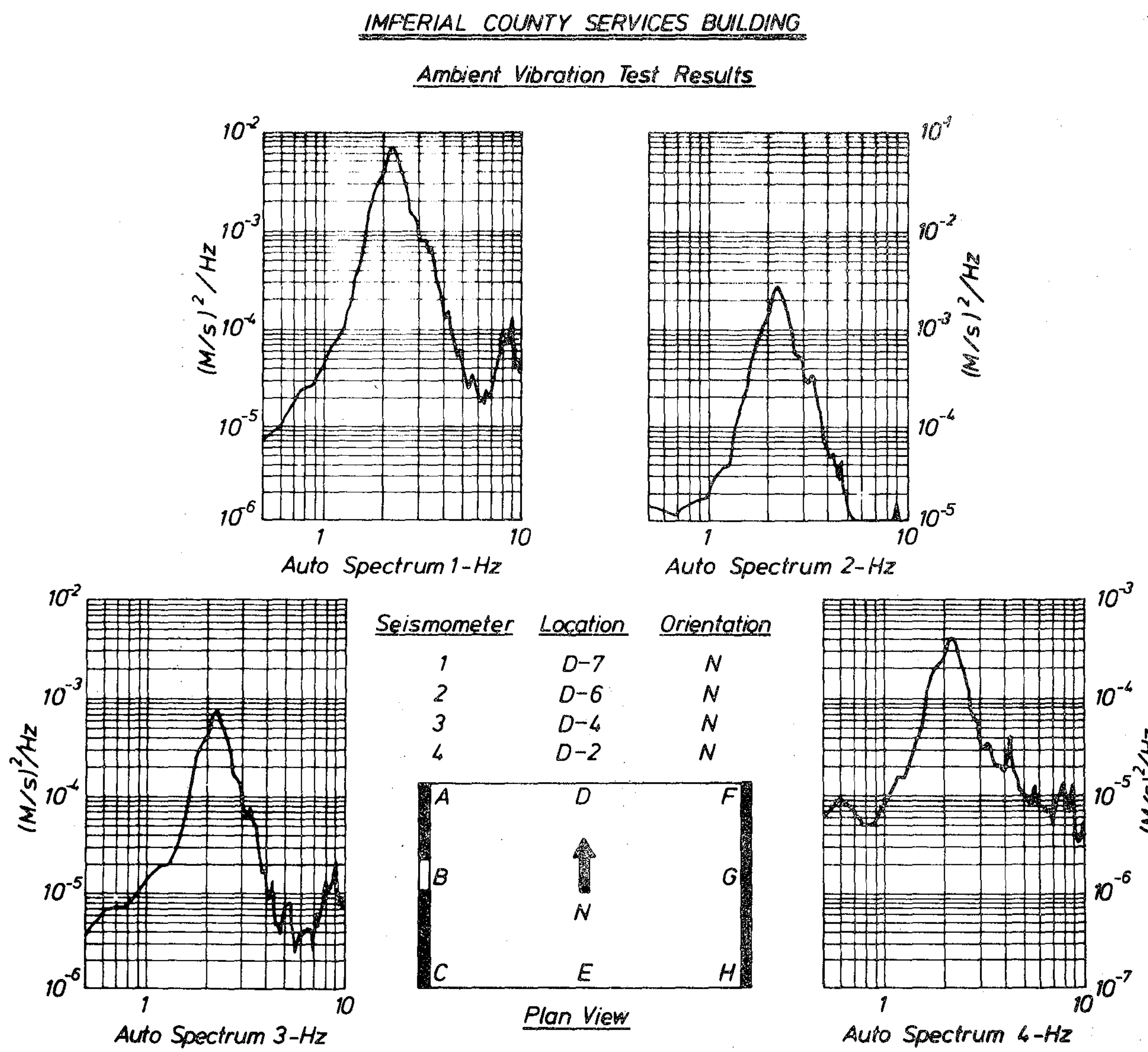


Figure 10 - N-S Power Spectrum Responses At Location D

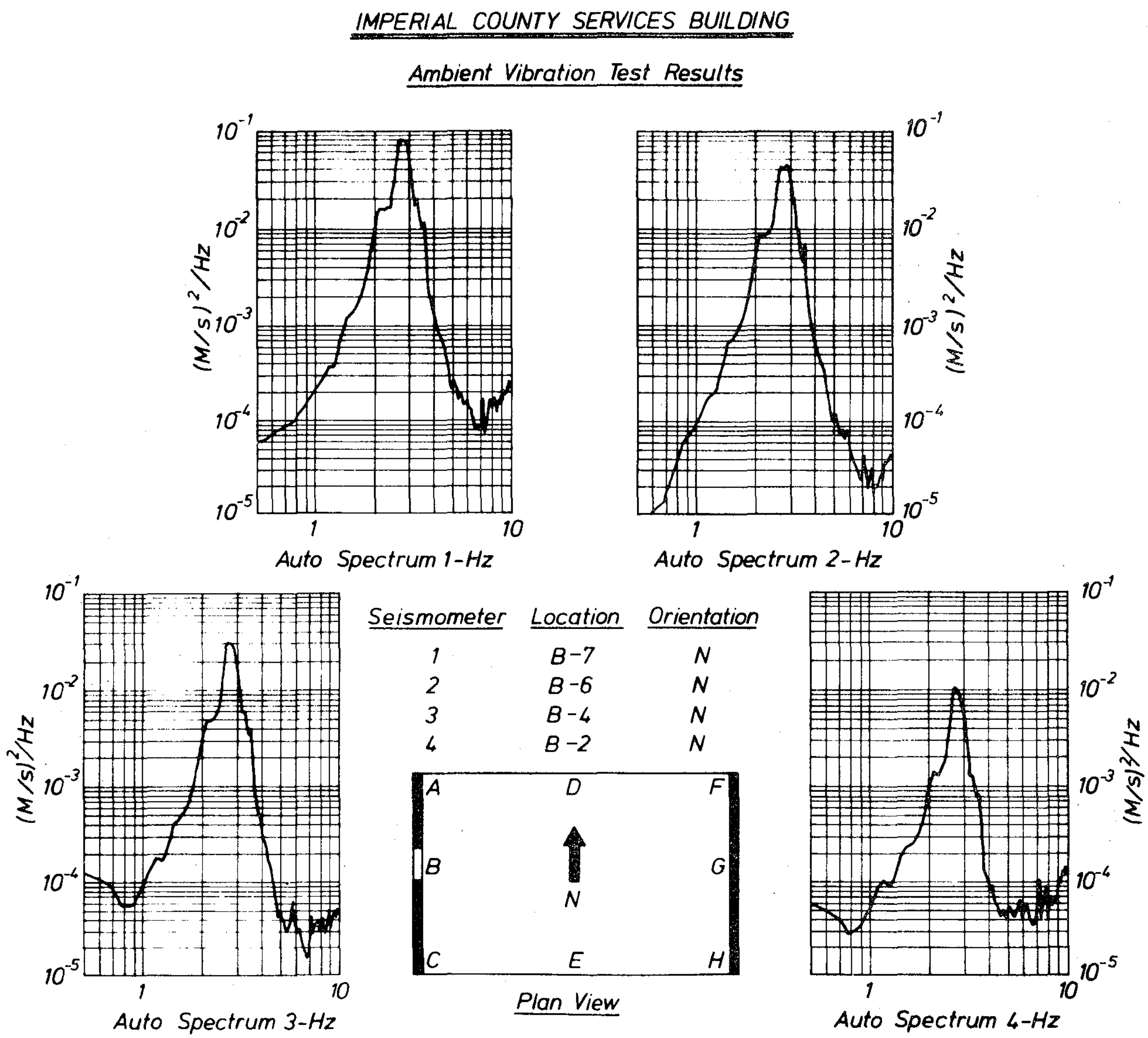


Figure 11 - N-S Power Spectrum Responses At Location B

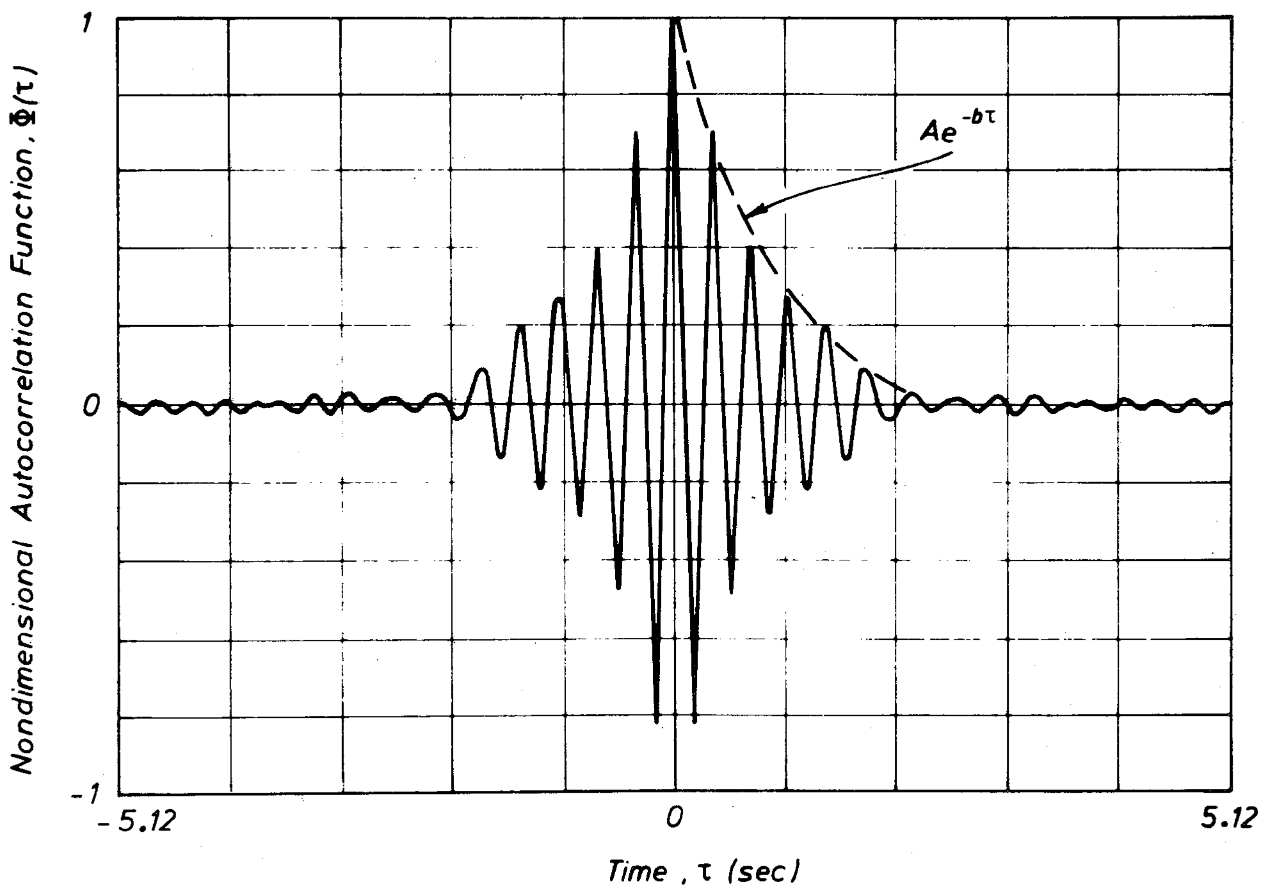
frequency determined from the power spectrum and auto-correlation would be identical. The frequency deviations indicate departures from these assumptions.

A second point is the discrepancy in frequency when the seismometers are oriented the north direction. With the seismometers positioned at points D-7, D-6, D-4, and D-2 the response energy is concentrated near 2.24 Hz whereas with the seismometers positioned at points B-7, B-6, B-4, and B-2 the response energy is concentrated near 2.81 Hz. This apparent discrepancy was resolved by investigating the response of the torsional tests in which the seismometers were oriented in a "pin wheel" fashion at roof locations A (seismometer pointed South), C (seismometer pointed East), H (seismometer pointed North), and F (seismometer pointed West). By comparing the response of the diametrically opposite seismometers via the frequency response function, it was found that the seismometers were 180° out of phase up until a frequency of 2.8 Hz at which point they became in phase. Thus the response energy concentrated near 2.24 Hz is attributed to the N-S lateral vibration whereas the response energy concentrated near 2.81 Hz is attributed to the torsional vibration.

Tests Results - Damping

The damping values associated with the ambient vibration tests of the Imperial County Services Building were computed by various means and are summarized in Table 2.

EXPONENTIAL CURVE FIT TO AUTOCORRELATION ENVELOPE



$$b = -\frac{\Sigma\tau_i \ln\Phi_i - \frac{1}{n}\Sigma\tau_i \Sigma \ln\Phi_i}{\Sigma\tau_i^2 - \frac{1}{n}(\Sigma\tau_i)^2} = 1.310hz = 2\pi f \xi$$

$$A = exp\left[\frac{b\Sigma\tau_i + \Sigma\Phi_i}{n}\right] = 1.056$$

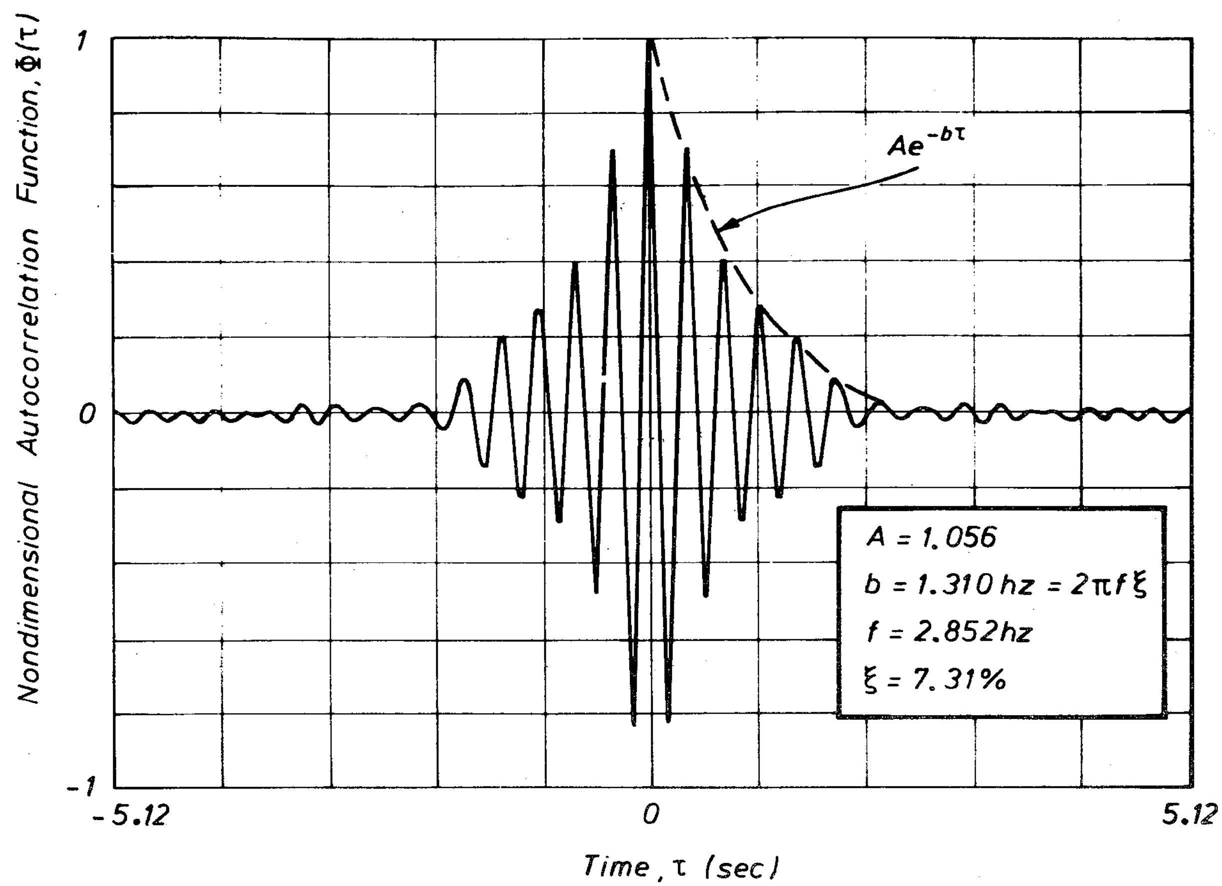
$$f = 2.852hz$$

$$\xi = 7.31\%$$

Figure 12 - Exponential Curve Fit To Autocorrelation Envelope

IMPERIAL COUNTY SERVICES BUILDING

Ambient Vibration Test Results



EXPONENTIAL CURVE FIT TO AUTOCORRELATION ENVELOPE

Figure 13 - Exponential Curve Fit To Autocorrelation Envelope

Configuration	Method	Damping (% Critical)
1	Power Spectrum Auto Correlation Random Decrement	6.42, 6.42, 6.42, 5.50 4.38, 4.78, 5.54, 5.54 *
2	Power Spectrum Auto Correlation Random Decrement	12.86, 11.96, 12.86, 17.22 7.67, 5.00, 7.14, 11.71 *
3	Power Spectrum Auto Correlation Random Decrement	9.15, 9.15, 9.15, 8.24 7.05, 8.83, 7.31, 6.48 *

Table 2 - Damping Values From Ambient Vibration Tests.

Using the power spectrum curves the damping ratio of the appropriate mode of vibration was obtained by measuring the width of the peak at half the peak value. This so-called bandwidth of the half-power point is equal to $2 \S f_n$.

Using the autocorrelation function curves the damping ratio of the appropriate mode of vibration was obtained by fitting an exponential curve to the auto correlation envelope and evaluating the appropriate curve fit parameters. The curve fit parameters[8] are depicted in Figure 12 whereas Figure 13 denotes the implementation of this curve fit procedure for seismometer 3 of test configuration 3.

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The damping value from the random decrement technique is obtained by measuring the curves' decay δ (P). It should be noted that the ambient vibration velocity data was integrated once to obtain the displacement data for the random decrement technique.

From the results it is quite apparent 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 these various techniques was to indicate the range of potential values and the variability according to the technique used.

Test Results - Mode Shape

The mode shape amplitudes were obtained by extracting the appropriate values of the transfer or frequency response function. For convenience the transfer function was defined as the ratio of the Fourier transform of a particular story level seismometer's output divided by the Fourier transform of the roof level seismometer. In actuality since an averaging process was used the numerator consisted of the averaged cross spectrum of the seismometers whereas the denominator consisted of the averaged power spectrum. The mode shape data is summarized in Table 3.

Test Configuration	Mode Shape Amplitudes			
	Roof	6th	4th	2nd
1	1.00	.62	.24	.15
2	1.00	.77	.60	.37
3	1.00	.93	.67	.28

Table 3 - Mode Shape Amplitudes From Ambient Vibration Tests

Although other ambient vibration tests were conducted no attempt is made here to fill in the amplitudes at the other story levels because it is felt that the ambient vibration tests may not be a good means of determining mode shape data. In the absence of a clearly defined input spectrum for each of the seismometers it is dangerous to put much credence in the mode shape data obtained by rationing the appropriate Fourier Components. Specifically the mode shape amplitudes may be misleading unless one could be assured that all seismometer locations were subjected to white noise or band limited white noise. The transfer function is 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.

Conclusion

The ambient vibration test results summarized in this report reflect one phase of the data base related to the Imperial County Services Building. Although these results reflect very low level excitation they do provide a means of comparison for the harmonic shaker and 15 October 79 earthquake results.

Acknowledgement

This research was sponsored by the National Science Foundation under its Research Initiation in Earthquake Hazards Mitigation Program. The support of the Foundation is gratefully acknowledged.

A special thanks go to two advisory board members of this project, Chris Rojahn of USGS and John Ragsdale of CDMG. Chris played a vital role in having the Imperial County Services Building instrumented with the strong motion accelerographs - his wisdom and foresight has now provided engineers and scientists with an excellent and significant strong motion data base for this building. John, under the direction of Tom Wootton of the office of Strong Motion Studies of CDMG, was helpful in enlisting the resources of CDMG for this project.

A particularly warm and generous thanks goes to Randy Rister, Assistant Director of Buildings and Grounds. Without Randy's enthusiasm, much less his permission to test the building, the building quite literally would not have

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APPENDIX A

Data Reduction System Description

The Zonic Technical Laboratories DMS 5003 system is the heart of the data reduction system. When used to process ambient or forced vibration data the DMS 5003 performs the following tasks for 2 channels of input data:

(a) Filters and converts the analog signals to digital form at prescribed sampling rates.

(b) Manipulates the digitized time data with a hard wired FFT micro-computer in order to calculate frequency domain functions such as autospectrum, cross spectrum, transfer function, etc.

(c) Displays up to 35 functions of processed data on the CRT.

(d) Provides a permanent copy of the processed data by
(i) displaying results on the CRT and making a hardcopy or
(ii) storing the results on a magnetic cassette tape.

The DMS 5003 is comprised of five major subsystems:

1. Data Memory System. The DMS is actually a modular stand-alone instrument generally known as a digital waveform recorder. Its purpose is to capture, store and playback time varying signals to a multitude of data display/analysis devices ranging from oscilloscopes to computers. These signals may have a duration from a few milliseconds to several seconds.

   One key to capturing data with the DMS is first estimating what digitization rate is necessary to properly define the waveform.

This rate is normally two to three times the highest frequency component in the data, although a rate of ten to twenty times may be desirable for high quality visual presentation. The Memory Control is the DMS module which allows the selection of sampling/ digitization rates. This module also contains the circuitry which determines when data are captured relative to a triggering event.

Another important consideration in digital data acquisition is duration of signal captured. This in turn determines the quantity of digital memory which must be allocated for signal storage. The Data Memory is the other main DMS module which contains the necessary storage. Data Memories, which are connected in parallel to the Memory Control, also contain individual signal conditioning, sample and hold, analog to digital convertor and digital to analog convertor. Although a Data Memory module may contain up to 4096 digital words, the DMS will utilize a maximum of 1024 words per Fourier transform.

The standard DMS utilizes a Memory Control and two Data Memories, because most structures analysis problems require a minimum of two simultaneous inputs. Due to the DMS' multichannel modularity, however, it is quite straightforward to configure an "n" channel FFT system. This approach may be taken to reduce data aquisition time, or when trigging occurs only once and analysis requires multi-location inputs.

A third DMS module, the Anti-Aliasing Filter is generally not required for conventional uses of digital wave-form-recorders. When the data captured will be analysed for frequency content, however, it is necessary to assure that the phenomenon known as aliasing does not introduce errors in the analysis. Aliasing refers to the fact that high frequency components of a time function can impersonate low frequencies if the digitization rate is too low. Figure A-1 below demonstrates this phenomenon by showing that a relatively low frequency can share the identical sample points as the sampled or higher frequency.

The Anti-Aliasing Filter is actually a dual change low pass filter device which attenuates the magnitude of any signals higher than a settable cutoff frequency. The attenuation is such that any possible aliasing, or "frequency foldback", has been reduced below the dynamic range of the Data Memory modules. These filters are also phase-matched so that the phase-lag introduced by the filtering process is equal for each channel. This is a necessary condition for dual channel analyses.

2. Microcomputer. The microcomputer is evolving rapidly in the electronic instrument field. It is being used to both replace existing technology, eg. minicomputers, as well as open new application areas. Its wide range of applicability, configuration and cost has convinced a number of equipment manufacturers to exploit this resource.

The DMS utilizes particular configurations of the Zonic Microcomputer, depending on the user's needs. All DMS microcomputers, however, require the following basic elements:

(i) CPU Board-the microprocessor chip is located on this printed circuit board and has the function of receiving, operating on, and outputting data. In this context, data may be viewed in its classical meaning or as program instructions.

(ii) Program Board (s)-instructions which determine the specific functions of the DMS are located on these boards. These instructions are physically "burned in" to an integrated circuit chip known as a Programmable Read Only Memory, or PROM.

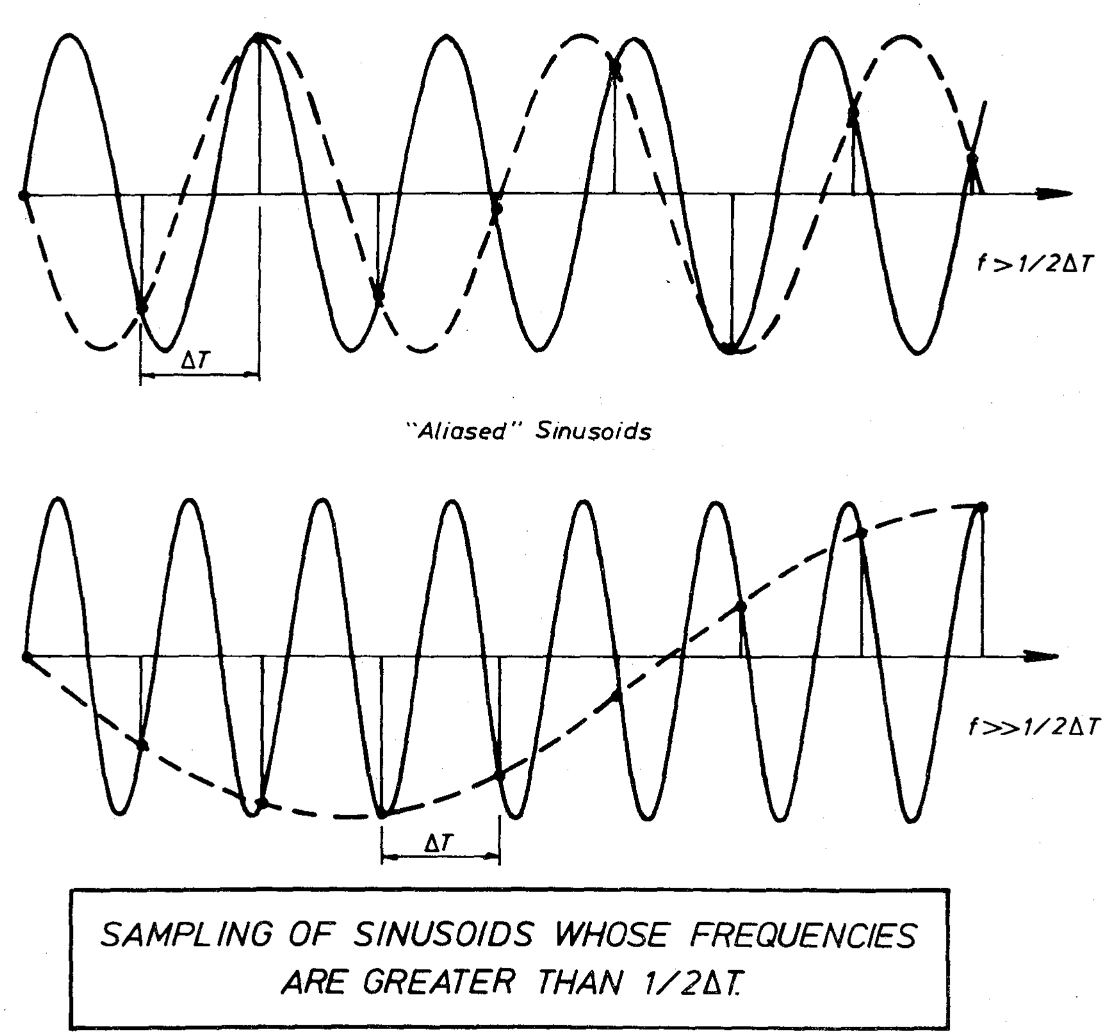

Figure A-1. - Aliasing - Sampling of Different Frequency Sinusoids Gives the Same Sampled Values time and frequency domain analyses without the need for timesharing.

5. Dual Cassette Tape Drive. The dual cassette drive provides a convenient and economical mass storage device for the DMS 5003 system. The tape unit supports storage and retrieval of averaged and unaveraged test data, batch routines (user keyboard programs), animated mode shapes, discrete point data, and epecutable machine code.

APPENDIX B

ISSN 0110—3326

RESEARCH REPORT

IMPERIAL COUNTY SERVICES BUILDING

Elastic and Inelastic Response Analyses

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December 1979 79-15

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IMPERIAL COUNTY SERVICES BUILDING

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by

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December 1979

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Elastic and Inelastic Response Analyses

Background

The success or failure of a structural design depends, to a great extent, upon the formulation of the appropriate conceptual and mathematical representation of the structure's characteristics. The design success or failure is particularly accentuated in an active earthquake zone. Success can be measured in terms of a cost effective, efficient, aesthetically pleasing design that meets or exceeds a demanding client's need while maintaining the structural integrity under severe seismic loading. Failure, on the other hand, may include overdesign, with its inherent increased costs, due to ignorance or uncertainty in order to meet severe seismic design loads or catastrophic structure failure resulting in injuries or fatalities. Inasmuch as a structure's analytical model indicates a significant part of the overall design, it is imperative, particularly in a high seismic area, that the model accurately represent the full-scale structure. One means of validating analytical procedures is to perform experimental studies of full-scale structures and compare these results with those of the analytical model.

Presently a research effort is being devoted to an in-depth experimental and analytical study of the Imperial County Services Building in El Centro, California (see Figure 1). This research project is being sponsored by the National Science Foundation under its Research Initiation in Earthquake Hazards Mitigation Program.

The experimental component of the research project has been devoted to low level structural excitations (ambient and forced vibration); however, the potential for experimental results (and the structure's inherent nonlinear behaviour) due to strong motion exists since the candidate structure is in a highly seismic area and is instrumented under the California Strong Motion

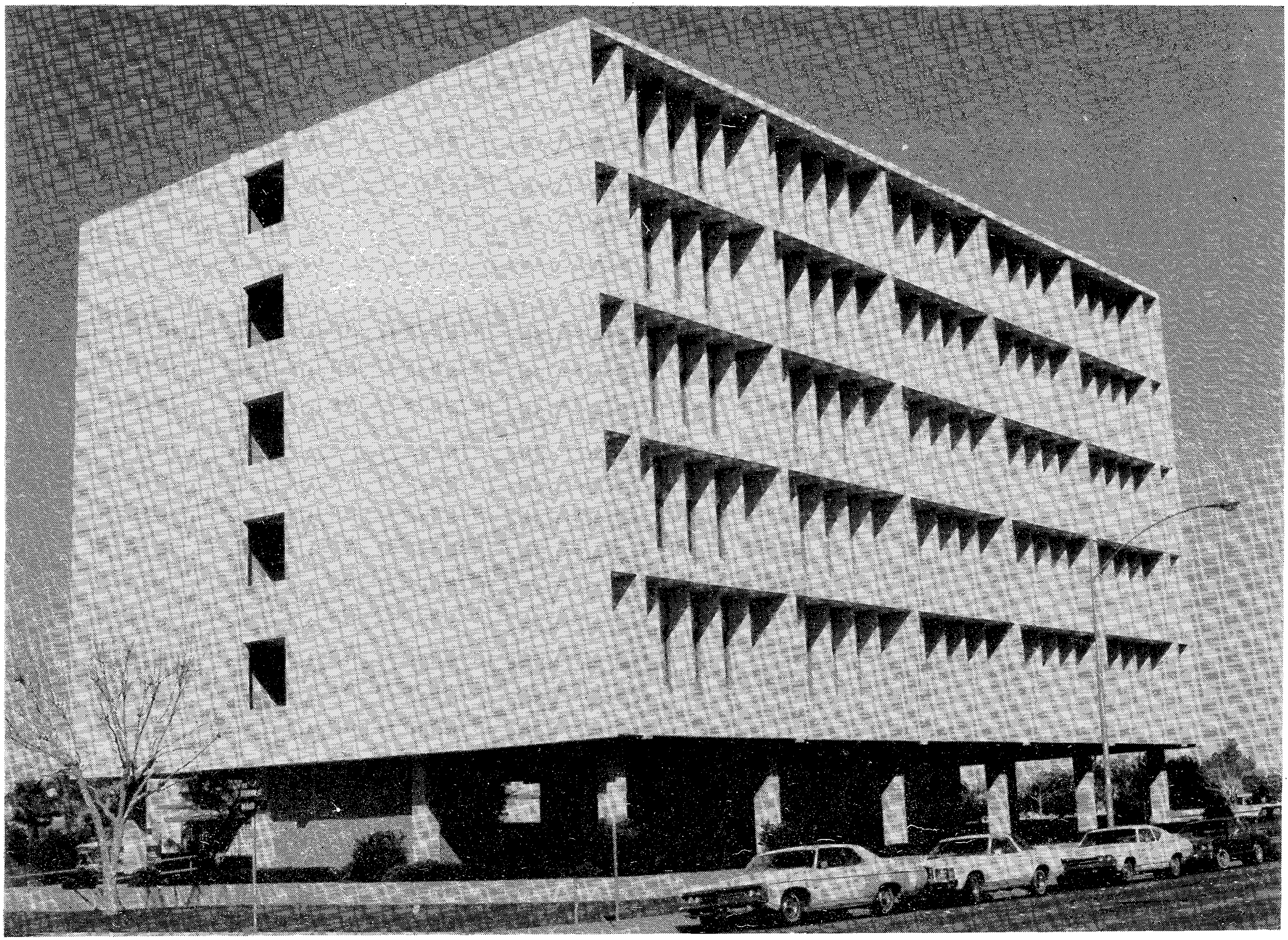


Figure 1 - Imperial County Services Building - View Looking Northeast

Instrumentation Program. On 15 October 1979 this earthquake potential became a reality when a powerful earthquake jolted Southern California and Northern Mexico causing extensive damage to the Imperial County Services Building. The earthquake, which measured 6.4 on the Richter scale, was centred on the Imperial Fault near the US-Mexican border. A brief, but informative report (see Appendix A) on the strong motion data of the 15 October earthquake was filed by Professor Paul C. Jennings of the California Institute of Technology. A more detailed report is given in Reference 1 by EERI reconnaissance team.

The analytical component of the research project is being devoted to developing a valid mathematical model to accurately represent the low level forced vibration while investigating methods and techniques needed to represent the nonlinear structural behaviour due to the strong motion records of 1934, 1940, and now 1979.

This report, one of several related to the Imperial County Services Building, is devoted to the presentation of the computer model results for the elastic and inelastic analyses. In light of the 15 October 1979 earthquake these results are being disseminated to the engineering community as rapidly as possible. However the complete investigation of the computer model results will, no doubt, be an ongoing project for engineers and researchers in the years to come.

Significance of the Imperial County Services Building

Although the correlation of an analytical model with low level forced vibration experimental results is not new, the Imperial County Services Building does offer some amenities that are unique and attractive. Consider the following characteristics of this site:

1. This particular 6-storey building has been instrumented by the California Department of Mines and Geology in accordance with its building instrumentation criteria for the California Strong Motion Instrumentation Program. The instrumentation in this reinforced concrete frame and shear wall building (see Figures 2 and 3) consists

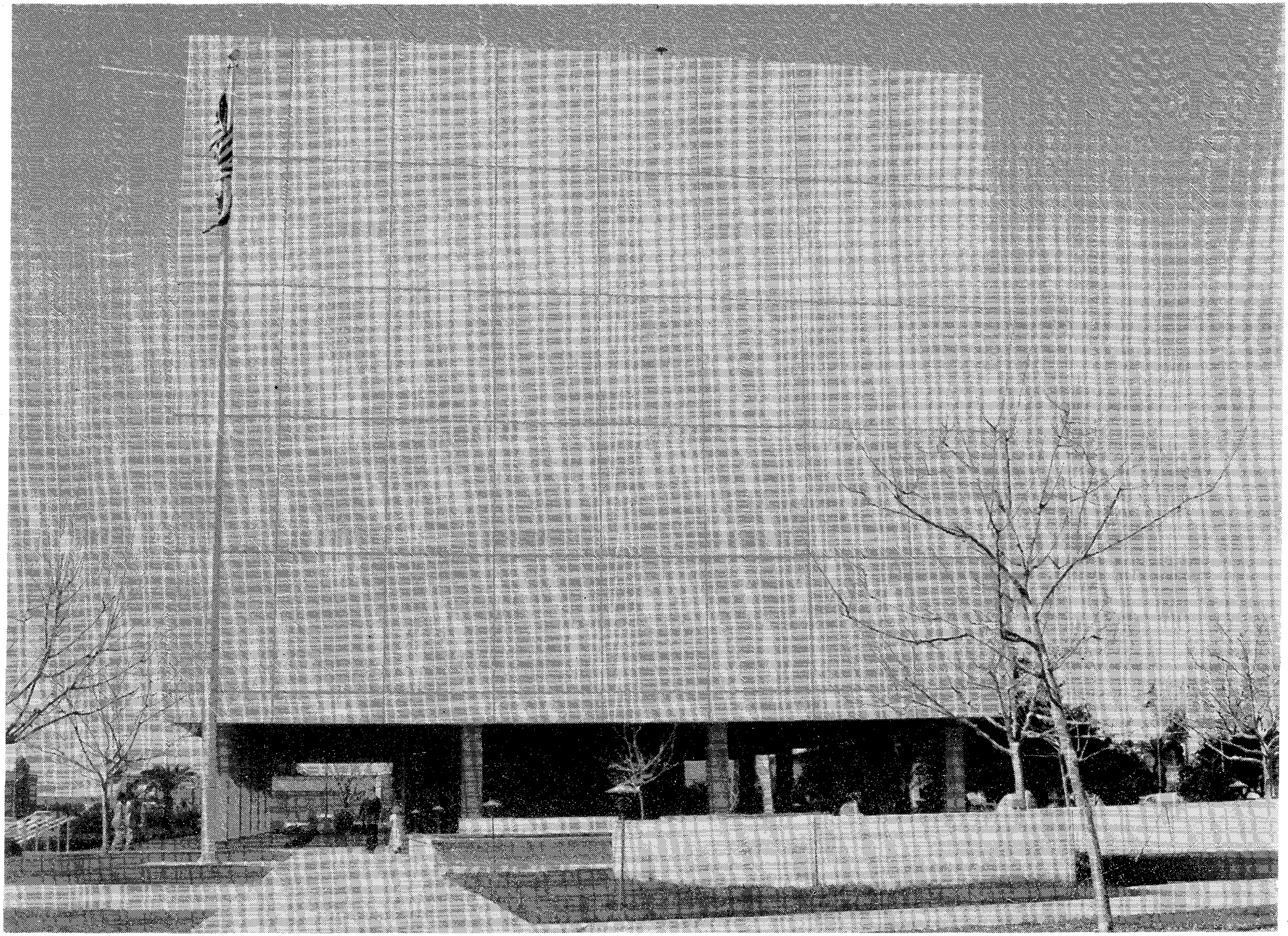


Figure 2 - Imperial County Services Building - East Shear Wall

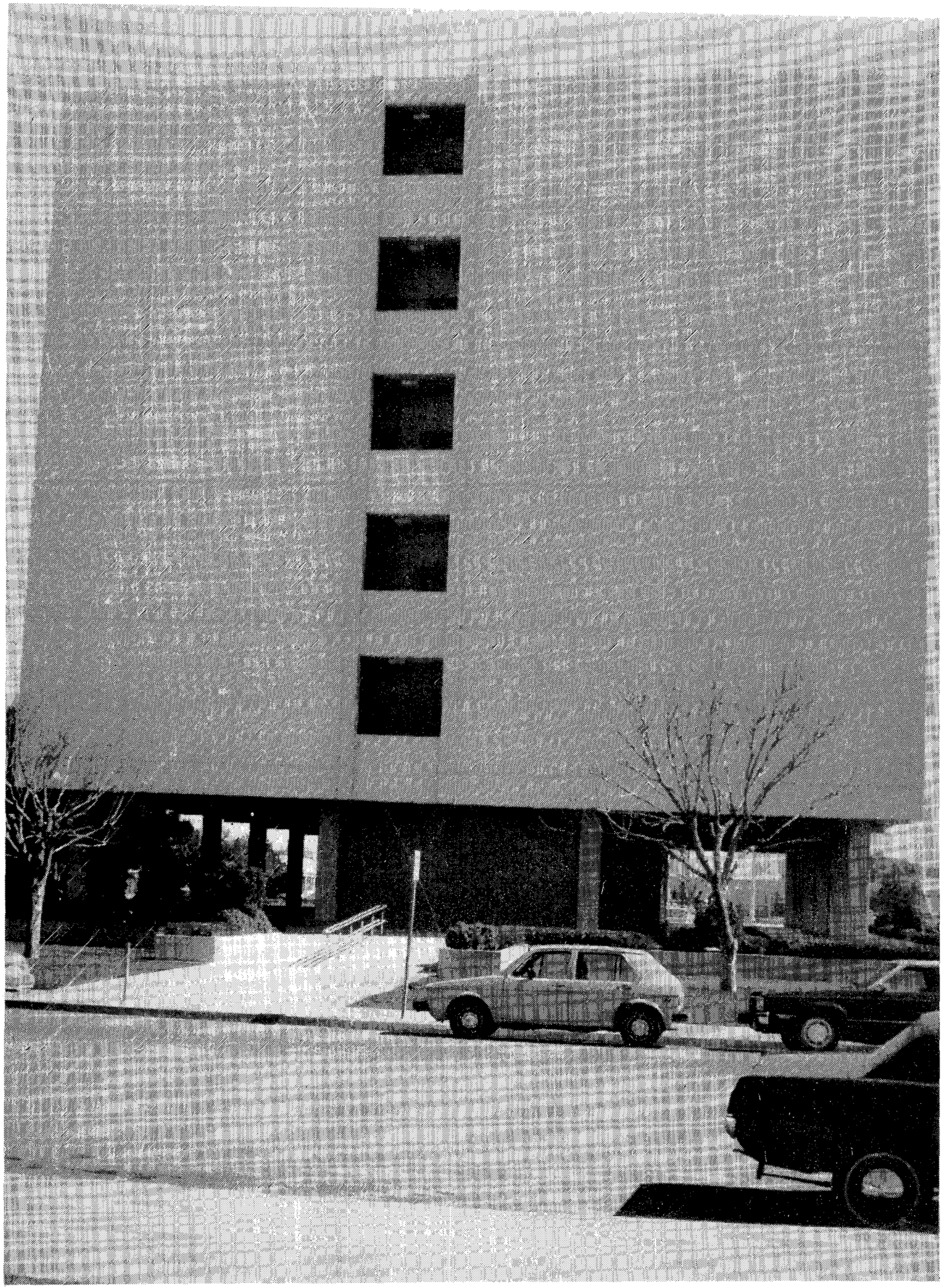


Figure 3 - Imperial County Services Building - West Shear Wall

of a triaxial package of accelerometers at ground level, four single axis horizontal accelerometers at the second floor level, one at the fourth floor, and four at the roof level. In all the building is instrumented with 13 accelerometers (see Figure 4). Lateral forces are resisted by shear walls in the N-S direction and by frame action in the E-W direction. There is a shear wall discontinuity at the second floor.

2. Approximately 104 meters east of the building is a "free field" accelerograph maintained by USGS. The location of this accelerograph is particularly significant when one considers the potential characteristics of the ground motion in soil-structure interaction.

3. El Centro is in a highly seismic region (see Figure 5). The earthquakes of 1934 and 1940 serve as benchmarks for scientists and engineers throughout the world. More recently, in November 1976, a swarm of more than 400 earthquakes occurred near Calipatria in the Imperial Valley. A total of 18 strong motion records were recovered from 7 accelerograph stations located within 32 km of the epicenter. These instruments are owned by the U.S. Geological Survey (USGS), the California Division of Mines and Geology (CDMG), and the Earthquake Engineering Research Laboratory of the California Institute of Technology (CIT). This cooperative effort includes the development of a specialized strong motion network in the Imperial Valley to fulfil such specific research needs as source-mechanism and ground-motion attenuation studies. The relatively dense instrumentation coverage in this region of recurring small - to moderate - size events provides an ideal situation in which those data necessary to implement studies can be accumulated. It should be noted that the Imperial County Services Building's instruments have triggered no less than a half dozen times since November 1976, with the last strong motion prior to the 15 October 1979 earthquake occurring on May 5, 1978.

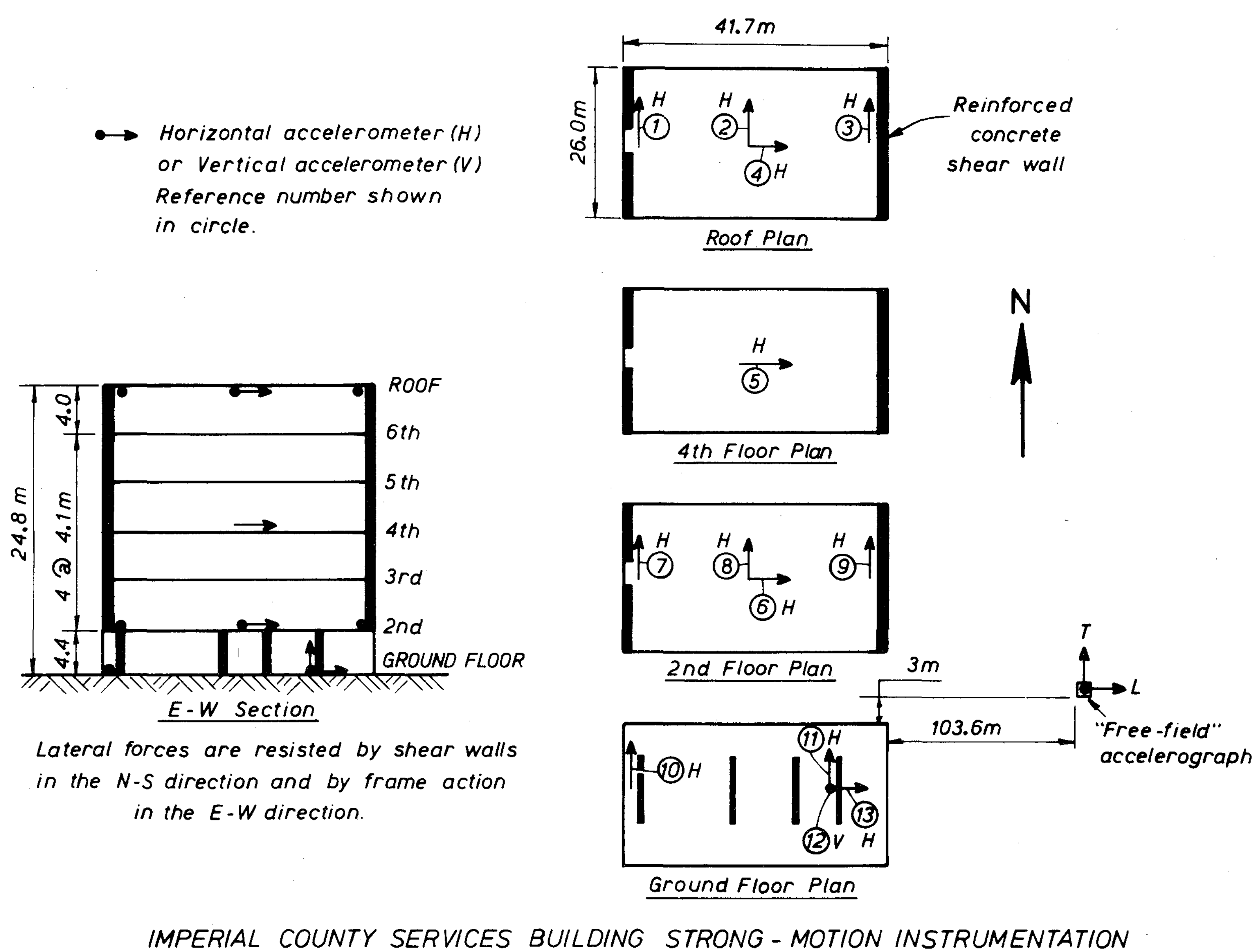
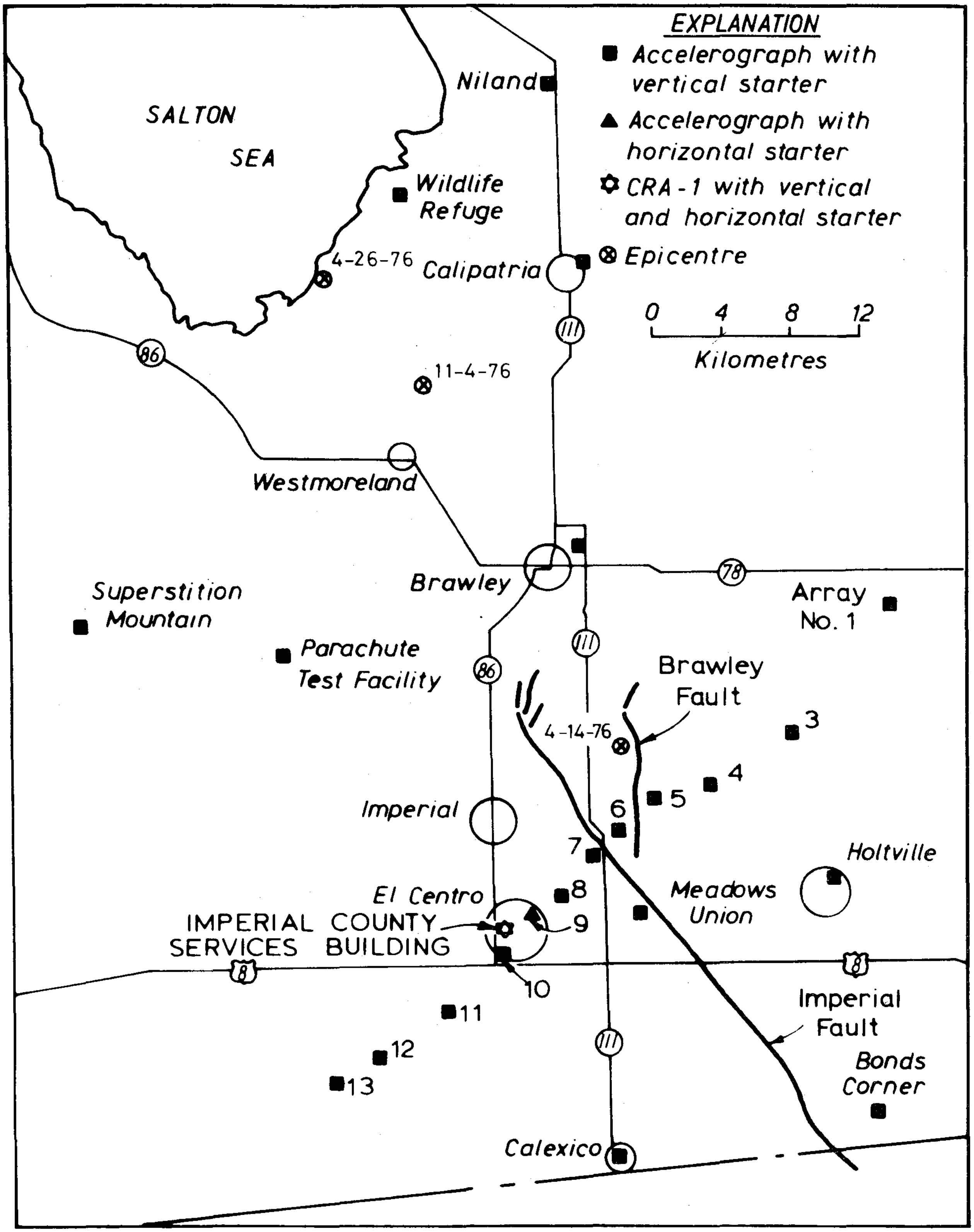


Figure 4 - Imperial County Services Building - Strong-Motion Instrumentation



STRONG-MOTION STATIONS IN THE IMPERIAL VALLEY AREA.

Figure 5 - Strong Motion Stations In the Imperial Valley Area

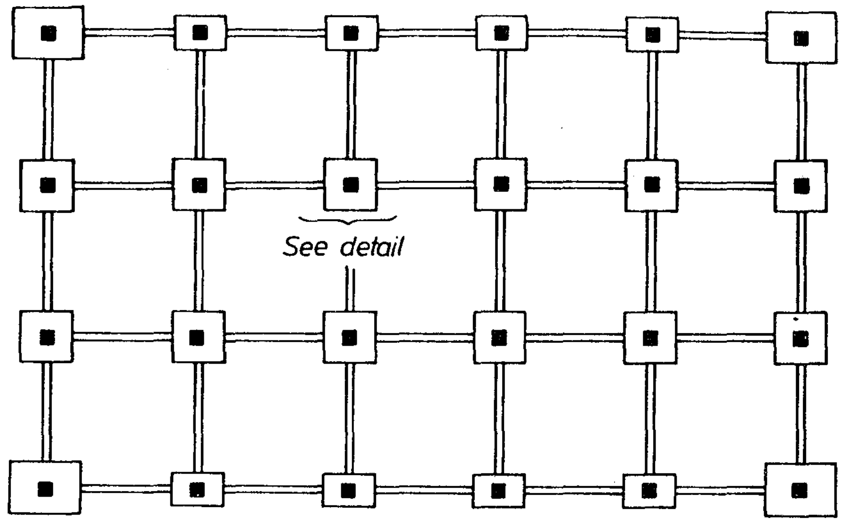
4. The candidate building has the unusual architectural feature of having the shear walls discontinuous at the second floor level. This feature does offer some analytical modeling challenges in correlating the analytical and experimental results.

Building Description[1]

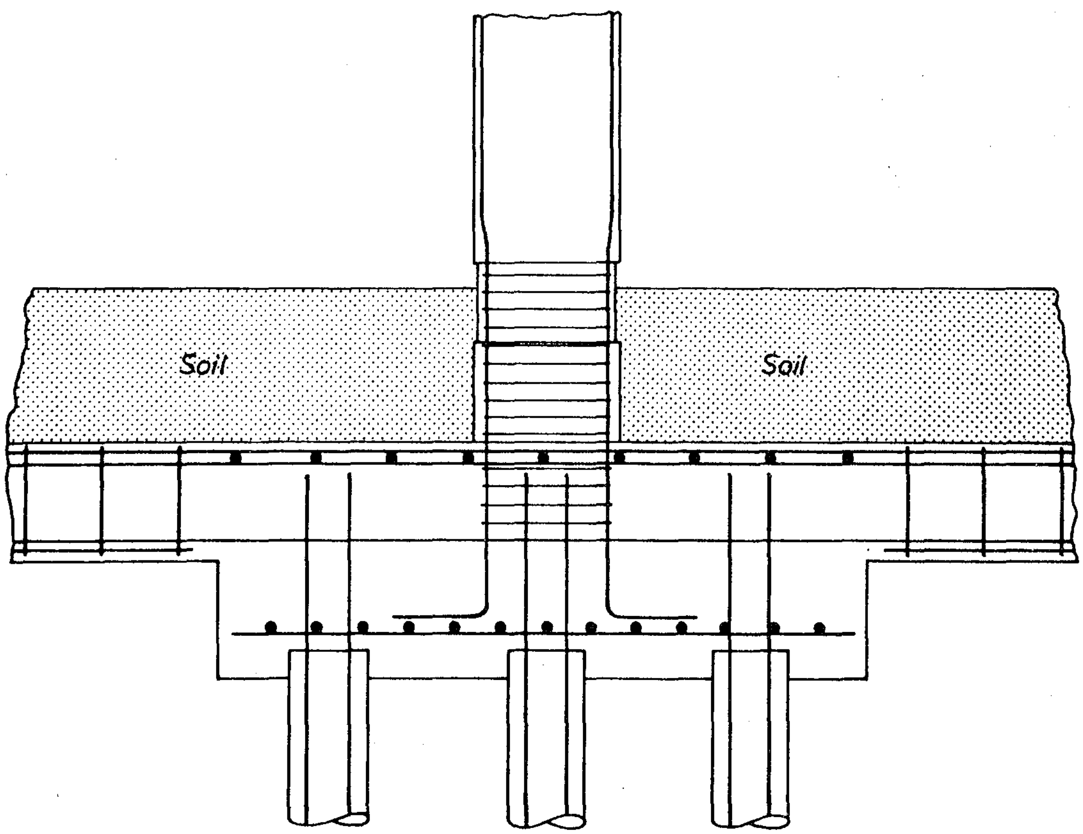
The Imperial County Services Building (ICSB) serves as an office building for Imperial County. It was designed in 1968 (using the 1967 edition of the Uniform Building Code) and was completed in 1971 at a construction cost of $1.87 million. The building is 136 feet 10 inches by 85 feet 4 inches in plan and is founded on a Raymond step-taper concrete pile foundation (see Figure 6). The piles are interconnected with reinforced-concrete link beams; they extend 45 feet to 60 feet into the alluvium foundation material composed primarily of sand with interbeds of clay to 60 feet (based on logs from 4 soil borings at the site).

Vertical loads are carried by reinforced-concrete floor slabs (5 inches thick at the second floor and 3 inches thick at the upper floors) supported by reinforced-concrete 5 1/2 inch-wide by 14 inch-deep pan joists spanning in the north-south (transverse) direction; the joists are supported by four longitudinal reinforced-concrete frames at 25 feet on center. The frame columns are typically 2 feet square, and the beams vary in size. Beams in the two interior frames are 2 feet wide by 2 feet 6 inches deep at all levels; those in the two exterior frames are 2 feet by 2 feet 5 inches at the second-floor level, 10 inches by 4 feet 5 inches at the third-floor through sixth-floor levels, and 10 inches by 4 feet 2 inches at the roof level.

Lateral loads are resisted by the four reinforced-concrete frames in the east-west direction and reinforced-concrete shear walls in the north-south direction. The shear walls are discontinuous at the second-floor level. Below the second floor are three interior and one exterior 1-foot thick shear walls, and above the second floor, shear walls exist only at the east and west ends. Between the second and third floors, the walls are 7



PILE AND FOUNDATION LAYOUT



TYPICAL PILE CAP

Figure 6 - Pile and Foundation Layout

1/2 inches thick, and above the third floor, they are 7 inches thick. According to the design calculations, the design "K" factor was 1.33 for the north-south shear walls, 0.67 for the east-west interior frames, and 1.0 for the east-west interior frames.

Scope of Analyses

With the unique opportunity of having ambient, forced, and earthquake response data of the ICSB, no doubt the ICSB is destined to become one of the most thoroughly analysed structures in earthquake history. This report documents the elastic and inelastic analyses conducted for the ICSB's analytical model due to several loading conditions.

The elastic analysis considers the ICSB as a three dimensional shear wall and framed structure subjected to the following loading conditions:

A) The 1967 uniform building code's implementation of psuedo dynamic forces by considering equivalent horizontal static loads 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 Oct 1979 Imperial County Earthquake recorded by the free field accelerograph located adjacent to the ICSB.

The inelastic analysis considers the E-W lateral load resistance of the ICSB with a planar frame model subjected to the following loading conditions:

A) The N-S component of the 18 May 1940 Imperial Valley earthquake.

B) The E-W component of the 15 Oct 1979 Imperial Valley earthquake.

A further study will consider the two dimensional inelastic response of the ICSB due to the above and other loading conditions.

Description of the Elastic Model

While the exact three dimensional structural analysis is required for only a limited number of buildings, the ETABS computer code (see Appendix B) makes two assumptions that greatly simplify the preparation of input data and significantly reduces the computational effort while retaining the inherent characteristics of a 3-D model.

The assumption that the floors of a typical building are rigid in their own plane is a realistic approximation for most buildings. Although the floors are rigid in their own plane, the bending deformations in the horizontal beams and in the floor slabs can be included (see Figure 7). The second simplification assumes that the horizontal lateral loads act at the floor levels. Thus the lateral loads are transferred to the columns and shear wall elements through the rigid floor diaphragms. These assumptions reduce the computational complexity of the analytical model to that of three degrees of freedom at each floor level - translations in the horizontal plane and a rotation about the vertical axis.

In defining the analytical model of the ICSB for the ETABS code, the complete building is assumed to be composed of structural elements which can 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 are 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 building is treated as an independent substructure in which the complete structure stiffness matrix is then formed under the assumption that all frames are connected at each floor level by a diaphragm which is rigid on its own plane.

Each joint of the structure has six degrees of freedom (displacement in, and rotation about, each coordinate axis). Within each frame three of these

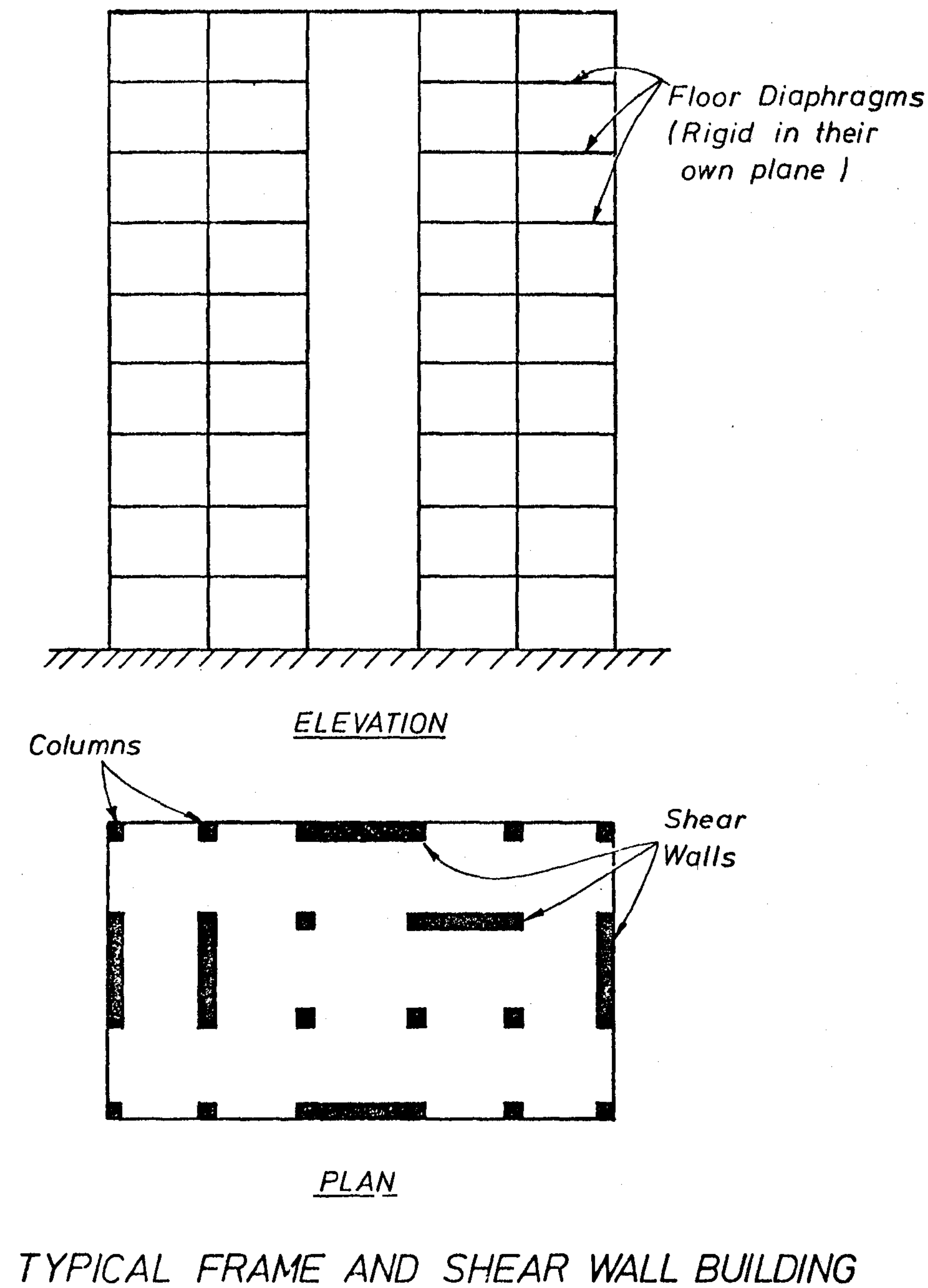


Figure 7 - Typical Frame and Shear Wall Building

degrees of freedom (the two horizontal translations and the rotation in the floor plane) can be transformed, using the assumption of the rigid in plane floor diaphragms, to the frame degrees of freedom at that floor level (see Figure 7). The three remaining degrees of freedom are eliminated by static condensation before each frame stiffness is added to the total structural stiffness matrix. Thus the final structural stiffness matrix corresponds to the three degrees of freedom per floor level.

Some of the significant aspects of the ETABS model of the ICSB include:

A) The North exterior frame, the South exterior frame, as well as the East and West discontinuous shear walls, are assumed to act as a single continuous structural unit. This unit is designated as Frame 1 for the ETABS model. The East discontinuous shear wall is sectioned to account for the fire escape (see Figure 8).

B) The North interior and South interior frames are assumed to be identical. The structural properties representing these frames are designated as frame 2 (see Figure 8).

C) The single story shear walls at column lines B,D, and F are denoted by the frame 3 properties (see Figure 8).

D) The single story shear wall at column line E is denoted by the frame 4 properties (see Figure 7).

E) Since deformations within joints are neglected, the effective length of both the beams and columns are reduced by the 'rigid end zone' lengths associated with each structural component (see the shaded areas of Figure 9). The 'rigid end zones' for the beams are calculated to be equal to half the width of the columns below. The top 'rigid end zone' for the columns are taken as the average depth of the girders on either side whereas the bottom 'rigid end zone' for the columns are taken as zero.

F) The concrete strength of the columns is taken as 4000 psi whereas the concrete strength of the beams and shear walls is taken as 3000 psi. The basic sectional properties of the beams and columns are given in Table 1. Note the frame numbers refer to the R/C frames shown in Figure 10 and not the frame numbers of the analytical model shown in Figure 8.

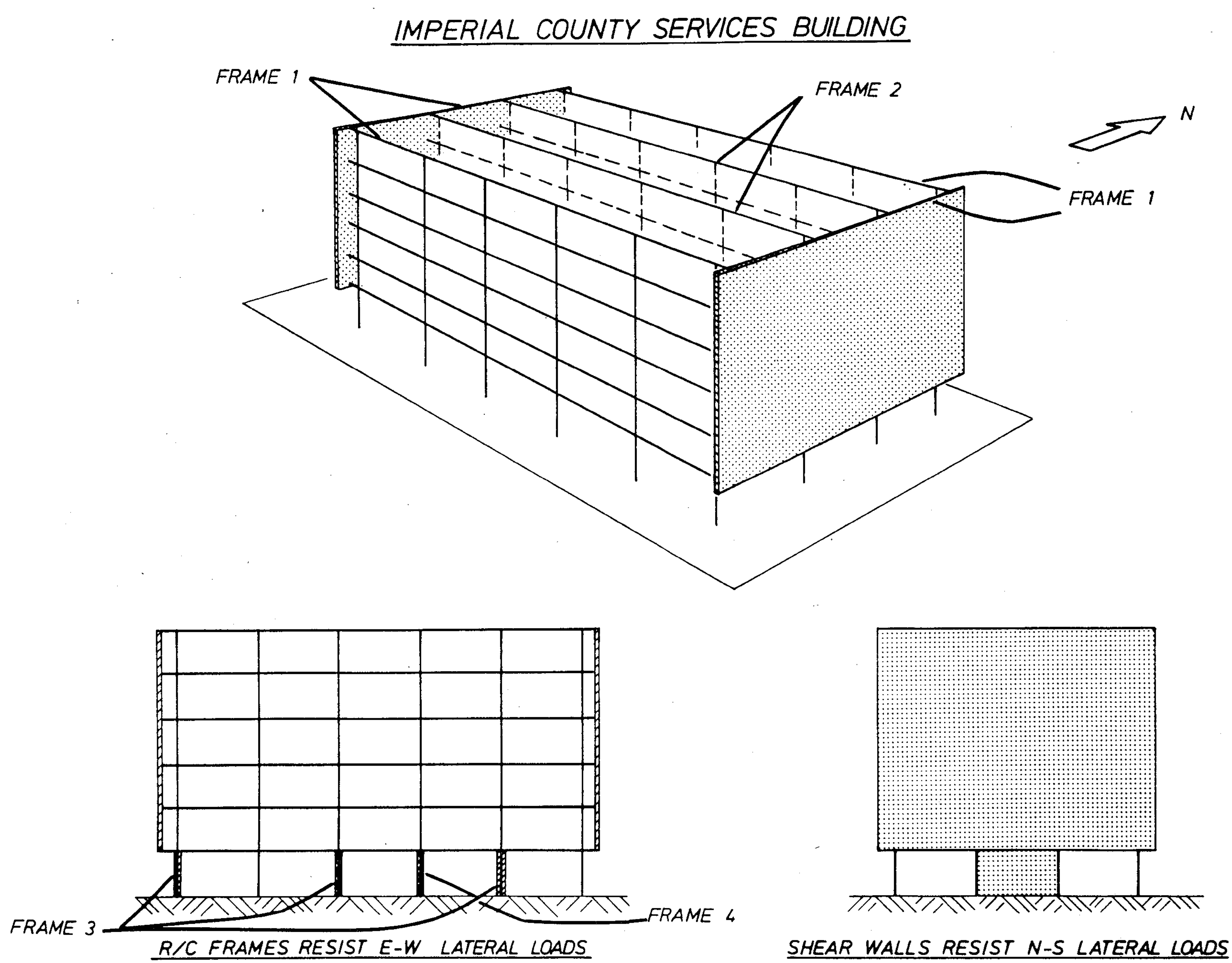


Figure 8 - ETABS Model of Imperial County Services Building

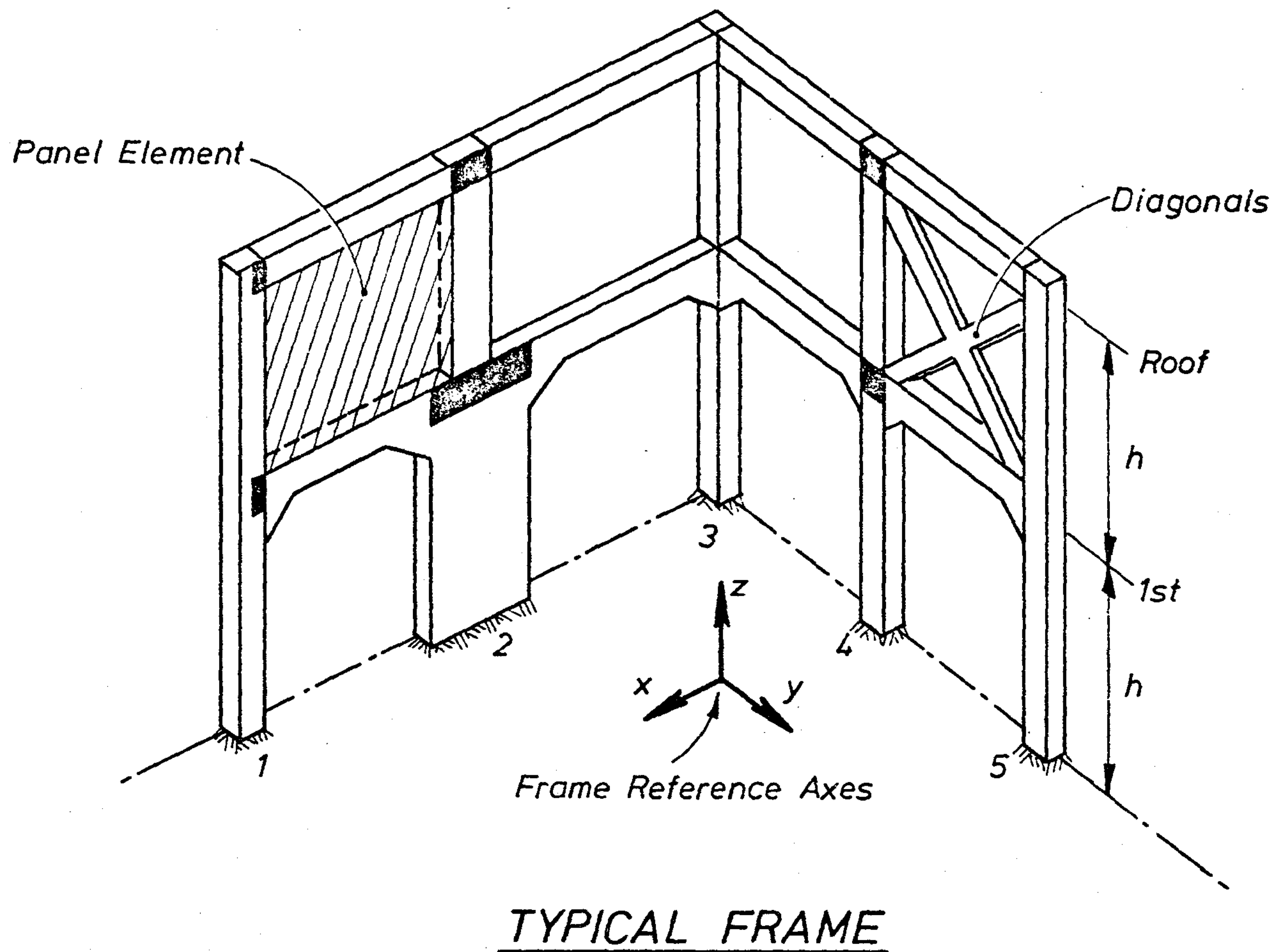
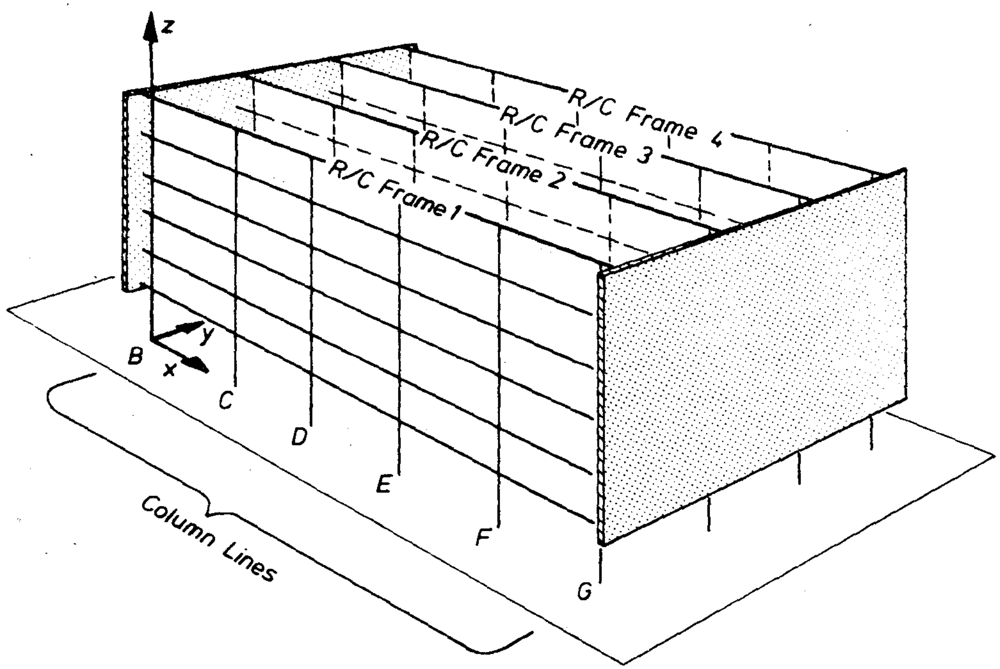
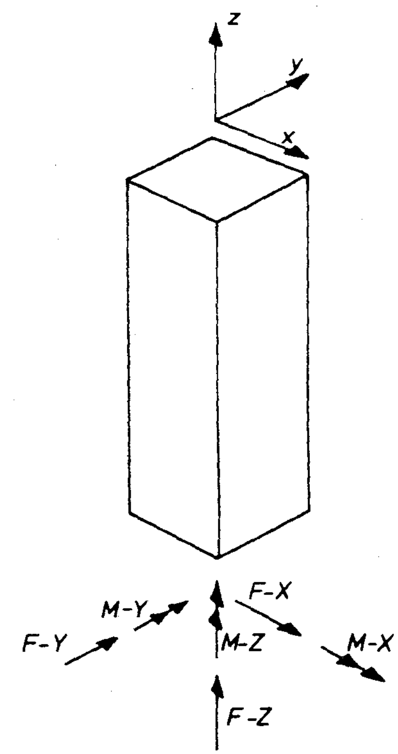


Figure 9 - Typical Frame With Rigid End Zones

IMPERIAL COUNTY SERVICES BUILDING



COLUMN DESIGNATIONS



GROUND LEVEL COLUMN FORCES

Figure 10 - Identification of Columns and Column Forces

IMPERIAL COUNTY SERVICES BUILDING

BEAM PROPERTIES

Frame	Floor Level	Iyy	Rigid End Zone at Each End
1 & 4	2	54000	9
1 & 4	3 → 6	124064	9
1 & 4	Roof	104167	9
2 & 3	2 → roof	54000	12

COLUMN PROPERTIES

Frame	Storey Level	A	Ixx	Iyy	Top Rigid End Zone
1 & 4	1	576	27648	27648	30
1 & 4	2 → 5	1028	420917	20177	53
1 & 4	6	1028	420917	20177	50
2 & 3	1	576	27648	27648	30
2 & 3	2 → 5	576	27648	27648	53
2 & 3	6	576	27648	27648	50

(Units: $A = in^2$, $I_{xx}, I_{yy} = in^4$, Rigid End Zone = in)

TABLE 1

G) The second floor slab thickness is 5 inches whereas the slab thickness for the remaining floors is 3 inches. The roof slab thickness is nominally 3 inches except for the Penthouse area which has a thickness of 6 inches.

H) The shear walls have a thickness of 7.5 inches from the second floor to the third floor and a 7 inch thickness elsewhere.

Assumptions of the Elastic Model

In addition to the inherent capabilities and limitations of the ETABS computer code certain stiffness and mass assumptions are associated with the ICSB analytical model. These assumptions include:

A)An uncracked concrete cross section is used for all beam, column, and shear wall properties.

B) Section properties are based solely upon the concrete, i.e., no transformed section properties are used to account for the reinforcing steel.

C) The center of mass for each floor is assumed to be at the geometric centre.

D) The rotational inertia of each floor is assumed to be based upon a uniformly distributed mass on a rectangular area.

E) The weight of the Penthouse structure is assumed to be 20 kips whereas the weight of the Penthouse machinery is assumed to be 50 kips.

Inasmuch as the mass properties are probably the most variable quantities in modelling the ICSB by different researchers, the detailed mass property calculations are offered in the spirit that other researchers can confirm or deny these mass property assumptions. See Appendix C for details.

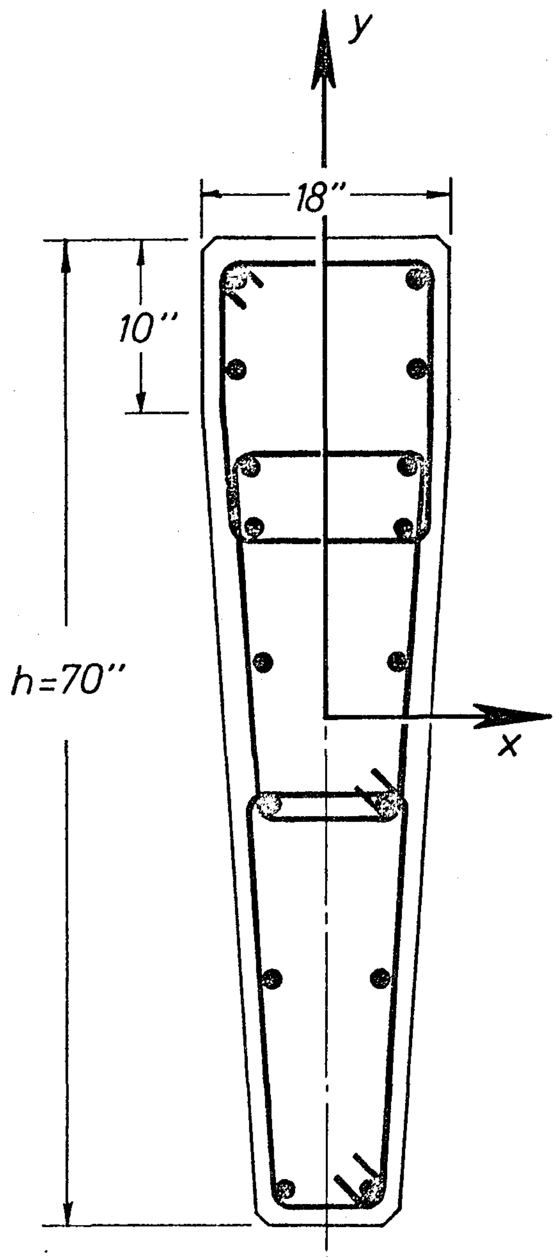
In addition to the aforementioned stiffness and mass property assumptions, there is still enough latitude in modelling assumptions that one can "fine tune" the analytical model to achieve certain response characteristics. As such the analytical model developed for the ETABS computer code was the one which most nearly represented the experimentally derived frequencies[3]. Specifically the fundamental frequencies predicted by the analaytical model depicted in Figure 8 and those derived experimentally were

Direction	Analytical	Experimental
N-S	2.27 Hz	2.25 Hz
E-W	1.49 Hz	1.54 Hz

As a measure of the analytical model's sensitivity to reasonable modelling assumptions, one need only consider the effect of the exterior frames' columns on the fundamental frequency. The reinforced concrete columns from the ground level up to the 2nd floor are 2 feet square in cross section whereas the columns from the 2nd floor level up to the roof are somewhat trapazoidal shaped. The bases of the trapazoid are approximately 10" and 18" whereas the depth is approximately 70" (see Figure 11). For such a cross section one might be tempted to consider an "effective depth" of the column for resisting bending about the X-axis. For instance does one choose h = 70" or some smaller dimension such as h = 40" (as the original ICSB design calculations indicate)? While it is recognised that certain local discontinuities would occur near the junction of the two different column cross sections at the 2nd floor level, it is felt that h = 70" probably represents the bending rigidity of the columns above the 2nd floor level and, as such, is the cross sectional property used in the analyses. Nevertheless one can see from the table of frequencies in Figure 11 that there is a certain degree of response sensitivity due to just the exterior column modelling assumption.

Additional stiffness parameters could equally alter the fundamental frequency results. For instance, the original design concrete strengths of 4000 psi and 5000 psi were specified for the

EFFECT OF EXTERIOR COLUMN PROPERTIES ON VIBRATION CHARACTERISTICS



h	I_{xx}	I_{yy}	E-W Frequency	N-S Frequency
70"	420917	20177	1.49 Hz	2.27 Hz
40"	86994	15653	1.47 Hz	2.14 Hz
18"	8620	8553	1.37 Hz	1.81 Hz

Figure 11 - Typical Exterior Column

beams and columns respectively. However after 8 years of construction one could expect these 28 day strengths to be easily 20% higher which would affect the elastic modulus by about 10% which, in turn, would effect the fundamental frequencies by about 5 %. In an effort to avoid a plethora of analytical models, one "base line" model was chosen in order to interpret the different responses due to different input loads. Thus no sensitivity analysis of the analytical model due to a single load condition is reported within this document. Certainly the trends from the different load conditions should still be apparent for analytical models which deviate somewhat from the "base line". Thus the ETABS analytical model representing the ICSB is depicted in Figure 8 and is characterized by the stiffness parameter in Table 1 and the mass properties in Appendix C. For completeness a card image of the ETABS data deck representing the ICSB is given in Appendix D. This appendix should be useful to those who chose to compare elastic and inelastic results reported within this report.

Description of Load Conditions

The ETABS analyses of the ICSB consists of 7 load conditions - 3 static and 4 time history analyses. The static analyses reflect the two lateral load conditions due to the 1967 Uniform Building Code requirements and a vertical load condition due to the structure's dead weight. The time histories reflect the appropriate components of the 1940 and 1979 Imperial Valley earthquakes considered in the N-S or E-W directions of the ICSB. 5% critical damping is assumed for all time history analyses.

The 1967 UBC[4] requires a static analysis to be conducted for the building due to the effect of a force applied horizontally at each floor or roof level above the foundation. The force should be assumed to come from any horizontal direction. The code requires that the lateral force at the jth level be determined from

$$F_j = (V - F_T)\, w_j h_j \Big/ \sum_{i=1}^{n} w_i h_i \qquad (1)$$

where

F_j = force at jth level

V = base shear

F_T = portion of V considered concentrated at the top of the structure (equal to 0 for the ICSB)

w_j = portion of the total dead load which is located at level j

h_j = height (in feet) above the base to the level j

n = number of levels to the uppermost level in the main portion of the structure.

The base shear has had an interesting evolution[5] since seismic and wind effects were first included in the 1924 Los Angeles building code. In 1967 the base shear was defined by UBC as

$$V = Z\,K\,C\,W \qquad (2)$$

where

Z = zone factor

K = ductility factor

C = numerical coefficient = $.05/\sqrt[3]{T}$

T = building period

W = total dead weight of the structure

Inasmuch as the period T and ductility factor K differ for the N-S and E-W directions of the ICSB, then there is a base shear V which corresponds to the N-S direction and a different one corresponding to the E-W direction. The lateral forces were derived from the following basic data:

Z = 1.0

K = 1.0 for E-W direction; 1.33 for N-S direction

C = .0577 for E-W direction; .0655 for N-S direction

T = (1/1.54 Hz) for E-W direction; (1/2.25 Hz) for N-S direction

W = 10420 Kips (from Appendix C)

Thus the base shear for the E-W direction is V = 601 Kips and the base shear for the N-S direction is V = 907 Kips. Implementation of equation (1) gives the distribution of lateral forces for the N-S and E-W directions as

Storey Level	j	F_j	F_{E-W}	F_{N-S}
Roof	6	.2280 V	137.0 K	206.9 K
6th Floor	5	.2529 V	152.0 K	229.4 K
5th Floor	4	.2030 V	122.0 K	184.3 K
4th Floor	3	.1532 V	92.1 K	139.0 K
3rd Floor	2	.1038 V	62.4 K	94.2 K
2nd Floor	1	.0591 V	35.5 K	53.7 K

The vertical load condition consists of approximating the dead load of each storey with a uniformly distributed load on the beams at each storey level.

The time histories considered were the first 15 seconds of the 1940[6,7] (N-S component) and the 1979[8] (E-W and N-S components) Imperial Valley earthquakes. These three time histories are depicted in Figures 12 - 14. It should be noted that the 1979 records represent the free field accelerograph response which was located approximately 100 meters east of the ICSB.

Discussion of Elastic Analyses Results

The elastic analyses results provide one with some 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 behaviour 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. While no attempt will be made to tabularize the force resultants for all structural members due to all load conditions, attention will be devoted to the significantly loaded

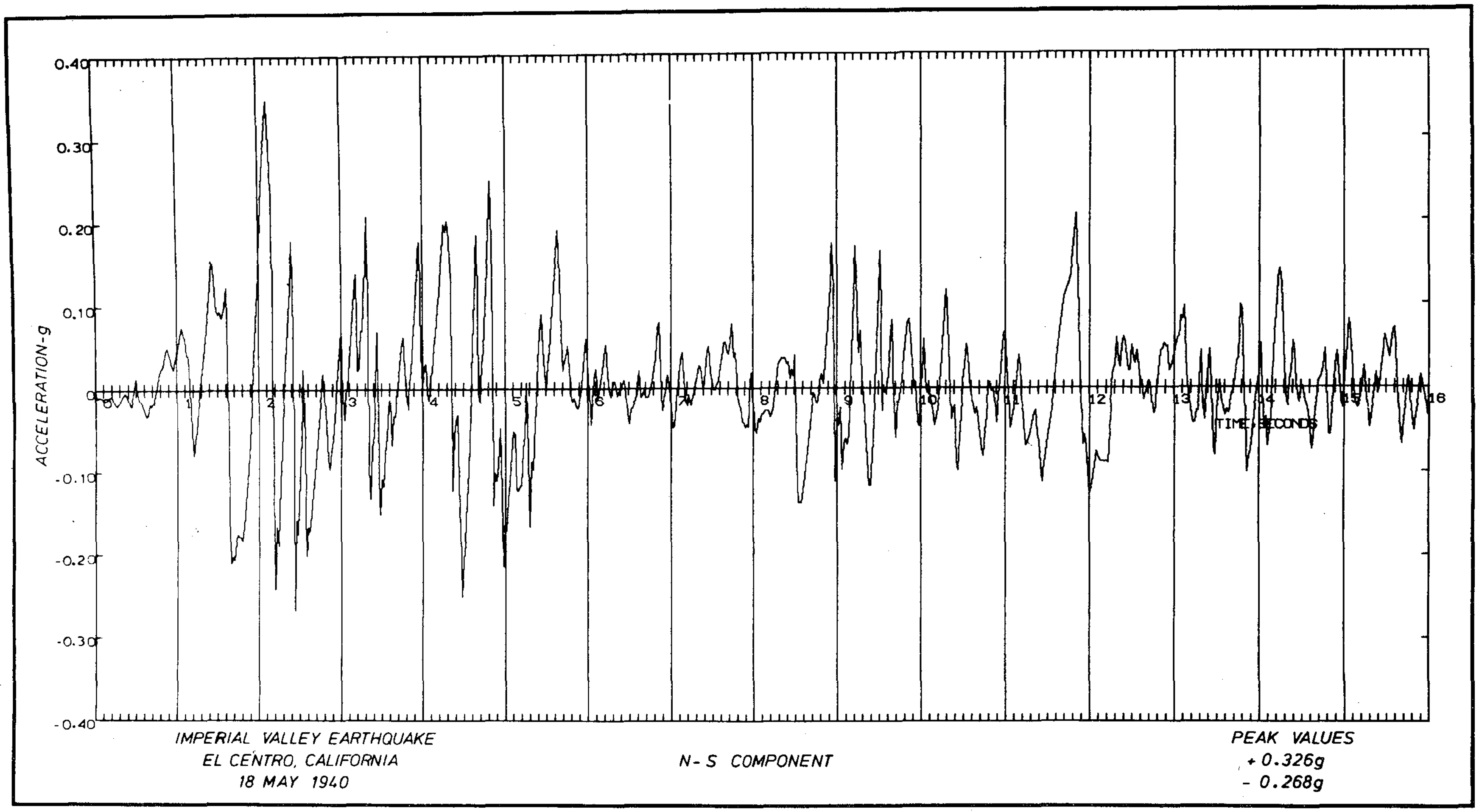


Figure 12 - 1940 Imperial Valley Earthquake - NS Component

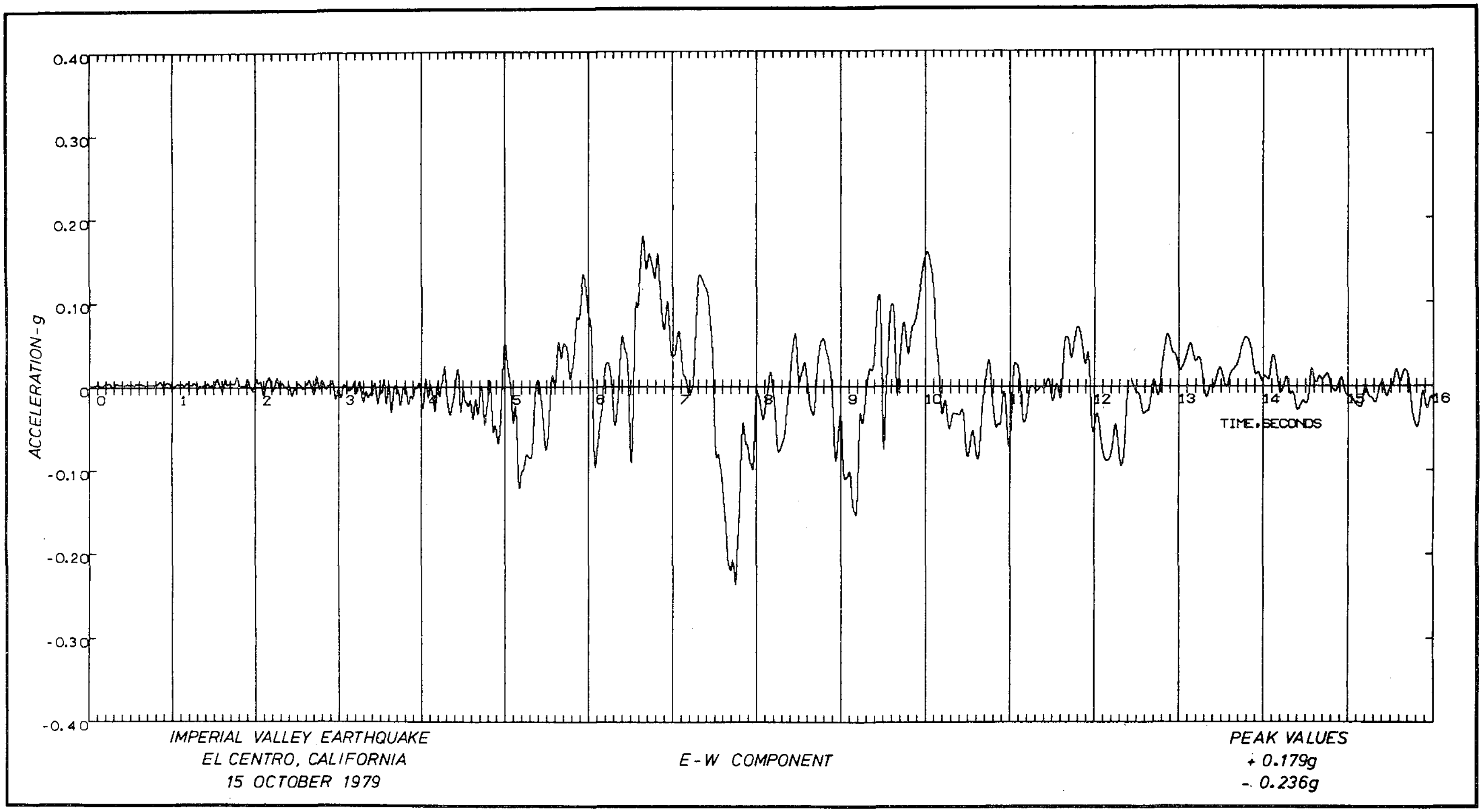


Figure 13 - 1979 Imperial Valley Earthquake - EW Component

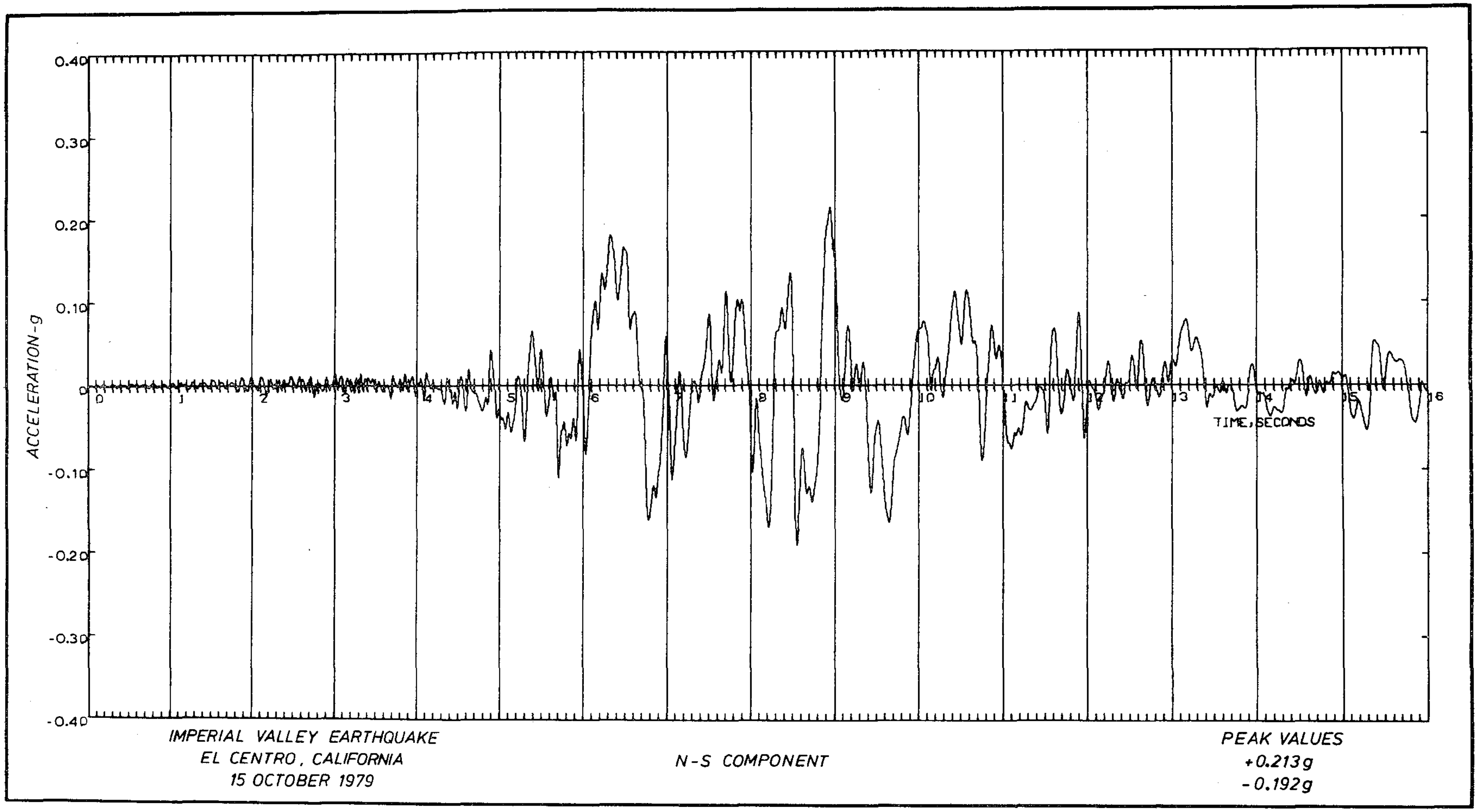


Figure 14 - 1979 Imperial Valley Earthquake - NS Component

members due to any particular load condition. However due to the catastrophic failure of the 4 columns along column line G (see Figure 10), particular attention is given to the ground level columns due to the various load conditions.

In determining the moment carrying capacity of the beams for the ICSB one must consider the steel reinforcement pattern throughout the building. In general the ultimate bending moment capacity of the beam can be found from[9]

$$M_u = .85f'_c ab\ (d - a/2) + A'_s f'_s\ (d - d') \qquad (3)$$

where the appropriate dimensions are detailed in Figure 15 with

M_u = ultimate moment

f'_c = concrete compressive strength

a = fraction of distance to centroid

b = beam width

d = distance of tension steel from beam surface

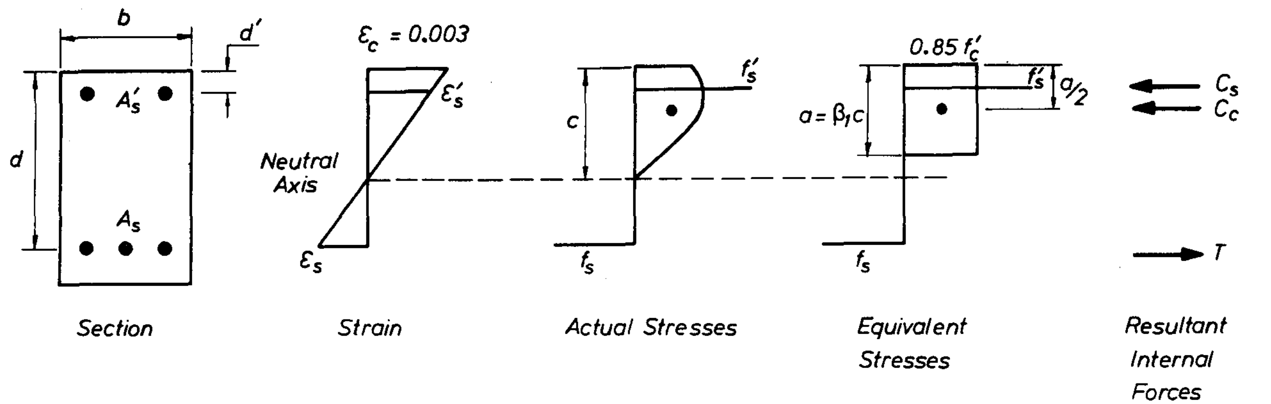
d' = distance of compression steel from beam surface

A'_s = area of compression steel

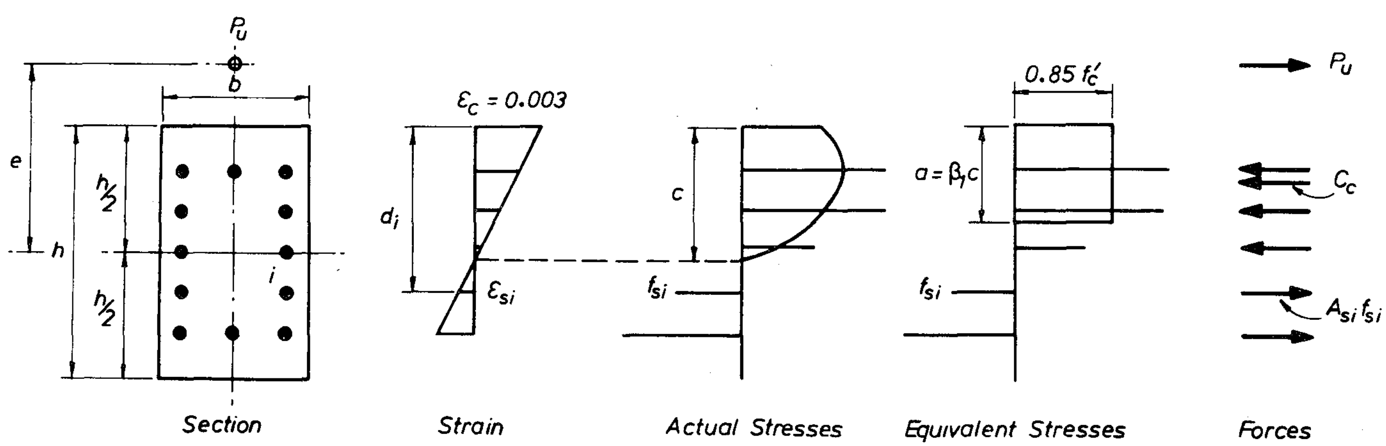
f'_s = stress in compression steel at ultimate moment

The ultimate moment cannot be found explicitly by direct substitution into Equation (3) because the neutral axis is undefined. Assuming a maximum strain of .003 in the concrete and a stress in the tension and compression steel no greater than its yield strength enables one to determine the magnitude and location of the concrete and steel force resultants shown in Figure 15. Using f'_c = 4000 psi concrete, the ultimate bending moment for the girders of the ICSB were determined and are displayed in Table 2. Note that for each end of the beam there are two moment values - one corresponding to a positive and negative moment. It should be noted that the ultimate moment values in Table 2 reflect the implementation of Equation (3) - no attempt has been made to account for the in-situ concrete strength

FLEXURAL STRENGTH OF BEAMS AND COLUMNS



DOUBLY REINFORCED CONCRETE SECTION WHEN THE FLEXURAL STRENGTH IS REACHED



ECCENTRICALLY LOADED COLUMN SECTION WITH BARS AT FOUR FACES AT THE ULTIMATE LOAD.

Figure 15 - Flexural Strength of Beams and Columns

TABLE 2

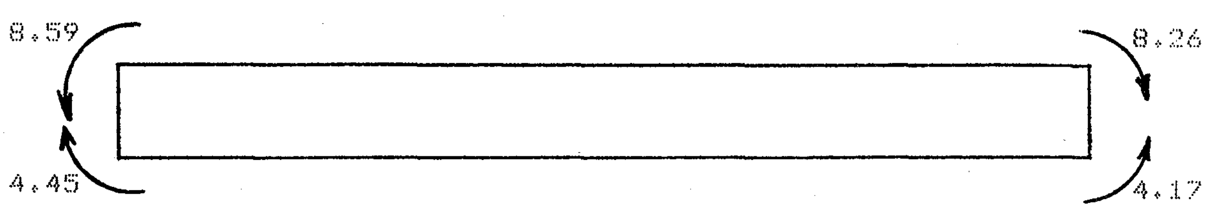
IMPERIAL COUNTY SERVICES BUILDING

MAXIMUM POSITIVE AND NEGATIVE ULTIMATE BEAM BENDING MOMENTS

FRAME	FLOOR	BEAM BC		BEAM CD		BEAM DE		COMMENT
EXT	ROOF	8.70	8.70	5.81	5.81	5.81	5.81	SYMMETRIC
		4.74	4.74	4.74	4.74	4.74	4.74	
EXT	6TH	8.53	8.53	6.18	6.18	6.18	6.18	SYMMETRIC
		5.05	5.05	5.05	5.05	5.05	5.05	
EXT	5TH	8.53	8.53	6.18	6.18	6.18	6.18	SYMMETRIC
		5.05	5.05	5.05	5.05	5.05	5.05	
EXT	4TH	8.72	8.72	7.00	7.00	7.00	7.00	SYMMETRIC
		5.05	5.05	5.05	5.05	5.05	5.05	
EXT	3RD	9.44	9.44	7.73	7.73	6.18	6.18	SYMMETRIC
		5.05	5.05	5.05	5.05	5.05	5.05	
EXT	2ND	8.59	8.26	8.26	7.67	7.67	7.67	SYMMETRIC
		4.45	4.17	4.17	4.17	4.17	4.17	
INT	ROOF	5.09	5.09	5.09	6.73	6.73	6.73	SPAN EF MOMENTS
		3.67	3.67	3.67	3.67	3.67	3.67	6.73 & 3.88
INT	6TH	6.73	6.73	6.73	6.73	6.73	6.73	SYMMETRIC
		3.67	3.67	3.67	3.67	3.67	3.67	
INT	5TH	6.73	6.73	6.73	6.73	6.73	6.73	SYMMETRIC
		3.67	3.67	3.67	3.67	3.67	3.67	
INT	4TH	7.38	7.38	7.38	7.38	7.38	7.38	SYMMETRIC
		4.09	4.09	4.09	4.09	4.09	4.09	
INT	3RD	8.59	7.65	7.65	7.98	7.98	7.98	SYMMETRIC
		4.09	4.09	4.09	4.09	4.09	4.09	
INT	2ND	10.00	9.18	9.18	9.18	9.18	9.18	SYMMETRIC
		4.92	4.59	4.59	4.59	4.59	4.59	

** NOTE ** ALL TABULAR MOMENT VALUES TO MULTIPLIED BY: 1000 KIP-IN

B E N D I N G   M O M E N T   S I G N   C O N V E N T I O N



BEAM BC, 2ND FLOOR, EXTERIOR FRAME

of the beams (probably somewhat greater than 4000 psi) nor for a capacity reduction factor ϕ to allow for variations in material strengths, workmanship, and dimensions. The compression and tension steel quantities used in Equation (3) reflect the steel distribution at the probable plastic hinge points of the beam.

Whereas the axial forces within the girders are assumed to be negligible and thus do not affect the load carrying capacity of the beams, one must consider the combination of axial load and moment for the columns. The resultant moment M_u and axial load P_u can always be replaced by an eccentrically loaded column at a distance $e = M_u/P_u$ (see Figure 15). For a cross section with uniaxial bending reinforced with n bars, the force and moment equilibrium equations are

$$P_u = .85f'_c ab + \sum_{i=1}^{n} f_{si} A_{si} \qquad (4)$$

$$P_u e = .85f'_c ab(h - a)/2 + \sum_{i=1}^{n} f_{si} A_{si} (h/2 - d_i) \qquad (5)$$

Figure 16 reflects the implementation of Equations (4) and (5) into an interaction curve for the ground floor level columns of the ICSB. Although the curve assumes bending about the N-S axis (e.g. due to horizontal E-W forces), the curves should suffice for the other principal direction of bending as well as for columns of other storey levels. Again no attempt is made to account for an in-situ concrete column strength greater than f'_c = 5000 psi nor for a capacity reduction factor.

Inasmuch as the ground level columns are of particular interest, Appendix E is provided in order to summarize the internal column forces. The results provided in Appendix E reflect the internal force resultants at ground level for each of the 24 columns due to the 7 elastic load conditions. Whereas the sign convention for the internal force

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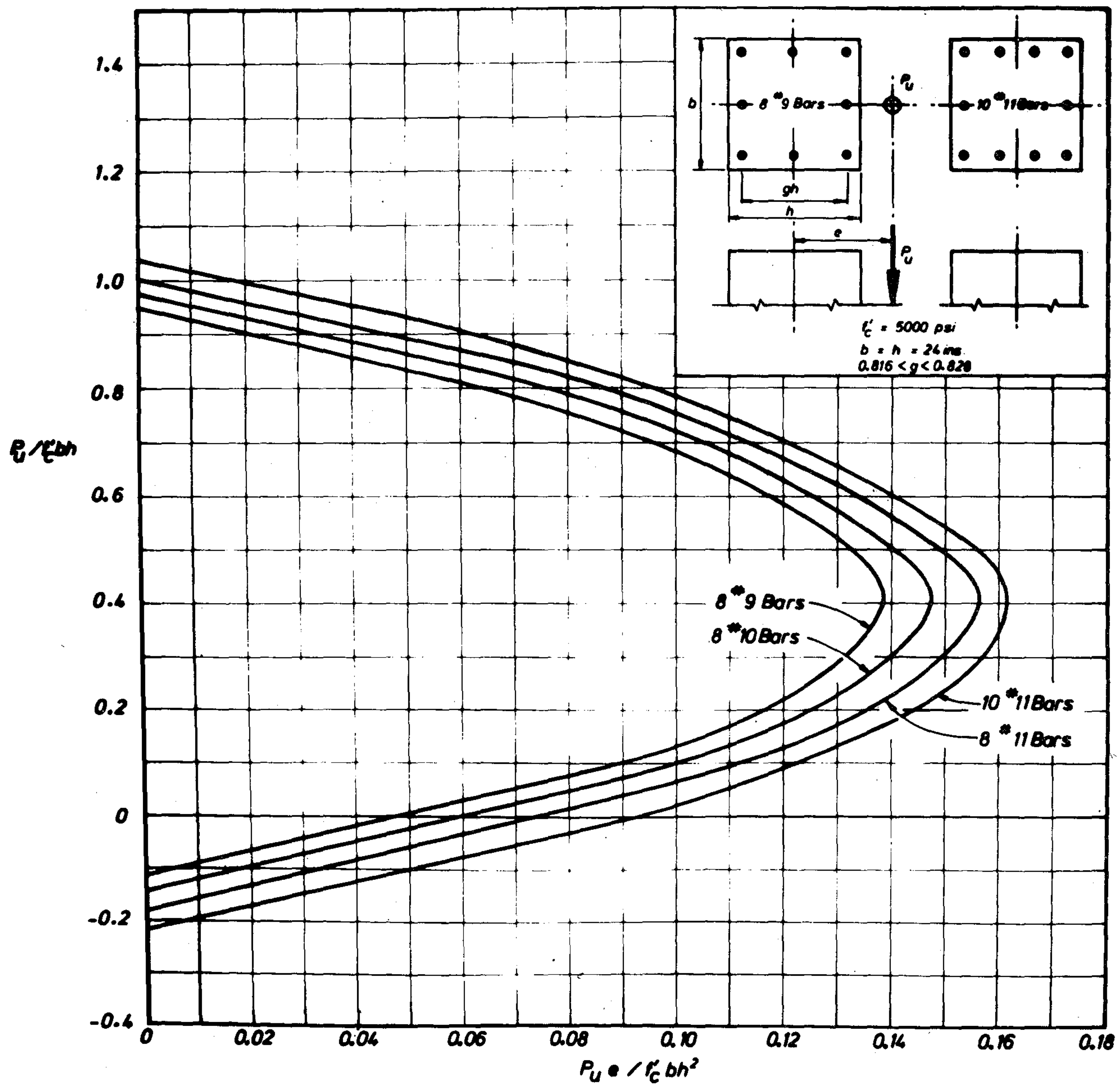
IMPERIAL COUNTY SERVICES BUILDING



Interaction Curve - Eccentrically Loaded R/C Ground Level Columns

Figure 16 - Interaction Curve for Ground Level Columns

resultant is given in Figure 10 it should be noted that no algebraic signs are provided for the time history results - only maximum values of a particular time history are reported.

The following comments reflect the elastic analysis results.

a) All girders respond elastically due to the 1967 UBC and dead load conditions (including the appropriate load factors).

b) All girders of the two interior frames respond elastically when the 1940 N-S record is considered in the N-S or shear wall direction. Plastic hinges are predicted for all exterior frames, 2nd floor girders as well as for all girders framing into the shear walls. The exterior bay girders of the exterior frames indicate possible hinging at the 3rd - 6th floor levels whereas the 3 interior bay girders of the exterior frames indicate elastic response at these floor levels. Additionally all roof level girders of the exterior frames indicate elastic response.

c) Although the predicted internal force resultants for the girders due to the 1979 N-S record are obviously different from the predicted force resultants due to the 1940 N-S record, the plastic hinge pattern for the girders is similar to that noted above in b).

d) All girders, except possibly those at the roof level, of the interior and exterior frames would probably respond inelastically if the ICSB were subjected to the 1940 N-S record in the E-W direction. The same macroscopic behaviour could be expected of the ICSB's girders when subjected to the E-W record of the 1979 earthquake. At this writing the preliminary reconnaissance[1,10,11] reports of the 1979 earthquake tend to substantiate these predictions.

e) The combination of axial force and moment for all ground level columns lie within the interaction curve envelope defined in Figure 16 due to the combination of the 1967 UBC and vertical load cases (with appropriate load factors). It should be noted that the reinforcing pattern for the columns are as follows:

Interior Frames - Columns B, C, D, F, G - 8 # 9 Bars

Interior Frames - Column E - 8 # 10 Bars

Exterior Frames - Columns C, D, E, F - 8 # 11 Bars

Exterior Frames - Columns B, G - 10 # 11 Bars

As expected the columns in lines B and G carry the lateral x-direction forces whereas only the 4 corner columns carry the lateral y-direction forces. Thus for an earthquake consisting of significant x- and y-direction components such as the 1979 event one would expect the 4 corner columns to be severely loaded.

Superficially the ICSB appears to be a simple, rectangular, symmetrical structure and, as such, one would expect the 4 corner columns to react on a symmetrical fashion. However, inspection of the first floor plan in Figures 8 and 17 indicates the unsymmetrical distribution of first story shear walls which account for the difference in column behaviour for those at the east end from those at the west end. Reference 10 provides an excellent background which describes the difference of east and west end column behaviour which is reflected on the numerical results in Appendix E as well as the catastrophic column failure at the east end during the 1979 earthquake:

"Reference to the building plans, sections and elevations (Figure 17) shows that at the West elevation the shear wall from roof to second floor is offset horizontally some five feet at the second floor level before continuing to the foundations (in the center bay only). At the East elevation a similar shear wall stops at the second floor: shear forces are then transferred through the second floor diaphragm some thirty feet to a center bay shear wall that runs from the second floor to the foundations.

Detailed study of the building damage shows that the four free-standing columns at the East end of the building failed by overturning, since the outer pair of columns show more distress than the inner. At the same time the end shear wall, the floor diaphragm, and the inner shear wall show no signs (on superficial inspection) of shear distress.

Thus the major difference in damage between the West and East ends of the building are paralleled by a major difference in architectural configuration. It should be noted that the center pair of first floor columns at the West end of the building represent the only place where a perimeter shear wall occurs. It should also be noted that the floor to floor height for the first floor height at 14'-6" is only one foot greater than the typical floor. It should be noted that the open first floor is not of any significance in itself: if it were enclosed with a light curtain wall its

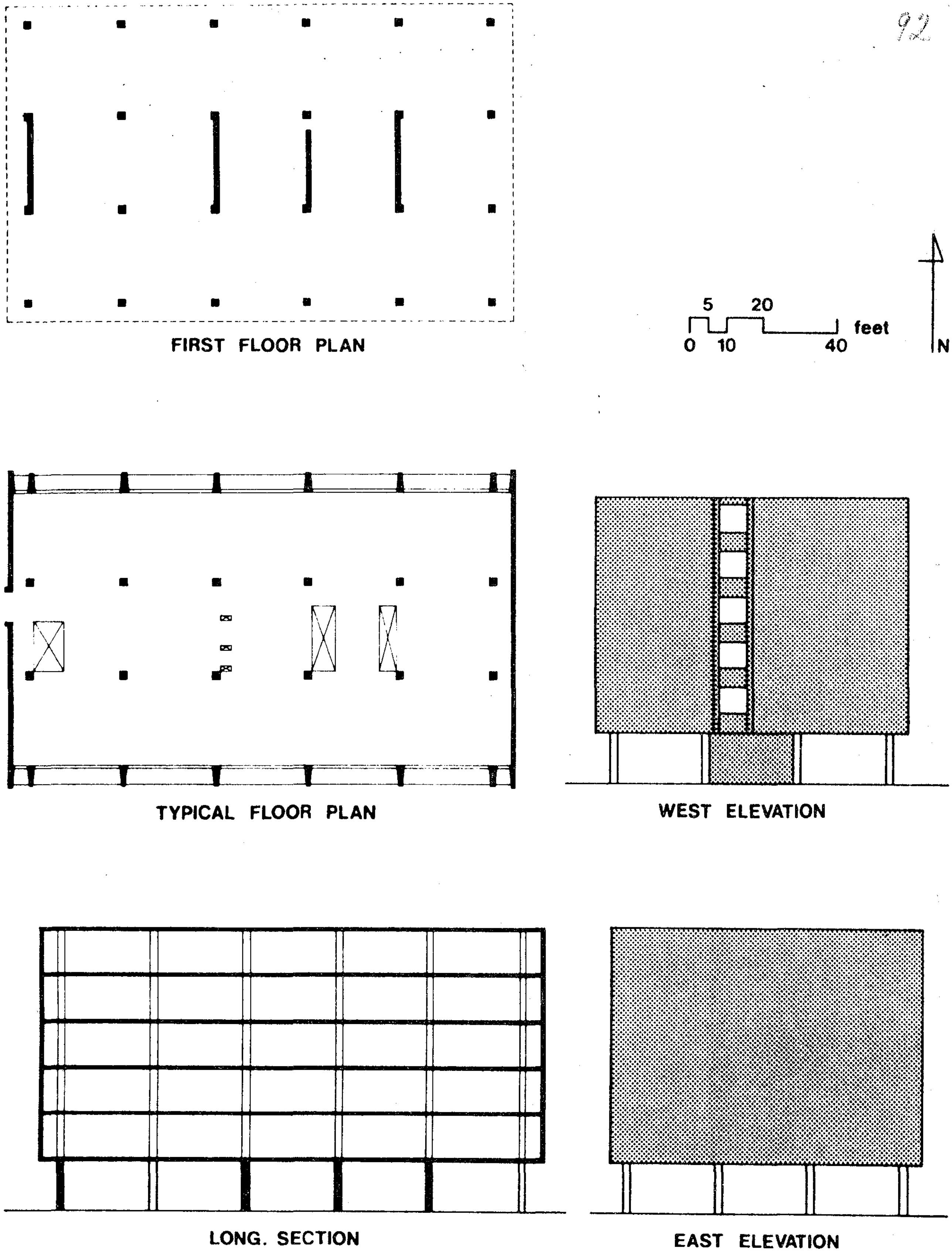


Figure 17 - Plan and Elevation Views of the Imperial County Services Building

structural performance would be essentially similar.

The condition at the East end represents a classic instance of shear wall discontinuity, in which an abrupt change of strength and stiffness occurs at the point where the shear wall, weighting approximately 300 tons, terminates at the second floor. Its vertical - overturning - loads must be brought down through four slightly offset columns, while its shear forces must be transferred to the smaller shear wall at the next bay.

Figure 18 schematically indicates the East end's two horizontal load paths: to resist translation, shear stresses can be carried in the plane of the second level floor structure in to an interior ground level wall. However, rotational (overturning) forces (which are perpendicular to the second floor) are not transferred across in this manner, and only the columns beneath the end wall can supply the tension-compression couple required to resist the overturning moment. The corner columns take the bulk of these alternating axial forces.

At the West end, the stiff ground level shear wall beneath the upper wall prevents large axial forces from reaching the columns. While there is a five foot "kink" in the load path from upper portion of West wall to ground level wall, compared to the East end this is more nearly a continuous shear wall. Holes in the upper West end wall comprise another geometric irregularity or discontinuity. At the force levels experienced in this particular moderate-sized earthquake, however, these holes at stairway landings did not apparently affect behaviour.

It should be made clear that the Uniform Building Code in force when the building was designed permits discontinuous shear walls: and the Code in force at present also permits such a condition. No mention was made of such configurational issues until the 1973 UBC when a clause was inserted (2312-C-3) that now reads:

> "3. Structures having irregular shapes or framing systems. The distribution of the lateral forces in structures which have highly irregular shapes, large differences in lateral resistance or stiffness between adjacent stories or other unusual structural features shall be determined considering the dynamic characteristics of the structure".

This clause in effect makes the designer responsible for evaluating the dynamic characteristics of the structure and using judgement in his design response (note that this clause carefully does not <u>mandate</u> a dynamic analysis).

In passing it should be noted that the Field Act uses a slightly stronger language that the UBC:

> "When the design of a structure or parts of a structure result in unusual configuration or irregular distribution of lateral stiffness, evidence shall be presented to show that equivalent safety to that established by these regulations is provided or the office shall withhold its approval".

In SEAOC's 1975 Recommended Lateral Force Requirements and Commentary, the discontinuous shear wall problem is specifically identified in the commentary in Section 1(E)3, Buildings with abrupt changes in Lateral Resistance:

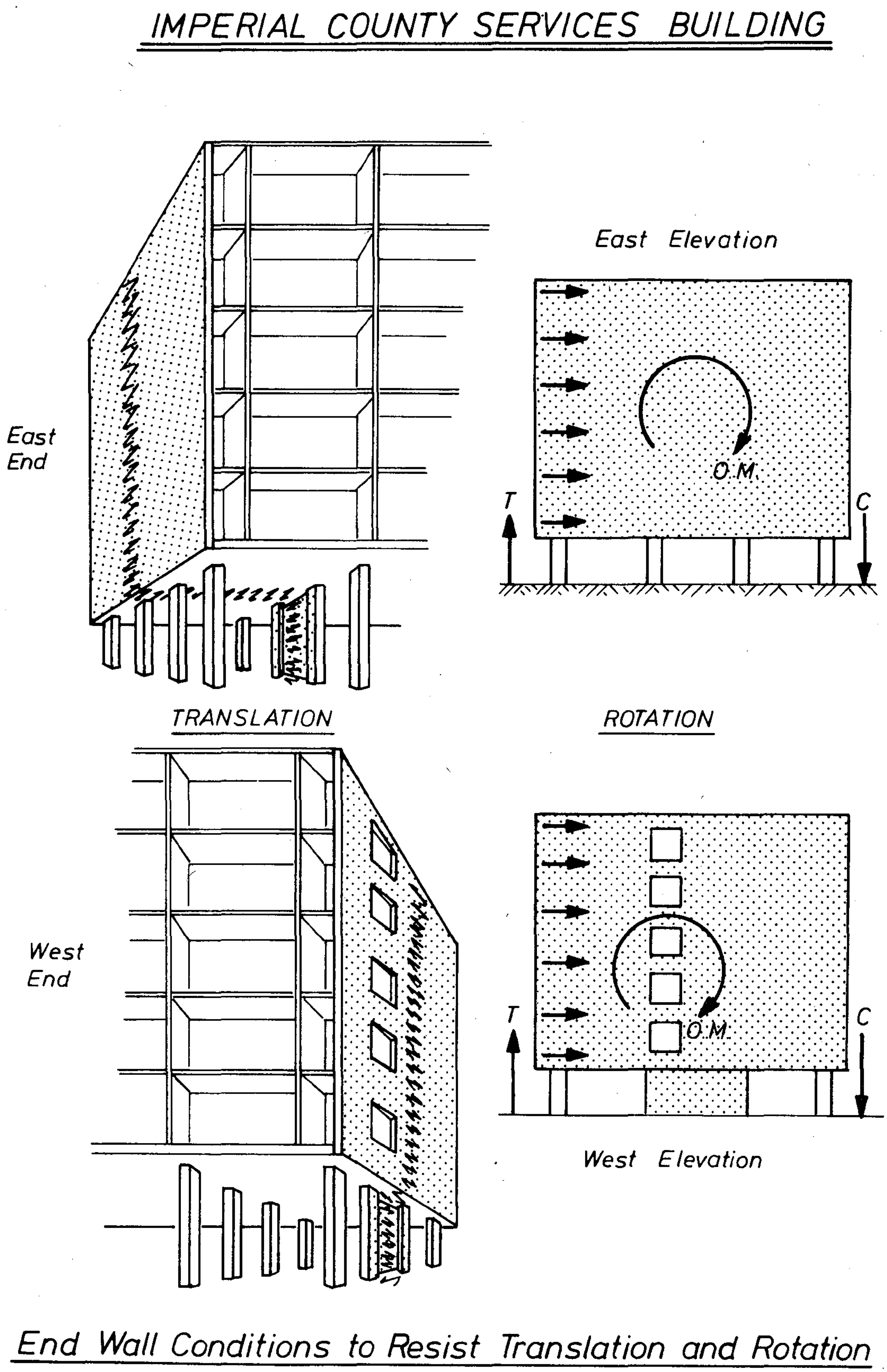


Figure 18 - End Conditions to Resist Translation and Rotation

> "the practice of providing open or "soft" lower levels by discontinuous shear walls or otherwise making certain levels weaker than the adjacent levels both for shear and overturning has frequently shown poor performance. The anticipated large distortions and energy absorption occurring in the weaker level make the proper distribution of design forces by dynamic analysis desirable. The new SEAOC Code Requirement only specifies a distribution of forces in accordance with the structures dynamic properties. It is realised that this distribution may not, by itself, indicate excessive energy absorption requirements". (underline added)

The experience of the Imperial County Services Building suggests that it may be timely to review the way in which the code deals with configuration issues".

(f) the combination of axial force and moment for all ground floor columns lie considerably outside the interaction curve envelope defined in Figure 16 due to the combination of the vertical dead loads and either the 1940 or 1979 records. However if the ICSB were subjected to say 25% levels of the 1940 or 1979 records then the combination of axial force and moment for all ground floor columns would lie within the interaction envelope. These 25% levels represent a design basis for performing a dynamic analysis in which the structure should be expected to respond elastically and that sufficient ductility could be incorporated to enable the structure to absorb higher earthquake loads.

One interesting item relates to the ductility of the east end ground floor columns. Figure 6 denotes a typical column detail in which column reinforcement ties are provided up to a point near the top of the soil. However a concrete slab encases the interior frames' columns near the top of the reinforcement ties so that at least half the columns do not have sufficient ties above the rigid slab. Thus a further explanation of the east end's column failure is that the corner columns (G-1, G-4) are sufficiently loaded due to the significant E-W and N-S records whereas the interior columns (G-2, G-3) failed due to insufficient ductility at the base.

Description of the Inelastic Model

Though a three-dimensional structural analysis may be required for a limited number of buildings, such inelastic time-history analyses are, in general, prohibitively expensive. As a result, most inelastic dynamic analyses are limited to equivalent plane or two-dimensional models of the structure.

In the East-West direction of the Imperial County Services Building there are four parallel frames and for the inelastic dynamic analyses it was proposed to take one interior and one exterior frame and impose a coupling so that the horizontal displacements at corresponding floors in each frame were to be the same. In the real structure, such coupling would be imposed by the floor diaphragm which is relatively stiff in its own plane. This model is shown in Figure 19. The coupling here is achieved by equivalent rigid links and rather than impose common horizontal displacements to all nodes on each floor level, only the ends of the frames were inter-connected so that the bandwidth of the stiffness matrix was minimised. This model has 108 nodes and 176 members. The disadvantage of this particular model is the large number of degrees of freedom and high expected computational cost and it was therefore set aside in favour of a further simplification.

This simplified model is shown in Figure 20 where one half of each of an interior and an exterior frame is coupled at the centre. This model would require two analyses to cover the variations in the column axial forces, depending on whether the excitation was in the West to East direction, or vice versa; however, the low, squat nature of the frame is such that these variations should be small and the assumption of antisymmetry at the centre under lateral loading seemed justifiable. It later transpired that the horizontal roller support at the centre, which could not be removed for the initial static analysis and then reimposed for the lateral time-history analyses, led to its removal altogether leaving only the mid-span hinge

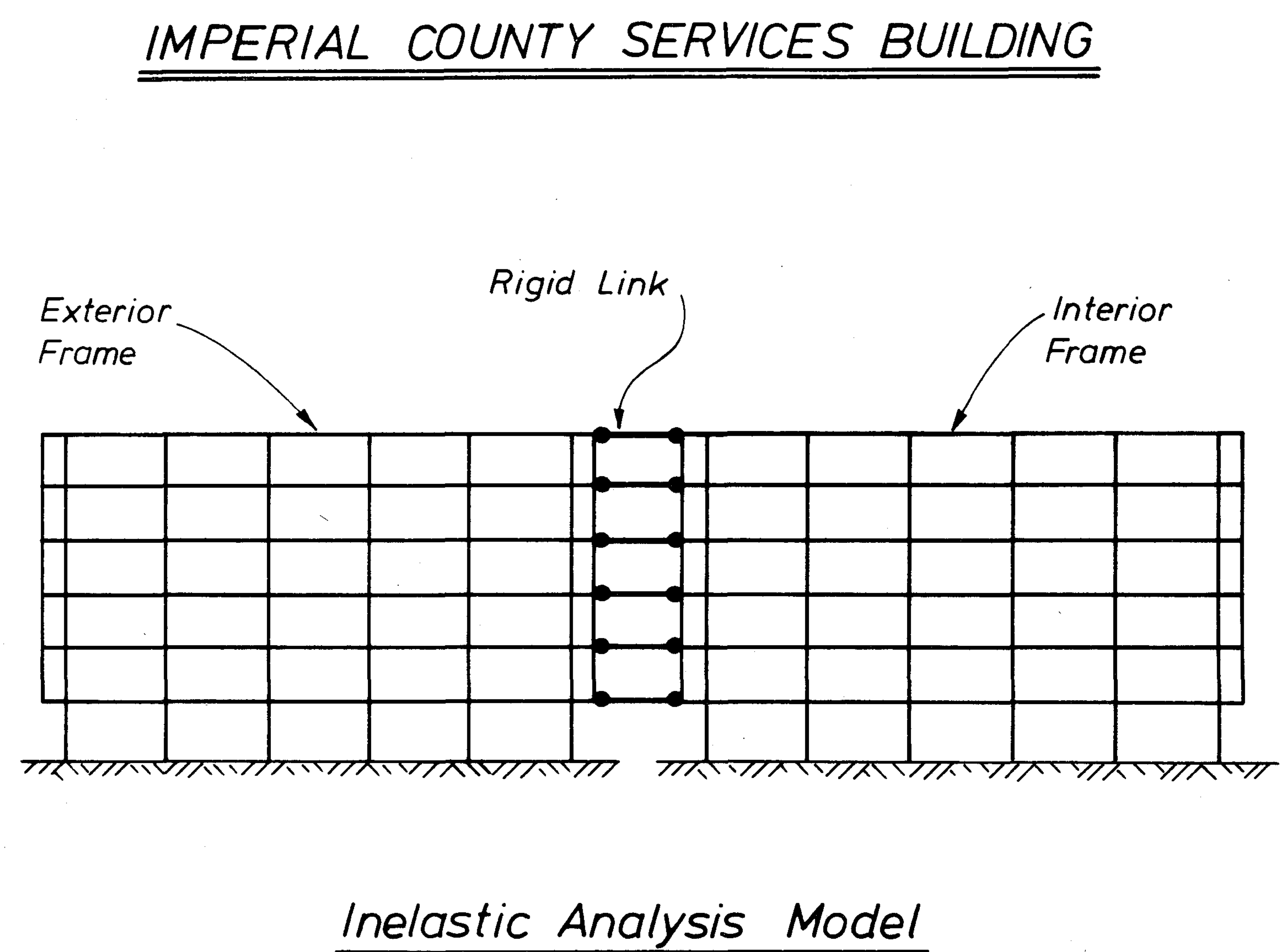


Figure 19 - Initial Model for Inelastic Analysis

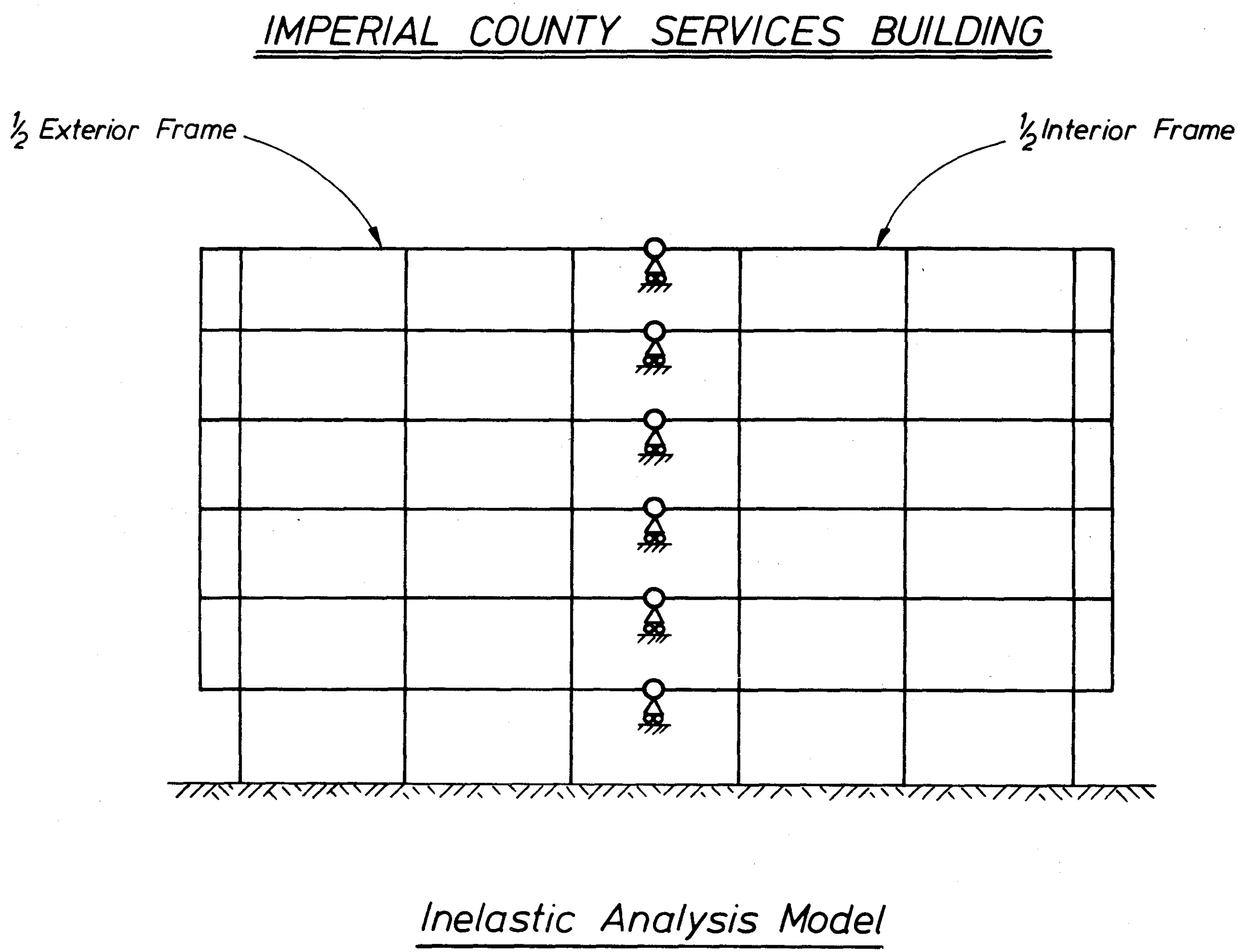


Figure 20 - Simplified Model Used for Inelastic Analyses of Imperial County Services Building

since the effect of girder shear on column axial loads appears to be minimal. The effect of roller support on the initial gravity load analyses led to peculiar moment patterns in the girders which, in turn, led to bizarre hinge formation patterns during the dynamic analysis.

Each joint of the structure has three degrees of freedom (being the x- and y- displacements and the z- rotation). The mass model may be a consistent mass or a lumped mass model, the latter having been generally used in analyses at the University of Canterbury was used for these analyses also. In the first attempts at analysing this structure, the only masses used were those associated with the horizontal degrees of freedom of each floor and these were taken to be acting at the joint connecting the two half-frames. However, after initial difficulties in obtaining a stable inelastic time-history integration the lumped mass model was modified to associate mass with the vertical and horizontal degrees of freedom of each node together with a representative rotational inertia obtained from equivalent terms from the consistent mass matrix. This was done in order to give the joint some enhanced capacity to absorb moment overshoot effects associated with a hinge forming during a given time-step. After a careful study of the apparent instability in the analysis, it was found that the problem was caused by machine accuracy and word length difficulties associated with the use of inches and kips as units, where the member lengths are of the order of 300 inches. Upon conversion of the dimensions and weights to metres and Kilo Newtons, the numerical stability problems immediately vanished.

The damping model chosen for the analyses was that of Rayleigh damping where the damping matrix was a linear combination of the mass matrix and the initial elastic stiffness matrix. The fractions of damping prescribed were 8% of critical damping in natural modes of free vibration one and six. This ensures sub-critical damping in the higher modes of vibration, with damping in the second and third modes being approximately 4% of critical damping.

The members in the model were such that the stub beams supporting the end walls, the end wall members and all the columns above the ground floor were assumed to remain elastic while the remaining members (that is all the interior beams of both frames and the ground floor columns of both frames) were assumed to have bilinear moment curvature relationships where the post elastic stiffness was 3% of the elastic stiffness. The columns used an axial force - moment interaction curve as similar to that shown in Figure 16 but where the actual curves were approximated by a straight line from the axial tension yield point to the pure moment yield point, a central section representing a cubic variation of moment with axial compression and a final straight line segment to the axial compression yield point. For the beams, the positive and negative yield moments for each end of the members are given in Table 2.

The analyses were carried out using the two dimensional frame analysis program "RUAUMOKO" (described in Appendix F) which has been developed at the University of Canterbury for the earthquake analysis of planar building frames.

The joint regions of all members were treated as rigid end blocks and the floor displacements determined by the analysis are those at the centroidal axis level of the girders at each floor level. This is different to the ETABS program since it is stated on P.14 of the ETABS manual [2] that the displacements are those of the floor level and goes on to recommend using zero length rigid end blocks at the base of each column even though there is no lateral offset of the girder nodes to provide a consistent model. In this respect, ETABS should use half the girder depth as the rigid end blocks at each end of the columns with the displacements therefore being those at the level of the girder centroidal axes.

The end shear wall members were treated as wide columns for the analyses and caused no modelling difficulties.

All the analyses carried out with the RUAUMOKO program used a time-step of 0.01 seconds with Newmark constant average acceleration ($\beta = \frac{1}{4}$) method since this had been shown to be satisfactory in many previous analyses carried out [12 - 15]. Elastic and inelastic dynamic analyses of the building were carried out, the former to provide a comparison with the results (elastic) from the ETABS program.

Discussion of Elastic and Inelastic Plane Frame Analyses

A total of four analyses were carried out, two each for the first 14 seconds of the N-S component of the May 1940 El Centro record and the E-W component (on the E-W frames) of the October 1979 free-field record at the Imperial County Services Building site; one of these analyses was carried out assuming an elastic structure while the other used the inelastic model. The purposes of the elastic analyses were to confirm the agreement between the ETABS analyses and that of the RUAUMOKO program and also to obtain a measure of the time-wise variation of the building response since the ETABS program only produces a response envelope.

(i) Elastic Analysis

The moments and shear forces appear to be in good agreement between the ETABS and RUAUMOKO analyses but differences occur in the predicted column axial forces. In order to compare the column forces, it is necessary to combine the dead load and earthquake axial forces derived from the ETABS model. The column forces in the exterior frame as determined by the plane frame analysis are lower than those from the ETABS analysis whereas for the interior frame, the plane frame axial forces were higher in the exterior column but lower in the interior columns than those determined by ETABS. The trends were the same under both earthquakes. This discrepancy in predicted column axial loads is probably accounted for by the fact that ETABS does not ensure compatibility of vertical deflection and rotation of joints about horizontal axes for intersecting frames.

The maximum displacement at the roof level was found to be 3.29 inches under the 1940 El Centro earthquake record and 2.69 inches under the 1979 earthquake record. Time history plots of the displacements under the two earthquake records are shown in Figure 21.

Time-history plots of the bending moments in the end columns of the exterior and interior frames at the ground level as determined using the RUAUMOKO analysis are illustrated in Figures 22 - 23. Here, the solid lines represent the moment at the bottom of the column while the dotted lines represent the moment at the top. These are also summarized in Appendix G.

(ii) Inelastic Analyses

Under the 1940 and 1979 earthquake records, the deflection at the roof level was found to be 2.36 inches and 2.85 inches respectively. On the basis of the elastic results where the 1979 earthquake produced a displacement response that was only some three-quarters of that of the 1940 earthquake, it is somewhat surprising that for the inelastic analyses the displacement response was greater under the 1979 earthquake. Comparing the time-history plots for the inelastic analyses (Figure 24) with those from the elastic analyses (Figure 21) it can be seen that for the 1940 earthquake the plots are similar for both analyses; however, for the 1979 earthquake the inelastic analysis shows that the acceleration peak just before the time of 8 seconds (see also Figure 13) produces sufficient plasticity in the frames that significant permanent deformation occurs.

Time history plots for the column bending moments at the top (dashed line) and bottom (solid line) in the end columns of the exterior and interior frames are shown in Figures 25 and 26. From these Figures and Appendix G, it can be seen that, as would be expected, the bending moments from the inelastic analysis are considerably lower than the comparable values from the elastic analysis. For the 1940 earthquake, it can be seen that the behaviour of the end columns of the two frames are slightly different. In the case of the exterior frame, it can be seen that after the first inelastic excursion at 1.9 seconds, the bending moments in the columns remain essentially in the same direction. For the interior frame, the moment at the top of the column maintains the same direction after first yield but the bottom moment continues to alternate in direction. The 1979 earthquake produces moments in the end columns of both frames that fluctuate in direction generally in a manner similar to the elastic analysis. Diagrams showing the plastic hinge formation and floor deflection profiles at various times through the 1979 earthquake record are shown in Figure 27.

Under both earthquakes, the member ductility demand in the ground floor columns of both interior and exterior frames was found to be low being a maximum of 2.6 under the 1940 earthquake record and 3.1 under the 1979 record. The girder ductility demand was a maximum of 7.2 under the 1940 earthquake record and 7.4 under the 1979 record.

Conclusions

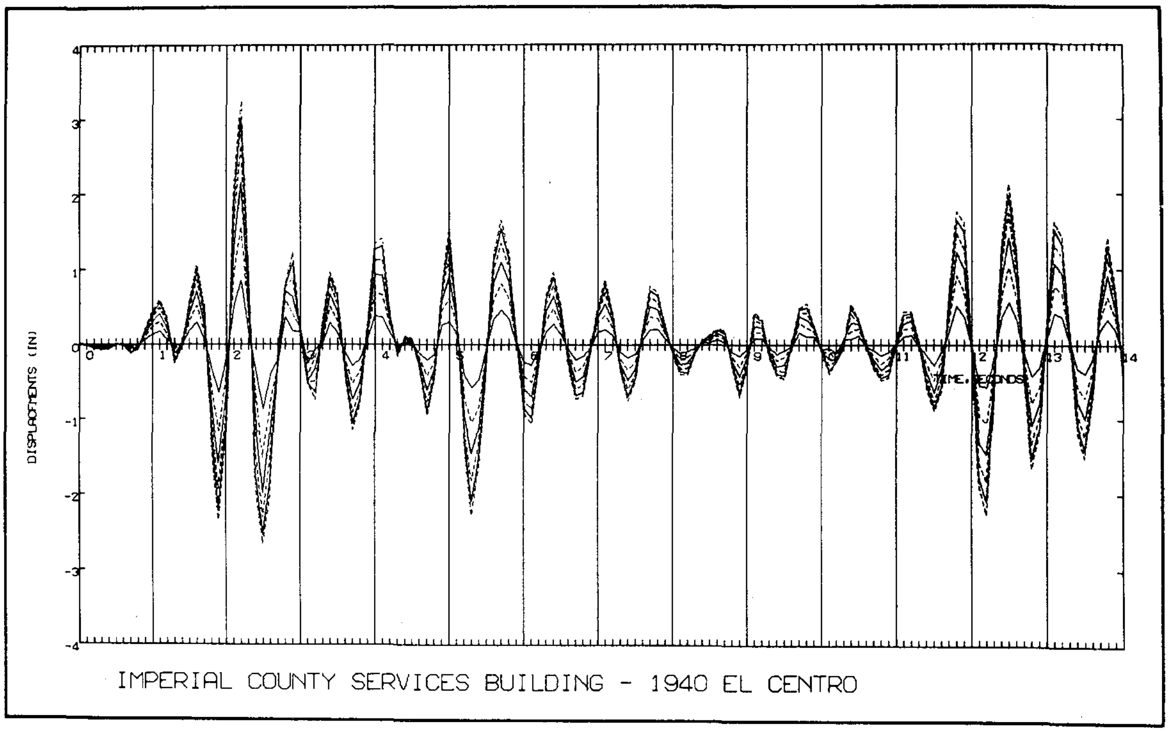
For elastic analyses, the ETABS program facilitates a reasonably economic analysis tool for three dimensional structures. However, for the ICSB difficulties were caused by the end shear walls which did not continue to the ground floor and hence required modelling along with other frames into a single frame. The model chosen here, which included these shear walls into an external box frame, is not necessarily the optimum model but did enable the ETABS program to incorporate these unusual structural elements. A major shortcoming of the program for a research orientated analysis is that a time-history of deflections and moments is not available whereas for design purposes the provided envelopes of the results are satisfactory. Further, if a fine time step is required for more accurate envelopes, the consequence is a greatly increased demand for computer memory.

Reasonable agreement between computed deflections and moments for the three dimensional ETABS and the two dimensional elastic analyses was obtained. This shows that for loading in a single direction, the two dimensional model is perfectly adequate. However, should it be desired to analyse the building under simultaneous earthquake components in the N-S and E-W directions, then a full three-dimensional analysis would be required particularly if inelastic effects in the corner columns were to be taken into account. It should be noted in this respect that ETABS can only consider a single earthquake component in any single analysis.

It is interesting to note that whereas the elastic analysis indicate that the 1940 earthquake places a more severe demand on the building than does the 1979 earthquake, the reverse is indicated for the inelastic analyses and also that in the inelastic analyses the 1979 record leads to a small permanent drift in the building whereas no significant displacements are evident from the 1940 record.

For the 1979 earthquake comparisons with deflection histories for various floors have not been made because an investigation of the recorded ground flow-accelerations are significantly different from the "free field" record used in these analyses. Though some foundation-structure interaction effects on the ground flow motion were to be expected the ground floor accelerations appear to be considerably larger than the "free field" though with a similar time-wise variation. Further analyses are to be carried out using this measured ground flow acceleration as the input motion with a view to carrying out further comparisons.

The inelastic deflections and the member ductility demands for both the 1940 and 1979 earthquake in the East-West direction appear to be small and would not indicate the level of damage observed in the building after the 1979 earthquake. The greater ground floor accelerations, mentioned above, will raise these somewhat but a large increase in the corner column axial forces will come from the overturning moments in the end shear-walls resulting from the North-South component of the earthquake. These effects together with the unfortunate placing of the ground level slab above the detailed column hinge reinforcing probably led to the failure of these corner columns.



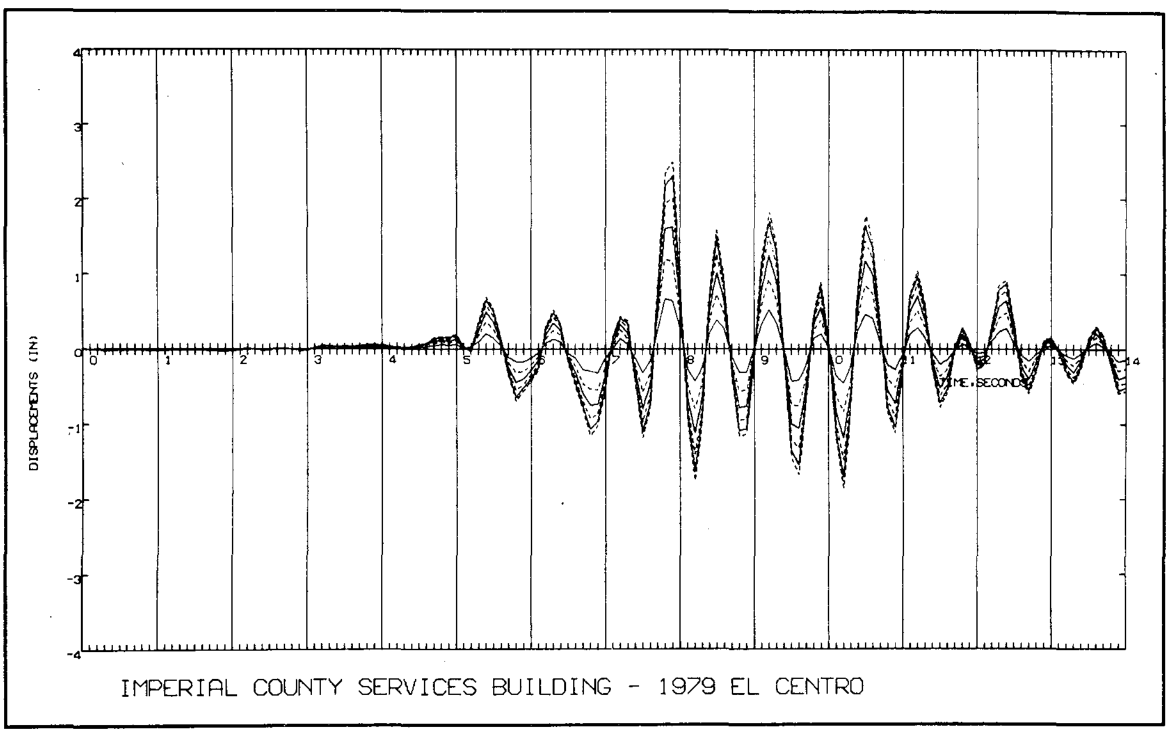
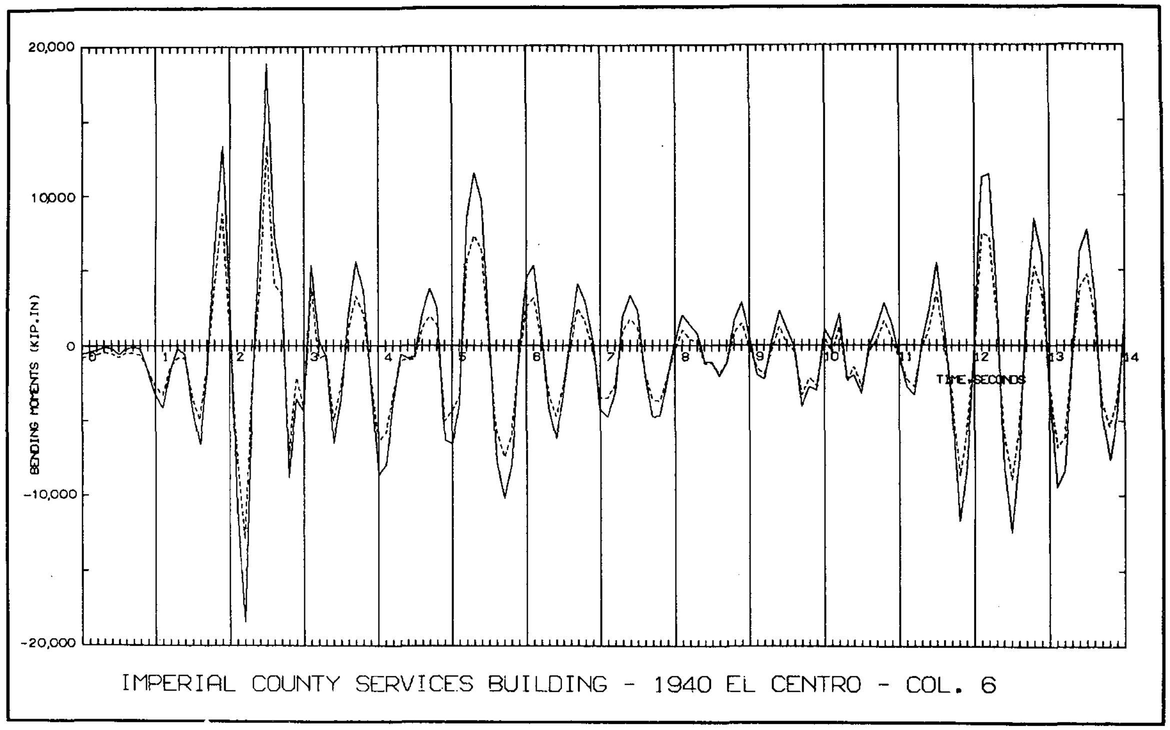


Figure 21 - Displacement Time-History Plots for Elastic Analyses Using 1940 and 1979 Earthquake Records



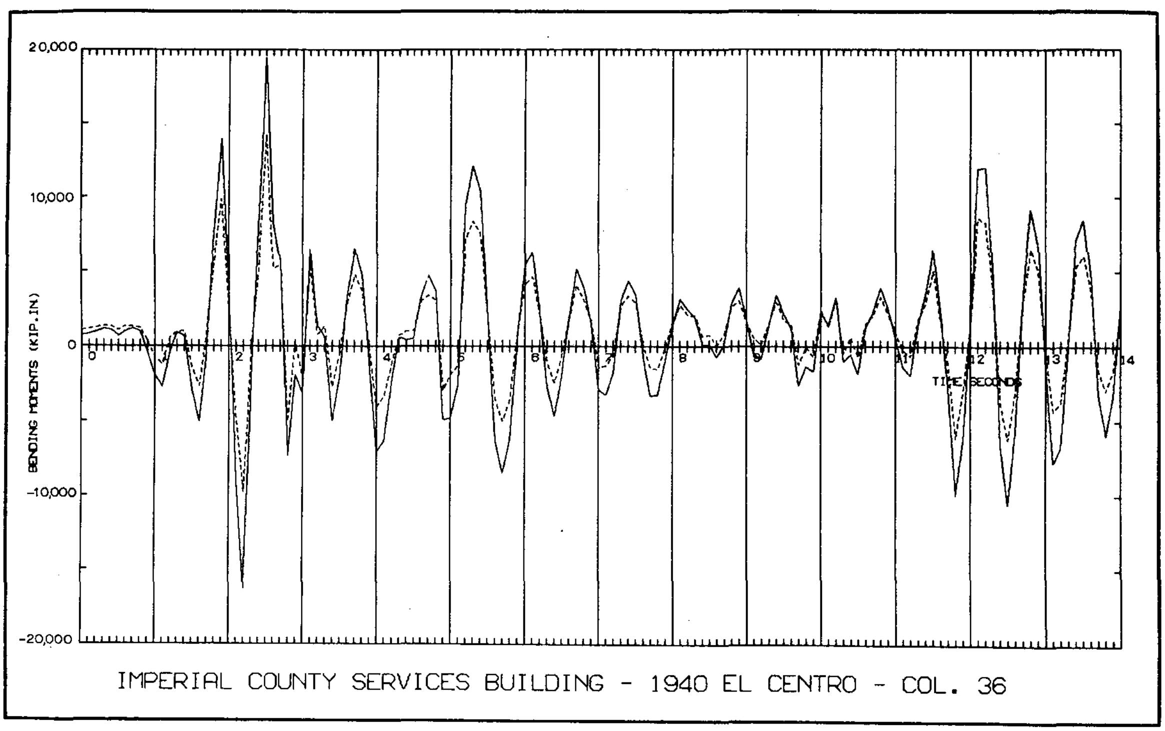
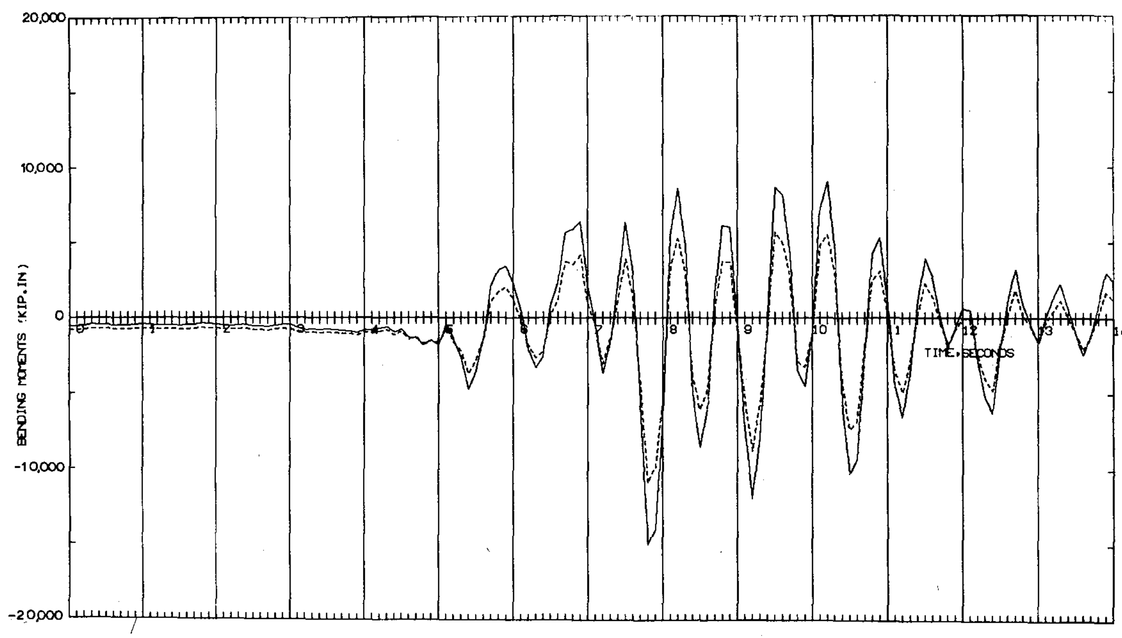
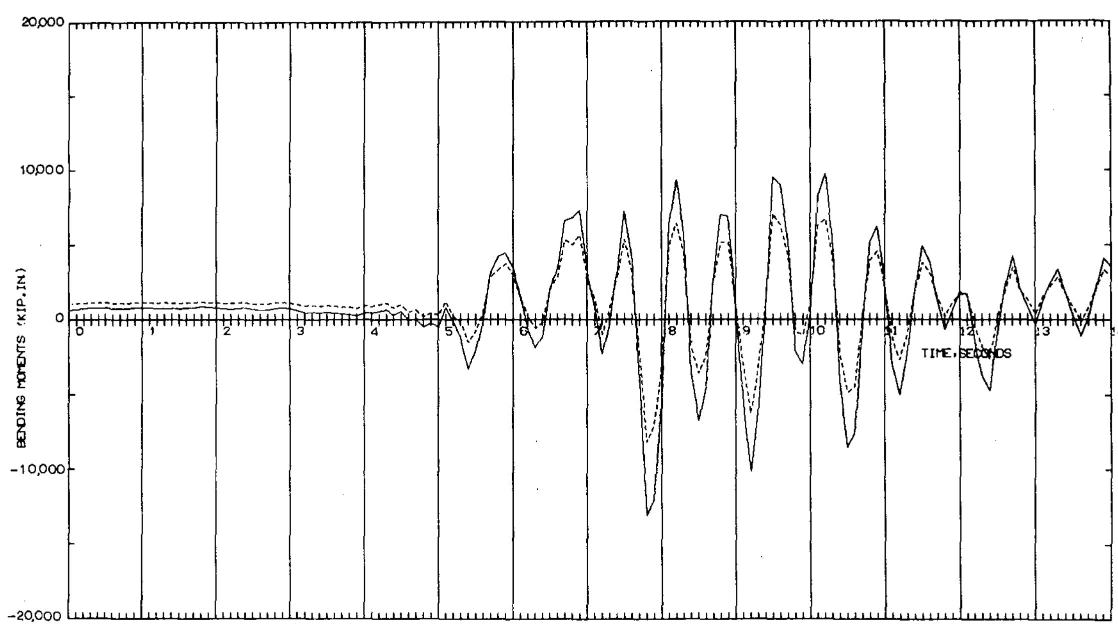


Figure 22 - Elastic Bending Moment Time-History Plots for the Outermost Column in the Exterior (Top) and Interior (Bottom) Frames Under El Centro 1940 Earthquake Record

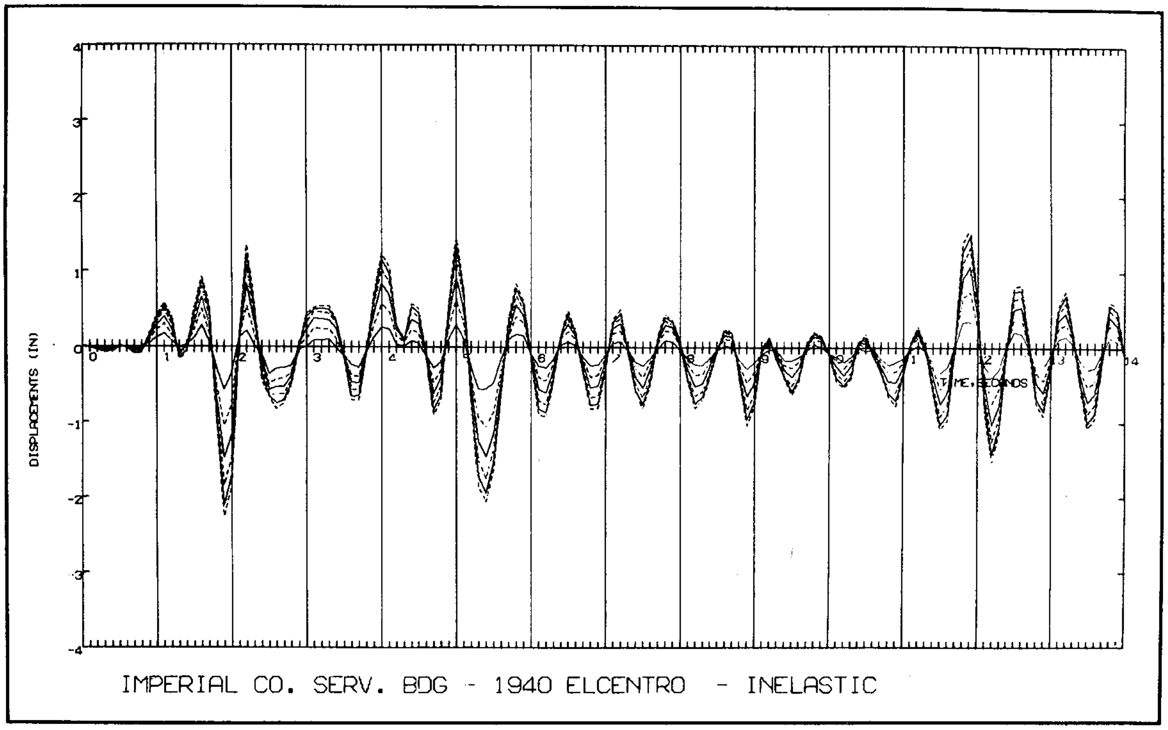


IMPERIAL COUNTY SERVICES BUILDING - 1979 EL CENTRO - COL. 6



IMPERIAL COUNTY SERVICES BUILDING - 1979 EL CENTRO - COL. 36

Figure 23 - Elastic Bending Moment Time-History Plots for the Outermost Column in the Exterior (Top) and Interior (Bottom) Frames Under 1979 ICSB "Free Field" Earthquake Record



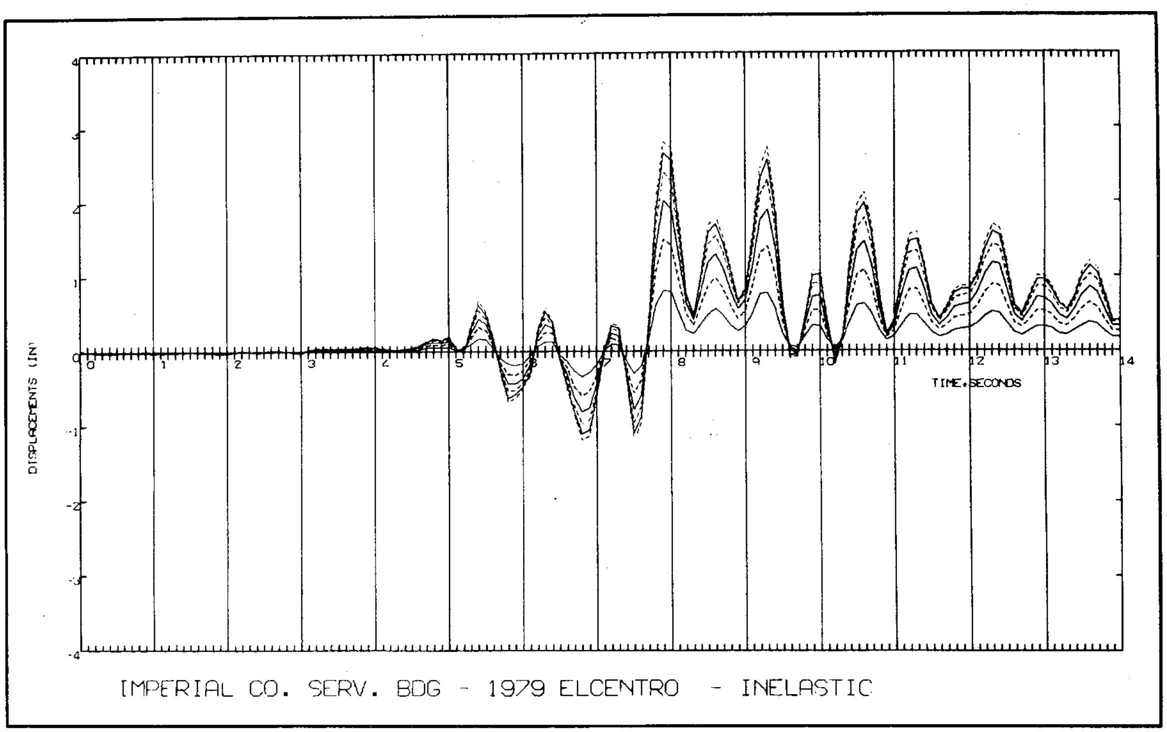
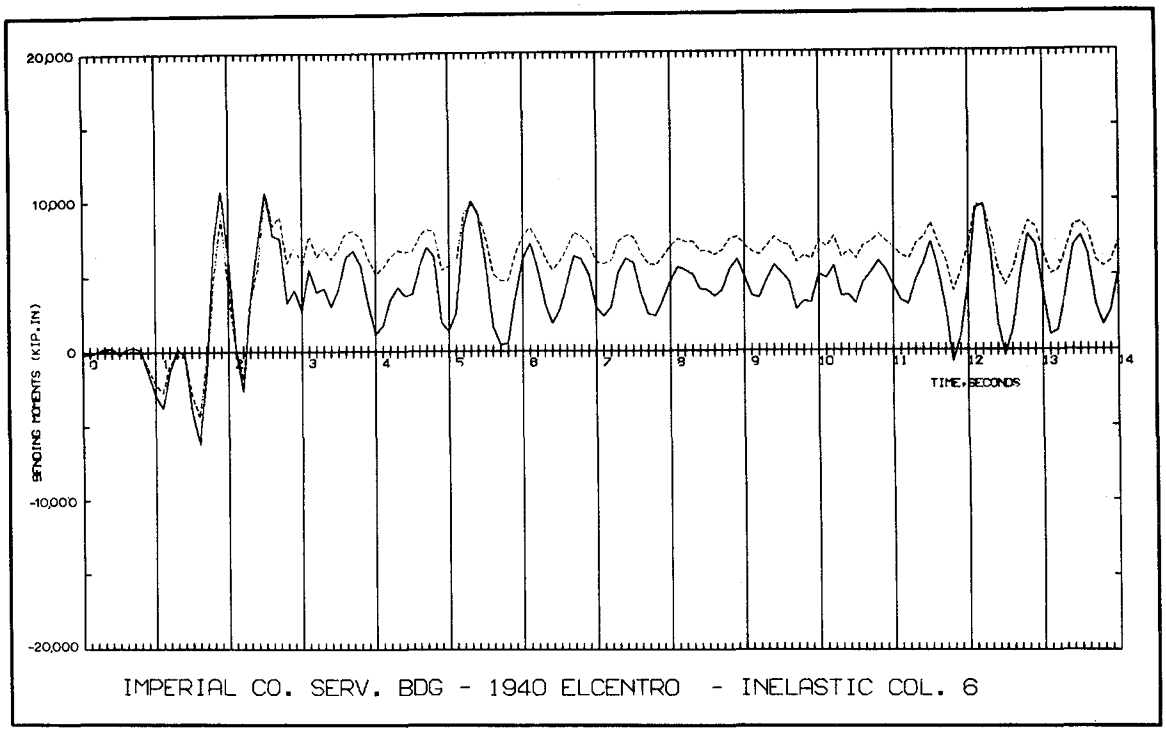


Figure 24 - Displacement Time-History Plots for Inelastic Analyses Using 1940 and 1979 Earthquake Records



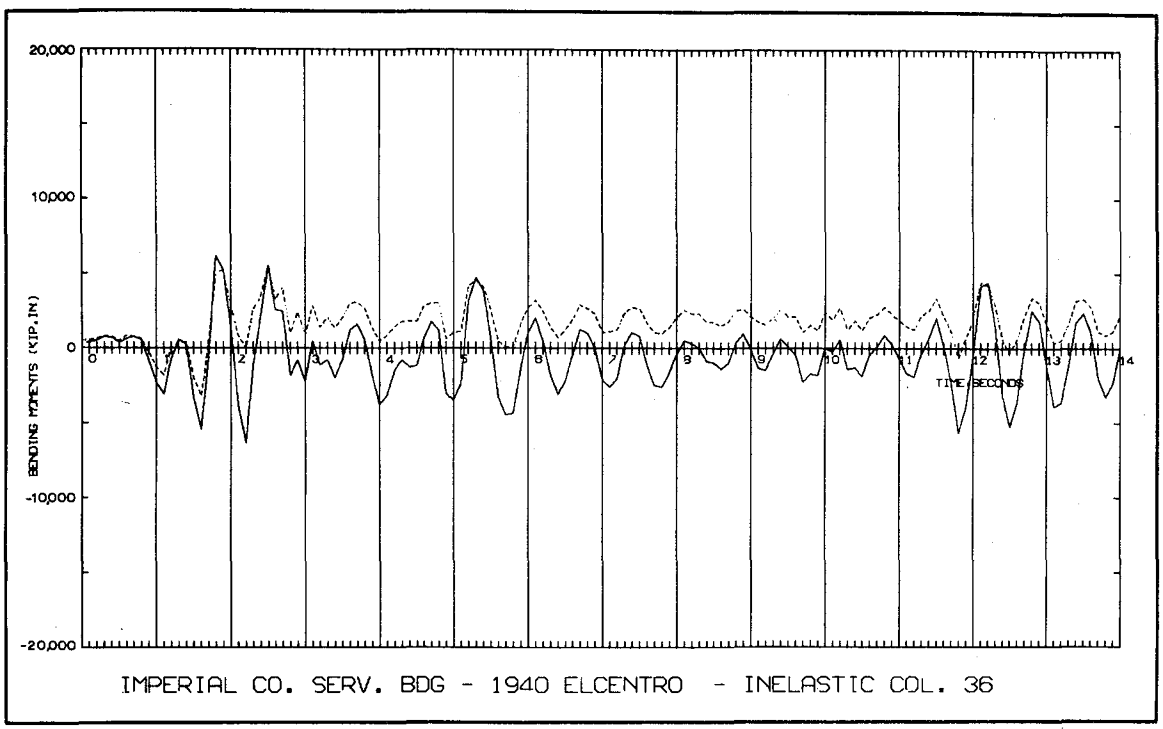
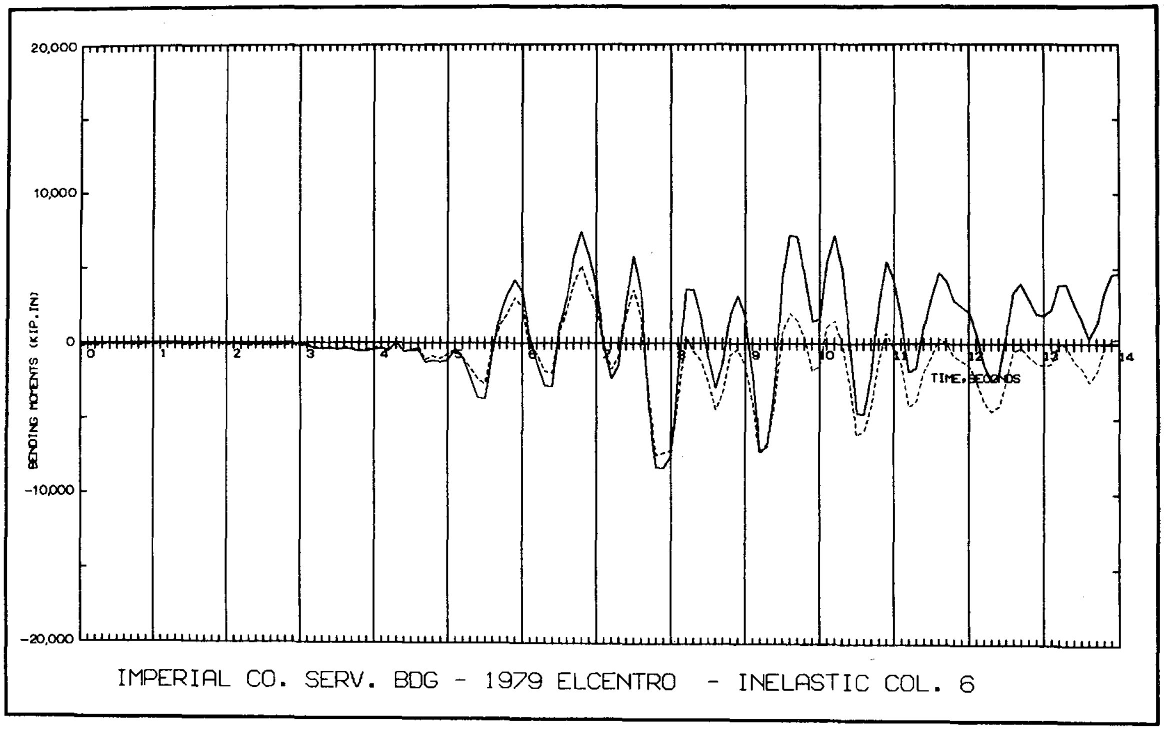


Figure 25 - Inelastic Bending Moment Time-History Plots for the Outermost Column in the Exterior (Top) and Interior (Bottom) Frames Under El Centro 1940 Earthquake Record.



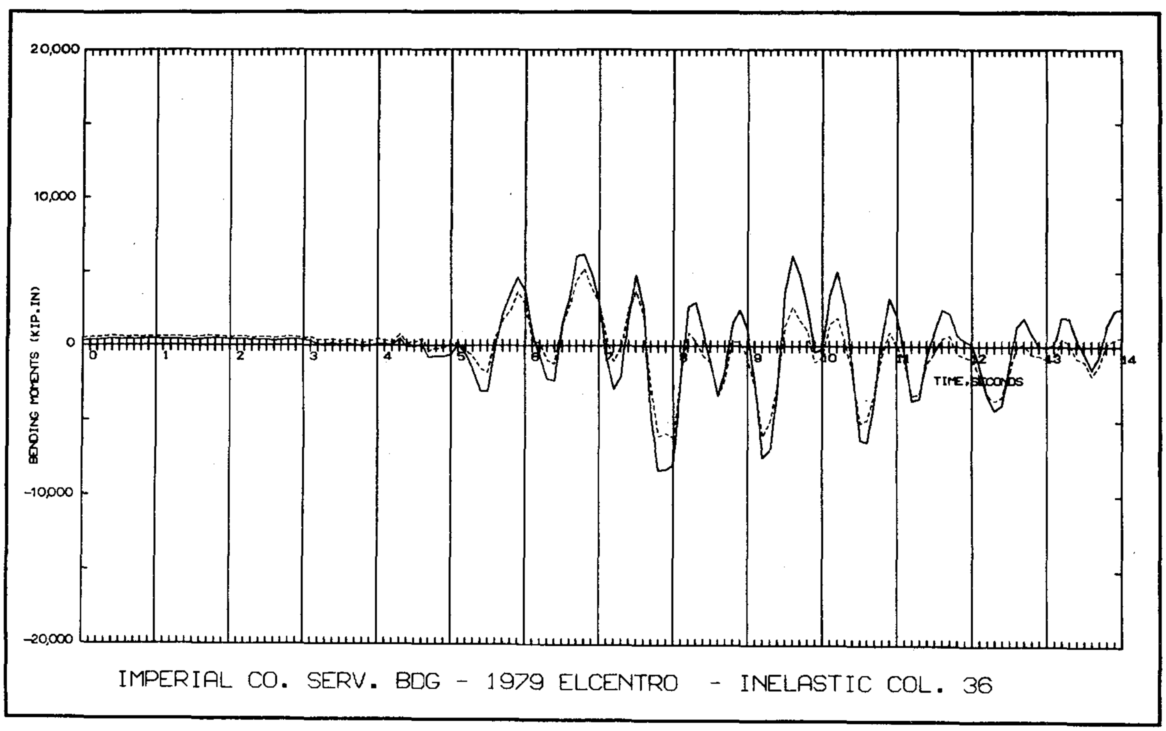


Figure 26 - Inelastic Bending Moment Time-History Plots for the Outermost Column in the Exterior (Top) and Interior (Bottom) Frames Under 1979 ICSB "Free Field" Earthquake Record

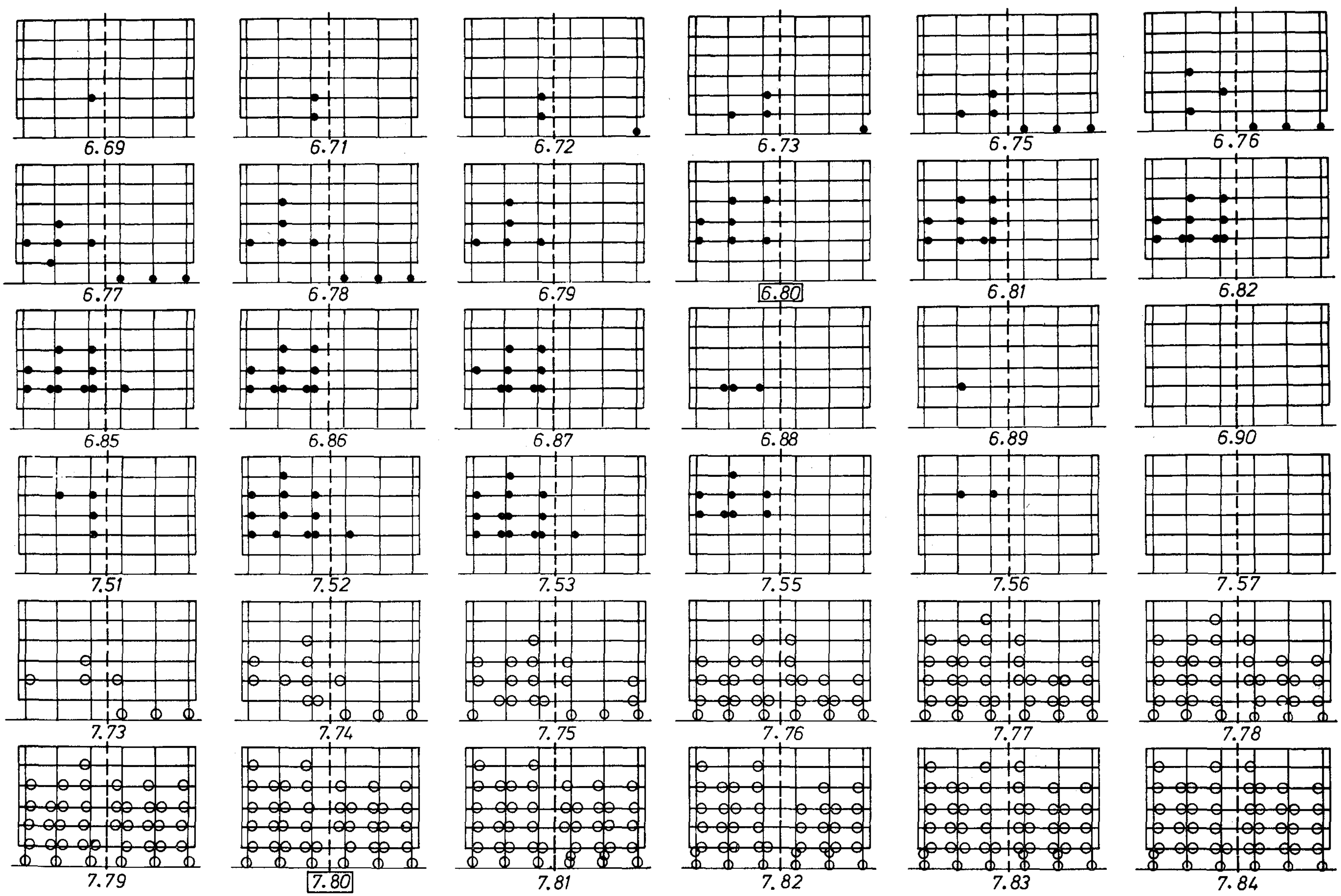


Figure 27 - Plastic Hinge Formation and Deflection Profiles Under the 1979 Free Field Earthquake Record (cont....)

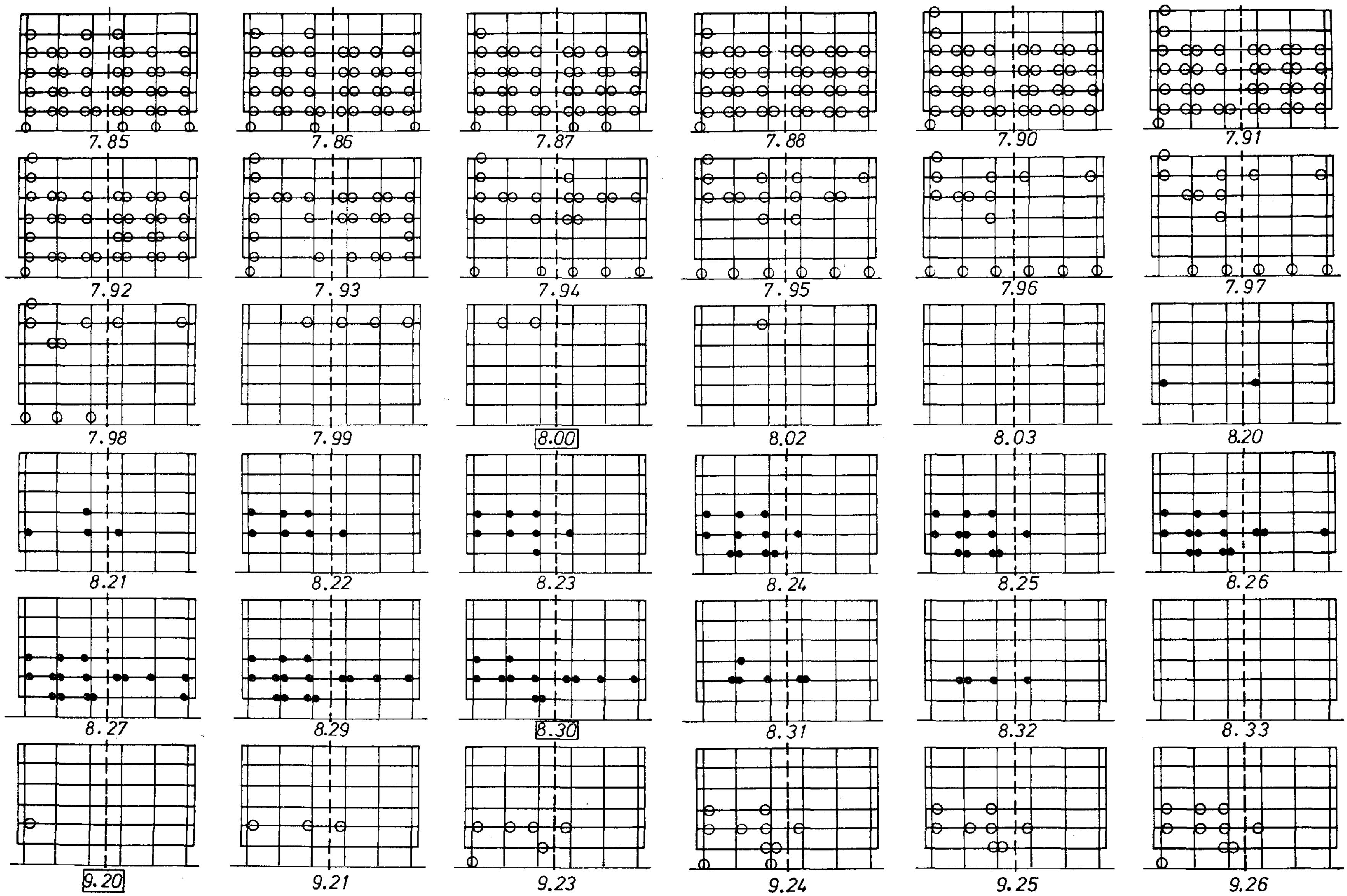


Figure 27 - Plastic Hinge Formation and Deflection Profiles Under the 1979 Free Field Earthquake Record

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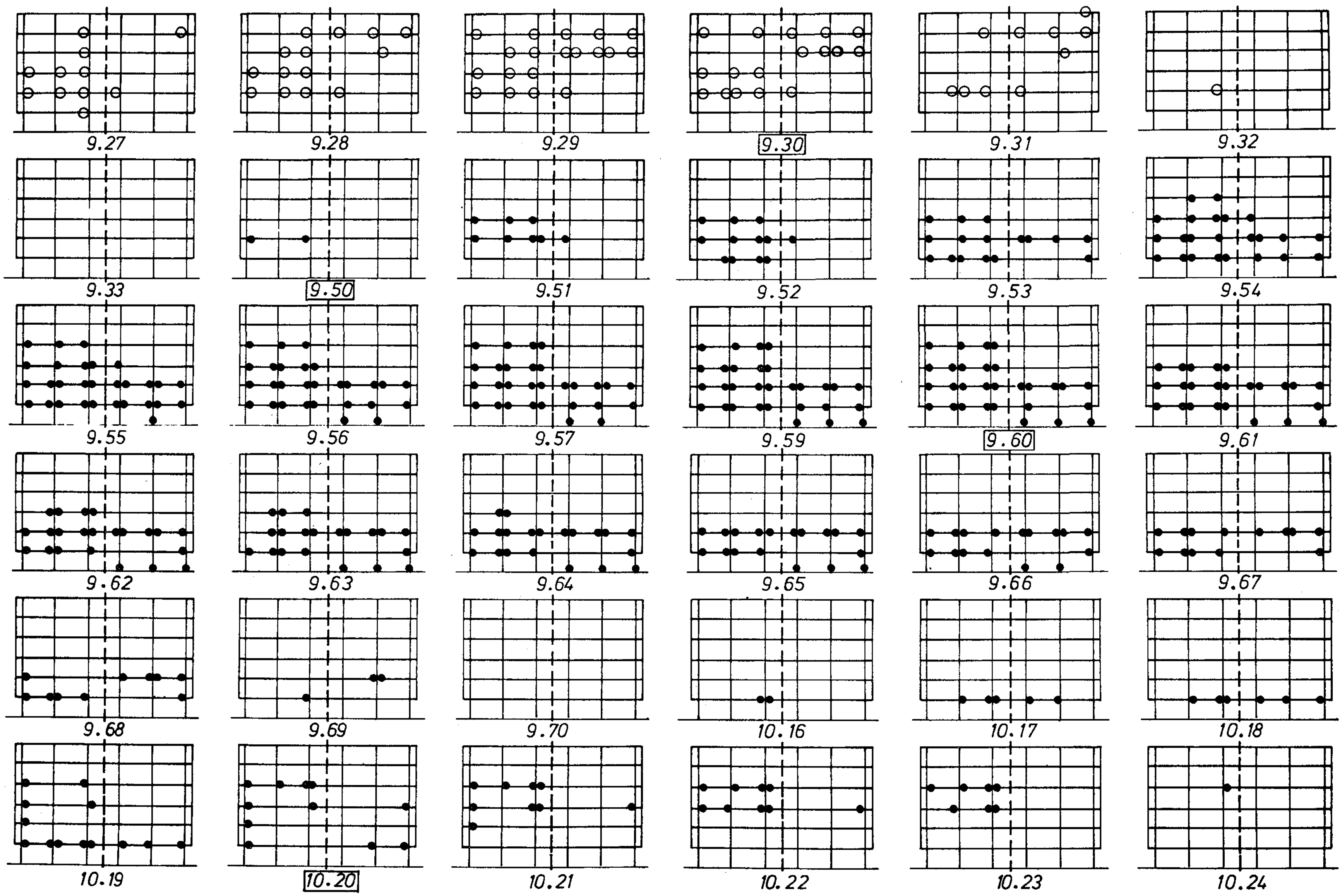

Figure 27 - Plastic Hinge Formation and Deflection Profiles Under the 1979 Free Field Earthquake Record (cont....)

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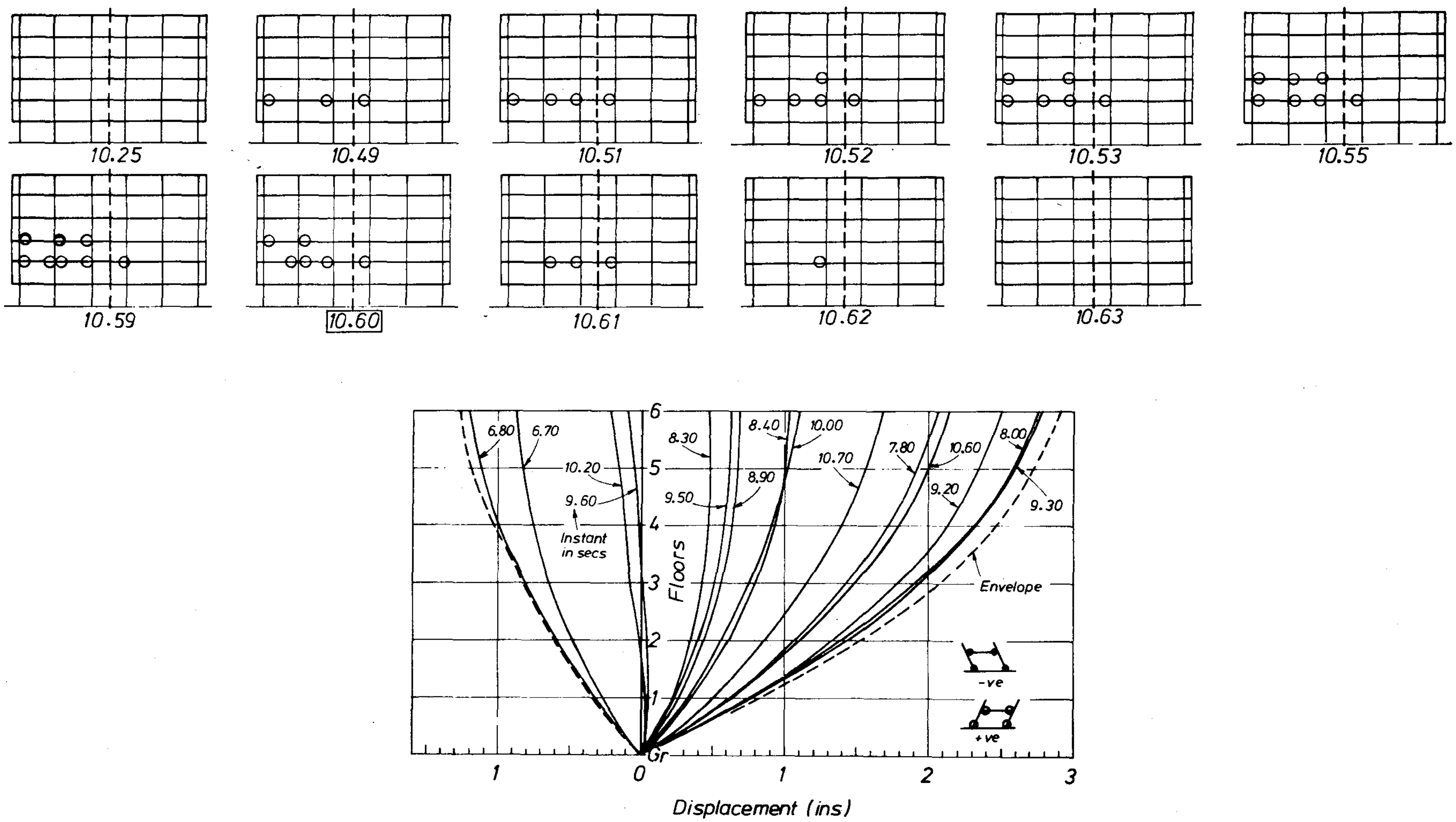

Figure 27 - Plastic Hinge Formation and Deflection Profiles Under the 1979 Free Field Earthquake Record

Acknowledgements

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A special thanks go to two advisory board members of this project, Chris Rojahn of USGS and John Ragsdale of CDMG. Chris played a vital role in having the Imperial County Services Building instrumented with strong motion accelerographs - his wisdom and foresight has now provided engineers and scientists with an excellent and significant strong motion data base for this building. John, under the direction of Tom Wootton of the Office of Strong Motion Studies of CDMG, was helpful in enlisting the resources of CDMG for this project. Additionally Larry Porter of CDMG was instrumental in disseminating the corrected strong motion records of the ICSB and the free field station.

A particular warm and generous thanks goes to Randy Rister, Assistant Director of Buildings and Grounds. Without Randy's enthusiasm, much less his permission to test the building, the building quite literally would not have been tested. Additionally Randy has been quite helpful in providing the original structural drawings and design calculations of the ICSB.

The Computer facilities were provided by the Computer Centre of the University of Canterbury.

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APPENDIX A

Strong Motion Data - Paul C. Jennings

The records of strong ground motion during the Imperial Valley earthquake of October 15 are a uniquely valuable contribution to earthquake engineering and seismology. Most of the data comes from the programs of the USGS Seismic Engineering Branch and the State of California's Office of Strong-Motion Studies. Several records were also obtained from a U.S.-Mexico cooperative program operated by the University of California, San Diego. The first two organizations have already issued preliminary reports describing the records, as listed below:

"Preliminary Summary of the U.S. Geological Survey Strong-Motion Records from the October 15, 1979 Imperial Valley Earthquake", Open File Report 79-1654. U.S. Geological Survey, 345 Middlefield Rd, Menlo Park, Calif. 94025.

"Preliminary Data - Partial Film Records and File Data - Imperial Valley Earthquake of 15 October 1979", Office of Strong-Motion Studies, California Division of Mines and Geology, 2811 O Street, Sacramento, California 95816.

The accelerograms obtained by these two agencies in the main shock are expected to be available in processed, digital form in approximately three months. Significant aftershock data will probably be processed over a longer time. The data from the array operated by the University of California, San Diego, which is almost all from digital accelerographs, will also be made available in the course of the research project. James Brune is the principal U.S. investigator for the project.

Significant accelerograms were obtained from approximately fifty sites in the Imperial Valley, including five special arrays.

1. A 13-accelerograph array transverse to the Imperial fault at El Centro. The instruments are spaced at about 3 km intervals at distances from about one kilometer up to 12 km from the fault (USGS).

2. A 13-transducer array in the Imperial County Services Building. This set of data is the first response ever measured in a building that received major structural damage (CDMG).

3. A 26-transducer array at the Mellowland Freeway Overpass on Interstate 8. The freeway structure, apparently undamaged by the earthquake, is approximately 4 km from the fault (CDMG).

4. A 6-instrument, linear array of digital accelerographs spanning 1000 ft. This array, which is near El Centro, is intended to measure the lateral variation of strong ground motion (USGS).

5. An array of fourteen strong-motion accelerographs located in Mexico. These instruments are in the Mexican portion of the Imperial Valley from the border south to near the Gulf of California (UCSD).

The measurements obtained from instruments 10 km or more from the fault resemble those obtained from other earthquakes of this magnitude range. The records obtained nearer the fault, however, show several unusual features. These include a high-frequency, vertical peak acceleration of 1.7g from an instrument one kilometer from the fault; a large, unusually shaped pulse in the horizontal motion at another site also at one kilometer from the fault; generally stronger shaking in the vertical direction than the horizontal at some near-field sites; and large amplitude high frequency components on some of the records obtained in Mexico.

The records obtained from the Imperial County Services Building show the initial, linear response of the building, the large elongation of period in the E-W direction that is associated with hinging in the first floor column; and high frequency pulses that probably record the point when failure of the easternmost bay of columns occurred, and this end of the building settled approximately a foot. These records therefore represent unprecedented data of earthquake response and will be of great value in both research and practice.

The records of strong ground motion and structural response obtained in this earthquake represent a major advance in our efforts to understand earthquakes and their effects. The quantity and quality of the data is truly exceptional and shows convincingly the value of the basic instrumentation programs in earthquake engineering.

APPENDIX B

Program Name: ETABS.....EXTENDED THREE-DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS.

Developed by: E.L. Wilson, J.P. Hollings, and H.H. Dovey
Department of Civil Engineering
University of California
Berkeley, California.

Program Date: April 1975.

Summary: ETABS is designed to perform linear structural analysis of frame and shear wall buildings subjected to both static and earthquake loadings. The building is idealised by a system of independent frame and shear wall elements interconnected by floor diaphragms which are rigid in their own plane. Frame and shear wall elements of arbitrary plan may be specified, within which full kinematic compatibility is enforced. Bending, axial, and shearing deformations are included within each column. Beams, girders, and vertical diagonal braces may be non-prismatic, and bending and shearing deformations are included. Special panel elements allow discontinuous shear walls to be modelled. Finite column and beam widths are included in the formulation. Nonsymmetric, non-rectangular buildings that have frames and shear walls located arbitrarily in plan can be considered. Axial deformations of common column lines of different frames are treated as uncoupled by the program.

Three independent vertical and two lateral static loading conditions are possible. The static loads may be combined with a lateral earthquake input that is specified either as an acceleration spectrum response or as a ground acceleration record. Three dimensional mode shapes and frequencies are evaluated.

The program is written in FORTRAN IV with dynamic storage allocation for major arrays in blank COMMON.

Data Preparation: Frame and shear walls are considered as substructures in the basic formulation; therefore, for many structures input data preparation can be minimised and a significant reduction in computational effort can result.

Output: In addition to a printout of all input data, the following output is given by the program: storey displacements, mode shapes and periods, lateral frame displacements, frame member forces at each level of the frame. Results can be printed for as many modes under consideration as are desired.

Machine Versions Available: CDC 6400, IBM 370.

Reference: Wilson, E.L., Hollings, J.P. and Dovery, H.H. (1975), "Three Dimensional Analysis of Building Systems (Extended Version)," Earthquake Engineering Research Center, Report No. EERC 75-13, University of California, Berkeley, April 1975.

NISEE Program Number: 24-675

APPENDIX C

MASS CALCULATIONS FOR

IMPERIAL COUNTY SERVICES BUILDING

UNIVERSITY OF CANTERBURY SCHOOL OF ENGINEERING

PROJECT: Imperial County Services Building
SUBJECT: Mass Properties - 2nd Floor
PAGE NO: 1 OF 6
BY: G. C. Pardoen
DATE:

Shear Walls

East Shear Wall

$(85'4'') \times \frac{1}{2}(13'6'') \times 7\frac{1}{2}'' = 360.00\ ft^3$

West Shear Wall

$(85'4'' - 8') \times \frac{1}{2}(13'6'') \times 7\frac{1}{2}'' = 326.25\ ft^3$

$(8') \times (74'') \times 10'' = 41.11\ ft^3$

$2(9'') \times \frac{1}{2}(13'6'') \times 16'' = 13.50\ ft^3$

Total Concrete Volume: $740.86\ ft^3$ ←

Slab

$136'10'' \times 75' \times 5'' = 4276.04\ ft^3$ ←

Joists

$\frac{1}{2}(5\frac{1}{2}'' + 7\frac{1}{2}'') \times 14'' \times 25' = 15.80\ ft^3/\text{joist}$

Assume 44 joists in N and S bays (reasonable estimate)
Assume 44 joists in middle bay (some guesswork here)

Total Concrete Volume $= (132) \times (15.80\ ft^3/\text{joist})$
$= 2085.60\ ft^3$ ←

Beams

$2' \times 2'6'' \times 136'10'' = 684.17\ ft^3/\text{frame}$

Total Concrete Volume $= (4\ \text{frames}) \times 684.17\ ft^3/\text{frame}$
$= 2736.68\ ft^3$ ←

Columns

Interior Columns

$$2' \times 2' \times [1/2(14'6" + 13'6")] = 56 \text{ ft}^3/\text{int. col.}$$

Exterior Columns

$$[1'6" \times 1' + 1/2(1'6" + 10") \times 4'10"] \times 1/2(13'6") +$$

$$2' \times 2' \times 1/2(14'6") = 77.19 \text{ ft}^3/\text{ext. col.}$$

$$\text{Total Concrete Volume} = (12 \text{ int. cols.}) \times (56 \text{ ft}^3/\text{int. col.})$$

$$+ (12 \text{ ext. col.}) \times (77.19 \text{ ft}^3/\text{ext. col.}) = 1598.28 \text{ ft}^3$$

Weight of Concrete

Shear Walls	740.86 ft³
Slab	4276.04 ft³
Joists	2085.60 ft³
Beams	2736.68 ft³
Columns	1598.28 ft³
	11437.46 ft³

Assume $\rho_{concrete} = .15^k/\text{ft}^3$

$$W_c = (.15^k/\text{ft}^3)(11437.46 \text{ ft}^3)$$

$$W_c = 1715.619 \text{ Kips}$$

Weight of Other Items (Estimates)

Ceiling/mechanical	10 lb/ft²
Partitions	10 lb/ft²
Flooring	2 lb/ft²
Miscellaneous	2 lb/ft²
	24 lb/ft²

$$\text{Area} = 75' \times 136'10" = 10262.5 \text{ ft}^2$$

$$W_m = (24 \text{ lb/ft}^2)(10262.5 \text{ ft}^2)$$

$$W_m = 246.300 \text{ Kips}$$

Total Weight

$$W = W_c + W_m = 1961.919 \text{ Kips}$$

Mass Properties

$$M = W/g = (1961.919 \text{ k})/386.4 \text{ in/sec}^2 = 5.077430124 \text{ K-sec}^2/\text{in}$$

$$I = (W/g)(a^2 + b^2)/12 = (5.077430124) \times (136'10"^2 + 75'^2)/12$$

$$I = 1483525.226 \text{ K-in-sec}^2$$

Shear Walls

East Shear Wall

(85'4") × (13'6") × 7" = 672 ft^3

(85'4") × ½ (13'6") × ½" = 24 ft^3 increment for 3rd floor only

West Shear Wall

(85'4" − 8') × (13'6") × 7" = 609 ft^3

(85'4" − 8') × ½ (13'6") × ½" = 21.75 ft^3 increment for 3rd floor only

2 (9") × 13'6" × 16" = 27 ft^3

(8') × 74" × 10" = 41.11 ft^3

Total Concrete Volume: 1394.86 ft^3 (3rd floor) ←

1349.11 ft^3 (4th, 5th, 6th floors) ←

Slab

136'10" × 75' × 3" = 2565.63 ft^3 ←

Joists

(132 joists) × 15.80 ft^3/joist = 2085.60 ft^3 ←

Beams

2' × 2'6" × 136'10" = 684.17 ft^3/int. frame

10" × 4'6" × 136'10" = 513.12 ft^3/ext. frame

Total Concrete Volume:

(2 int. frames) × 684.17 ft^3/int. frame

+ (2 ext. frames) × 513.12 ft^3/ext. frame = 2394.58 ft^3 ←

UNIVERSITY OF CANTERBURY SCHOOL OF ENGINEERING

PROJECT: Imperial County Services Building
SUBJECT: Mass Properties – 3rd, 4th, 5th, 6th Floors
BY: G. C. Pardoen DATE:

Columns

$2' \times 2' \times 13'6'' = 54\ ft^3/\text{int. col.}$

$[1'6'' \times 1' + \frac{1}{2}(1'6'' + 10'') \times 4'10''] \times 13'6'' = 96.38\ ft^3/\text{ext. col.}$

Total Concrete Volume = (12 int. cols) × 54 ft^3/int. col.

+ (12 ext. cols.) × (96.38 ft^3/ext. col) = 1804.56 ft^3 ←

Weight of Concrete

Shear Walls (3rd Floor)	1394.86 ft^3
Slab	2565.63 ft^3
Joists	2085.60 ft^3
Beams	2394.58 ft^3
Columns	1804.56 ft^3
	10245.23 ft^3 ← 3rd Floor

Assume $\rho_{concrete} = .15\,K/ft^3$

10199.48 ft^3 ← 4th, 5th, 6th Floors

$W_c = (.15^k/ft^3)\ 10245.23\ ft^3 = 1536.785^k$ ←
(3rd Floor)

$W_c = (.15^k/ft^3)\ 10199.48\ ft^3 = 1529.922^k$ ←
(4th, 5th, 6th Floor)

Weight of Other Items (Estimates)

Assume distributed loading of 24 lb/ft^2 (same as 2nd Floor) acting over 10262.5 ft^2

$W_m = 246.300\ K$ ←

Total Weight

$W = W_c + W_m = 1783.085^k$ (3rd Floor) or 1776.222^k (4th, 5th, 6th Floors)

Mass Properties

$M = W/g = (1783.085^k)/386.4\,in/sec^2 = 4.614609213\ K\text{-}sec^2/in$ ←
(3rd Floor)

$(1776.222^k)/386.4\,in/sec^2 = 4.596847826\ K\text{-}sec^2/in$ ←
(4th, 5th, 6th Floors)

$I = W/g\,(a^2 + b^2)/12 = 1348298.058\ K\text{-}in\text{-}sec^2$ ←
(3rd Floor)

$= 1343108.530\ K\text{-}in\text{-}sec^2$ ←
(4th, 5th, 6th Floors)

UNIVERSITY OF CANTERBURY SCHOOL OF ENGINEERING

PROJECT: Imperial County Services Building
SUBJECT: Mass Properties - Roof & Penthouse PAGE NO: 5 OF 6
BY: G. C. Pardoen DATE:

Shear Walls

East Shear Wall

(85'4") × (13'2" − ½ 13'6") × 7" = 319.41 ft^3

West Shear Wall

(85'4" − 8') × (13'2" − ½ 13'6") × 7" = 289.46 ft^3

2 (9") × (13'2" − ½ 13'6") × 16" = 12.83 ft^3

8' × 34" × 10" = 18.89 ft^3

Total Concrete Volume: 640.59 ft^3 ←

Slab

136'10" × 75' × 3" + 25' × 50' × 3" = 2878.13 ft^3 ←

Joists

½ (5½" + 7½") × 10" × 25' = 11.28 ft^3/joist

½ (5½" + 7½") × 14" × 25' = 15.80 ft^3/joist

Assume (110 joists) × 11.28 ft^3/joist

+ (22 joists) × 15.80 ft^3/joist = 1588.40 ft^3 ←

Beams

2' × 2'6" × 136'10" = 684.17 ft^3/int. frame

10" × 4'2" × 136'10" = 475.12 ft^3/ext. frame

Total Concrete Volume:

(2 int. frames) × 684.17 ft^3/int. frame

+ (2 ext. frames) × 475.12 ft^3/ext. frame = 2318.58 ft^3 ←

UNIVERSITY OF CANTERBURY SCHOOL OF ENGINEERING

PROJECT: Imperial County Services Building
SUBJECT: Mass Properties - Roof & Penthouse PAGE NO: 6 OF 6
BY: G.C. Pardoen DATE:

Columns

Interior Columns

$2' \times 2' \times (13'2" - \frac{1}{2}\, 13'6") = 25.67\ ft^3/\text{int. col.}$

Exterior Columns

$[1'6" \times 1' + \frac{1}{2}(1'6" + 10") \times 4'10"] \times (13'2" - \frac{1}{2}\, 13'6") = 45.81\ ft^3/\text{ext. col}$

Total Concrete Volume: $(12\ \text{int. cols}) \times (25.67\ ft^3/\text{int. col.})$
$+ (12\ \text{ext. cols}) \times (45.81\ ft^3/\text{ext. col.}) = 857.73\ ft^3$ ←

Weight of Concrete

Assume $\rho_{concrete} = .15^k/ft^3$

Shear Walls	640.59 ft^3
Slab	2878.13 ft^3
Joists	1588.40 ft^3
Beams	2318.58 ft^3
Columns	857.73 ft^3
	8283.43 ft^3

$W_c = (8283.43\ ft^3) \times .15^k/ft^3 = 1242.515^k$ ←

Weight of Other Items (Estimates)

Roofing: $3\ lb/ft^2 \times (136'10") \times 75' = 30.788^k$

Penthouse Structure: 20 K

Machinery: 50^k

$W_m = 100.788^k$

Mass Properties

$M = W/g = (1343.303/386.4) = 3.476455745\ K\text{-}sec^2/in$ ←

$I = (W_c/g)(a_1^2 + b_1^2)/12 + (W_m/g)(a_2^2 + b_2^2)/12$

$= (1242.515^k/386.4\ in/sec^2)(136'10"^2 + 75'^2)/12$

$+ (100.788/386.4\ in/sec^2)(25'^2 + 50'^2)/12$

$I = 949321.9380\ K\text{-}in\text{-}sec^2$ ←

APPENDIX D

ETABS INPUT DATA

CARD IMAGE OF **E T A B S** INPUT DATA

STATIC ANALYSIS--1967 UBC & VERTICAL LOADING

```
   6    4    7    3    2    8    0IMPERIAL CO SERVICES BLDG*1967 UBC & VERTICAL
ROOF       158.       3.47646   949321.94  821.      450.
 137.04146   0.         821.       450.      0.      206.90411   821.       450.
6TH        162.       4.59685  1343108.53  821.      450.
 151.99215   0.         821.       450.      0.      229.47654   821.       450.
5TH        162.       4.59685  1343108.53  821.      450.
 122.03749   0.         821.       450.      0.      184.25123   821.       450.
4TH        162.       4.59685  1343108.53  821.      450.
  92.08284   0.         821.       450.      0.      139.02593   821.       450.
3RD        162.       4.61461  1348298.06  821.      450.
  62.36823   0.         821.       450.      0.       94.16306   821.       450.
2ND        174.       5.07743  1483525.23  821.      450.
  35.53714   0.         821.       450.      0.       53.65368   821.       450.
    1    6   18   14    3    3    4   20    0**EXTERIOR FRAME WITH SHEAR WALLS**
    1    3.5        0.
    2   71.0        0.
    3  371.0        0.
    4  671.0        0.
    5  971.0        0.
    6 1271.0        0.
    7 1571.0        0.
    8 1638.5        0.
    9 1638.5      900.
   10 1571.0      900.
   11 1271.0      900.
   12  971.0      900.
   13  671.0      900.
   14  371.0      900.
   15   71.0      900.
   16    3.5      900.
   17    3.5      570.
   18    3.5      458.
    1 4030.50865      1028.      0.      0.   441094.78   20177.33  420917.45  50.
    2 4030.50865      1028.      0.      0.   441094.78   20177.33  420917.45  53.
    3 4030.50865       576.      0.      0.    55296.     27648.     27648.    30.
    1 3604.99653        0.  108333.33 104166.67  4.    4.    2.    9.    9.
    2 3604.99653        0.  128480.83 124064.17  4.    4.    2.    9.    9.
    3 3604.99653        0.   88560.00  54000.00  4.    4.    2.    9.    9.
```

1	0							.224632609
2	0							.297027090
3	0							.298174749
4	0							.328080100
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1	1	2	2	2	2	0	0	
1	1	2	2	0	3	0	0	
1	1	2	3	0	4	0	0	
2	2	3	1	0	1	0	0	
2	2	3	2	2	2	0	0	
2	2	3	2	0	3	0	0	
2	2	3	3	0	4	0	0	
3	3	4	1	0	1	0	0	
3	3	4	2	2	2	0	0	
3	3	4	2	0	3	0	0	
3	3	4	3	0	4	0	0	
4	4	5	1	0	1	0	0	
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4	4	5	2	0	3	0	0	
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6	6	7	2	0	3	0	0	
6	6	7	3	0	4	0	0	
7	7	8	1	0	1	0	0	
7	7	8	2	2	2	0	0	
7	7	8	2	0	3	0	0	
7	7	8	3	0	4	0	0	
8	9	10	1	0	1	0	0	
8	9	10	2	2	2	0	0	
8	9	10	2	0	3	0	0	
8	9	10	3	0	4	0	0	
9	10	11	1	0	1	0	0	
9	10	11	2	2	2	0	0	
9	10	11	2	0	3	0	0	
9	10	11	3	0	4	0	0	

10	11	12	1	0	1	0	0
10	11	12	2	2	2	0	0
10	11	12	2	0	3	0	0
10	11	12	3	0	4	0	0
11	12	13	1	0	1	0	0
11	12	13	2	2	2	0	0
11	12	13	2	0	3	0	0
11	12	13	3	0	4	0	0
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12	13	14	2	0	3	0	0
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13	14	15	1	0	1	0	0
13	14	15	2	2	2	0	0
13	14	15	2	0	3	0	0
13	14	15	3	0	4	0	0
14	15	16	1	0	1	0	0
14	15	16	2	2	2	0	0
14	15	16	2	0	3	0	0
14	15	16	3	0	4	0	0
1	0	2	5				
2	1	3	0				
2	2	3	3				
2	3	3	0				
3	1	4	0				
3	2	4	3				
3	3	4	0				
4	1	5	0				
4	2	5	3				
4	3	5	0				
5	1	6	0				
5	2	6	3				
5	3	6	0				
6	1	7	0				
6	2	7	3				
6	3	7	0				
7	1	8	0				
7	2	8	3				
7	3	8	0				
8	0	7	5				

9	0	10	5			
10	1	11	0			
10	2	11	3			
10	3	11	0			
11	1	12	0			
11	2	12	3			
11	3	12	0			
12	1	13	0			
12	2	13	3			
12	3	13	0			
13	1	14	0			
13	2	14	3			
13	3	14	0			
14	1	15	0			
14	2	15	3			
14	3	15	0			
15	1	16	0			
15	2	16	3			
15	3	16	0			
16	0	15	5			
17	0	18	5			
18	0	17	5			
6	8	93604.99653	6300.	0.	5250.	1567.38980
5	8	93604.99653	6300.	0.	5250.	1567.38980
4	8	93604.99653	6300.	0.	5250.	1567.38980
3	8	93604.99653	6300.	0.	5250.	1567.38980
2	8	93604.99653	6750.	0.	5625.	1567.38980
6	16	173604.99653	2310.	0.	1925.00	1567.38980
5	16	173604.99653	2310.	0.	1925.00	1567.38980
4	16	173604.99653	2310.	0.	1925.00	1567.38980
3	16	173604.99653	2310.	0.	1925.00	1567.38980
2	16	173604.99653	2475.	0.	2062.50	1567.38980
6	17	183604.99653	425.32	0.	354.43	1567.38980
5	17	183604.99653	438.52	0.	365.43	1567.38980
4	17	183604.99653	438.52	0.	365.43	1567.38980
3	17	183604.99653	438.52	0.	365.43	1567.38980
2	17	183604.99653	438.52	0.	365.43	1567.38980
6	18	13604.99653	3206.	0.	2671.67	1567.38980
5	18	13604.99653	3206.	0.	2671.67	1567.38980
4	18	13604.99653	3206.	0.	2671.67	1567.38980

133

```
3   18     13604.99653  3206.      0.        2671.67  1567.38980
2   18     13604.99653  3435.      0.        2862.50  1567.38980
2    6     8     7     3     1     4     0    0**INTERIOR FRAME FOR E-W LOADING**
1    3.5          0.
2   71.0          0.
3  371.0          0.
4  671.0          0.
5  971.0          0.
6 1271.0          0.
7 1571.0          0.
8 1638.5          0.
1 4030.50865          576.      0.    0.   55296.     27648.     27648.    50.
2 4030.50865          576.      0.    0.   55296.     27648.     27648.    53.
3 4030.50865          576.      0.    0.   55296.     27648.     27648.    30.
1 3604.99653        0.  88560.     54000.     4.    4.   2.  12.  12.
1    0                                              .224632609
2    0                                              .297027090
3    0                                              .298174749
4    0                                              .328080100
1    1     2     1     0     1     0     0
1    1     2     1     2     2     0     0
1    1     2     1     0     3     0     0
1    1     2     1     0     4     0     0
2    2     3     1     0     1     0     0
2    2     3     1     2     2     0     0
2    2     3     1     0     3     0     0
2    2     3     1     0     4     0     0
3    3     4     1     0     1     0     0
3    3     4     1     2     2     0     0
3    3     4     1     0     3     0     0
3    3     4     1     0     4     0     0
4    4     5     1     0     1     0     0
4    4     5     1     2     2     0     0
4    4     5     1     0     3     0     0
4    4     5     1     0     4     0     0
5    5     6     1     0     1     0     0
5    5     6     1     2     2     0     0
5    5     6     1     0     3     0     0
5    5     6     1     0     4     0     0
6    6     7     1     0     1     0     0
```

```
6   6   7   1   2   2   0   0
6   6   7   1   0   3   0   0
6   6   7   1   0   4   0   0
7   7   8   1   0   1   0   0
7   7   8   1   2   2   0   0
7   7   8   1   0   3   0   0
7   7   8   1   0   4   0   0
1   0   2   5
2   1   3   0
2   2   3   3
2   3   3   0
3   1   4   0
3   2   4   3
3   3   4   0
4   1   5   0
4   2   5   3
4   3   5   0
5   1   6   0
5   2   6   3
5   3   6   0
6   1   7   0
6   2   7   3
6   3   7   0
7   1   8   0
7   2   8   3
7   3   8   0
8   0   7   5
3   1   2   0   0   0   0   1    0INT SHEAR WALL AT COL LINES B,D,F
1   0.       0.
2   0.     300.
1   0   2   0
2   0   1   0
1   1   23604.99653  3600.     0.      3000.    1567.38980
4   1   2   0   0   0   0   1    0INT SHEAR WALL AT COL LINE E
1   0.       0.
2   0.     246.
1   0   2   0
2   0   1   0
1   1   23604.99653  2952.     0.      2460.    1567.38980
1   0   0.        0.        0.    **EXTERIOR FRAME FOR E-W AND N-S LOADS**
```

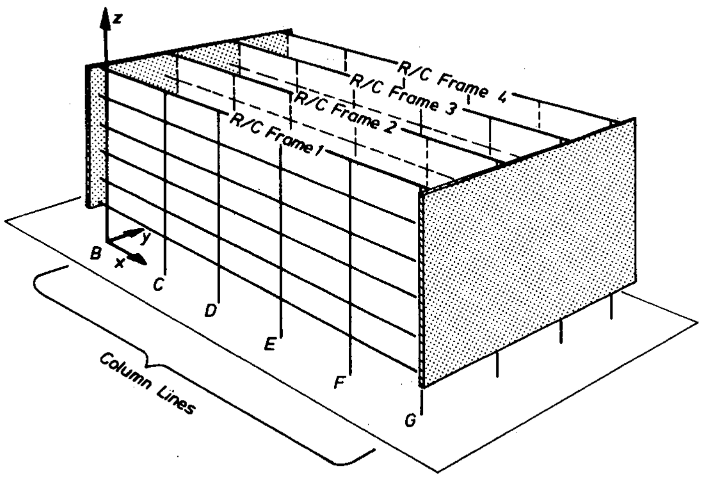
```
2     0     0.        300.          0.     **SOUTH INTERIOR FRAME FOR E-W LOADING**
2     0     0.        600.          0.     **NORTH INTERIOR FRAME FOR E-W LOADING**
3     0    71.        300.          0.     **INTERIOR SHEAR WALL AT COLUMN LINE B**
3     0   671.        300.          0.     **INTERIOR SHEAR WALL AT COLUMN LINE D**
3     0  1271.        300.          0.     **INTERIOR SHEAR WALL AT COLUMN LINE F**
4     0   971.        300.          0.     **INTERIOR SHEAR WALL AT COLUMN LINE E**
 0.          0.         0.         1.         0.         0.         0.         0.
 0.          0.         0.         0.         1.         0.         0.         0.
 1.          0.         0.         0.         0.         0.         0.         0.
```

APPENDIX E

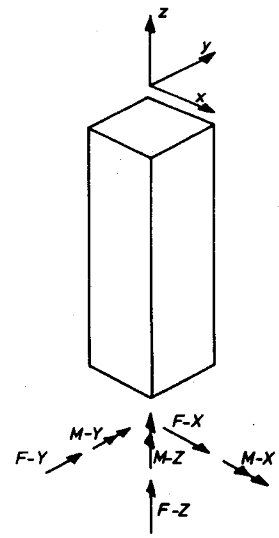
COLUMN FORCES AT GROUND LEVEL

ELASTIC ANALYSES

IMPERIAL COUNTY SERVICES BUILDING



COLUMN DESIGNATIONS



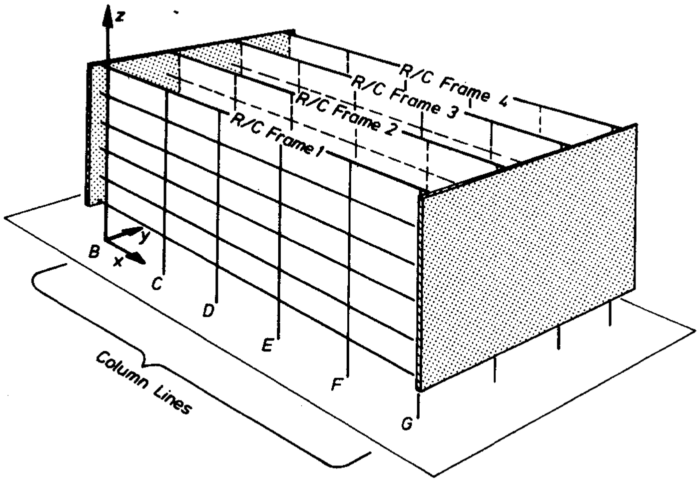
GROUND LEVEL COLUMN FORCES

1967 UBC CODE--LATERAL FORCES IN X-DIRECTION

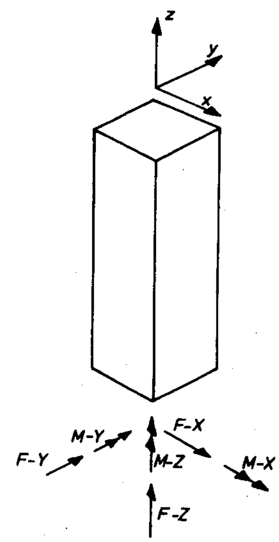
COLUMN FORCES AT GROUND LEVEL

FRAME	COLUMN LINE	F-X	F-Y	F-Z	M-X	M-Y	M-Z
1	B	21.1	0.7	-75.8	51.1	-1923.6	1.0
1	C	26.6	0.5	4.1	37.4	-2226.2	1.0
1	D	26.2	0.3	0.3	23.7	-2208.7	1.0
1	E	26.2	0.1	-0.2	10.0	-2206.7	1.0
1	F	26.8	0.0	-5.0	-3.6	-2237.7	1.0
1	G	19.5	-0.2	70.9	-17.3	-1839.0	1.0
2	B	21.1	0.3	-53.2	31.1	-1927.4	1.0
2	C	28.1	0.2	1.6	22.5	-2314.2	1.0
2	D	27.4	0.1	-0.1	14.3	-2276.6	1.0
2	E	27.4	0.1	0.1	6.1	-2276.6	1.0
2	F	28.1	0.0	-1.6	-2.2	-2314.2	1.0
2	G	21.1	-0.1	53.2	-10.8	-1927.4	1.0
3	B	21.2	0.3	-53.3	31.1	-1938.2	1.0
3	C	28.3	0.2	1.6	22.5	-2326.5	1.0
3	D	27.6	0.1	-0.1	14.3	-2288.7	1.0
3	E	27.6	0.1	0.1	6.1	-2288.7	1.0
3	F	28.3	0.0	-1.6	-2.2	-2326.5	1.0
3	G	21.2	-0.1	53.3	-10.8	-1938.2	1.0
4	B	20.2	0.7	-63.6	51.1	-1886.9	1.0
4	C	27.2	0.5	5.8	37.4	-2272.4	1.0
4	D	26.7	0.3	0.2	23.7	-2243.4	1.0
4	E	26.7	0.1	-0.3	10.0	-2242.8	1.0
4	F	27.2	0.0	-5.0	-3.6	-2273.8	1.0
4	G	20.0	-0.2	71.3	-17.3	-1873.2	1.0

** NOTE ** FORCES IN KIPS, MOMENTS IN KIP-IN



COLUMN DESIGNATIONS



GROUND LEVEL COLUMN FORCES

1967 UBC CODE---LATERAL FORCES IN Y-DIRECTION

COLUMN FORCES AT GROUND LEVEL

FRAME	COLUMN LINE	F-X	F-Y	F-Z	M-X	M-Y	M-Z
1	B	15.9	-7.4	-153.3	-489.2	-1313.6	-80.8
1	C	18.2	6.9	-2.8	585.3	-1439.7	-80.8
1	D	18.5	21.1	0.8	1658.1	-1456.3	-80.8
1	E	18.5	35.4	1.2	2731.0	-1453.6	-80.8
1	F	18.2	49.7	-1.7	3803.9	-1439.9	-80.8
1	G	29.3	64.0	-284.4	4878.3	-2049.9	-80.8
2	B	5.4	-3.7	-2.5	-283.3	-445.3	-80.8
2	C	6.6	3.5	0.2	402.0	-512.1	-80.8
2	D	6.4	10.2	0.0	1056.3	-503.7	-80.8
2	E	6.4	16.8	0.0	1708.8	-503.7	-80.8
2	F	6.6	23.5	-0.2	2363.1	-512.1	-80.8
2	G	5.4	30.7	2.5	3048.3	-445.3	-80.8
3	B	-5.0	-3.7	2.0	-283.3	414.0	-80.8
3	C	-6.1	3.5	-0.2	402.0	475.5	-80.8
3	D	-6.0	10.2	0.0	1056.3	467.8	-80.8
3	E	-6.0	16.8	0.0	1708.8	467.8	-80.8
3	F	-6.1	23.5	0.2	2363.1	475.5	-80.8
3	G	-5.0	30.7	-2.0	3048.3	414.0	-80.8
4	B	-11.6	-7.4	105.0	-489.2	1063.1	-80.8
4	C	-18.3	6.9	-0.1	585.3	1430.9	-80.8
4	D	-18.0	21.1	-0.4	1658.1	1419.0	-80.8
4	E	-18.1	35.4	-1.1	2731.0	1418.9	-80.8
4	F	-17.8	49.7	1.6	3803.9	1404.3	-80.8
4	G	-29.0	64.0	285.0	4878.3	2019.8	-80.8

** NOTE **   FORCES IN KIPS, MOMENTS IN KIP-IN

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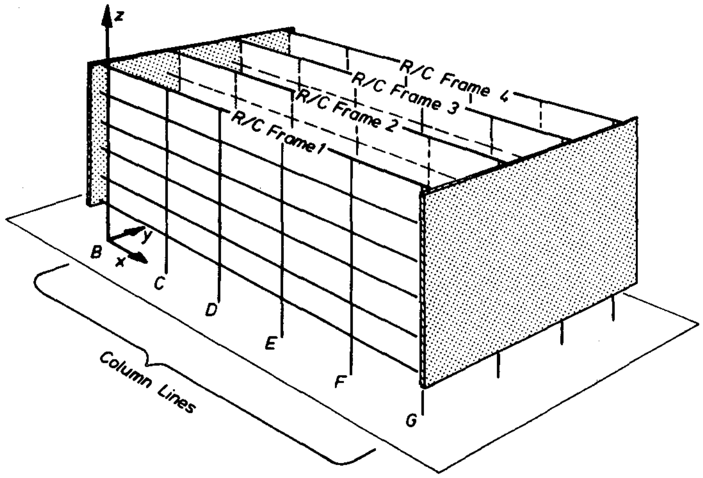
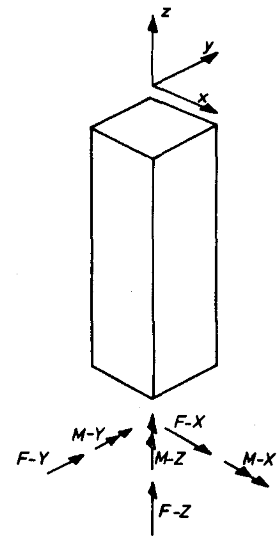
IMPERIAL COUNTY SERVICES BUILDING



COLUMN DESIGNATIONS



GROUND LEVEL COLUMN FORCES

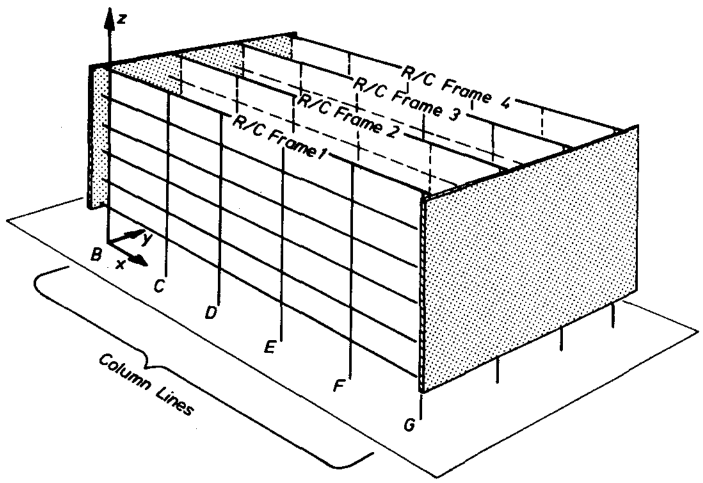
VERTICAL DEAD WEIGHT LOADING

COLUMN FORCES AT GROUND LEVEL

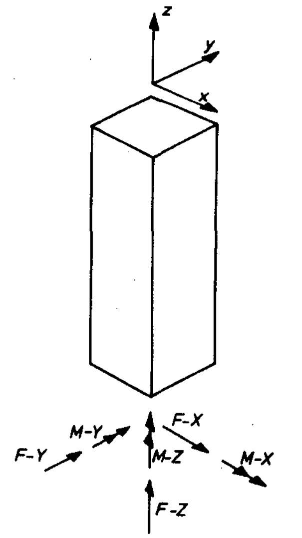
FRAME	COLUMN LINE	F-X	F-Y	F-Z	M-X	M-Y	M-Z
1	B	-4.2	-1.4	330.8	-106.8	222.6	-1.8
1	C	0.4	-1.1	488.0	-83.4	32.7	-1.8
1	D	0.3	-0.8	492.2	-60.1	-24.8	-1.8
1	E	0.4	-0.5	492.1	-36.7	-31.3	-1.8
1	F	-0.2	-0.2	489.6	-13.4	1.5	-1.8
1	G	8.1	0.1	333.1	10.0	-452.0	-1.8
2	B	-7.4	-0.7	320.0	-67.6	407.9	-1.8
2	C	0.6	-0.5	476.9	-52.5	-33.8	-1.8
2	D	0.0	-0.4	480.8	-38.0	-0.4	-1.8
2	E	0.2	-0.2	480.8	-23.6	-10.7	-1.8
2	F	-0.4	-0.1	476.9	-9.1	22.5	-1.8
2	G	7.6	0.1	320.0	6.0	-417.8	-1.8
3	B	-7.7	-0.7	320.0	-67.6	427.0	-1.8
3	C	0.3	-0.5	476.9	-52.5	-11.8	-1.8
3	D	-0.3	-0.4	480.8	-38.0	21.3	-1.8
3	E	-0.1	-0.2	480.8	-23.6	11.0	-1.8
3	F	-0.7	-0.1	476.9	-9.1	44.6	-1.8
3	G	7.3	0.1	320.0	6.0	-398.7	-1.8
4	B	-8.2	-1.4	337.2	-106.8	463.9	-1.8
4	C	0.0	-1.1	491.0	-83.4	9.7	-1.8
4	D	-0.6	-0.8	492.0	-60.1	41.7	-1.8
4	E	-0.4	-0.5	492.0	-36.7	31.9	-1.8
4	F	-1.0	-0.2	489.7	-13.4	67.0	-1.8
4	G	7.5	0.1	332.1	10.0	-401.8	-1.8

** NOTE ** FORCES IN KIPS, MOMENTS IN KIP-IN

IMPERIAL COUNTY SERVICES BUILDING



COLUMN DESIGNATIONS



GROUND LEVEL COLUMN FORCES

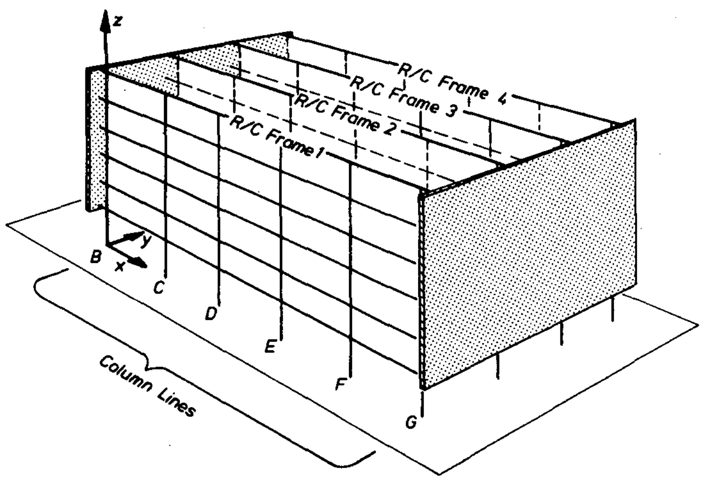
1940 IMPERIAL VALLEY EARTHQUAKE (N-S COMPONENT) IN X-DIRECTION

COLUMN FORCES AT GROUND LEVEL

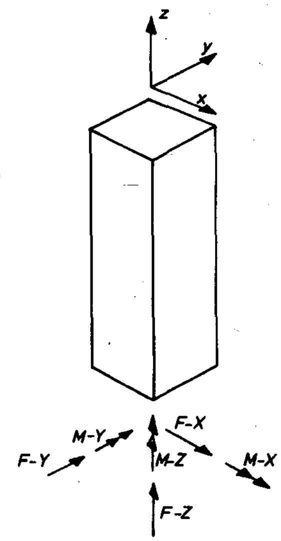
FRAME	COLUMN LINE	F-X	F-Y	F-Z	M-X	M-Y	M-Z
1	B	200.4	6.8	723.1	507.4	18227.1	8.0
1	C	251.0	5.4	38.8	405.2	21009.4	8.0
1	D	248.1	4.1	3.2	303.2	20851.5	8.0
1	E	247.7	2.7	2.2	201.2	20831.8	8.0
1	F	253.0	1.3	47.1	99.2	21122.9	8.0
1	G	185.3	2.0	671.1	147.0	17391.7	8.0
2	B	199.2	3.2	502.7	307.6	18184.8	8.0
2	C	265.2	2.5	14.8	243.6	21817.9	8.0
2	D	258.8	1.9	0.8	182.5	21464.5	8.0
2	E	258.8	1.2	0.8	121.6	21464.5	8.0
2	F	265.2	0.6	14.8	60.5	21817.9	8.0
2	G	199.2	1.0	502.7	90.2	18184.8	8.0
3	B	200.2	3.2	503.5	307.6	18264.6	8.0
3	C	266.4	2.5	14.8	243.6	21909.2	8.0
3	D	259.9	1.9	0.8	182.5	21554.4	8.0
3	E	259.9	1.2	0.8	121.6	21544.4	8.0
3	F	266.4	0.6	14.8	60.5	21909.2	8.0
3	G	200.2	1.0	503.5	90.2	18264.6	8.0
4	B	190.7	6.8	603.7	507.4	17769.1	8.0
4	C	256.2	5.4	54.8	405.2	21377.5	8.0
4	D	251.3	4.1	1.9	303.2	21105.8	8.0
4	E	251.2	2.7	2.5	201.2	21099.8	8.0
4	F	256.5	1.3	47.2	99.2	21393.1	8.0
4	G	187.9	2.0	681.3	147.0	17617.2	8.0

** NOTE ** FORCES IN KIPS, MOMENTS IN KIP-IN

IMPERIAL COUNTY SERVICES BUILDING



COLUMN DESIGNATIONS



GROUND LEVEL COLUMN FORCES

1940 IMPERIAL VALLEY EARTHQUAKE (N-S COMPONENT) IN Y-DIRECTION

COLUMN FORCES AT GROUND LEVEL

FRAME	COLUMN LINE	F-X	F-Y	F-Z	M-X	M-Y	M-Z
1	B	111.8	32.5	642.7	2039.8	9089.0	552.6
1	C	123.7	65.8	15.5	5344.1	9747.8	552.6
1	D	124.7	163.4	3.9	12681.0	9799.3	552.6
1	E	124.5	261.0	6.7	20017.7	9791.0	552.6
1	F	122.4	358.6	7.6	27354.6	9673.2	552.6
1	G	196.4	456.4	1685.3	34702.1	13747.5	552.6
2	B	35.4	17.2	15.5	1195.5	2918.2	552.6
2	C	43.2	32.5	1.4	3511.0	3350.1	552.6
2	D	42.2	78.1	0.2	7986.1	3295.6	552.6
2	E	42.2	123.5	0.2	12448.7	3295.6	552.6
2	F	43.2	169.1	1.4	16923.7	3350.1	552.6
2	G	35.4	218.6	15.5	21610.0	2918.2	552.6
3	B	35.7	17.2	16.0	1195.5	2956.5	552.6
3	C	43.8	32.5	1.5	3511.0	3401.2	552.6
3	D	42.8	78.1	0.2	7986.0	3345.5	552.6
3	E	42.8	123.5	0.2	12448.7	3345.5	552.6
3	F	43.8	169.1	1.5	16923.7	3401.2	552.6
3	G	35.7	218.6	16.0	21610.0	2956.5	552.6
4	B	80.8	32.5	562.0	2039.8	7402.5	552.6
4	C	128.3	65.8	7.2	5344.1	10019.2	552.6
4	D	124.6	163.4	2.4	12681.0	9814.4	552.6
4	E	125.2	261.0	6.3	20017.7	9845.8	552.6
4	F	123.0	358.6	7.7	27354.6	9723.9	552.6
4	G	196.7	456.4	1684.6	34702.1	13785.7	552.6

** NOTE ** FORCES IN KIPS, MOMENTS IN KIP-IN

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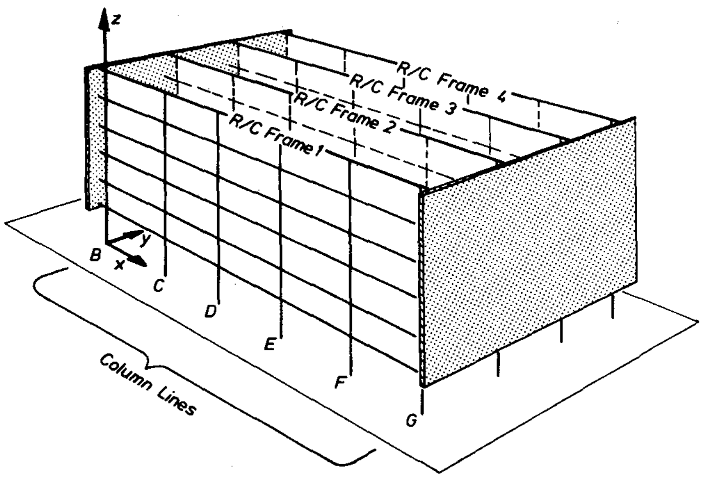
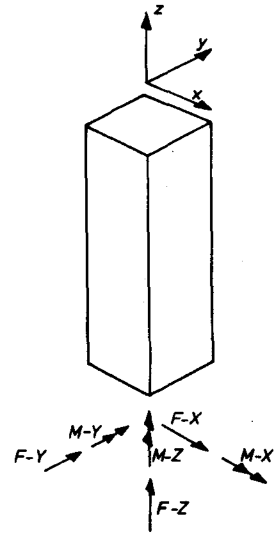
IMPERIAL COUNTY SERVICES BUILDING



COLUMN DESIGNATIONS



GROUND LEVEL COLUMN FORCES

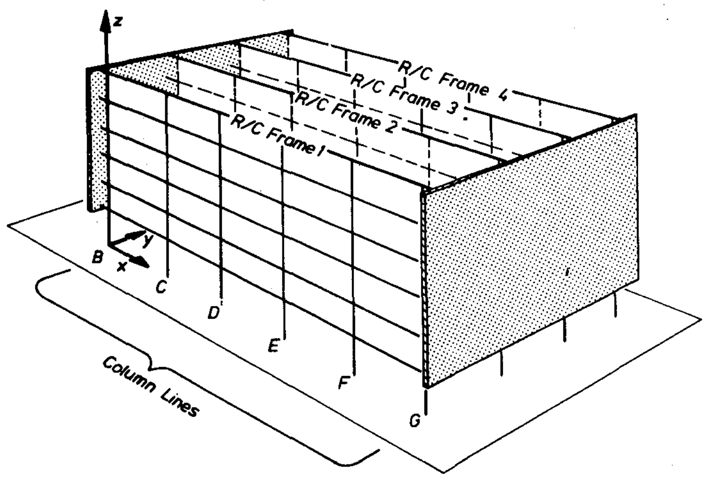
1979 IMPERIAL VALLEY EARTHQUAKE (E-W COMPONENT) IN X-DIRECTION

COLUMN FORCES AT GROUND LEVEL

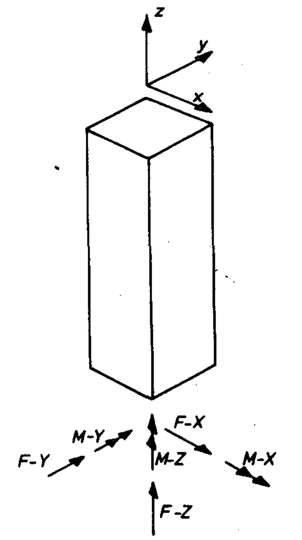
FRAME	COLUMN LINE	F-X	F-Y	F-Z	M-X	M-Y	M-Z
1	B	165.9	5.4	561.0	399.2	15088.2	6.7
1	C	208.3	4.1	29.9	306.3	17421.4	6.7
1	D	205.8	2.9	2.4	213.6	17282.3	6.7
1	E	205.5	1.6	1.5	120.8	17266.5	6.7
1	F	210.0	0.6	37.0	45.1	17513.0	6.7
1	G	153.7	1.4	523.4	104.9	14415.4	6.7
2	B	165.3	2.5	394.5	242.5	15079.5	6.7
2	C	220.0	1.9	12.3	184.2	18088.2	6.7
2	D	214.6	1.3	0.6	128.6	17792.7	6.7
2	E	214.6	0.7	0.6	73.0	17792.7	6.7
2	F	220.0	0.3	12.3	27.2	18088.2	6.7
2	G	165.3	0.7	394.5	65.1	15079.5	6.7
3	B	166.2	2.5	395.1	242.5	15152.2	6.7
3	C	221.1	1.9	12.4	184.2	18171.5	6.7
3	D	215.7	1.3	0.6	128.6	17874.7	6.7
3	E	215.7	0.7	0.6	73.0	17874.7	6.7
3	F	221.1	0.3	12.4	27.2	18171.5	6.7
3	G	166.2	0.7	395.1	65.1	15152.2	6.7
4	B	158.5	5.4	468.2	399.2	14749.0	6.7
4	C	212.9	4.1	43.4	306.3	17746.4	6.7
4	D	208.7	2.9	1.3	213.6	17515.4	6.7
4	E	208.6	1.6	1.8	120.8	17510.7	6.7
4	F	213.1	0.6	37.2	45.1	17757.7	6.7
4	G	156.4	1.4	526.3	104.9	14633.8	6.7

** NOTE **   FORCES IN KIPS, MOMENTS IN KIP-IN

IMPERIAL COUNTY SERVICES BUILDING



COLUMN DESIGNATIONS



GROUND LEVEL COLUMN FORCES

1979 IMPERIAL VALLEY EARTHQUAKE (N-S COMPONENT) IN Y-DIRECTION

COLUMN FORCES AT GROUND LEVEL

FRAME	COLUMN LINE	F-X	F-Y	F-Z	M-X	M-Y	M-Z
1	B	100.5	29.0	577.8	1818.3	8161.2	495.9
1	C	110.7	59.5	14.1	4831.1	8721.2	495.9
1	D	111.6	147.1	3.5	11416.4	8770.3	495.9
1	E	111.4	234.7	5.9	18001.5	8762.5	495.9
1	F	109.5	322.3	6.3	24586.7	8655.3	495.9
1	G	175.9	410.1	1500.3	31181.4	12310.0	495.9
2	B	31.4	15.4	13.2	1066.3	2596.4	495.9
2	C	38.4	29.4	1.3	3170.8	2980.8	495.9
2	D	37.6	70.3	0.2	7187.2	2932.3	495.9
2	E	37.6	111.1	0.2	11192.5	2932.3	495.9
2	F	38.4	152.0	1.3	15208.9	2980.8	495.9
2	G	31.4	196.4	13.2	19414.9	2596.4	495.9
3	B	32.3	15.4	14.9	1066.3	2676.1	495.9
3	C	39.6	29.4	1.3	3170.8	3078.4	495.9
3	D	38.7	70.3	0.2	7187.2	3028.1	495.9
3	E	38.7	111.1	0.2	11192.5	3028.1	495.9
3	F	39.6	152.0	1.3	15208.9	3078.4	495.9
3	G	32.3	196.4	14.9	19414.9	2676.1	495.9
4	B	72.8	29.0	505.2	1818.3	6665.1	495.9
4	C	115.5	59.5	6.5	4831.1	9018.7	495.9
4	D	112.1	147.1	2.1	11416.4	8833.1	495.9
4	E	112.7	234.7	5.6	18001.5	8861.6	495.9
4	F	110.7	322.3	6.6	24586.7	8751.3	495.9
4	G	176.7	410.1	1498.2	31181.4	12388.2	495.9

** NOTE ** FORCES IN KIPS, MOMENTS IN KIP-IN

APPENDIX F

Program Name: RUAUMOKO ... INELASTIC DYNAMIC ANALYSIS OF TWO DIMENSIONAL BUILDING FRAMES

Developed by: A.J. Carr and R.D. Sharpe
Department of Civil Engineering
University of Canterbury
Christchurch, New Zealand.

Program Date: January 1980.

Summary: RUAUMOKO (Maori God of Fire and Earthquakes) is designed to perform non-linear dynamic time-history analyses of frame and shear wall type plane structures. The building is idealised as an assemblage of beams, columns and shear wall elements connected to form an arbitrary two dimensional structure. The beams and beam-column members may have different yield properties at each end. Elasto-plastic, bi-linear, Ramberg-Osgood and Degrading stiffness models are available and Taylor's shear wall model is also incorporated. Viscous damping members may also be included in the structure.

Five different structural damping options are available and consistent or lumped mass matrices may be used. Further options include non-linear geometric effects or a simplified P-delta model.

The analyses may include an initial static analysis as well as a modal analysis to obtain all natural frequencies and mode shapes. Both vertical and horizontal earthquake excitation is possible and the program has a restart option.

At specified time steps selected results may be printed as well as stored on magnetic tape for post-processing and at every time step where a change in yield state of any member of the structure changes a picture is produced showing the location and sign of every plastic hinge or yield state in the structure.

At the end of the analysis a complete summary of all enveloped results is presented.

The program is written in FORTRAN 4 with dynamic storage allocation for major arrays.

Data Preparation: All input is in a free-format and data interpolation is used wherever possible.

Output: In addition to a printout of all input data, the following output is given by the program: approximate line printer picture of the structure, all natural frequencies of the structure and the amount of damping associated with each initial elastic mode, mode shapes as required, results of static loading. For specified time steps, displacements of selected nodes, together with damping and inertia forces, for selected members, moments, axial forces and plastic displacements. At every change of state of structure a picture showing all plastic hinges and axial yield of members. One completion of the analysis, envelopes of member forces, nodal displacements, member plastic displacements and ductilities are computed.

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Machine Versions Available: B6700, Program largely machine independent.

References:

(1) Sharpe, R.D., "The Seismic Response of Inelastic Structures", Research Report 74-13, Department of Civil Engineering, University of Canterbury, November 1974.

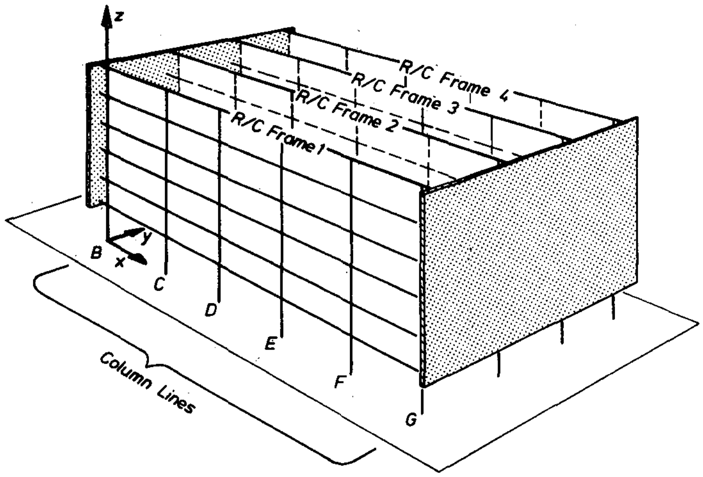
(2) Taylor, R.G., "The Non-Linear Seismic Response of Tall Shear Wall Structures", Research Report 77-12, Department of Civil Engineering, University of Canterbury, November 1977.

(3) Carr, A.J. and Moss, P.J., "The Effects of Large Displacements on the Earthquake Response of Tall Structures", paper submitted to the 7th World Conference on Earthquake Engineering, Istanbul, Turkey, 1980.

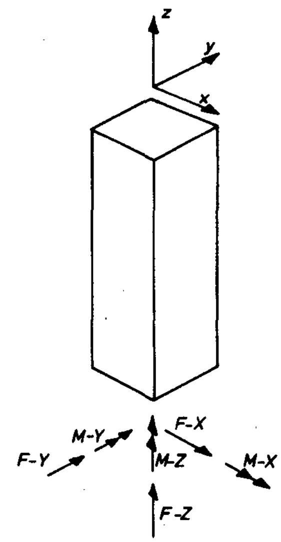
APPENDIX G

COLUMN FORCES AT GROUND LEVEL -
PLANE FRAME ANALYSES

IMPERIAL COUNTY SERVICES BUILDING



COLUMN DESIGNATIONS



GROUND LEVEL COLUMN FORCES

1940 IMPERIAL VALLEY EARTHQUAKE (N-S COMPONENT) IN X-DIRECTION
PLUS VERTICAL DEAD LOAD

COLUMN FORCES AT GROUND LEVEL

ELASTIC

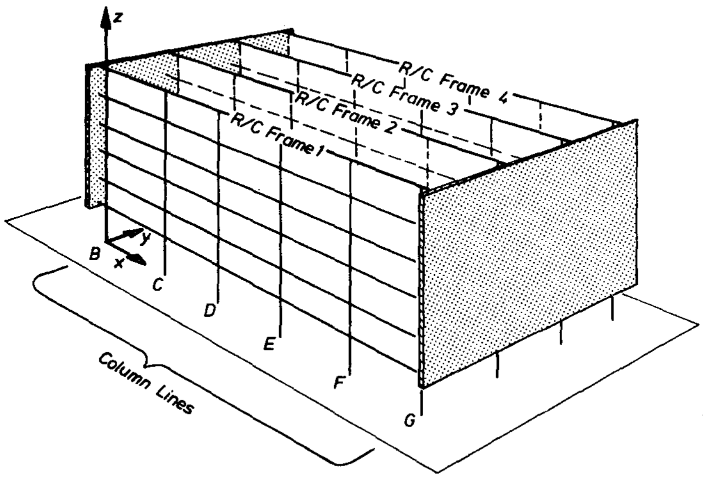
FRAME	COLUMN LINE	F-X	F-Z	M-Y
1	B	223.9	-917.6	18894.0
1	C	235.4	-470.4	19569.4
1	D	237.7	-365.3	19722.4
2	B	234.0	-876.5	19426.3
2	C	243.5	-419.5	19977.7
2	D	247.6	-368.5	20282.4

INELASTIC

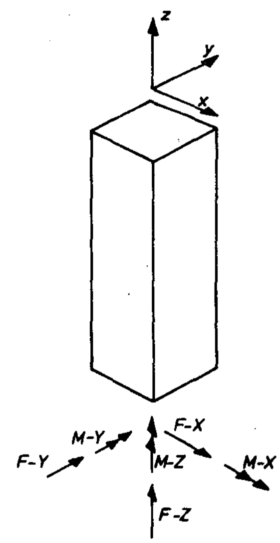
FRAME	COLUMN LINE	F-X	F-Z	M-Y
1	B	127.5	-744.6	10566.9
1	C	115.8	-535.1	-9120.3
1	D	116.6	-534.4	-9119.2
2	B	91.8	-674.0	-8262.5
2	C	106.0	-508.6	-7706.0
2	D	104.1	-524.4	-7850.0

** NOTE ** FORCES IN KIPS, MOMENTS IN KIP-IN

IMPERIAL COUNTY SERVICES BUILDING



COLUMN DESIGNATIONS



GROUND LEVEL COLUMN FORCES

1979 IMPERIAL VALLEY EARTHQUAKE (N-S COMPONENT) IN X-DIRECTION
PLUS VERTICAL DEAD LOAD

COLUMN FORCES AT GROUND LEVEL

ELASTIC

FRAME	COLUMN LINE	F-X	F-Z	M-Y
1	B	-179.6	-502.2	-14959.9
1	C	-178.7	-460.9	-14922.5
1	D	-182.2	-365.1	-15133.6
2	B	-147.7	-774.9	-13082.3
2	C	-185.2	-415.6	-15189.6
2	D	-183.2	-367.8	-15148.0

INELASTIC

FRAME	COLUMN LINE	F-X	F-Z	M-Y
1	B	-111.1	-699.7	-8340.9
1	C	-112.8	-539.0	-9213.9
1	D	-115.6	-528.4	-9296.3
2	B	-104.8	-682.9	-8483.4
2	C	-106.4	-498.4	-7795.0
2	D	-106.8	-514.5	-7938.2

** NOTE ** FORCES IN KIPS, MOMENTS IN KIP-IN

Classn:

IMPERIAL COUNTY SERVICES BUILDING : Elastic and Inelastic Response Analyses

Athol J. Carr, Peter J. Moss and Gerard C. Pardoen

ABSTRACT: The Imperial County Services Building is described and the analyses carried out on it are outlined. The model considered for an elastic analysis and the assumptions involved are outlined. A different model was used for carrying out inelastic analyses. The results of these two analyses are discussed and compared. The earthquake records from El Centro 1940 and the 1979 "free field" record were used in the dynamic analyses.

Department of Civil Engineering, University of Canterbury
Civil Engineering Research Report No. 79/15, December 1979.

Seismic Behavior of the Imperial County Services Building in El Centro, California during the Imperial Valley Earthquake

Gerard C. Pardoen*

Introduction

The full scale ambient and forced structural dynamic measurement of buildings has been an ongoing research activity for the past two decades, whereas the last decade has seen the emergence of strong motion data recorded by a number of building's seismographs due to significant seismic events. Recently the response of the Imperial County Services Building (ICSB) due to the 15 October 1979 Imperial Valley earchquake, has stimulated interest within the earthquake engineering community because the structures response represents the first response measured in a building that has received major structural damage. This paper is devoted to the pre- and post-earthquake dynamic response data of the ICSB as well as providing an explanation for the structure's significant damage.

Building Description

The Imperial County Services Building serves as an office building for Imperial County. It was designed in 1968 (using the 1967 edition of the Uniform Building Code) and was completed in 1971 at a construction cost of $1.87 million. The building is 136 feet 10 inches by 85 feet 4 inches in plan and is founded on a Raymond step-taper concrete pile foundation. The piles are interconnected with reinforced-concrete link beams; they extend 45 feet to 60 feet into the alluvium foundation material composed primarily of sand with interbeds of clay to 60 feet (based on logs from 4 soil borings at the site).

Vertical loads are carried by reinforced-concrete floor slabs (5 inches thick at the second floor and 3 inches thick at the upper floors) supported by reinforced-concrete 5-1/2 inch-wide by 14 inch-deep pan joists spanning in the north-south (transverse) direction; the joists are supported by four longitudinal reinforced-concrete frames at 25 feet on center. The frame columns are typically 2 feet square, and the beams vary in size. Beams in the two interior frames are 2 feet wide by 2 feet 6 inches deep at all levels; those in the two exterior frames are 2 feet by 2 feet 5 inches at the second-floor level, 10 inches by 4 feet 5 inches at the third-floor

*Assistant Professor, Civil Engineering, UC Irvine, Irvine,Calif. USA

Table 1 PRE AND POST EARTHQUAKE AMBIENT VIBRATION PERIODS

Date of Test	Frequency	Period (sec)	Direction
February 1979	1.55	.65	E
February 1979	2.24	.45	N
February 1979	2.81	.36	Torsion
March 1980	1.20	.83	E
March 1980	1.92	.52	N
March 1980	2.32	.43	Torsion

Note: Building was shored up when ambient natural frequencies were measured in March 1980

'When the design of a structure or parts of a structure result in unusual configuration or irregular distribution of lateral stiffness, evidence shall be presented to show that equivalent safety to that establishment by these regulations is provided or the office shall withhold its approval.'

"In SEAOC's 1975 Recommended Lateral Force Requirements and Commentary, the discontinuous shear wall problem is specifically identified in the commentary in Section 1(E)3, Buildings with abrupt changes in Lateral Resistance:

'the practice of providing open or "Soft" lower levels <u>by discontinuous shear walls</u> or otherwise making certain levels weaker than the adjacent levels both for <u>shear and overturning has frequently shown poor performance</u>. The anticipated large distortions and energy absorption occurring in the weaker level make the proper distribution of design forces by dynamic analysis desirable. The new SEAOC Code Requirement only specifies a distribution of forces in accordance with the structures dynamic properties. It is realized that this distribution may not, by itself, indicate excessive energy absorbtion requirements.' (underline added)

"The experience of the Imperial County Services Building suggests that it may be timely to review the way in which the code deals with configuration issues."

One interesting item relates to the ductility of the east end ground floor columns. Figure 7 denotes a typical column detail in which column reinforcement ties are provided up to a point near the top of the soil. However a concrete slab encases the interior frames' column near the top of the reinforcement ties so that at least half the columns do not have sufficient ties above the rigid slab. Thus a further explanation of the east end's column failure is that the corner columns (G-1, G-4) are sufficiently loaded due to the significant E-W and N-S records whereas the interior columns (G-2, G-3) failed due to insufficient ductility at the base.

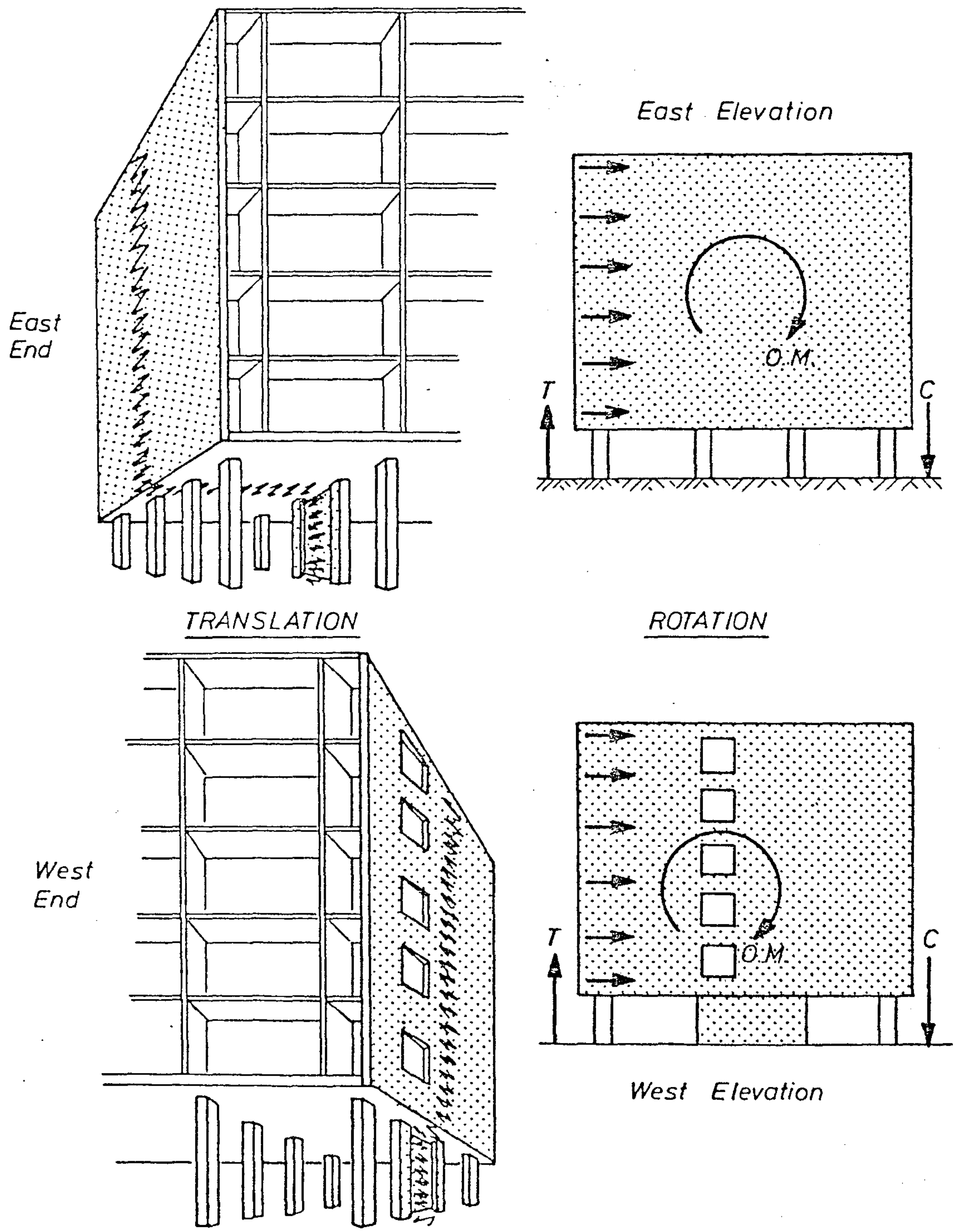
Conclusion

This note presents an in-depth analysis of the predicted structural response of the Imperial County Services Building due to several static and synamic load excitations. Seven representative loat conditions were considered in which a numerical as well as a commentary summary are provided. These computed response results of the ICSB, which is destined to become the most thoroughly investigated structure in earthquake engineering history, help explain the catastrophic failure of the building during the 15 October 1979 earthquake.

References

1. Arnold, C., "Architectural Implications", EERI Reconnaissance Report - Imperial County, California, Earthquake, October 15, 1979 - Feb 1980, pp 111-138.

IMPERIAL COUNTY SERVICES BUILDING



End Wall Conditions to Resist Translation, and Rotation

Figure 7 - End Conditions to Resist Translation and Rotation

in itself: if it were enclosed with a light curtain wall its structural performance would be essentially similar.

"The condition at the East end represents a classic instance of shear wall discontinuity, in which an abrupt change of strength and stiffness occurs at the point where the shear wall, weighing approximately 300 tons, terminates at the second floor. Its vertical - overturning - loads must be brought down through four slightly offset columns, while its shear forces must be transferred to the smaller shear wall at the next bay.

"Figure 7 schematically indicates the East end's two horizontal load paths: to resist translation, shear stresses can be carried in the plane of the second level floor structure in to an interior ground level wall. However, rotational (overturning)forces (which are perpendicular to the second floor) are not transferred across in this manner, and only the columns beneath the end wall can supply the tension-compression couple required to resist the overturning moment. The corner columns take the bulk of these alternating axial forces.

"At the West end, the stiff ground level shear wall beneath the upper wall presents large axial forces from reaching the columns. While there is a five foot "kink" in the load path from upper portion of West wall to ground level wall, compared to the East end this is more nearly a continuous shear wall. Holes in the upper West end wall comprise another geometric irregularity or discontinuity. At the force levels experienced in this particular moderate-sized earthquake, however, these holes at stairway landings did not apparently affect behavior.

"It should be made clear that the Uniform Building Code in force when the building was designed permits discontinuous shear walls: and the Code in force at present also permits such a condition. No mention was made of such configurational issues until the 1973 UBC when a clause was inserted (2312-C-3) that now reads:

> '3. Structures having irregular shapes or framing systems. The distribution of the lateral forces in structures which have highly irregular shapes, large differences in lateral resistance or stiffness between adjacent stories or other unusual structural features shall be determined considering the dynamic characteristics of the structure.'

"This cause in effect makes the designer responsible for evaluating the dynamic characteristics of the structure and using judgment in his design response (note that this clause carefully does not <u>mandate</u> a dynamic analysis).

"In passing it should be noted that the Field Act uses a slightly stronger language than the UBC:

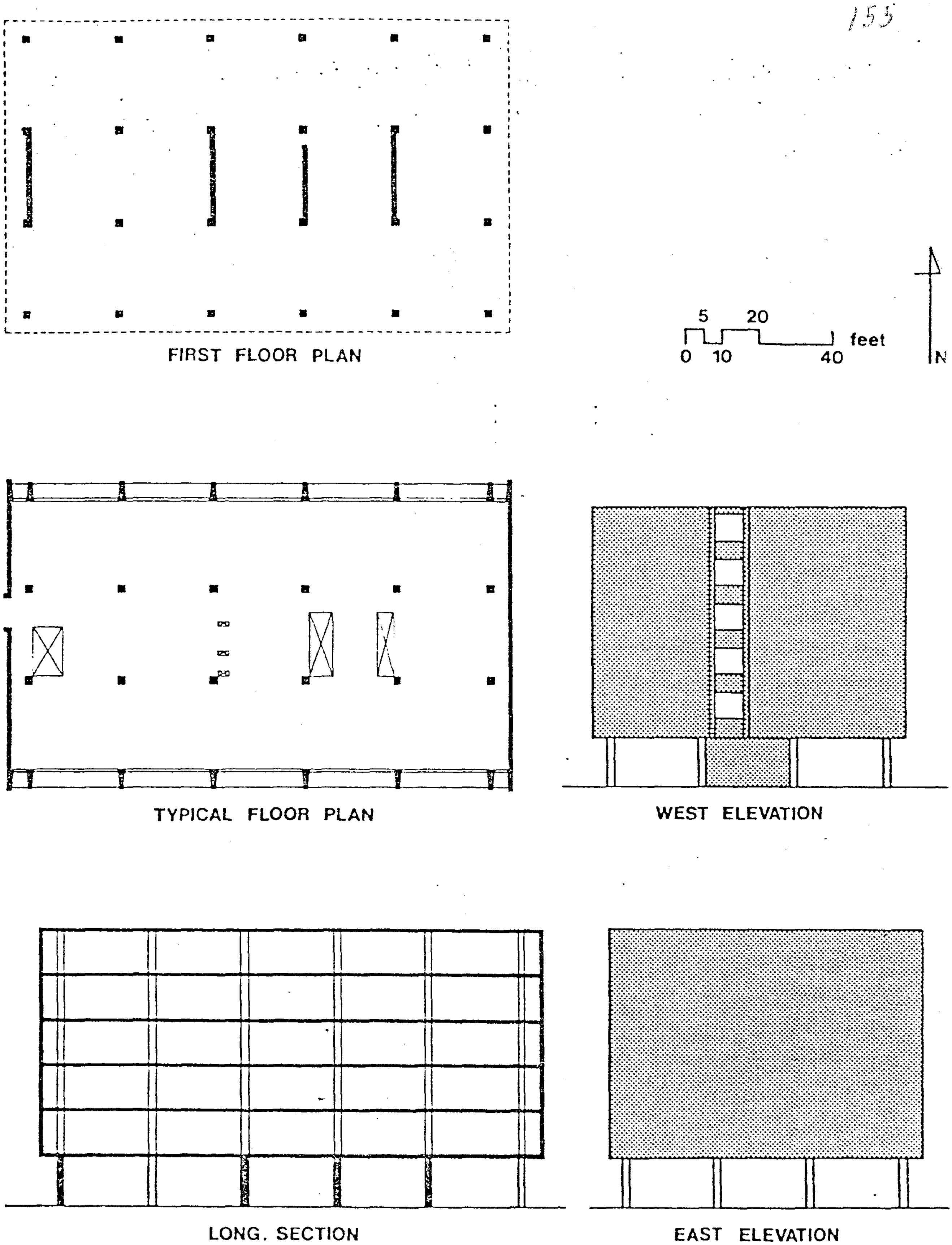


Figure 6 - Plan and Elevation Views of the Imperial County Services Building

A more probable, but tentative, explanation subject to a detailed nonlinear dynamic analysis of the four column's failure is due to overturning since the outer column pair showed more distress than the inner pair while the east end shear wall, the floor diaphragm, and the inner shear wall showed no signs of distress. The major difference in damage between the east and west ends of the building is due to the shear load and overturning moment resistance. The west end ground level shear wall beneath the upper wall prevents large axial forces from developing on the columns. However, the east end ground level shear wall is offset by some 30 feet from the upper wall requiring that the four slightly offset columns must resist the vertical and overturning loads.

Analysis of Column Failure

Superficially the ICSB appears to be a simple, rectangular, symmetrical structure and, as such, one would expect the 4 corner columns to react on a symmetrical fashion. However, inspection of the first floor plan indicates the unsymmetrical distribution of first story shear walls which account for the difference in column behavior for those at the east end from those at the west end. Reference 1 provides an excellent background which describes the difference of each and west end column behavior which is reflected in the catastrophic column failure at the east end furing the 1979 earthquake:

"Reference to the building plans, sections and elevations (Figure 6) shows that at the West elevation the shear wall from roof to second floor is offset horizontally some five feet at the second floor level before continuing to the foundations (in the center bay only). At the East elevation a similar shear wall stops at the second floor: shear forces are then transferred through the second floor diaphragm some thirty feet to a center bay shear wall that runs from the second floor to the foundations.

"Detailed study of the building damage shows that the four freestanding columns at the East end of the building failed by overturning, since the outer pair of columns show more distress than the inner. At the same time the end shear wall, the floor diaphragm, and the inner shear wall show no signs (on superficial inspection) of shear distress.

"Thus the major difference in damage between the West and East ends of the building are paralleled by a major difference in architectural configuration. It should be noted that the center pair of first floor columns at the West end of the building represent the only place where a perimeter shear wall occurs. It should also be noted that the floor to floor height for the first floor height at 14'-6" is only one foot greater than the typical floor. It should be noted that the open first floor is not of any greater

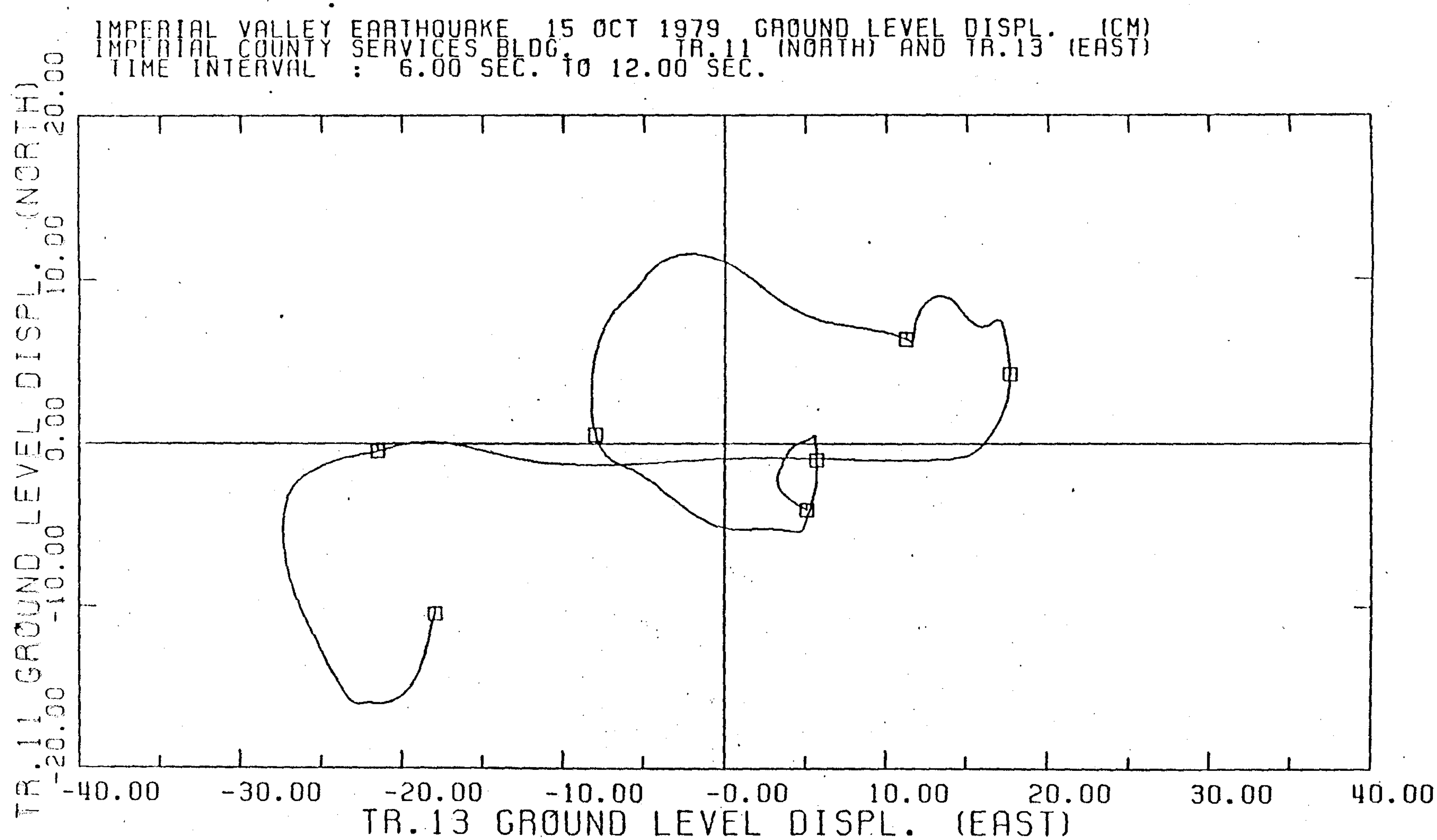


FIGURE 5 BUILDING GROUND LEVEL TIME HISTORY

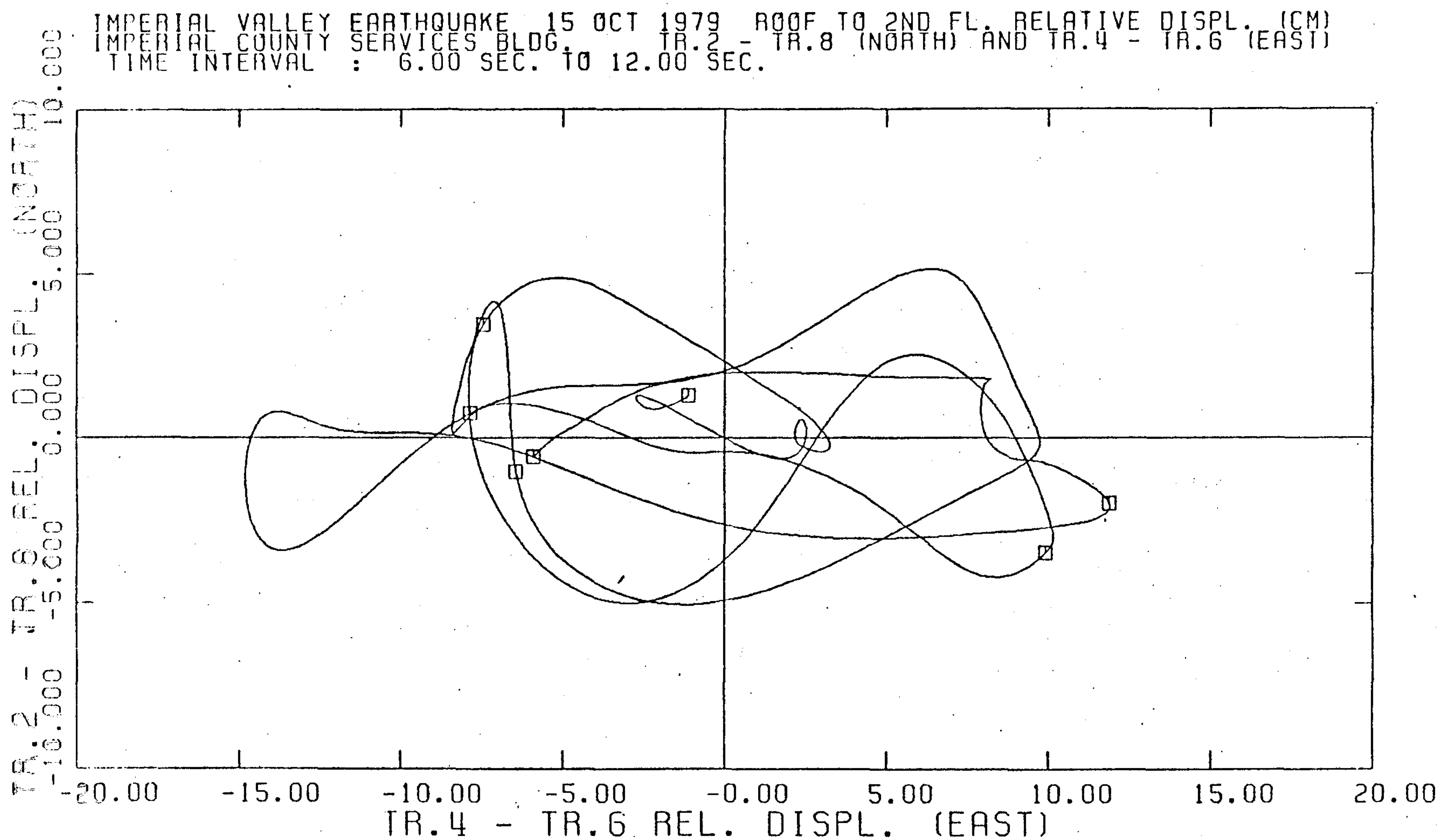


FIGURE 4 ROOF TO SECOND FLOOR RELATIVE DISPLACEMENT TIME HISTORY

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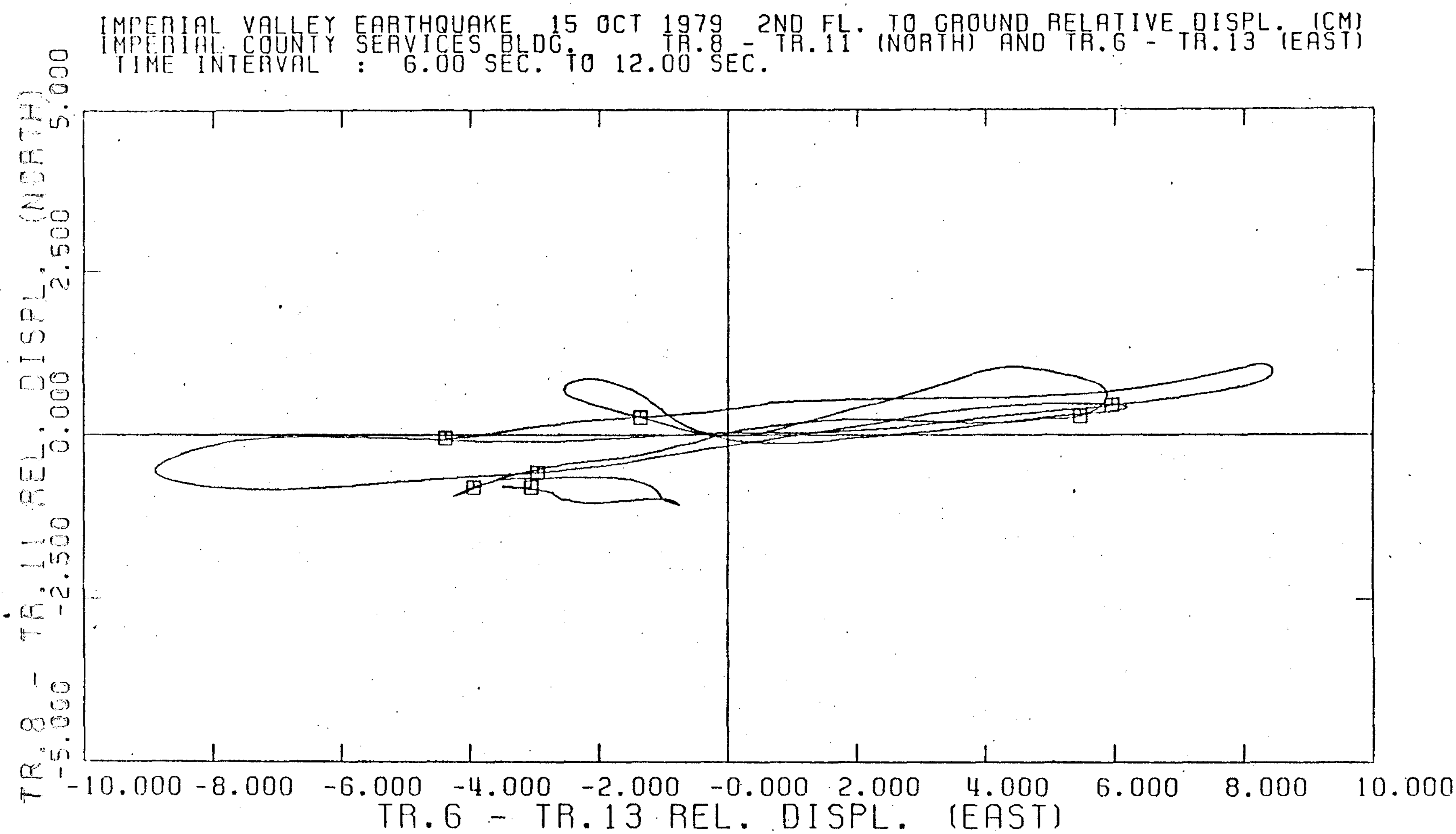

FIGURE 3 SECOND FLOOR TO GROUND RELATIVE DISPLACEMENT TIME HISTORY

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IMPERIAL COUNTY SERVICES BUILDING

Ambient Vibration Test Results

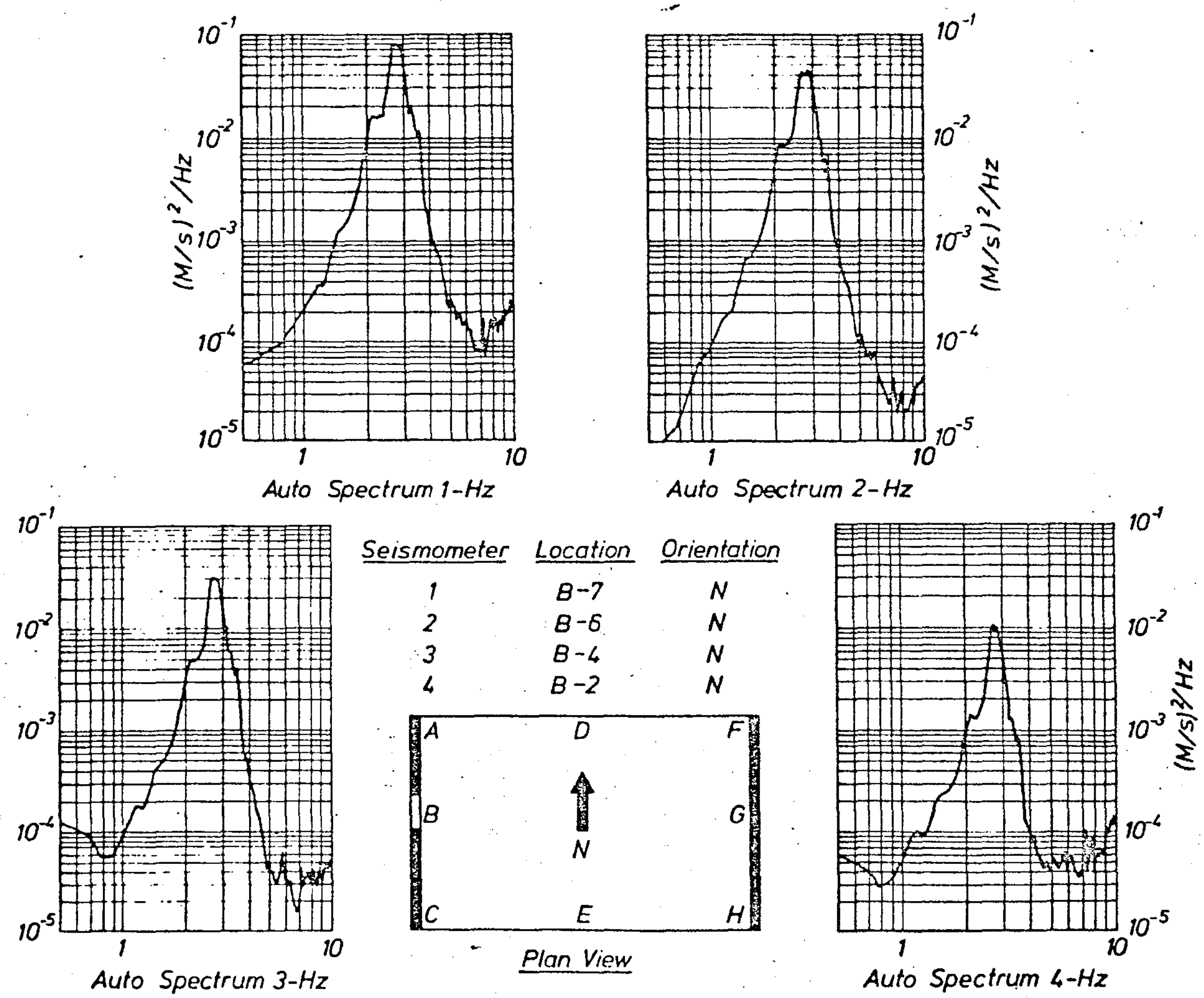

Figure 2c - N-S Power Spectrum Responses At Location B

IMPERIAL COUNTY SERVICES BUILDING

Ambient Vibration Test Results

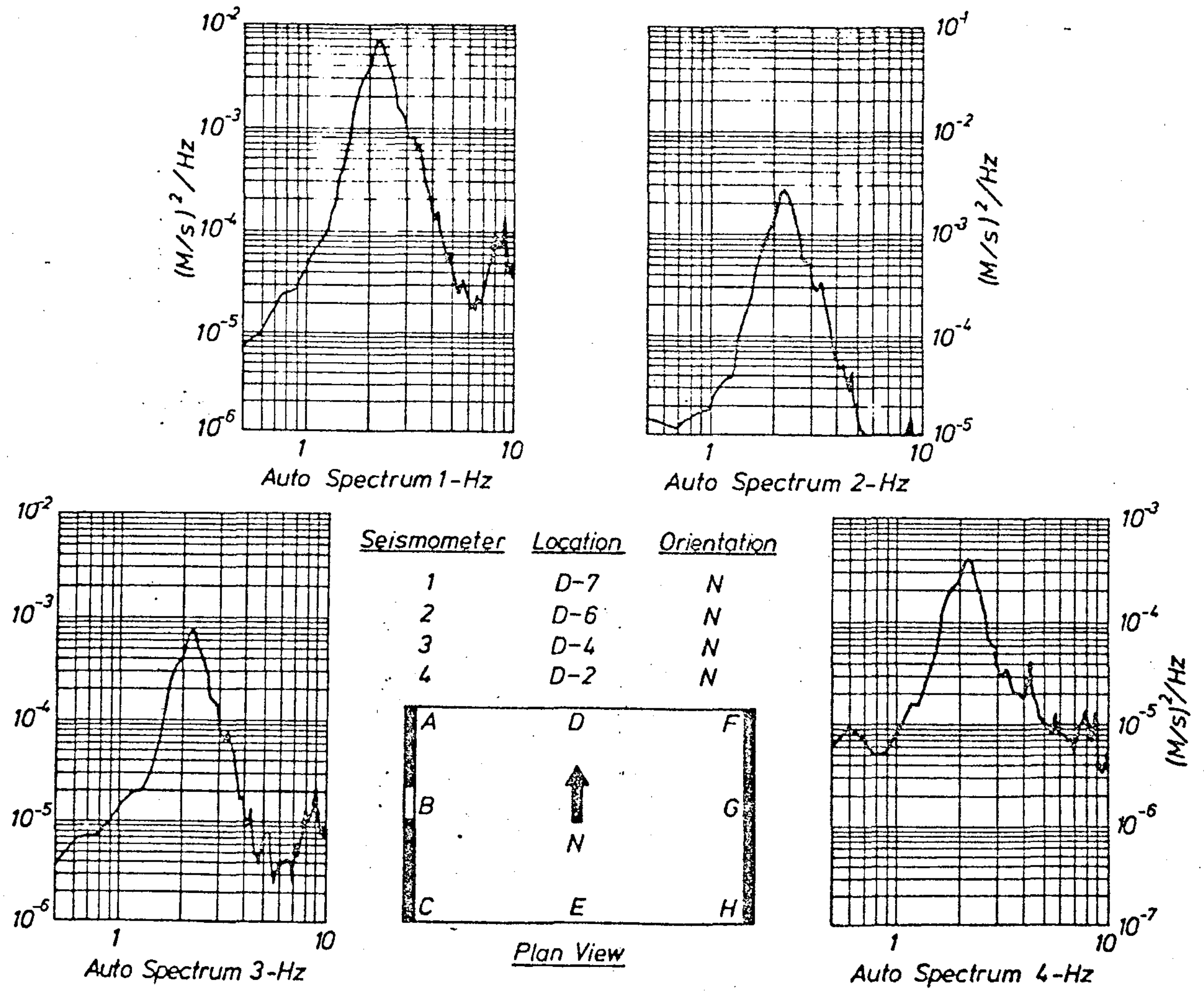

Figure 2b - N-S Power Spectrum Responses At Location D

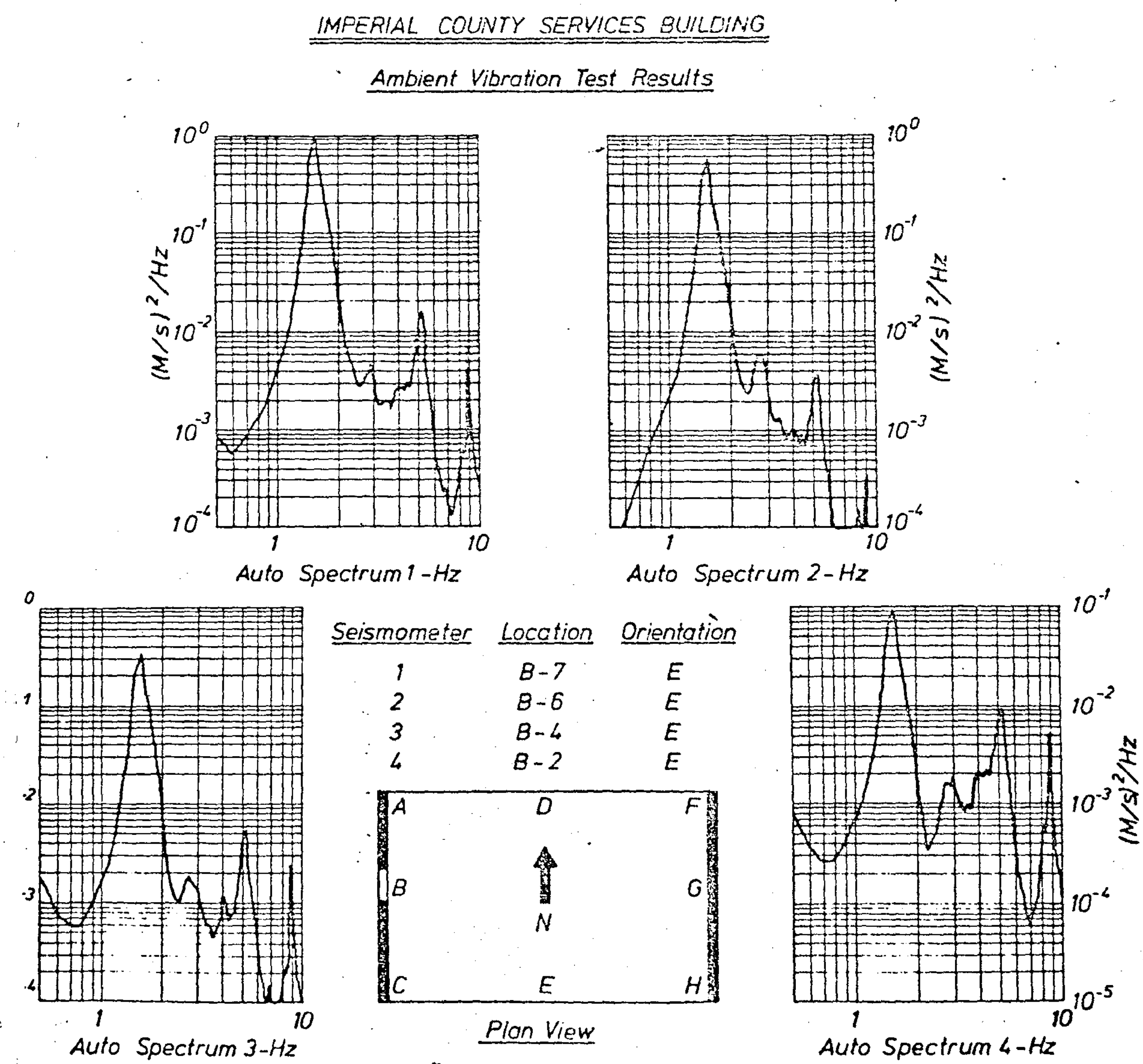

Figure 2a- E-W Power Spectrum Responses At Location B to second floor inter-story motion information. Similarly, the accelerometers at the ground floor, second floor, fourth floor, and roof levels in the more flexible east-west (frame) direction (accelerometers 4,5,6 and 13) provide east-west translational response, mode shape, and inter-story motion information. The two north-south-oriented accelerometers at ground level(accelerometers 10 and 11) are intended to identify collectively the extent to which differential horizontal ground motion has occurred, and the vertical accelerometer at ground level (accelerometer 12) provides information on vertical motion at this location. There are no vertically oriented accelerometers above ground level.

Pre- and Post-Earthquake Ambient Vibration Tests

Ambient vibration tests were performed on the building prior to and after the 15 October 1979 earthquake. The results of these tests, which were conducted as part of a cooperative effort of the Los Angeles and Irvine campuses of the University of California, are depicted in Figs. 2a,b,c and tabularized in Table 1. It should be noted that the post earthquake results reflect the building on its shored up configuration.

Analysis of Earthquake Records

By manipulating the appropriate traces of the strong motion records, one can obtain an approximate horizontal plane movement time history such as those depicted in Figures 3,4 and 5. Figure 3, for instance, 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 3 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. 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 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 fail. The lack of apparent frame flexural failure suggests that there was sufficient frame ductility despite the fact that comparing the moment developed by such interstory drift with column interaction diagrams was much greater than that needed to produce collapse.

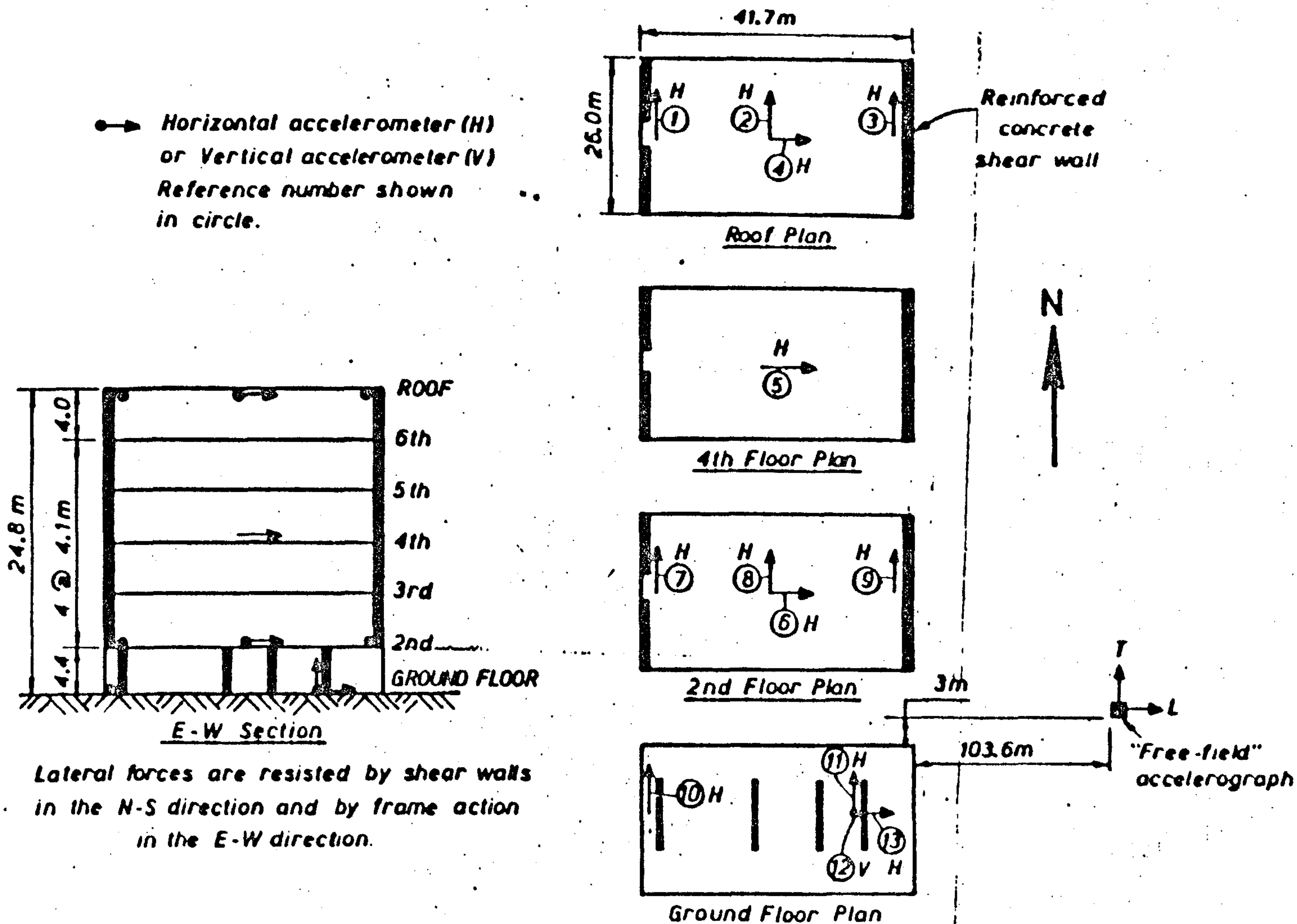

Figure 1 IMPERIAL COUNTY SERVICES BUILDING - STRONG-MOTION INSTRUMENTATION

through sixth-floor levels, and 10 inches by 4 feet 2 inches at the roof level.

Lateral loads are resisted by the floor reinforced-concrete frames in the east-west direction and reinforced-concrete shear walls in the north-south direction. The shear walls are discontinuous at the second-floor level. Below the second floor are three interior and one exterior 1-foot thick shear walls, and above the second floor, shear walls exist only at the east and west ends. Between the second and third floors, the walls are 7-1/2 inches thick, and above the third floor, they are 7 inches thick. According to the design calculations, the design "K" factor was 1.33 for the north-south shear walls, 0.67 for the east-west interior frames, and 1.0 for the east-west interior frames.

Instrumentation

This particular 6-story building has been instrumented by the California Division of Mines and Geology in accordance with its building instrumentation criteria for the California Strong Motion Instrumentation Program. In all the building is 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 consists of 9 FBA-1 single-axis force-balance accelerometers located at various locations throughout the upper stories of the structure, as east-west oriented HS-0 horizontal starter at roof level, and 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 have a natural frequency of approximately 50 Hz and are connected by low-voltage data cable to the central recording unit. The recording unit is battery powered, is triggered by horizontal or vertical motion that equals or exceeds .01 g, and records on 7-inch (178-mm) light-sensitive film. The system is designed to record acceleration with frequency components in the 0 to 50 Hz range and wi-h maximum amplitudes of 1 g (980 cm/sec^2). Real time is provided by a WWVB receiver and time-tick generator system; the recorder is not connected with the SMA-1 accelerograph located east of the building.

The FBA accelerometer location (Figure 1) were 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) is 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 provide translational and torsional response, mode shape, and ground

APPENDIX D

Earthquake and Ambient Response of El Centro County Services Building

by

Gerard C. Pardoen*, M.ASCE

Gary C. Hart**, M.ASCE

Barrett T. Bunce***

Introduction

The full scale ambient and forced structural dynamic measurement of buildings has been an ongoing research activity for the past two decades, whereas the last decade has seen the emergence of strong motion data recorded by a number of building's seismographs due to significant seismic events. Recently the response of the Imperial County Services Building, due to the 15 October 1979 Imperial Valley earthquake, has stimulated interest within the earthquake engineering community because the structures response represents the first response measured in a building that has received major structural damage. This paper is devoted to assessing the column damage received by the Imperial County Services Building during the '79 earthquake and relating this damage to its strong motion data.

Building Description

The Imperial County Services Building (ICSB) serves as an office building for Imperial County. It was designed in 1968 (using the 1967 edition of the Uniform Building Code) and was completed in 1971 at a construction cost of $1.87 million. The building is 136 feet 10 inches by 85 feet 4 inches in plan and is founded on a Raymond step-taper concrete pile foundation. The piles are interconnected with reinforced-concrete link beams; they extend 45 feet to 60 feet into the alluvium foundation material composed primarily of sand with interbeds of clay to 60 feet (based on logs from 4 soil borings at the site).

Vertical loads are carried by reinforced-concrete floor slabs (5 inches thick at the second floor and 3 inches thick at the upper floors) supported by reinforced-concrete 5 1/2 inch-wide by 14 inch-deep pan joists spanning in the north-south (transverse) direction; the joists are supported by four longitudinal reinforced-concrete frames at 25 feet on center. The frame columns are typically 2 feet square, and the beams vary in size. Beams in the two interior frames are 2 feet wide by 2 feet 6 inches deep at all levels; those in the two exterior frames are 2 feet by 2 feet 5 inches at the second-floor level, 10 inches by 4 feet 5 inches

* Assistant Professor, University of California, Irvine, CA.
** Professor, University of California, Los Angeles, and Principal, Ruthroff, Englekirk & Hart, Inc., Los Angeles.
*** Senior Associate, Ruthroff, Englekirk & Hart, Los Angeles.

at the third-floor through sixth-floor levels, and 10 inches by 4 feet 2 inches at the roof level.

Lateral loads are resisted by the four reinforced-concrete frames in the east-west direction and reinforced-concrete shear walls in the north-south direction. The shear walls are discontinuous at the second-floor level. Below the second floor are three interior and one exterior 1-foot thick shear walls, and above the second floor, shear walls exist only at the east and west ends. Between the second and third floors, the walls are 7 1/2 inches thick, and above the third floor, they are 7 inches thick. According to the design calculations, the design "K" factor was 1.33 for the north-south shear walls, 0.67 for the east-west interior frames, and 1.0 for the east-west interior frames.

Instrumentation

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The 13-channel CRA-1 system in the building consists of 9 FBA-1 single-axis force-balance accelerometers located at various locations throughout the upper stories of the structure, an east-west oriented HS-0 horizontal starter at roof level, and 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 have a natural frequency of approximately 50 Hz and are connected by low-voltage data cable to the central recording unit. The recording unit is battery powered, is triggered by horizontal or vertical motion that equals or exceeds .01 g, and records on 7-inch (178-mm) light-sensitive film. The system is designed to record acceleration with frequency components in the 0 to 50 Hz range and with maximum amplitudes of 1 g (980 cm/sec^2). Real time is provided by a WWVB receiver and time-tick generator system; the recorder is not connected with the SMA-1 accelerograph located east of the building.

The FBA accelerometer location (Figure 1) were 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) is 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 provide 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) provide east-west translational response, mode shape, and inter-story motion information. The two north-south-oriented accelerometers at ground level (accelerometers 10 and 11) are intended

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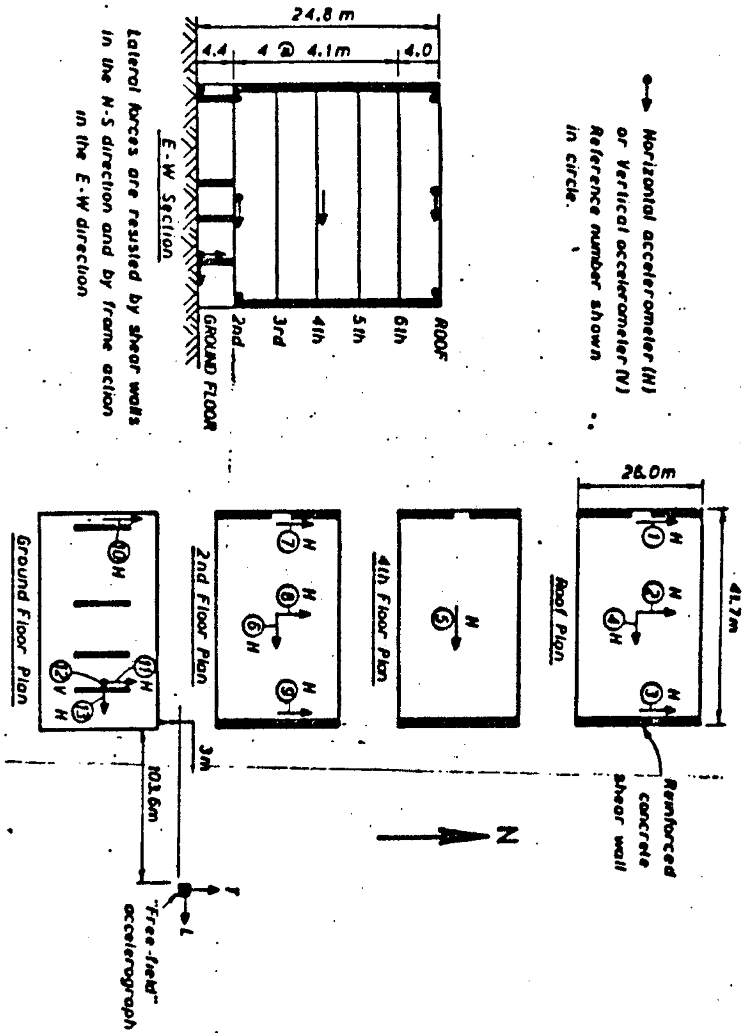

Figure 1 IMPERIAL COUNTY SERVICES BUILDING - STRONG-MOTION INSTRUMENTATION

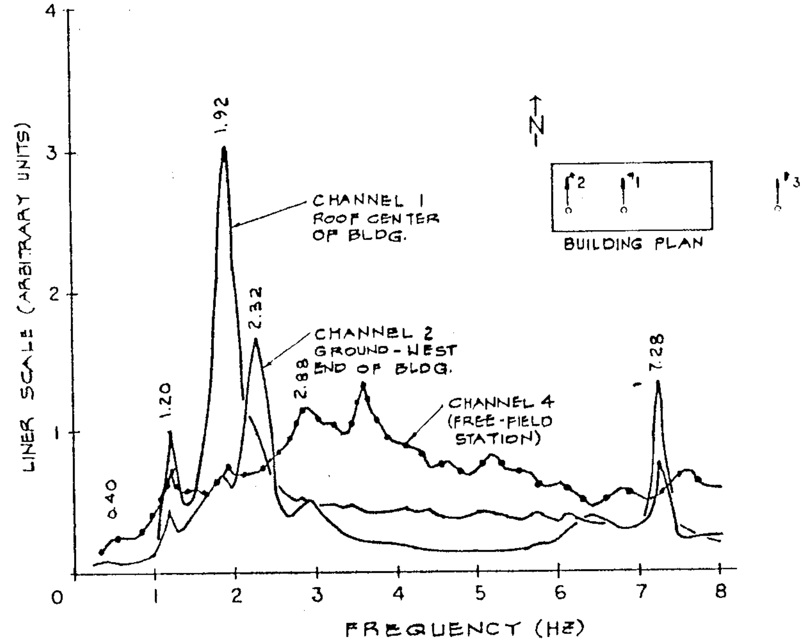

FIGURE 2 POST EARTHQUAKE AMBIENT GROUND LEVEL MOTIONS

Table 1 PRE AND POST EARTHQUAKE AMBIENT VIBRATION PERIODS

Date of Test	Frequency	Period (sec)	Direction
February 1979	1.55	.65	E
February 1979	2.24	.45	N
February 1979	2.81	.36	Torsion
March 1980	1.20	.83	E
March 1980	1.92	.52	N
March 1980	2.32	.43	Torsion

Note: Building was shored up when ambient natural frequencies were measured in March 1980

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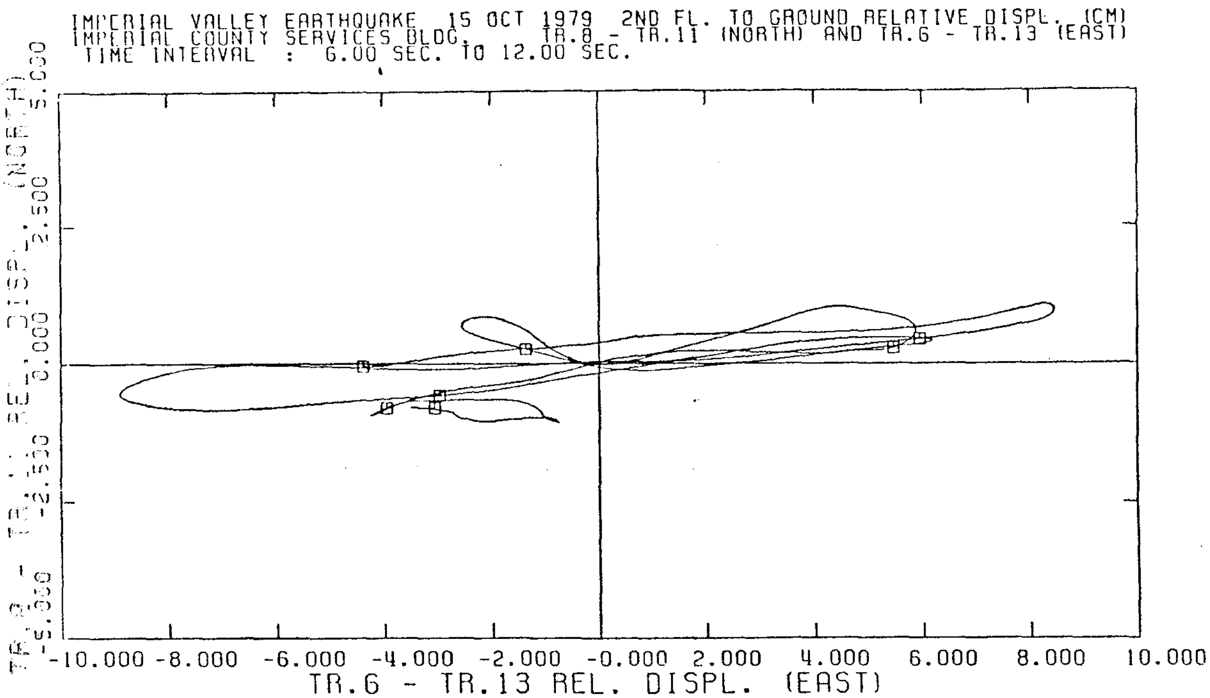

FIGURE 3 SECOND FLOOR TO GROUND RELATIVE DISPLACEMENT TIME HISTORY

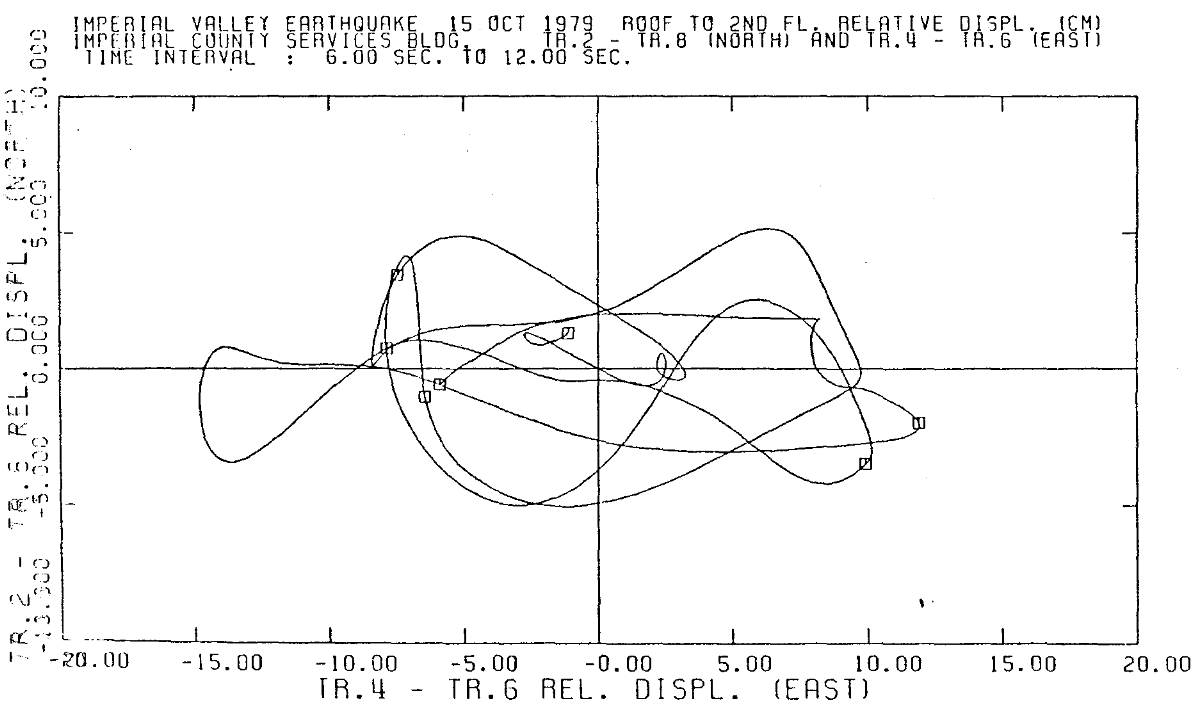

FIGURE 4 ROOF TO SECOND FLOOR RELATIVE DISPLACEMENT TIME HISTORY

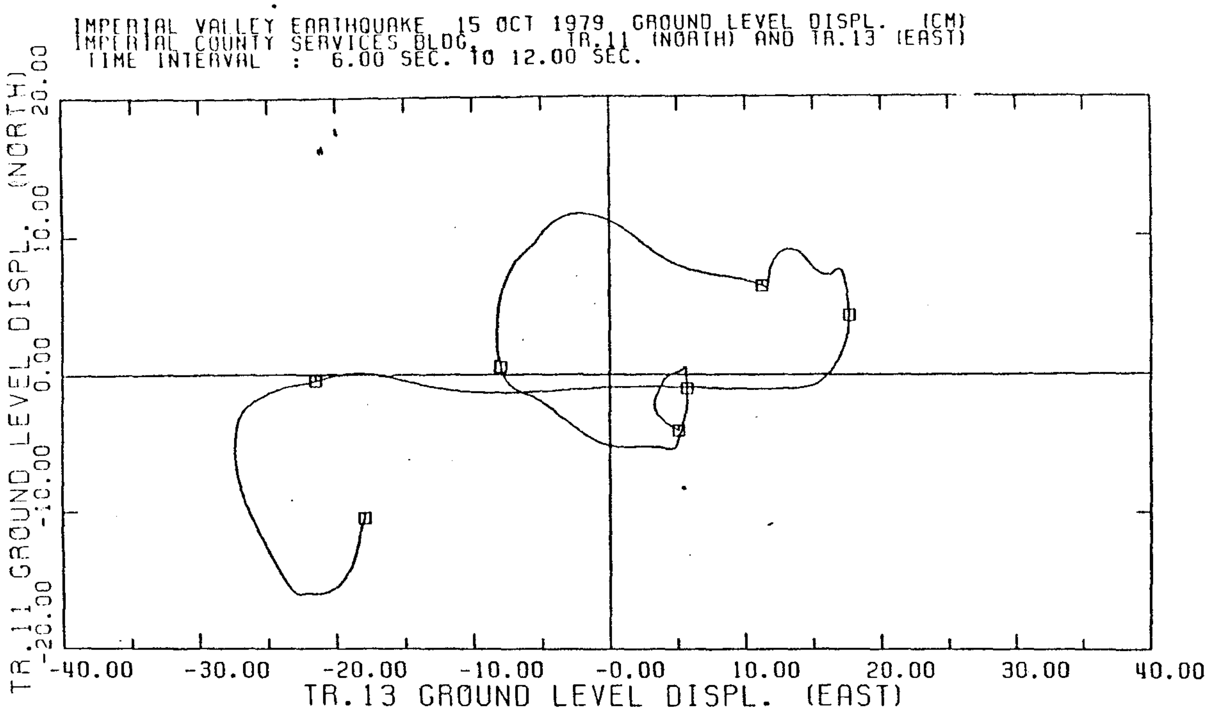

FIGURE 5 BUILDING GROUND LEVEL TIME HISTORY

BUILDING RESPONSE

Table 2 INTERSTORY DISPLACEMENTS (CM) AT MAXIMA TIME: (2ND FLOOR TO GROUND)

Time	Maximum (CM)	TR7 - TR10 (CM)	TR8 - TR11 (CM)	TR9 - TR11 (CM)	TR6 - TR13 (CM)
----	All Absolute	-2.648 (@6.58 sec)	-1.071 (@6.58 sec)	- 3.442 (@9.24 sec)	-8.904 (@10.34 sec)
6.28	TR9=14.50	-2.40	-0.98	1.49	-2.57
6.37	TR8=-17.06 TR11=-16.04	-2.45	-1.02	1.63	-2.34
6.38	TR7=-16.98 TR10=-14.52	-2.46	-1.03	1.64	-2.30
6.58	(TR7-TR10)=-2.65 (TR8-TR11)=-1.07	-2.65	-1.07	1.63	-0.76
6.71	TR13=-27.41	-2.34	-0.92	2.18	-1.06
6.72	TR6=-28.68	-2.29	-0.90	2.25	-1.07
9.24	(TR9-TR11)=-3.44	0.20	-0.02	-3.44	-2.24
10.34	(TR6-TR13)=-8.90	-0.28	-0.54	0.15	-8.90

Table 3 INTERSTORY DISPLACEMENTS (CM) AT MAXIMA TIME (ROOF TO 2ND FLOOR)

Time (Sec)	Maximum (CM)	TR1 - TR7 (CM)	TR2 - TR8 (CM)	TR3 - TR9 (CM)	TR4 - TR6 (CM)
----	All Absolute	4.59 (@6.89 sec)	5.16 (@9.68 sec)	6.74 (@10.18 sec)	14.80 (@10.28 sec)
6.28	TR9=-14.50	-0.24	-0.22	-4.17	0.42
6.32	TR1=-17.36 TR3=18.92	-0.59	-0.63	-4.46	1.63
6.33	TR2=-17.58	-0.56	-0.63	-4.47	1.87
6.37	TR8=-17.06	0.01	-0.18	-4.21	2.46
6.38	TR7=-16.98	0.19	-0.01	-4.10	2.51
6.72	TR6=-28.48	1.65	1.51	-3.52	1.35
6.89	(TR1-TR7)=4.59	4.59	4.87	-1.26	-5.05
8.05	TR4=33.46	-1.82	-3.01	2.93	10.15
9.68	(TR2-TR8)=5.16	2.50	5.16	5.30	6.32
10.18	(TR3-TR9)=6.74	-2.95	-3.43	6.74	-13.77
10.28	(TR4-TR6)=-14.80	-1.36	-1.34	-5.19	-14.80

to identify collectively the extent to which differential horizontal ground motion has occurred, and the vertical accelerometer at ground level (accelerometer 12) provides information on vertical motion at this location. There are no vertically oriented accelerometers above ground level.

Pre- and Post-Earthquake Ambient Vibration Tests

Ambient vibration tests were performed on the building prior to and after the 15 October 1979 earthquake. The results of these tests, which were conducted as part of a cooperative effort of the Los Angeles and Irvine campuses of the University of California, are depicted in Fig. 2, tabularized in Table 1. It should be noted that the post earthquake results reflect the building on its shored up configuration.

Analysis of Earthquake Records

By manipulating the appropriate traces of the strong motion records, one can obtain an approximate horizontal plane movement time history such as those depicted in Figures 3, 4, and 5. Figure 3, for instance, 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 3 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. 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 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 fail. The lack of apparent frame flexural failure suggests that there was sufficient frame ductility despite the fact that comparing the moment developed by such interstory drift with column interaction diagrams was much greater than that needed to produce collapse.

A more probable, but tentative, explanation subject to a detailed nonlinear dynamic analysis of the four column's failure is due to overturning since the outer column pair showed more distress than the inner pair while the east end shear wall, the floor diaphragm, and the inner shear wall showed no signs of distress. The major difference in damage between the east and west ends of the building is due to the shear load and overturning moment resistance. The west end ground level shear wall beneath the upper wall prevents large axial forces from developing on the columns. However, the east end ground level shear wall is offset by some 30 feet from the upper wall requiring that the four slightly offset columns must resist the vertical and overturning loads.

APPENDIX E

AMBIENT VIBRATION TEST RESULTS OF THE IMPERIAL COUNTY SERVICES BUILDING

Gerard C. Pardoen

SUMMARY

The ambient vibration test results conducted on the Imperial County Services Building prior to the 15 October 1979 Imperial Valley earthquake are presented. These results are of significant interest because the Imperial County Services Building has been the source of many investigations following the 1979 earthquake. This event was notable as for the first time a building instrumented with strong motion recorders suffered major structural failure and the response can be interpreted in the light of well defined dynamic characteristics.

Background

The success or failure of a structural design depends, to a great extent, upon the formulation of the appropriate conceptual and mathematical representation of the structure's static and dynamic characteristics. Inasmuch as a structure's analytical model represents a significant part of the overall design, it is imperative, that the model accurately represent the full-scale structure. One means of validating analytical procedures is to perform experimental studies of full-scale structures and compare these results with those of the analytical model. The ambient vibration test results conducted on the Imperial County Services Building (ICBS) prior to the 15 October 1979 earthquake, provide a data base of particular value in interpreting the subsequent behavior of the structure.

Significance of the Imperial County Service Building

Some of a structural and instrumentation characteristics of the building are:

1. This particular 6-story building had been instrumented by the California Department of Mines and Geology in accordance with its building instrumentation

criteria for the California Strong Motion Instrumentation Program. The instrumentation in this reinforced concrete frame and shear wall building (see Figure 1) consisted of a triaxial package of accelerometers at ground level, four single axis horizontal accelerometers at the second floor level, one at the fourth floor, and four at the roof level. In all the building was instrumented with 13 accelerometers (see Figure 2). All 13 accelerometers were triggered by the 15 October 1979 event.

2. Approximately 104 meters east of the building is a "free field" accelerograph maintained by USGS. The location of this accelerograph has proven to be particularly significant when one considers soil-structure interaction. All 3 "free field" accelerometers were also triggered.

3. The ICSB resisted E-W lateral forces by 4 reinforced concrete frames while resisting N-S lateral forces by massive end shear works discontinuous at the ground floor. Additionally there were 4 unsymmetrically placed N-S shear walls extending from the ground level up to the second story.

Test Apparatus

The apparatus associated with the ambient vibration tests consists of data acquisition and data reduction equipment.

The data acquisition equipment includes:

Kinemetric SS-1 Ranger Seismometers (4)

Kinemetric SC-1 Signal Conditioner

Hewlett-Packard 3960 Instrumentation Tape Recorder

The date reduction equipment includes:

Zonic Technical Laboratories DMS 5003 - The Data Memory System, which is used to manipulate the recorded ambient or forced vibration data, is composed of five major subsystems:

1. A multi-channel digitial data acquisition system, called the Data Memory System (DMS), which serves as the "front end" to the system.

2. A microcomputer which has been preprogrammed to perform various time and frequency domain fuctions.
3. A Tektronix 4006 graphics display terminal for communicating to the DMS as well as displaying graphical results.
4. A Tektronix 4631 hard copy unit.
5. A dual cassette tape drive system for storing and retrieving user programs, raw data, and processed data.

Test Procedure

The ambient vibration tests were conducted on three separate days in 1979: 26 February, 27 February and 17 May. With the exception of the late evening of the 26 February, all three days were characterized by light prevailing winds. Apart from the 5 minute calibration runs in the N-S and E-W directions at the beginning and end of the test sequence, all ambient vibration tests were conducted for approximately 18 minutes. The approximate 18 minute time frame was chosen so that there would be sufficient data to perform the spectral analyses of the 100 "snapshots" of time data of 10.24 seconds each.

The ambient vibration tests were conducted by placing the seismometers in strategic locations throughout the building on both the N-S and E-W directions. With the exception of the torsion tests the seismometers were usually located at a common point in a plan view but at different elevations, or 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. The seismometers were oriented in a "pinwheel" fashion for torsion tests, i.e. the seismometer in the NE corner was pointed north, the seismometer in the NW corner was pointed west, the seismometer in the SW corner was pointed south, and the seismometer in the SE corner was pointed east.

Apart from one ambient vibration test that was conducted by "tapping" into the accelerometers of the California Division of Mines and Geology, primarily velocity data was acquired and recorded for the ambient tests. By using the manufacturer's calibrated values for the velocity transducers, it was a relatively routine matter to obtain absolute magnitudes of velocity rather than the relative values.

Data Reduction - Fourier Transform Method

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 due to low level excitation. With the implementation of the Fast Fourier Transform (FFT) on hardwired microcomputers, such as the Zonic DMS 5003, the effort in calculating time and frequency domain functions related to these fundamental vibration characteristics has been tremendously decreased.

Test Results -Frequency

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

Configuration	Seismometer Direction	Seismometer Locations	Method	Frequency
1	East	B-7,B-6,B-4,B-2	Power Spectrum	1.54 Hz 1.56 Hz
2	North	D-7,D-6,D-4,D-2	Power Spectrum	2.24 Hz 2.25 Hz
3	North	B-7,B-6,B-4,B-2	Power Spectrum	2.81 Hz 2.85 Hz

Table 1 - Frequencies From Ambient Vibration Tests

The power spectum results reflecting the three basic test configurations are depicted in Figures 3, 4, and 5. Each power spectum curve represents the average power spectrum resulting from 100 "snapshots" of time data. The letter-number code denoting the seismometers locations refers to both a plan view and story level position - the letters A to H refer to specific plan view locations whereas the numbers refer to the story level (7 = roof, 6 = 6th floor, etc.).

A few aspects should be stressed. First, if the building were truly responding as a single degree-of-freedom subjected to white noise then the frequency determined from the power spectrum and auto-correlation would be identical. The frequency deviations indicate departures from these assumptions.

A second point is the discrepency in frequency when the seismometers are oriented the north direction. With the seismometers positioned at points D-7, D-6, D-4, and D-2 the response energy is concentrated near 2.24 Hz whereas with the seismometers positioned at points B-7, B-6, B-4, and B-2 the response energy is concentrated near 2.81 Hz. This apparent discrepancy was resolved by investigating the response of the torsional tests in which the seismometers were oriented in a "pin wheel" fashion at roof locations A (seismometer pointed South), C (seismometer pointed East), H (seismometer pointed North), and F (seismometer pointed West). By comparing the response of the diametrically opposite seismometers via the frequency response function, it was found that the seismometers were 180^{o} out of phase up until a frequency of 2.8 Hz at which point they become in phase. Thus the response energy concentrated near 2.24 Hz is attributed to the N-S lateral vibration whereas the response energy concentrated near 2.81 Hz is attributed to the torsional vibration.

Tests Results - Damping

The damping values associated with the ambient vibration tests of the Imperial County Services Building were computed by various means and are summarized in Table 2.

Configuration	Method	Damping (% Critical)
1	Power Spectrum	6.42, 6.42, 6.42, 5.50
	Auto Correlation	4.38, 4.78, 5.54, 5.54
2	Power Spectrum	12.86, 11.96, 12.86, 17.22
	Auto Correlation	7.67, 5.00, 7.14, 11.71
3	Power Spectrum	9.15, 9.15, 9.15, 8.24
	Auto correlation	7.05, 8.83, 7.31, 6.48

Table 2 - Damping Values From Ambient Vibration Tests

Using the power spectrum curves the damping ratio of the appropriate mode of vibration was obtained by measuring the width of the peak at half the peak value. The four damping values given per line correspond to the damping obtained from each of the four channels.

Using the auto correlation function curves the damping ratio of the appropriate mode of vibration was obtained by fitting an exponential curve to the auto correlation envelope and evaluating the appropriate curve fit parameters. The curve fit parameters are depicted in Figure 6 as well as denoting the implementation of this curve fit procedure for seismometer 3 of test configuration 3.

From the results it is quite apparent 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 these two 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 configuration 2. Although the power spectrum curves displayed in Figure 4 suggest a single degree of freedom system near 2.20 Hz, the broadband response gives almost unrealistically high damping values when using the half power point method.

Test Results - Mode Shape

The mode shape amplitudes were obtained by extracting the appropriate values of the transfer or frequency response function. For convenience the transfer function was defined as the ratio of the Fourier transform of a particular story level seismometer's output divided by the Fourier transform of the roof level seismometer. In actuality since an averaging process was used the numerator consisted of the averaged cross spectrum of the seismometers whereas the denominator consisted of the averaged power spectrum. The mode shape data is summarized in Table 3.

Test Configuration	Mode Shape Amplitudes			
	Roof	6th	4th	2nd
1	1.00	.62	.24	.15
2	1.00	.77	.60	.37
3	1.00	.93	.67	.28

Table 3 - Mode Shape Amplitudes From Ambient Vibration Tests

Although other ambient vibration tests were conducted the amplitudes at the other story levels is not assessed because, in the absence of a clearly defined input spectrum for each of the seismometers, it is not fully 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.

Conclusion

The ambient vibration test results summarized in this note reflect one phase of the data base related to the Imperial County Services Building. Although these results reflect very low level excitation they do provide a means of response comparison for the 15 October 1979 earthquake and various analtyical model results of the Imperial County Services Building and will be of particular value in the verification of elastic computer models which, in turn are useful in interpreting the post-earthquake condition of the building.

Acknowledgment

This research was sponsored by the National Science Foundation under its Research Initiation in Earthquake Hazards Mitigation Program. The support of the foundation, under the direction of Dr. Michael P. Gaus, is gratefully acknowledged.

A special thanks goes to two advisory board members of this project, Chris Rojahn of The Applied Technology Council and John Ragsdale of the California Division of Mines and Geology. The fact that the Imperial County Services Building was instrumented with the strong motion accelerographs has provided engineers and scientists with an excellent and significant strong motion data base for this building.

A particularly warm and generous thanks goes to Randy Reister, Assistant Director of Buildings and Grounds. Without Randy's enthusiasm, much less his permission to test the building, the project would have had no chance of success.

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LIST OF FIGURES

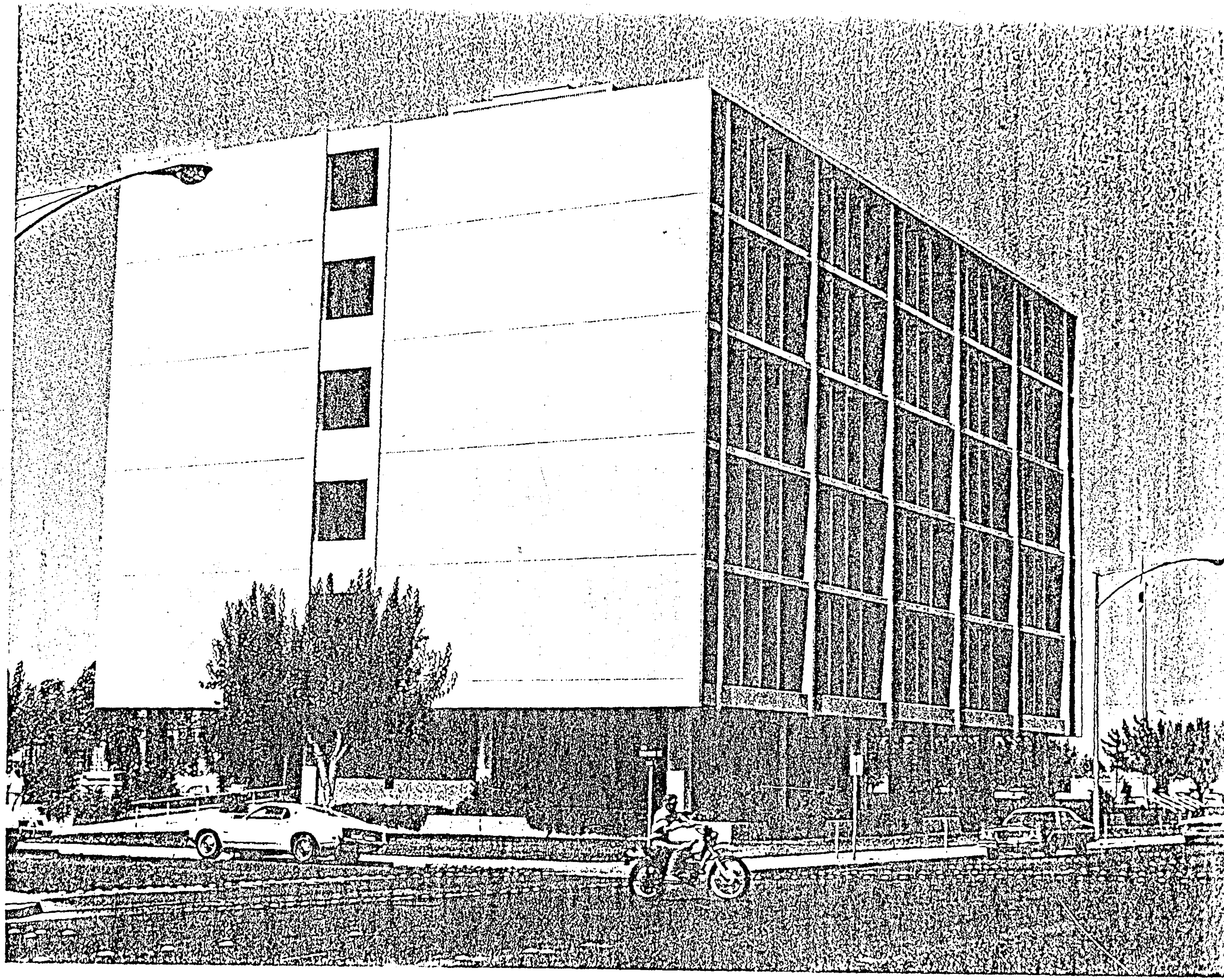

Figure 1

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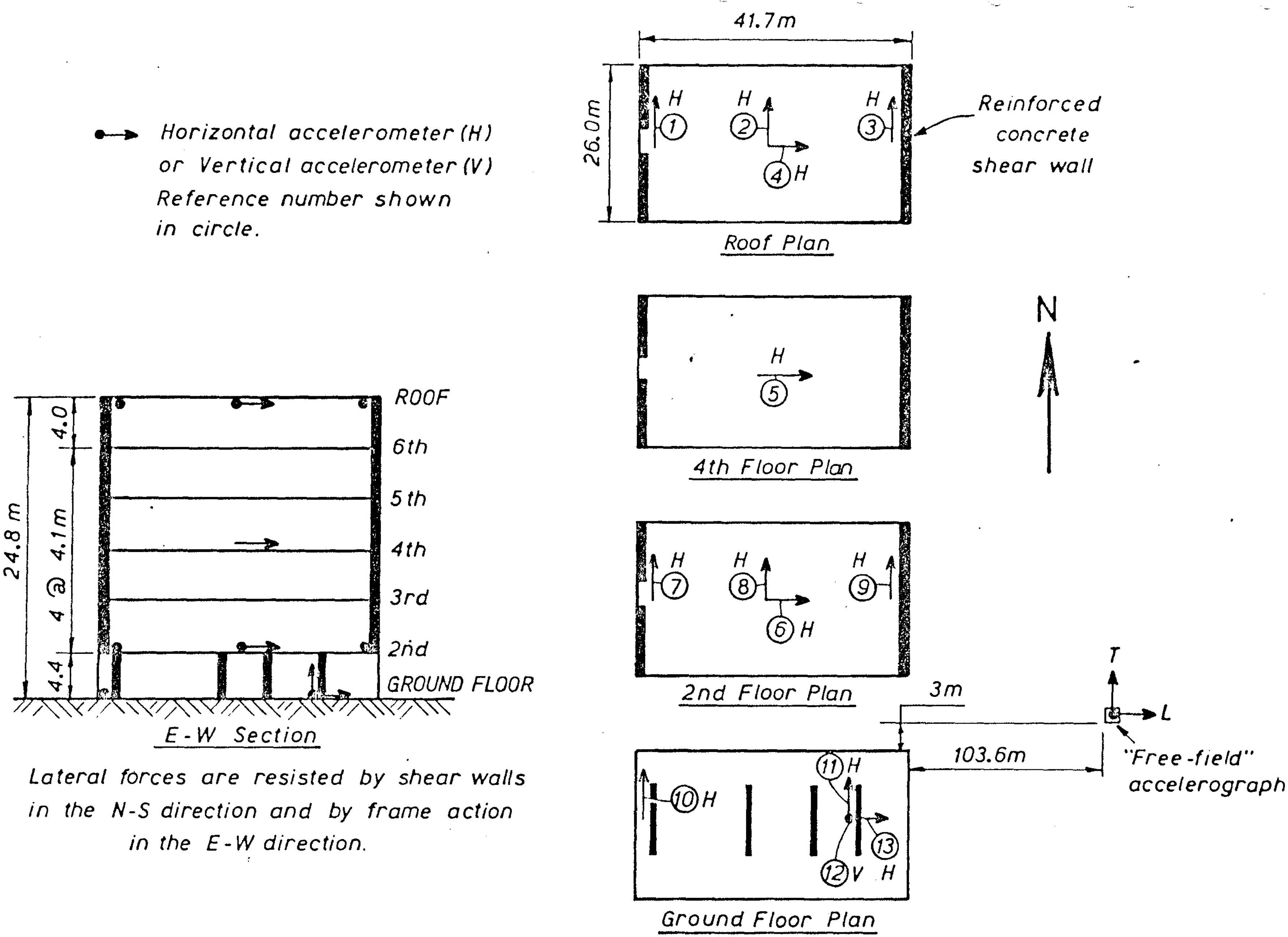
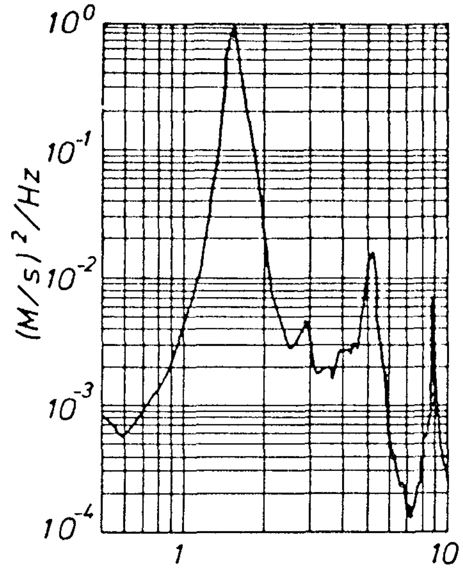


IMPERIAL COUNTY SERVICES BUILDING STRONG-MOTION INSTRUMENTATION

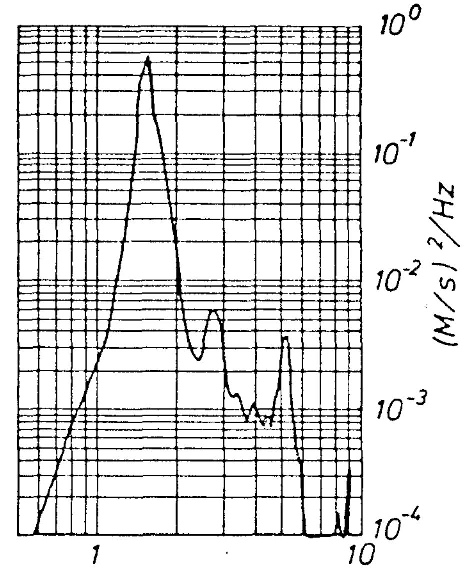
Figure 2

IMPERIAL COUNTY SERVICES BUILDING

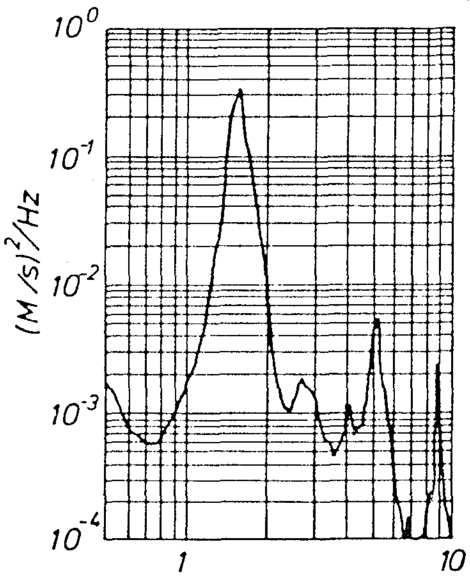
Ambient Vibration Test Results



Auto Spectrum 1-Hz

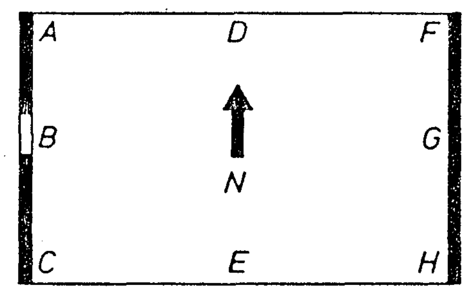


Auto Spectrum 2-Hz

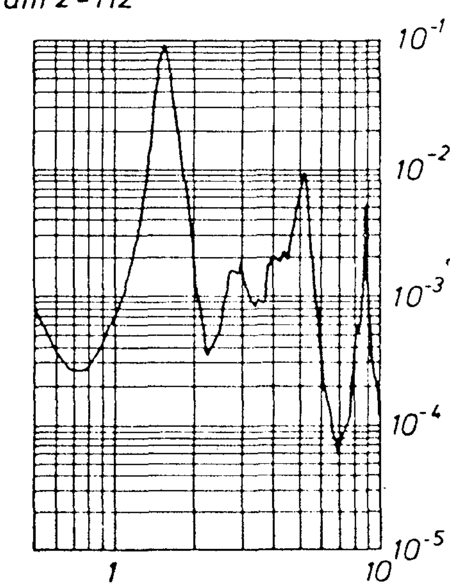


Auto Spectrum 3-Hz

Seismometer	Location	Orientation
1	B-7	E
2	B-6	E
3	B-4	E
4	B-2	E



Plan View



Auto Spectrum 4-Hz

Figure 3

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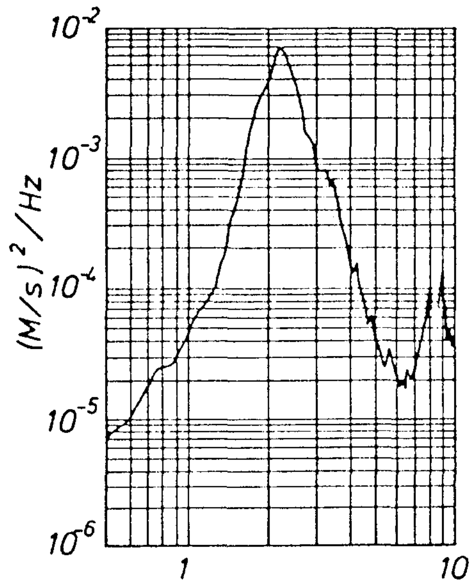
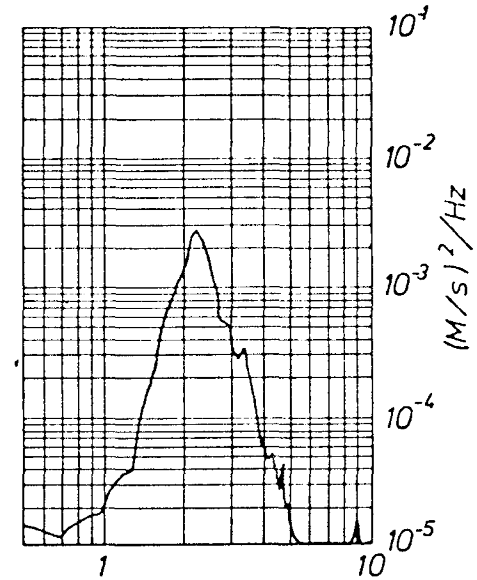
IMPERIAL COUNTY SERVICES BUILDING

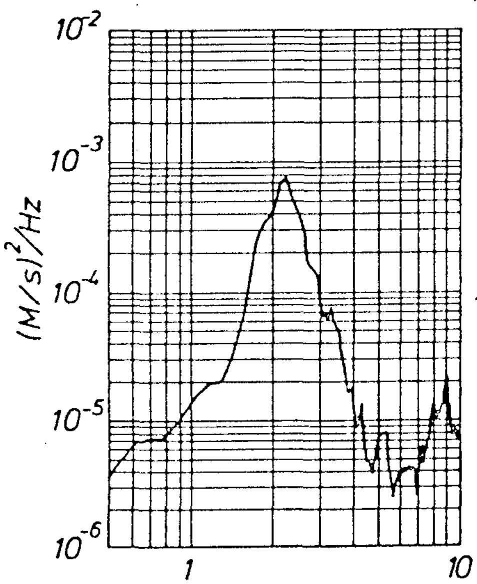
Ambient Vibration Test Results



Auto Spectrum 1-Hz

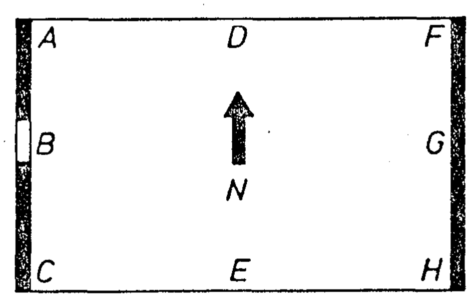


Auto Spectrum 2-Hz

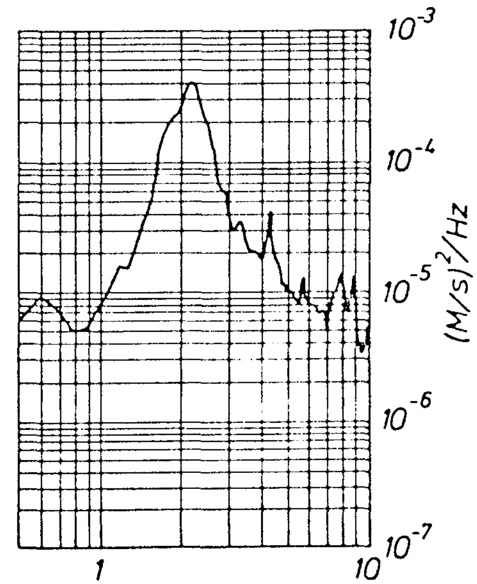


Auto Spectrum 3-Hz

Seismometer	Location	Orientation
1	D-7	N
2	D-6	N
3	D-4	N
4	D-2	N



Plan View

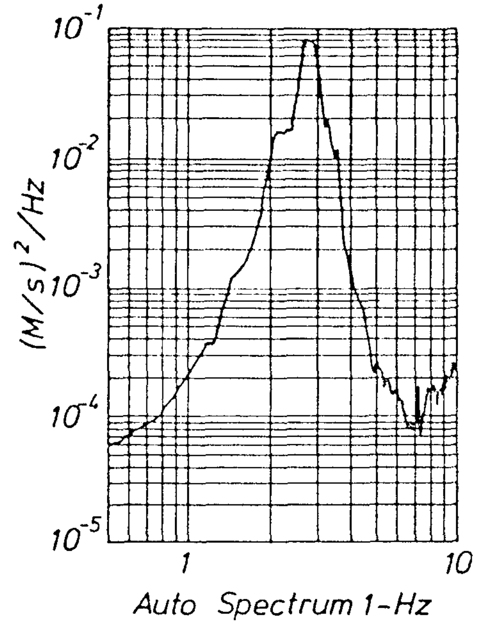


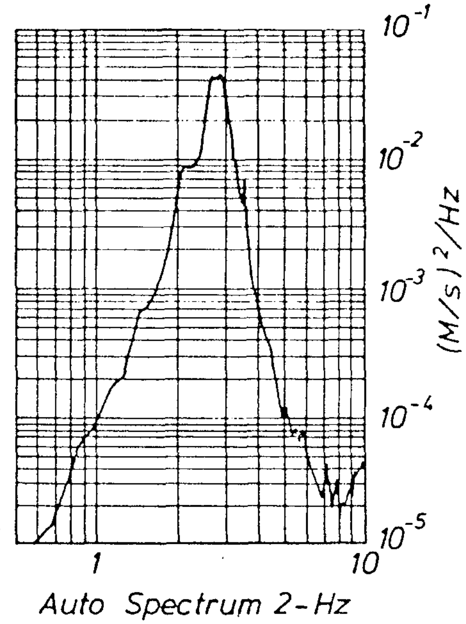
Auto Spectrum 4-Hz

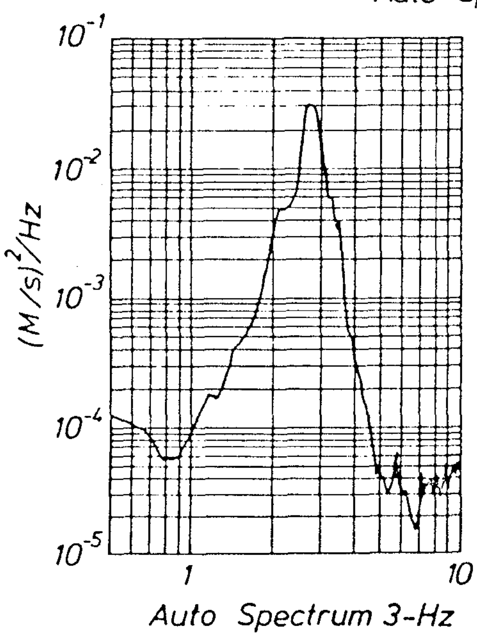
Figure 4

IMPERIAL COUNTY SERVICES BUILDING

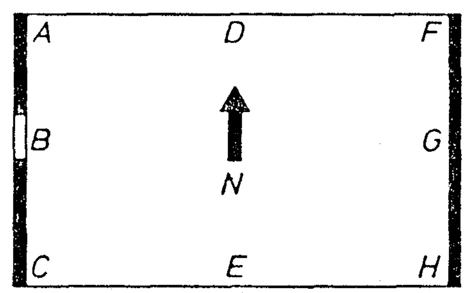
Ambient Vibration Test Results







Seismometer	Location	Orientation
1	B-7	N
2	B-6	N
3	B-4	N
4	B-2	N



Plan View

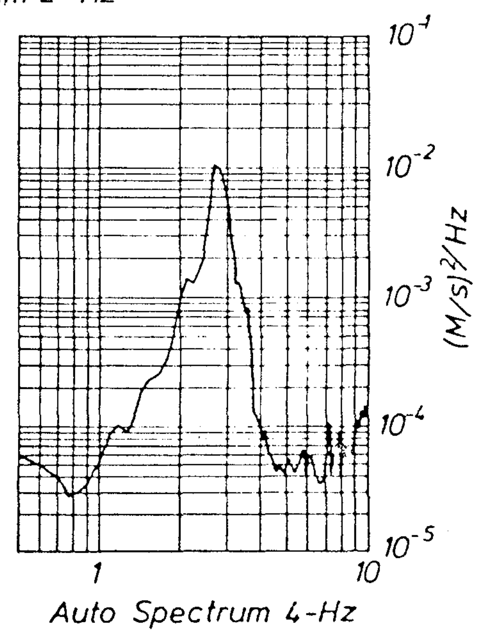
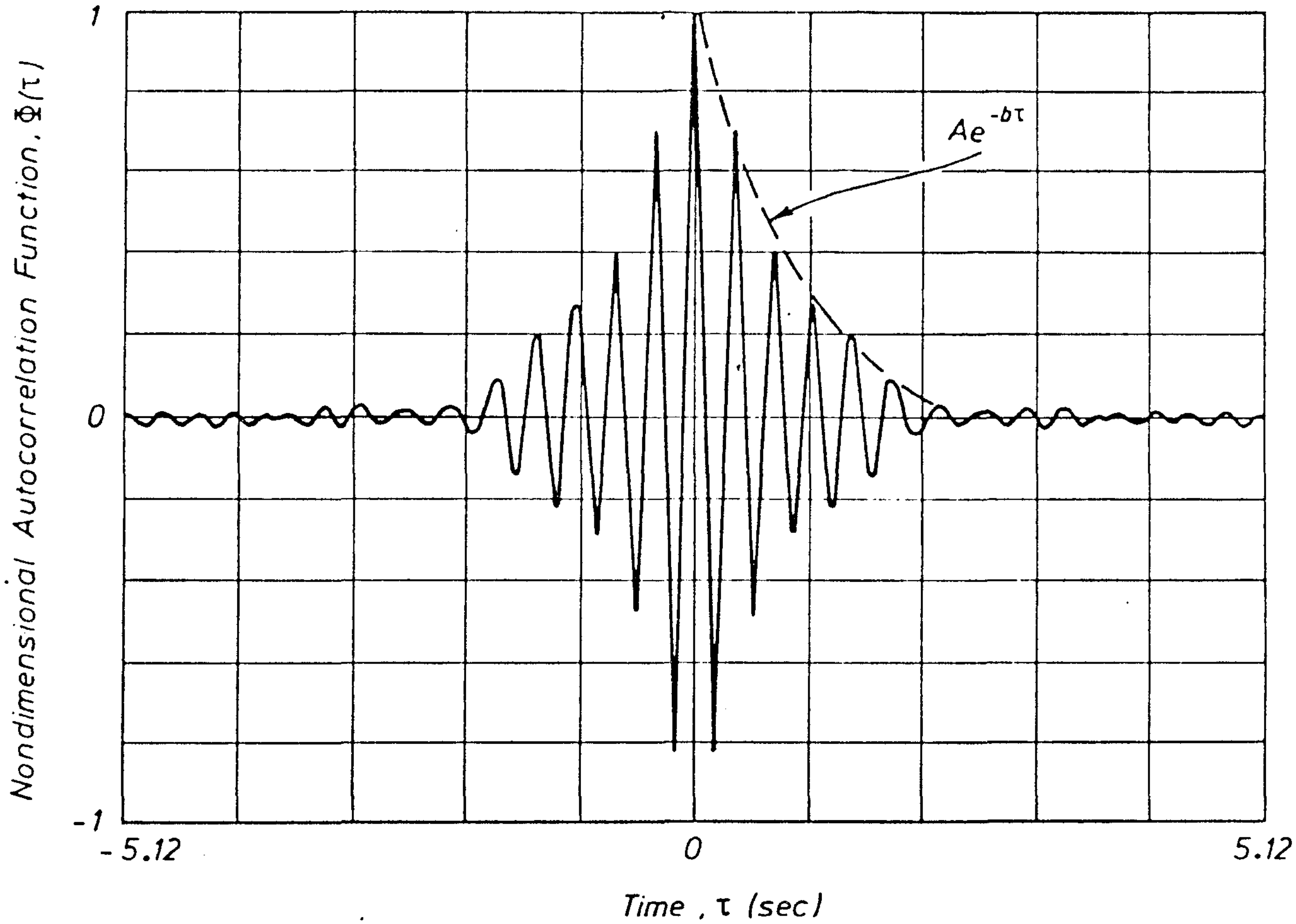


Figure 5

EXPONENTIAL CURVE FIT TO AUTOCORRELATION ENVELOPE



$$b = -\frac{\sum\tau_i \ln\Phi_i - \frac{1}{n}\sum\tau_i \sum \ln\Phi_i}{\sum\tau_i^2 - \frac{1}{n}(\sum\tau_i)^2} = 1.310hz = 2\pi f \xi$$

$$A = exp\left[\frac{b\sum\tau_i + \sum\Phi_i}{n}\right] = 1.056$$

$f = 2.852\,hz$

$\xi = 7.31\%$

Figure 6

APPENDIX F

ELASTIC ANALYSIS OF THE IMPERIAL COUNTY SERVICES BUILDING

Gerard C. Pardoen
Peter J. Moss
Athol J. Carr

SUMMARY

This paper presents the results of an elastic structural analysis of the Imperial County Services Building (ICSB) in El Centro, California. This analysis is of particular significance because the ICSB suffered major structural damage during the October 1979 Imperial Valley earthquake and the building's response was recorded on permanently installed strong motion accelerographs.

Background

The Imperial County Services Building (ICSB) has been and continues to be a topic of structural engineering research and investigation. The cause calibre for these investigations is the unique circumstances that the building was instrumented under the California Strong Motion Instrumentation Program and this instrumentation recorded the building's motion during the 15 October 1979 Imperial Valley earthquake. Additionally the free field motion of a triaxial accelerograph recorded the strong motion enabling researchers to perform soil structure interaction studies.

This paper presents the results of an elastic structural analysis of the ICSB undertaken to provide a basis of comparison for those examining the experimentally recorded response and for those conducting inelastic analyses.

Building Description[10]

The Imperial County Services Building (ICSB) served as an office building for the governmental services provided by Imperial County (see Figure 1). It was designed in 1968 (using 1967 edition of the Uniform Building Code) and completed in 1971 at a construction cost of $1.87 million. Due to the extensive damage it suffered during the October 1979 earthquake, the ICSB was demolished in 1980. The building was 136 feet 10 inches by 85 feet 4 inches in plan and founded on a Raymond step-taper concrete pile foundation (see Figure 2). The piles were interconnected with reinforced-concrete link beams; they extended 45 feet to 60 feet into the alluvium foundation material composed primarily of sand with interbeds of clay to 60 feet (based on logs from 4 soil borings at the site).

Vertical loads were carried by reinforced-concrete floor slabs (5 inches thick at the second floor, 3 inches thick at all the upper floors except 6 inches thick in the Penthouse area) supported by reinforced-concrete 5 1/2 inch-wide by 14 inch-deep pan joists spanning in the north-south (transverse) direction; the joists were supported by four longitudinal reinforced-concrete frames at 25 feet on center. The frame columns were typically 2 feet square whereas the beams varied in size. Beams in the two interior frames were 2 feet wide by 2 feet 6 inches deep at all levels; those in the two exterior frames were 2 feet by 2 feet 5 inches at the second-floor level, 10 inches by 4 feet 5 inches at the third-floor through sixth-floor levels, and 10 inches by 4 feet 2 inches at the roof level.

Lateral loads were resisted by the four reinforced-concrete frames in the east-west direction and reinforced-concrete shear walls in the north-south direction. The shear walls were discontinuous at the second-floor level. Below the second floor were three interior and one exterior 1-foot thick shear walls, and above the second floor, shear walls existed only at the east and west ends. Between the second and third floors, the walls were 7 1/2 inches thick, and above the third floor, they were 7 inches thick. According to the design calculations, the design "K" factor used was 1.33 for the north-south shear walls, 0.67 for the east-west interior frames, and 1.0 for the east-west interior frames.

Scope of Analyses

The elastic analysis herein considers 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[6] lateral forces for both the N-S and E-W directions.

B) The N-S component of the 18 May 1940 Imperial Valley[3,4] 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.

Description of the Elastic Model

Whereas a three-dimensional structural analysis model of the ICSB is inherently complex, the ETABS[12] computer code, which was used in these analyses, makes two assumptions that greatly simplify the preparation of input data and significantly reduces the computational effort while retaining the significant characteristics of the building's 3-D model.

The assumption that the floors of the ICSB are rigid in their own plane is a realistic approximation for most buildings. The second simplification assumes that the horizontal lateral loads act at the floor levels. Thus the lateral loads are transferred to the columns and shear wall elements through the rigid floor diaphragms. These two assumptions reduce the computational complexity of the analytical model to that of three degrees of freedom at each floor level--translations in the horizontal plane and a rotation about the vertical axis.

In defining the analytical model of the ICSB for the ETABS code, the complete building is assumed to be composed of structural elements which can 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, are 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 is treated as an independent substructure in which the complete structure stiffness matrix is then formed under the assumption that all frames are connected at each floor level by a diaphragm which is rigid on its own plane.

Some of the significant aspects of the ETABS model of the ICSB include:

A) The North exterior frame, the South exterior frame, as well as the East and West discontinuous shear walls, are assumed to act as a single continuous structural unit. This unit is designated as Frame 1 for the ETABS model. The East discontinuous shear wall is sectioned to account for the fire escape (see Figure 1).

B) The North interior and South interior frames are assumed to be identical. The structural properties representing these frames are designated as Frame 2 for the ETABS model (see Figure 3).

C) The single story shear walls at column lines B, D, and F are denoted by the Frame 3 properties for the ETABS model (see Figure 3).

D) The single story shear wall at column line E is denoted by the Frame 4 properties for the ETABS model (see Figure 3).

E) Since deformations within joints are neglected, the effective length of both the beams and columns are reduced by the 'rigid end zones' lengths associated with each structural component. The 'rigid end zones' for the beams are calculated to be equal to half the width of the columns below. The top 'rigid end zone' for the columns are taken as the average depth of the girders on either side whereas the bottom 'rigid end zone' for the columns are taken as zero.

F) The concrete strength of the columns is taken at 5000 psi whereas the concrete strength of the beams and shear walls is taken at 4000 psi. The basic sectional properties of the beams and columns are given in Table 1. It should be noted that the frame numbers refer to the R/C frames shown in Figure 4 and distinct from the frame numbers of the analytical model shown in Figure 3.

G) The ground floor columns are assumed to be fixed at the ground level and not at the pile cap. It should be noted that some of the ICSB's ground level columns could be considered fixed at the ground (those interior columns tied in with the ground floor slab) and the others fixed at the pile cap (those perimeter columns encased in soil).

Assumptions of the Elastic Model

In addition to the modeling choices imposed by use of the ETABS computer code certain stiffness and mass assumptions are selected for the ICSB analytical model. These assumptions include:

A) An uncracked concrete cross section is used for all beam, column, and shear wall properties.

B) Section properties are based solely upon the concrete, i.e., no transformed section properties are used to account for the reinforcing steel.

C) The center of mass for each floor is assumed to be at the geometric center.

D) The rotational inertia of each floor is assumed to be based upon a uniformly distributed mass on a rectangular area.

E) The weight of the Penthouse structure is assumed to be 20 kips whereas the weight of the Penthouse machinery is assumed to be 50 kips.

In addition to the aforementioned stiffness and mass property assumptions, the justifiable latitude in modeling assumptions allows one to "fine tune" the analytical model. 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[7] and those predicted by the analytical model depicted in Figure 3 after "fine tuning" are compared.

	ANALYTICAL		EXPERIMENTAL	
Direction	N-S	E-W	N-S	E-W
Frequency	2.27 Hz	1.49 Hz	2.25 Hz	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

As a measure of the analytical model's sensitivity to reasonable modelling assumptions, the effect of the exterior frames' columns on the fundamental frequency may be examined. The reinforced concrete columns from the ground level up to the 2nd floor are 2 feet square in cross section whereas the columns from the 2nd floor level up to the roof are somewhat trapezoidal shaped. The bases of the trapezoid are approximately 10" and 18" whereas the depth is approximately 70" (see Figure 5). For such a cross section one might be tempted to consider an "effective depth" of the column for resisting bending about the x-axis. For instance does one choose h=70" or some smaller dimension such as h=40" (as the original ICSB design calculations indicate)? While it is recognized that certain local discontinuities would occur near the junction of the two different column cross sections at the 2nd floor level, it is felt that h=70" probably represents the bending rigidity of the columns above the 2nd floor level and, as such, is the cross sectional property used in the analyses. Nevertheless one can see from the table of frequencies in Figure 5 that there is a certain degree of response sensitivity due to just the exterior column modeling assumption.

Additional stiffness parameters could equally alter the fundamental frequency results. For instance, the original design concrete strengths of 4000 psi and 5000 psi were specified for the beams and columns respectively. However after 8 years of construction one could expect these 28 day strengths to be easily 20% higher which would affect the elastic modulus by about 10% which, in turn, would effect the fundamental frequencies by about 5%. In an effort to avoid a plethora of analytical models, one "base line" model was chosen in order to interpret the different responses due to different input loads. The trends for the different load conditions are apparent for analytical models which deviate somewhat from the "base line." Thus the ETABS analytical model representing the ICSB is depicted in Figure 3 and is characterized by the stiffness parameters in Table 1 and the mass properties in Table 2.

Description of Load Conditions

The ETABS analyses of the ICSB cover 7 load conditions--3 static and 4 time history analyses. The static analyses reflect the two lateral load conditions due to the 1967 Uniform Building Code requirements and a vertical load condition arising from the structure's dead weight. The time histories reflect the appropriate components of the 1940 and 1979 Imperial Valley earthquakes considered in the N-S or E-W directions of the ICSB. Five percent critical damping is assumed for all time history analyses.

The 1967 UBC required a static analysis to be conducted for the building due to the effect of a force applied horizontally at each floor or roof level above the foundation. The lateral force at the jth level is was

$$F_j = (V - F_T)\, w_j h_j / \sum_{i=1}^{n} w_i h_i \qquad (1)$$

The 1967 UBC-defined base shear was[5]

$$V = Z\ K\ C\ W \qquad (2)$$

Inasmuch as the period T and ductility factor K differ for the N-S and E-W directions of the ICSB, then there is a base shear V corresponding to each of the N-S and E-W directions. The laterial forces were derived from the following basic data:

$Z = 1.0$

$K = 1.0$ for E-W direction; 1.33 for N-S direction

$C = .0577$ for E-W direction; .0655 for N-S direction

$T = (1/1.54\ Hz)$ for E-W direction; $(1/2.25Hz)$ for N-S direction

$W = 10420$ Kips (from Table 2)

Thus the base shear for the E-W direction is $V = 601$ Kips and the base shear for the N-S direction is $V=907$ Kips. Implementation of equation (1) gives the distribution of lateral forces for the N-S and E-W directions as

Story Level	j	F	F_{E-W}	F_{N-S}
Roof	6	.2280 V	137.0 K	206.9 K
6th Floor	5	.2529 V	152.0 K	229.4 K
5th Floor	4	.2030 V	122.0 K	184.3 K
4th Floor	3	.1532 V	92.1 K	139.0 K
3rd Floor	2	.1038 V	62.4 K	94.2 K
2nd Floor	1	.0591 V	35.5 K	53.7 K

The vertical load condition was approximated by adding the dead load of each story to a uniformly distributed load on the beams at each story level.

The time histories considered were the first 15 seconds of the 1940 (N-S component) and the 1979 (E-W and N-S compoments) Imperial Valley earthquakes. It should be noted that the 1979 records represent the free field accelerograph response which was located approximately 100 meters east of the ICSB.

Discussion of Elastic Analyses Results

The elastic analyses results provide 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 is made to tabularize the force resultants for all structural members due to all load conditions, but attention is devoted to the signficantly loaded members and, in view of the catastrophic failure of the 4 columns along column line G (see Figure 4), particularly the ground level columns.

The moment carrying capacity of the beams in the ICSB is determined using[9]

$$M_u = .85f'_c\,ab\,(d - a/2) + A'_s f'_s (d - d') \tag{3}$$

where the appropriate dimensions are detailed in Figure 6.

However the ultimate moment cannot be found explicitly by direct substitution into Equation (3) because the neutral axis is undefined. Assuming a maximum strain of .003 in the concrete and a stress in the tension and compression steel no greater than its yield strength, the magnitude and location of the concrete and steel force resultants can be determined. Using f = 4000 psi concrete, the ultimate bending moment for the girder of the ICSB were determined and are displayed in Table 3. Note that for each end of the beam there are two moment values--one corresponding to a positive and a negative moment. It should be noted that the ultimate moment values in Table 3 reflect the implementation of Equation (3)--in which neither the in-situ concrete strength of the beams (probably somewhat greater than 4000 psi) nor the normal capacity reduction factor is included. The compression and tension steel quantities used in Equation (3) reflect the steel distribution at the probable plastic hinge points of the beam.

Whereas the axial forces within the girders are assumed to be negligible and thus do not affect the load carrying capacity of the beams, for the columns the combination of axial load and moment is included. For a cross section with uniaxial bending reinforced with n bars, the force and moment equilibrium equations are

$$P_u = .85f'_c ab + \sum_{i=1}^{n} f_{si} A_{si} \qquad (4)$$

$$P_u e = .85f'_c ab(h - a)/2 + \sum_{i=1}^{n} f_{si} A_{si} (h/2 - d_i) \qquad (5)$$

where e is the eccentricity of applied axial load.

Figure 6 reflects the implementation of Equations (4) and (5) into an interaction curve for the ground floor level columns of the ICSB. Although the curve assumes bending about the N-S axis (e.g. due to horizontal E-W forces), the curves are valid for the other principal direction of bending as well as for columns of other story levels.

Detailed numerical results have been presented in Reference 8 and are summarized as follows:

a) All girders respond elastically due to the 1967 UBC and dead load conditions (including the appropriate load factors).

b) All girders of the two interior frames respond elastically when the 1940 N-S record is considered in the N-S or shear wall direction. Plastic hinges are predicted for all exterior frames, 2nd floor girders as well as for all girders framing into the shear walls. The exterior bay girders of the exterior frames indicate possible hinging at the 3rd-6th floor levels whereas the 3 interior bay girders of the exterior frames indicate elastic response at these floor levels. Additionally all roof level girders of the exterior frames indicate elastic response.

c) Although the predicted internal force resultants for the girders due to the 1979 N-S record are obviously different from the predicted force resultants arising from application of the 1940 N-S record, the plastic hinge pattern for the girders is similar to that noted above in b).

d) All girders, except possibly those at the roof level, of the interior and exterior frames would probably respond inelastically if the ICSB were subjected to the 1940 N-S record in the E-W direction. The same macroscopic behavior could be expected of the ICSB's girders when subject to the E-W record of the 1979 earthquake. The reconnaissance reports of the 1979 earthquake tend to substantiate these predictions.[2]

e) The combination of axial force and moment for all ground level columns lie within the interaction curve envelope defined in Figure 6 due to the combination of the 1967 UBC and vertical load cases (with appropriate load factors). It should be noted that the reinforcing pattern for the columns are as follows:

Interior Frames - Columns B, C, D, F, C	-	8 # 9 Bars
Interior Frames - Column E	-	8 #10 Bars
Interior Frames - Columns C, D, E, F	-	8 #11 Bars
Interior Frames - Columns B, G	-	10 #11 Bars

As expected the columns in lines B and G carry the lateral x-direction forces whereas only the 4 corner columns carry the lateral y-direction forces. Thus for an earthquake consisting of significant x- and y-direction components such as the 1979 event on the basis of these elastic analyses one would expect the 4 corner columns to be severely loaded.

Superficially the ICSB appears to be a simple, rectangular, symmetrical structure and, as such, one would expect the 4 corner columns to react on a symmetrical fashion. However, inspection of the first floor plan in Figure 3 indicates the unsymmetrical distribution of first story shear walls which account for the difference in column behavior for those at the east end from those at the west end. Arnold's[1] contribution to the EERI reconnaisance report describes the difference of east and west end column behavior which is reflected on the numerical results in Reference 8 as well as the catastrophic column failure at the east end during the 1979 earthquake.

f) The combination of axial force and moment for all ground floor columns lie considerably outside the interaction curve envelope defined in Figure 6 due to the combination of the vertical dead loads and either the 1940 or 1979 records. However if the ICSB were subject to say 25% levels of the 1940 or 1979 records then the combination of axial force and moment for all ground floor columns would lie within the interaction envelope. These 25% levels represent a design basis for performing a dynamic analysis in which the structure should be expected to respond elastically and that sufficiently ductility could be incorporated to enable the structure to absorb higher earthquake loads.

With respect to the ductility of the east end ground floor columns it may be noted that figure 2 denotes a typical column detail in which column reinforcement ties are provided up to a point near the top of the soil. However a concrete slab encases the interior frames' columns near the top of the reinforcement ties so that at least half the columns do not have sufficient ties above the rigid slab. Thus a further explanation of the east end's column failure is that the corner columns (G-1, G-4) are sufficiently loaded due to the significant E-W and N-S records whereas the interior columns (G-2, G-3) failed due to insufficient ductility at the base. Note that the elastic analyses considered the column height from the slab to the 2nd story rather than from the pile cap to the 2nd story.

Conclusion

This paper presents predictions of the structural response of the Imperial County Services Building due to several static and dynamic load excitations using elastic analysis techniques. Seven representative load conditions are considered 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 elastic behavior of the ICSB, the fact remains that elastic analyses clearly play a useful role in post-earthquake "post mortems" by clarifying structural action. In particular the elastic analyses clearly pin-pointed the east end column weaknesses which were subsequently confirmed by the damage sustained. These computed response results of the ICSB, which is destined to become the one of the most thoroughly investigated structure in earthquake engineering history, help explain the catastrophic failure of the building during the 15 October 1979 earthquake.

STIFFNESS PROPERTIES - IMPERIAL COUNTY SERVICES BUILDING

BEAM PROPERTIES

Frame	Floor Level	Iyy	Rigid End Zone at Each end
1 & 4	2	54000	9
1 & 4	3 → 6	124064	9
1 & 4	Roof	104167	9
2 & 3	2 → roof	54000	12

COLUMN PROPERTIES

Frame	Story Level	A	Ixx	Iyy	Top Rigid End Zone
1 & 4	1	576	27648	27648	30
1 & 4	2 → 5	1028	420917	20177	53
1 & 4	6	1028	420917	20177	50
2 & 3	1	576	27648	27648	30
2 & 3	2 → 5	576	27648	27648	53
2 & 3	6	576	27648	27648	50

(Units: A = in , Ixx,Iyy = in , Rigid End Zone = in)

TABLE 1

MASS PROPERTIES - IMPERIAL COUNTY SERVICES BUILDING

Floor Level	$M = W/g$	$I = W/g\ (a + b\ )/12$
2	5.071	1.492×10^6
3	4.614	1.348×10^6
4	4.597	1.343×10^6
5	4.597	1.343×10^6
6	4.597	1.343×10^6
Roof	3.476	9.493×10^5

(Units: $M = \text{K-Sec}^2/\text{in}$ $I = \text{K-in-Sec}^2$)

TABLE 2

MAXIMUM POSITIVE AND NEGATIVE ULTIMATE BEAM BENDING MOMENTS

FRAME	FLOOR	BEAM BC		BEAM CD		BEAM DE		COMMENT
EXT	ROOF	8.70	8.70	5.81	5.81	5.81	5.81	SYMMETRIC
		4.74	4.74	4.74	4.74	4.74	4.74	
EXT	6TH	8.53	8.53	6.18	6.18	6.18	6.18	SYMMETRIC
		5.05	5.05	5.05	5.05	5.05	5.05	
EXT	5TH	8.53	8.53	6.18	6.18	6.18	6.18	SYMMETRIC
		5.05	5.05	5.05	5.05	5.05	5.05	
EXT	4TH	8.72	8.72	7.00	7.00	7.00	7.00	SYMMETRIC
		5.05	5.05	5.05	5.05	5.05	5.05	
EXT	3RD	9.44	9.44	7.73	7.73	6.18	6.18	SYMMETRIC
		5.05	5.05	5.05	5.05	5.05	5.05	
EXT	2ND	8.59	8.26	8.26	7.67	7.67	7.67	SYMMETRIC
		4.45	4.17	4.17	4.17	4.17	4.17	

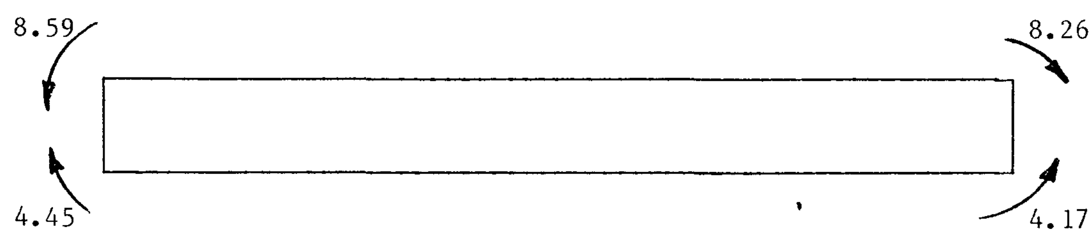
****

INT	ROOF	5.09	5.09	5.09	6.73	6.73	6.73	SPAN EF MOMENTS
		3.67	3.67	3.67	3.67	3.67	3.67	6.38 & 3.88
INT	6TH	6.73	6.73	6.73	6.73	6.73	6.73	SYMMETRIC
		3.67	3.67	3.67	3.67	3.67	3.67	
INT	5TH	6.73	6.73	6.73	6.73	6.73	6.73	SYMMETRIC
		3.67	3.67	3.67	3.67	3.67	3.67	
INT	4TH	7.38	7.38	7.38	7.38	7.38	7.38	SYMMETRIC
		4.09	4.09	4.09	4.09	4.09	4.09	
INT	3RD	8.59	7.65	7.65	7.98	7.98	7.98	SYMMETRIC
		4.09	4.09	4.09	4.09	4.09	4.09	
INT	2ND	10.00	9.18	9.18	9.18	9.18	9.18	SYMMETRIC
		4.92	4.59	4.59	4.59	4.59	4.59	

****

NOTE: All TABULAR MOMENT VALUES TO MULTIPLIED BY: 1000 KIP-IN

B E N D I N G M O M E N T S I G N C O N V E N T I O N



BEAM BC, 2ND FLOOR, EXTERIOR FRAME

TABLE 3

NOTATION

a = fraction of distance to centroid

A'_s = area of compression steel

b = beam width

C = $.05/\sqrt[3]{T}$

d = distance of tension steel from beam surface

d' = distance of compression steel from beam surface

F_j = force at jth level

f'_c = concrete compressive strength

f'_s = stress of compression steel at ultimate moment

F_T = portion of V considered concentrated at the top the structure (equal to 0 for the ICSB)

h_j = height above the base of the level j

K = ductility factor

M_u = ultimate moment

n = number of levels to the uppermost level in the main portion of the structure

T = building period

V = base shear

w_j = portion of the total dead load which is loaded at level j

W = total dead weight of the structure

Z = zone factor

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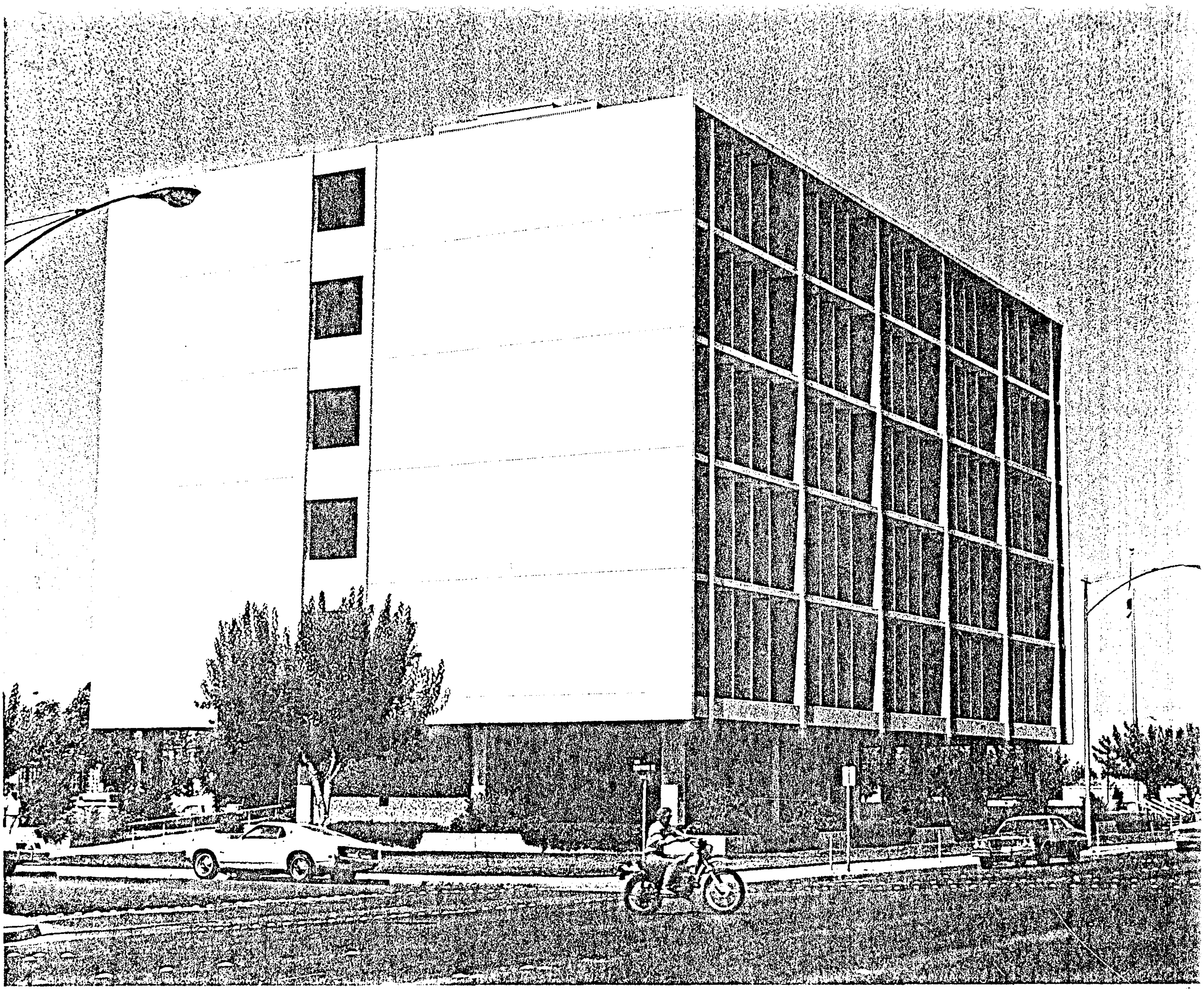
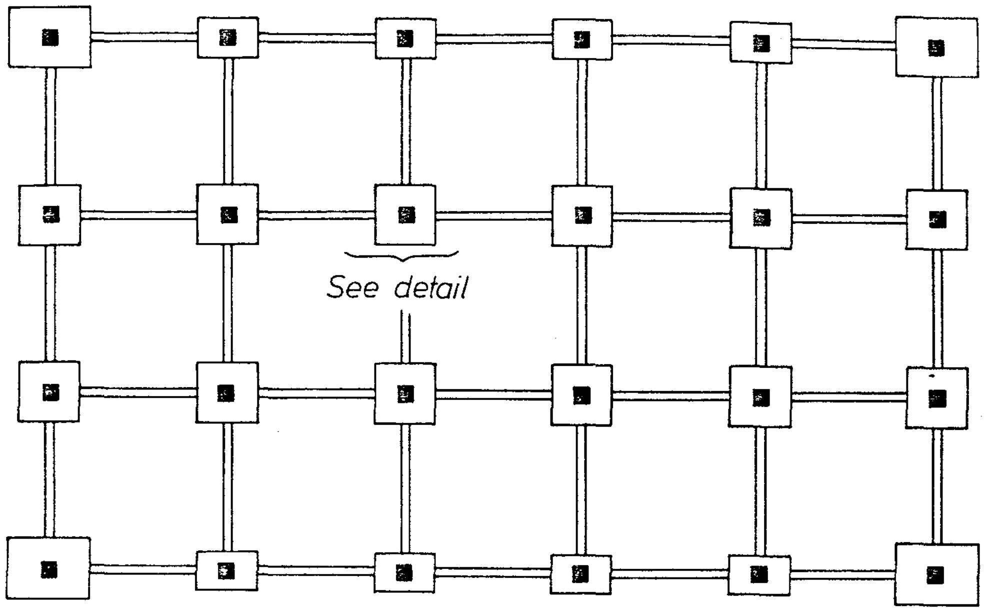
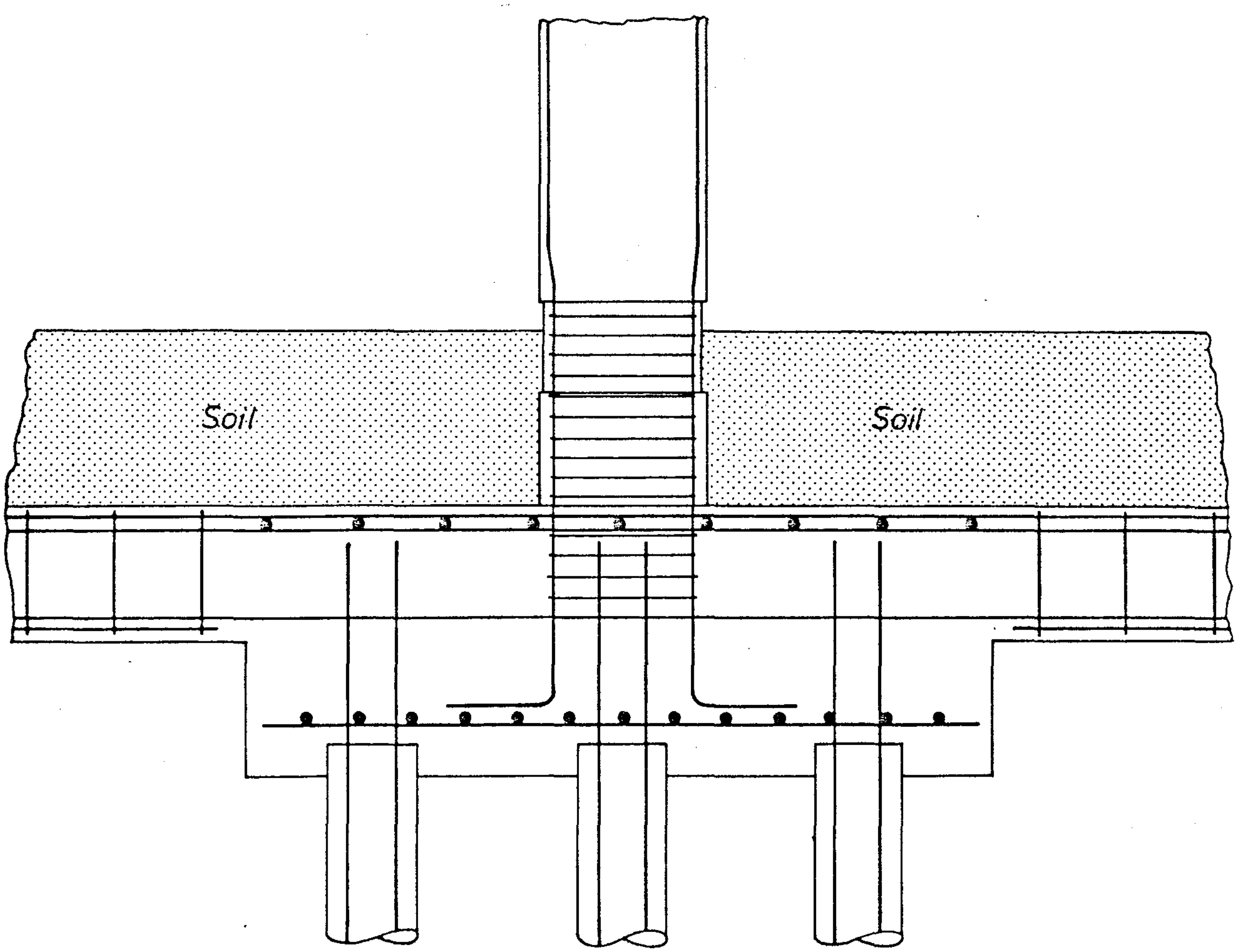


Figure 1



PILE AND FOUNDATION LAYOUT



TYPICAL PILE CAP

Figure 2

IMPERIAL COUNTY SERVICES BUILDING

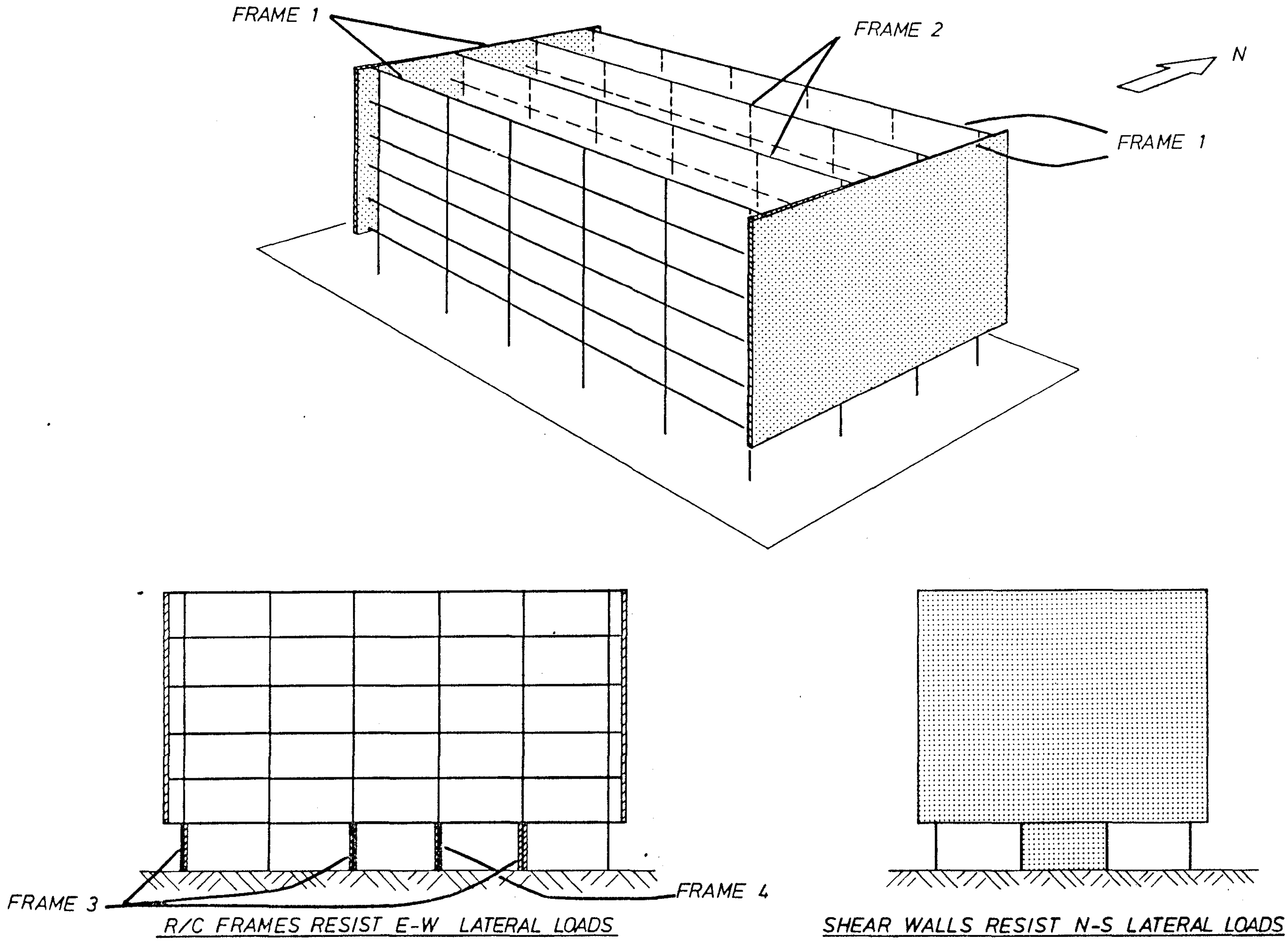
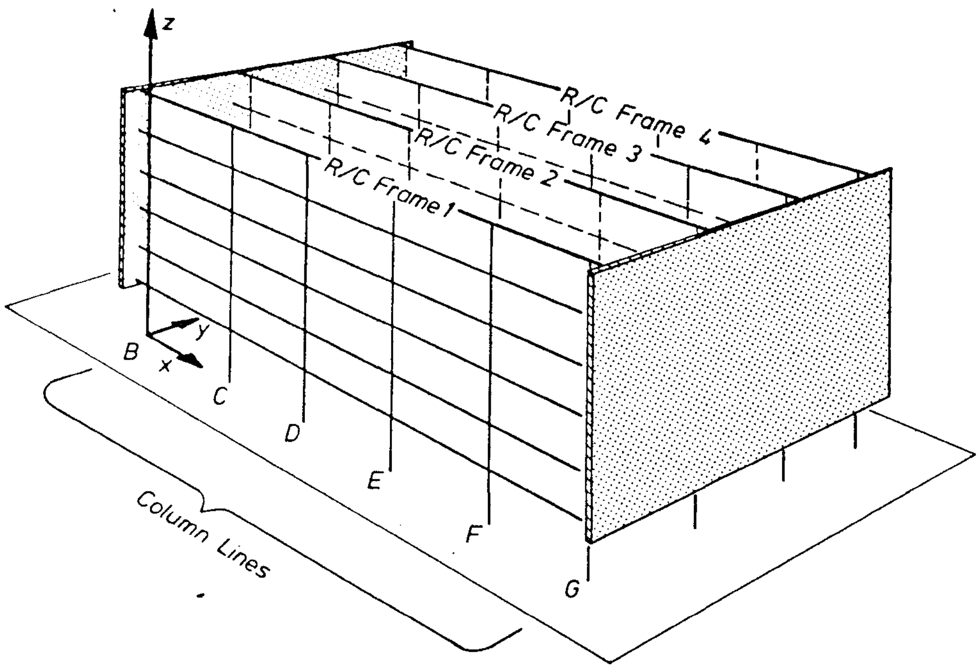
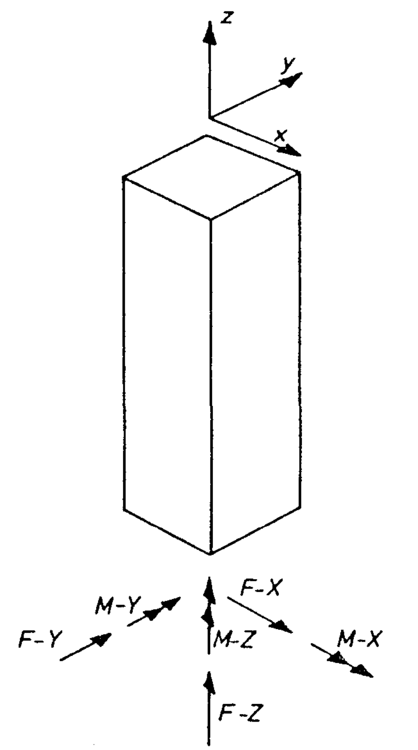


Figure 3

IMPERIAL COUNTY SERVICES BUILDING



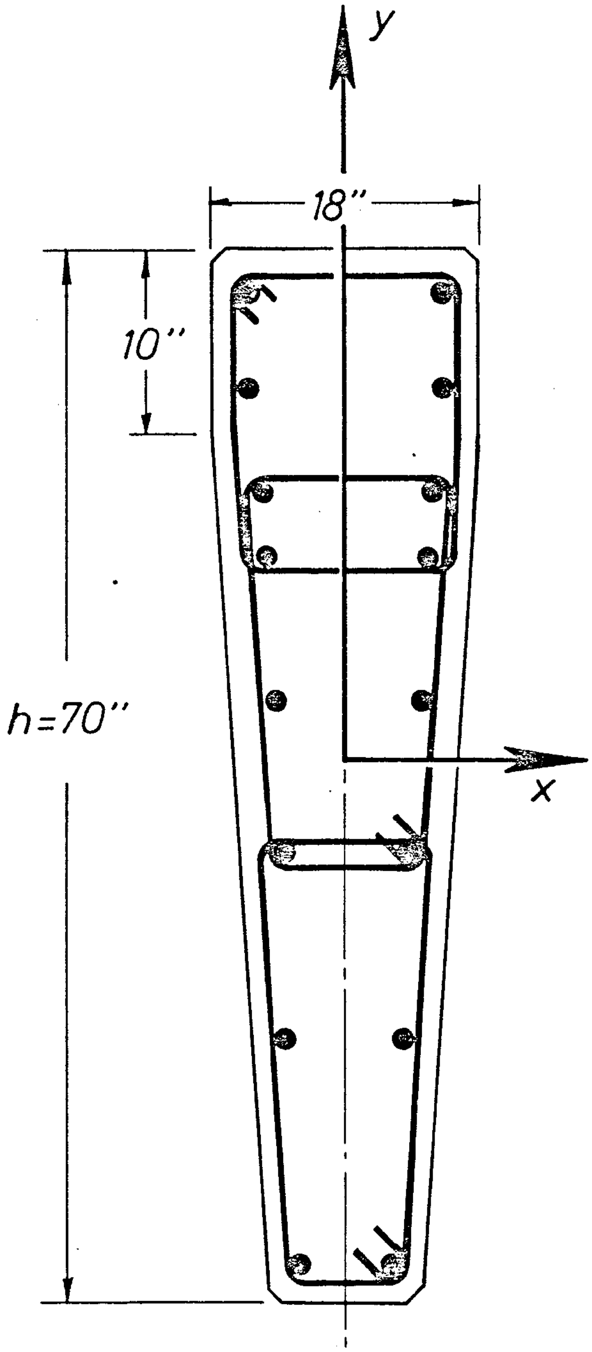
COLUMN DESIGNATIONS



GROUND LEVEL COLUMN FORCES

Figure 4

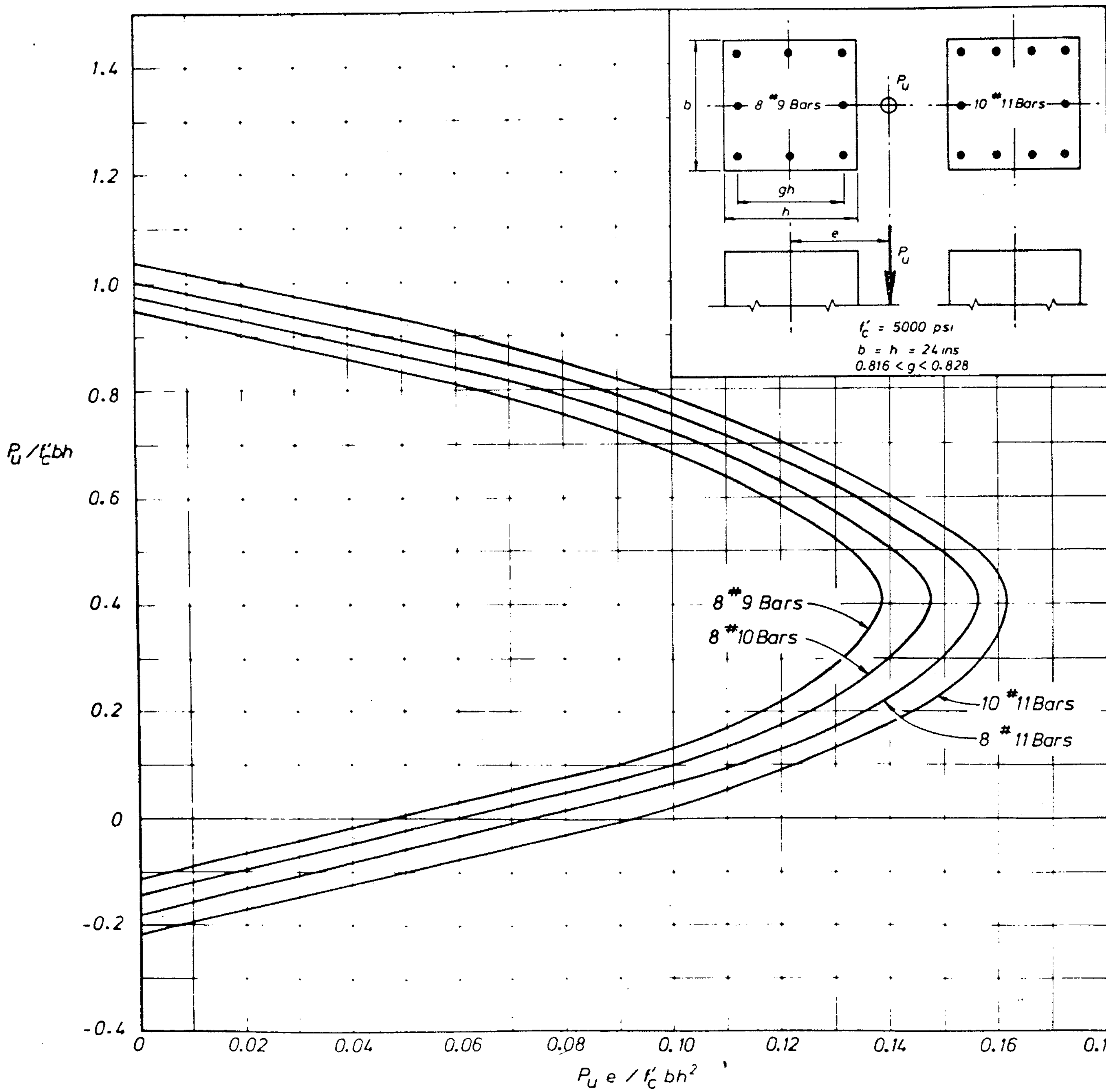
EFFECT OF EXTERIOR COLUMN PROPERTIES ON VIBRATION CHARACTERISTICS



h	I_{xx}	I_{yy}	E-W Frequency	N-S Frequency
70"	420917	20177	1.49 Hz	2.27 Hz
40"	86994	15653	1.47 Hz	2.14 Hz
18"	8620	8553	1.37 Hz	1.81 Hz

Figure 5

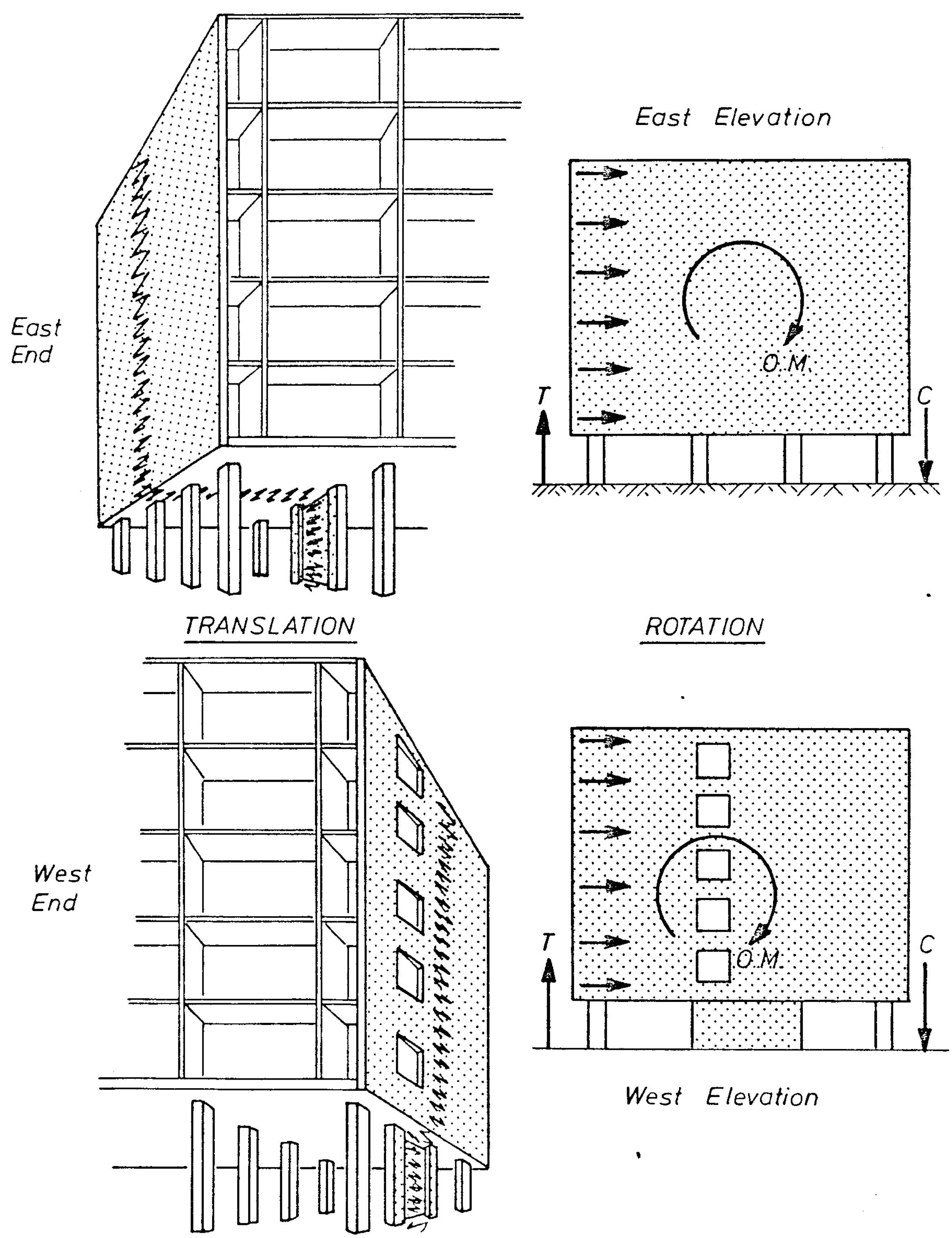
IMPERIAL COUNTY SERVICES BUILDING



Interaction Curve - Eccentrically Loaded R/C Ground Level Columns

Figure 6

IMPERIAL COUNTY SERVICES BUILDING



End Wall Conditions to Resist Translation and Rotation

Figure 7