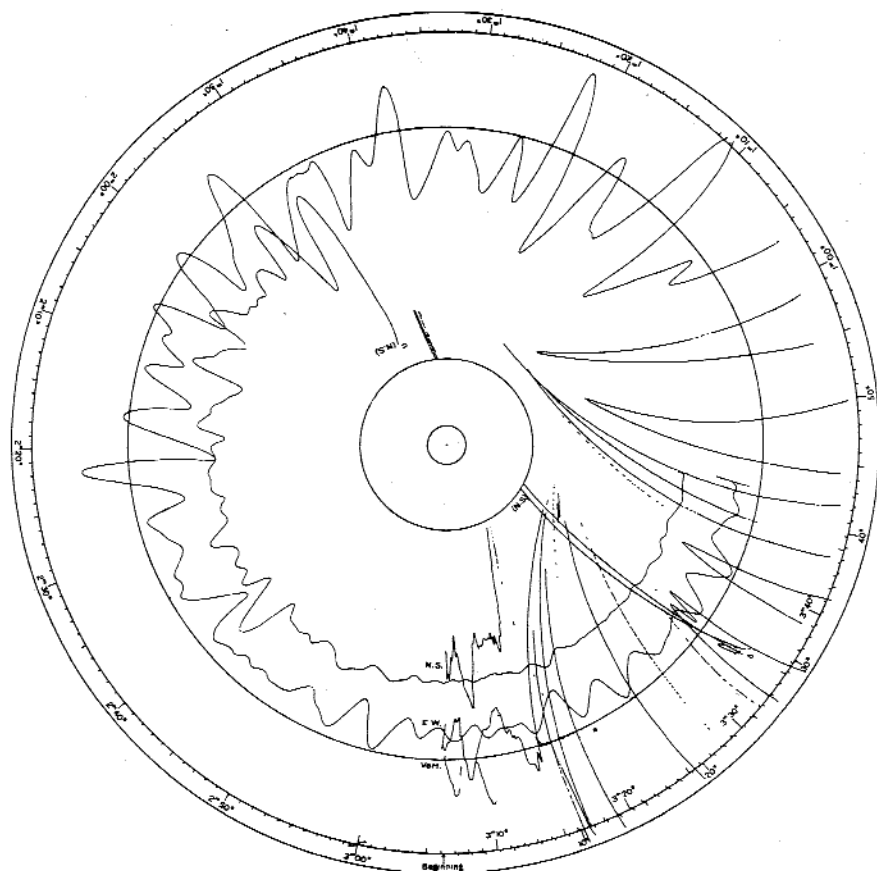


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The Professional Journal of the Earthquake Engineering Research Institute



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5.

Seismic Rocking Response of A Triangular Building Founded on Sand

G. Bongiovanni, M. Celebi, M.EERI, and E. Safak, M.EERI

A twenty-two story, triangular in plan, symmetrical, reinforced concrete building on the beachfront in Viña del Mar, Chile, was temporarily instrumented in August 1985 following the 3 March 1985 Central Chile earthquake ($M_s = 7.8$) and aftershocks were recorded. Ambient and free vibration tests were also performed. The paper reports studies of the records of the responses of the building, the primary modes of vibration of the structure, its excellent performance during the earthquake and results from analyses using mathematical models and system identification techniques.

INTRODUCTION

The 3 March 1985 Central Chile earthquake ($M_s = 7.8$) caused a variety of damage to structures in the townships of San Antonio, Melipilla, Valparaiso, Viña del Mar as well as the capital, Santiago. The general location of the epicenter of the main shock, some of the important aftershocks and the heavily affected main population centers are depicted in Figure 1.

Some of the uniquely engineered structures at the coastal town of Viña del Mar suffered extensive damage while others, such as Edificio de Miramar, survived the earthquake without any damage. Edificio de Miramar is one of a series of twenty-two story, triangular in plan, symmetrical, reinforced concrete buildings decorating the beachfront of Viña del Mar, Chile. During the $M_s = 7.8$ Central Chile earthquake of 3 March 1985, these buildings constructed from a typical design were not damaged while other architecturally unique structures in close proximity (Edificio H'Angoroa and Edificio Acapulco) were damaged extensively. Detailed accounts of the post-earthquake damage surveys related to the 3 March 1985 Chile earthquake are reported by Wyllie and others (1986), Monge and others (1986), and Celebi (1985) and will not constitute the scope of this paper. The scope of the paper will be restricted to Edificio de Miramar only.

It is important to study structures that perform well during earthquakes as well as those structures that do not. Another triangular-in-plan structure in Mexico City (Lottery Building—steel construction—in the lakebed zone of Mexico City) also performed well during the disastrous Michoacan, Mexico earthquake of 19 September 1985 ($M_s = 8.1$).

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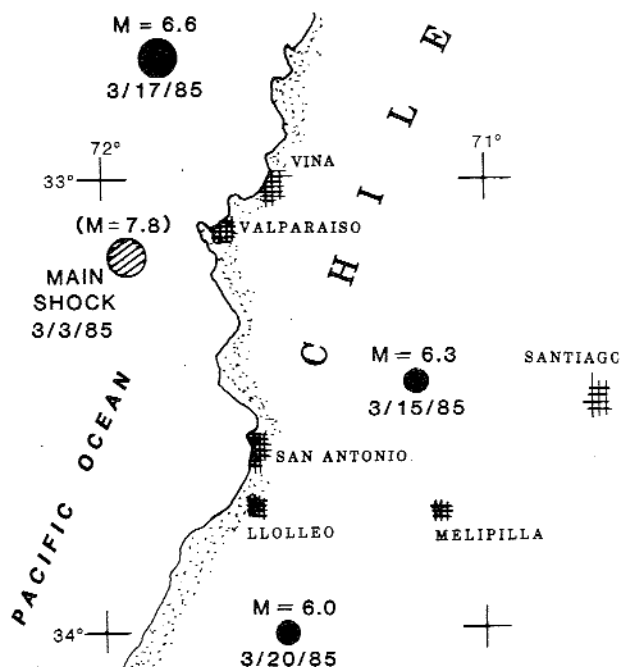


Figure 1. Main population centers affected by the main shock and aftershocks of the 3 March 1985 Chile earthquakes.

The purpose of this paper is to study the earthquake response of Edificio de Miramar which was temporarily instrumented to record earthquake motions in August, 1985—five months after the main event. Along with the data obtained from ambient and free vibration tests performed on the structure, the aftershock records from the building are used to determine the performance and vibrational characteristics of the structure. In addition, records of the same aftershock obtained at a reference rock station nearby are used to determine frequency-dependent amplification of motions at the site of the building with respect to the rock site. Finally, analyses using mathematical models that incorporate frequency independent soil springs to account for the soil-structure interaction and vibrational parameters derived by using system identification techniques are presented.

THE SITE

Edificio de Miramar is founded on beach sand. The depth to bedrock is unknown. A general location map of Edificio de Miramar is provided in Figure 2. The building is approximately 6 km north of the strong-motion station, VAL (an amphibolite gneiss rock site at the University of Santa Maria in Valparaiso) and about 2 km from the strong-motion station in Viña del Mar (VIÑA—an alluvial site), both of the Chilean strong-motion network (Saragoni and others, 1985). Records obtained during the 3 March 1985 earthquake show that there was considerable amplification of motion between 1–2 Hz in VIÑA compared with VAL (Celebi, 1987). Peak acceleration in VIÑA reached 0.36 g while at VAL the peak was 0.29 g (although not in the same direction).



Figure 2. Locations of stations in Viña del Mar.

THE BUILDING

Edificio de Miramar is a 56-meter-high, twenty-two story, equilateral triangular and therefore symmetrical about an axis in plan, reinforced concrete building. A general view, a vertical section and a plan view, respectively, of the building are provided in Figures 3-5. The building sits on a mat foundation without piles. There are two levels below the ground level with the lowest level filled in. Translational and torsional rigidity of the building is provided by three channel shaped central shear walls and the triangular columns around the perimeter of the building. The shear walls around the elevator shaft provide only a small eccentricity.

TEMPORARY INSTRUMENTATION

While performing other studies in Chile in August of 1985 and since sizeable aftershocks were still occurring at that time, with the hope of recording the response of a building, sensors and recorders that were not otherwise used were temporarily installed in three locations of the basement and two locations on the top floor of Edificio de Miramar. These locations, called stations TRA, TRB, TRC (in the basement), TRD, and TRE (on the top floor), are shown in Figure 6. These stations were not synchronized. Within two days, two small earthquakes with magnitudes less than 5 were recorded at the building stations as well as at the reference rock station, VAL. In addition, ambient and free vibration tests were performed using the sensors at the top floor.

To record the earthquakes and the response of the building, General Earthquake Observation System (GEOS) recorders and related peripherals were used. GEOS is discussed in detail by Borchardt and others (1985). During the course of these experiments, three-component Mark* Products L22-3D velocity transducers and three-component Kinematics* FBA-13 acceleration transducers were used.

*These are commercial names of instruments used only and do not constitute endorsement of these products.

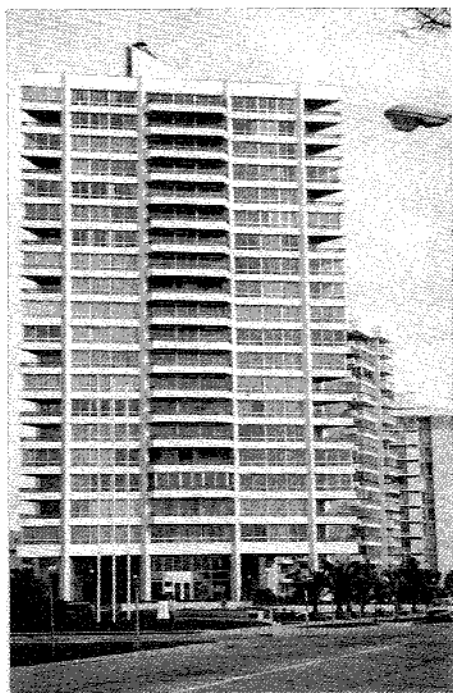
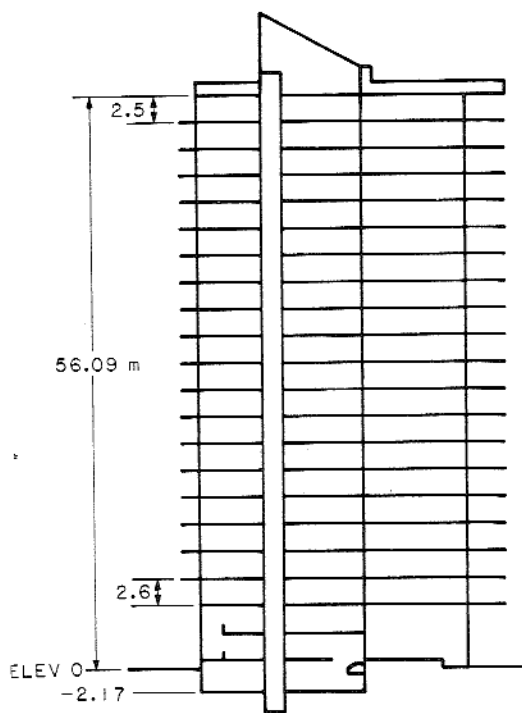


Figure 3. General view of Edificio de Miramar.



EDIFICIO TORRES DE MIRAMAR SECTION

Figure 4. Vertical section of the building.

RECORDS, FOURIER SPECTRA AND RATIOS

For the sake of brevity, three component velocity seismograms and corresponding Fourier spectra of one of the two aftershocks obtained at the reference station, VAL and at the basement stations (TRA, TRB, TRC), respectively are shown in Figure 7. Neglecting the effect of distance between the building and VAL (6 km), spectral ratios for the same event of records obtained at TRA with respect to VAL are shown in Figure 8. The spectral ratios are ratios of Fourier amplitude spectra and represent frequency-dependent amplification. These ratios as well as the seismograms show that there is considerable amplification of horizontal motion at the site of the building within the frequency range of 0.5–2.25 Hz with peaks at 1 Hz. This is significant because there are several midrise buildings in the vicinity of this building that have translational frequencies within this range, implying that they have been subjected to amplified motions during the main event; thereby one of the causes for the damage they suffered. The records from TRB and TRC both result with spectral ratios with similar trends; therefore, they will not be repeated herein.

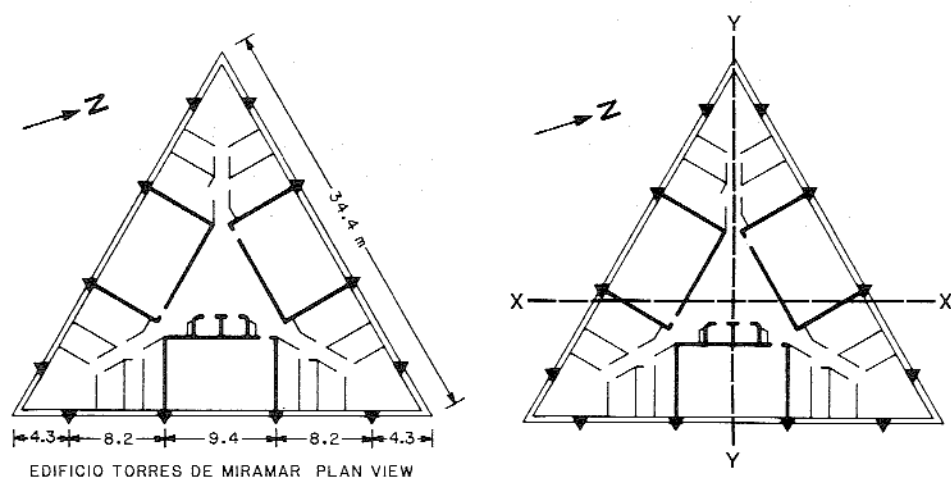


Figure 5. Plan-view of the building showing dimensions and locations of shear walls and columns as well as the main axes used for free vibration testing.

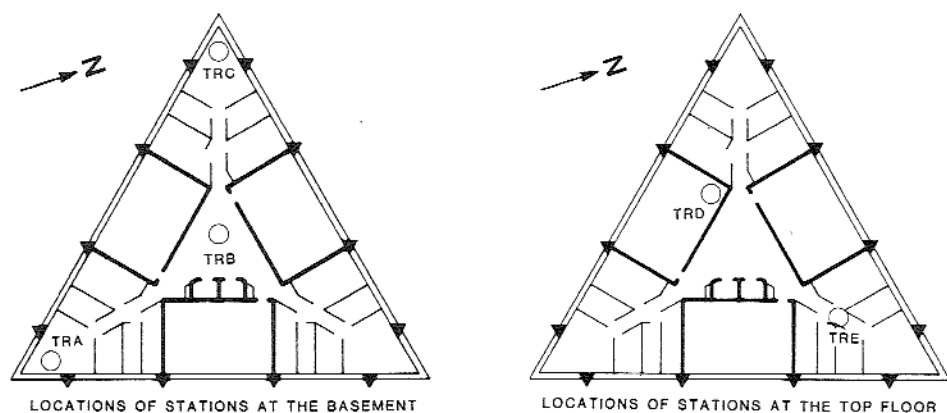


Figure 6. Location of stations at the basement and top floor of the building.

The Fourier spectra (Figure 7) of vertical motions at each of the three basement stations (TRA, TRB, TRC) all show peaks at 2.25 Hz. The spectral ratio TRA/VAL of vertical motions (Figure 8) shows amplification between 1–4 Hz with a peak at approximately 2.25 Hz.

In Figure 9, the velocity seismograms (vertical, N-S and E-W components) and corresponding Fourier spectra of the same event as in Figures 7 and 8 for the top floor stations TRD and TRE, respectively, are provided. These spectra derived from motions at the top floor (TRD and TRE) show two distinctive frequencies of horizontal vibration of the structure during the aftershock—at approximately 1 and 4 Hz.

VIBRATION TEST RESULTS

With two sets of recorders and sensors in operation on the top floor at locations TRD and TRE, the building was subjected to rhythmic human motions until satisfactory oscillation was achieved through oscilloscope observation and the responses were recorded. In Figures 10 and 11, the velocity responses and corresponding Fourier spectra of the building to excitations in the direction of both the "XX" and "YY" axes (see Figure 6) are shown. During the course of vibration tests, the excitation was applied at the intersection of the "XX" and "YY" axes. The velocity sensors were aligned with the "XX" and "YY" axes also. Again, distinctively, two modes of vibration are determined from Fourier spectra—at approximately 1 and 4 Hz, respectively. The mode at approximately 1 Hz is the dominant mode—similar to the one obtained from the aftershock records. The Fourier spectra of the horizontal orthogonal records of both stations TRD and TRE have identical peaks at this frequency, and the response amplitudes are different for the same direction at the two stations, thus indicating the effect of torsional rotation at this mode.

In addition to obtaining the dominant frequencies, the velocity records from the tests were integrated to obtain displacement histories which in turn were used to calculate equivalent critical viscous damping percentage corresponding to the 1 Hz mode. A sample of the displacement time-history used in calculating the damping is seen in Figure 12. Damping calculated using the logarithmic decrement relationship is approximately 1.5% (an average value between 1.4% and 1.6% from different records)—too low for translational mode of a reinforced concrete building. The usual range of damping for translationally vibrating reinforced concrete structures is 3–5% or higher. Thus the primary mode at 1 Hz (determined from horizontal motions) has small damping and therefore cannot be the sole contributor to the dissipation of energy. It is therefore possible that other actions such as rocking dissipates part of the incoming vibration energy.

VERTICAL AND HORIZONTAL MOTIONS— SYSTEM IDENTIFICATION

The vertical components of the records obtained at the building basement stations TRA, TRB and TRC were examined to investigate the possibility of rocking and the damping associated with it. The vertical components of the records from TRA, TRB and TRC were synchronized by two methods—one, by accepting that the first peaks occurred at the same instant and, two, by maximizing the cross correlation (Safak and Celebi, 1987). Each method showed that there were substantial differences and reversals in the displacements (integrated from velocity records) between any two of the three stations. Furthermore, the damping calculated for the rock-

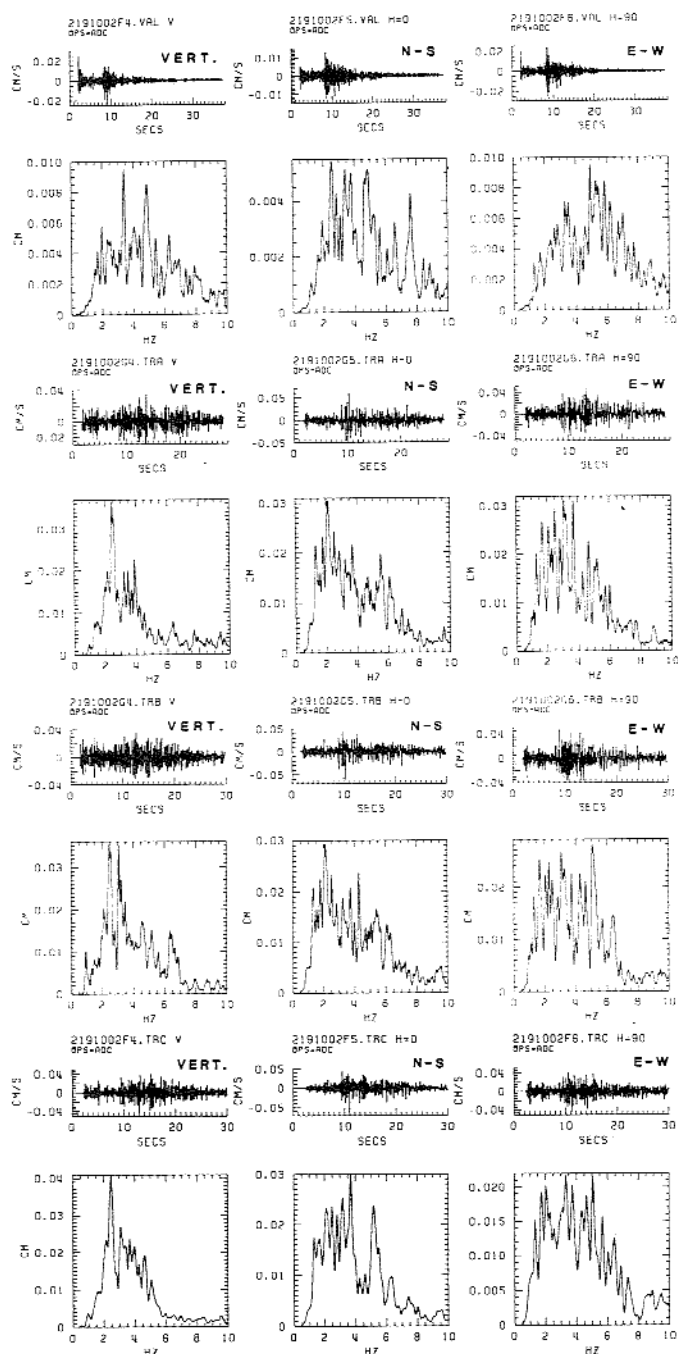


Figure 7. Typical set of velocity seismograms and corresponding Fourier spectra of event on August 7, 1985 (Julian 219) at 10:02 GMT for the vertical and horizontal components (N-S and E-W), respectively, of stations VAL, TRA, TRB, and TRC.

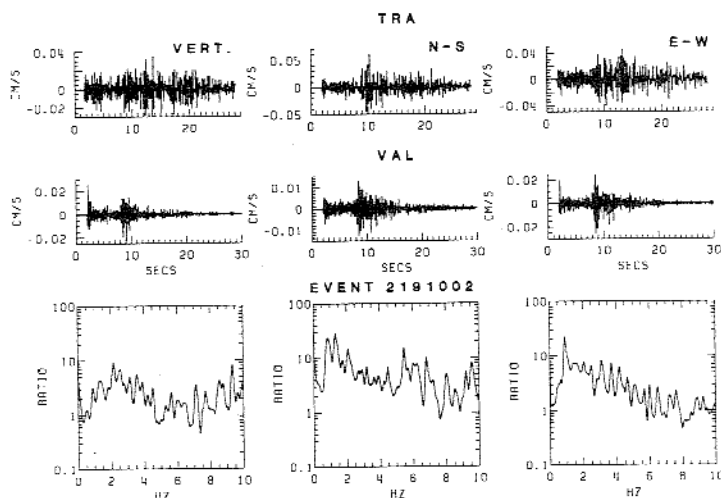


Figure 8.

Velocity seismograms and corresponding spectral ratios—event 2191002 corresponding to Julian 219 (August 7, 1985) at 10:02 GMT—for the vertical and horizontal components, respectively, of station TRA with respect to reference station VAL.

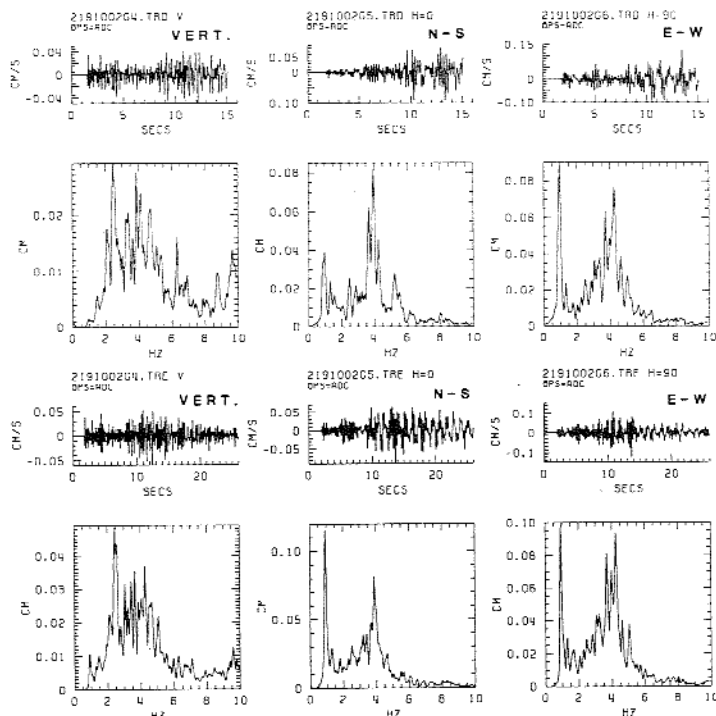


Figure 9.

Velocity seismograms and corresponding Fourier spectra of event on August 7, 1985 (Julian 219) at 10:02 GMT for the vertical and horizontal components (N-S and E-W), respectively, of top floor stations TRD, and TRE.

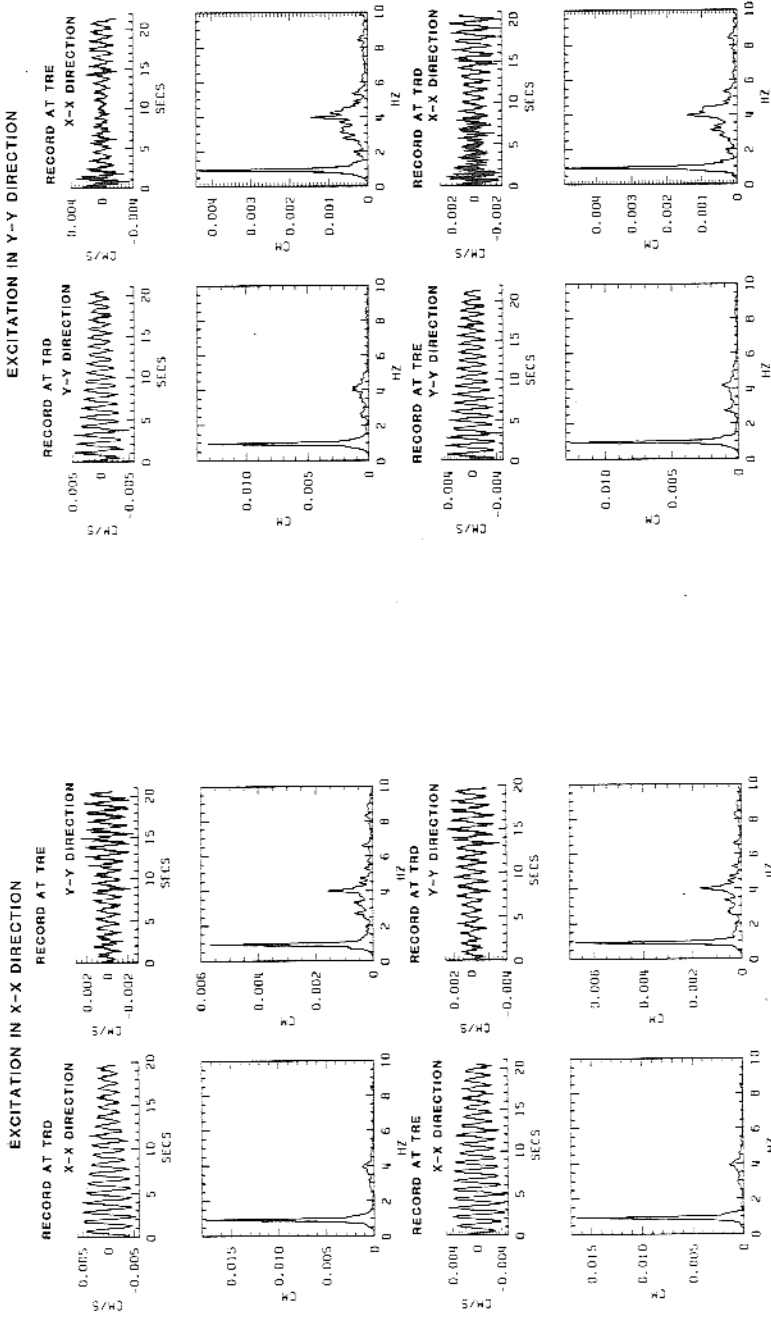


Figure 10. Velocity records and Fourier spectra of records from free vibration test (excitation in "XX" direction at the top floor). Records obtained both at TRD and TRE parallel to both "XX" and "YY" axes.

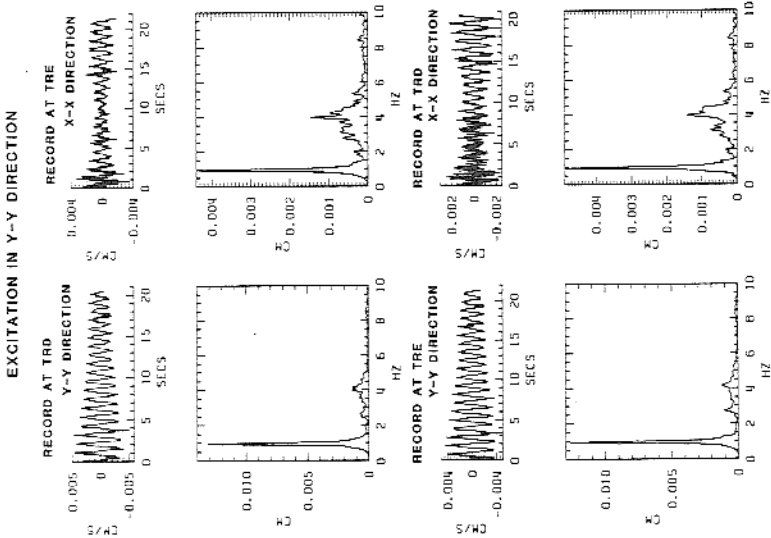


Figure 11. Velocity records and Fourier spectra of records from free vibration test (excitation in "YY" direction at the top floor). Records obtained both at TRD and TRE parallel to both "XX" and "YY" axes.

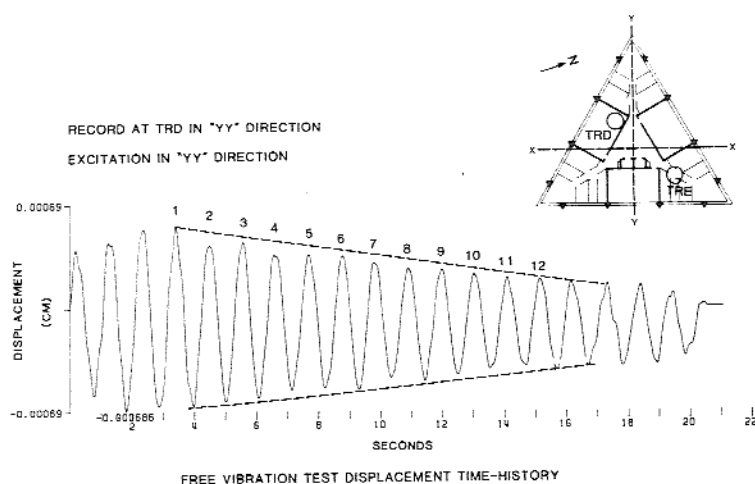


Figure 12.

Sample horizontal displacement time-history (in "YY" direction) derived from velocity records of the test conducted in "YY" direction. Average damping of 1.5% is calculated by using eight or nine successive peaks of similar plots..

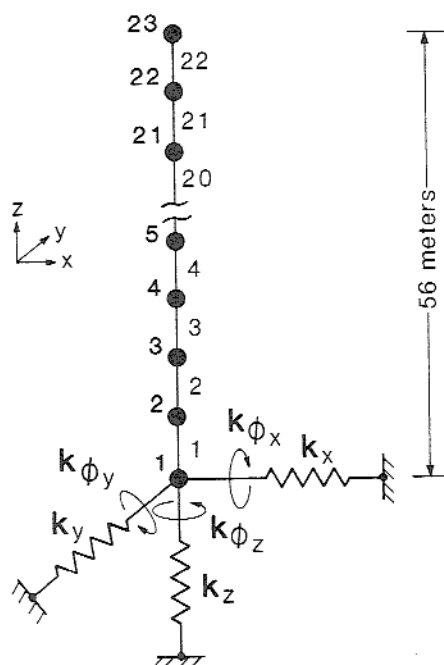


Figure 13.

Qualitative finite-element stick model incorporating frequency-independent soil-springs.

ing actions using the procedure described by system identification technique, based on the recursive prediction error method (Safak, 1987), is in the order of 14%. This same procedure was also used to calculate the frequencies and damping from the horizontal components of the aftershock response records of stations at the top floor (TRD and TRE). The frequencies were determined to be approximately 1 Hz and damping to be 1.4% -very close to the 1.5% calculated by the logarithmic decrement approach. However, damping calculated from the E-W component of records from the basement and the top floor showed a variation of 1.0-5.0%.

MATHEMATICAL MODEL

A finite element stick model (Figure 13) for the computer program SAP was developed utilizing only beam elements for the superstructure and boundary spring elements to account for soil-structure interaction. Material properties used for both the beam elements and the calculation of soil spring constants are summarized in Table 1. No material testing was performed. The properties for concrete were selected from standard references. The properties for sand were derived from an assumed shear wave velocity of 150 m/sec using studies made in the San Francisco Bay region (Fumal, 1978).

TABLE 1
Material Properties of Model

E (Concrete-Modulus of Elasticity, t/m^2)	2.0×10^6
ν (Concrete-Poisson's Ratio)	0.15
ρ (Concrete-Weight Density, t/m^3)	2.0
v (Sand-Shear Wave Velocity, m/sec)	150.0
ν (Sand-Poissons Ratio)	0.5
ρ (Sand-Weight Density, t/m^3)	1.8

The geometric properties of the stick model beam elements were hand calculated from the blue prints of the structure and are summarized in Table 2 which accounts for the changes in the cross section.

TABLE 2
Geometric Properties of Beam Elements

Description	Type	A (m^2)	I _{major} (m^4)	I _{minor} (m^4)
Beam Element 1	1	47.3	2430.0	2430.0
Beam Elements 2-3	2	31.2	884.0	884.0
Beam Elements 4-6	3	27.0	840.0	840.0
Beam Elements 7-11	4	25.0	800.0	800.0
Beam Elements 12-16	5	23.0	760.0	760.0
Beam Elements 17-22	4	20.0	700.0	700.0

The soil-springs were evaluated by using the recommended frequency independent soil spring constants (Clough and Penzien, 1975). The springs used in the analyses are summarized in Table 3. These springs are connected to node 1 corresponding to the basement level (Figure 13). Associated with these springs, the corresponding virtual soil masses were calculated and added to the structural masses of node 1. No particular effort was made to represent soil-springs at the ground level.

TABLE 3
Description of Soil Springs

Spring	Spring Constant
K_x -linear (t/m)*	0.29×10^6
K_y -linear (t/m)	0.29×10^6
K_z -linear (t/m)	0.44×10^6
K_{ϕ_x} -rotational (tm/rad)**	0.492×10^8
K_{ϕ_y} -(rotational) (tm/rad)	0.492×10^8
K_{ϕ_z} -(rotational) (tm/rad)	0.489×10^8

* t/m (tons/meter)

**tm/rad (tons.meter/radian)

MODAL ANALYSES

Several parametric modal analyses were performed to identify the impact of soil-structure interaction. Some of the frequencies derived from these analyses for the first 6 modes are summarized in Table 4. When only linear springs are considered the calculated frequencies do not compare well with the measured frequencies (model b). The first mode frequency of the structure determined from the mathematical model is close to that determined from recorded small earthquake response recordings of the top floor of the building (Figure 9) and from vibration tests (Figure 10) only when rocking rotational springs are considered (models c, d and e). For example, from the model d, we obtain the first two translational modes to be at 0.88 Hz. and 1.12 Hz., the vertical mode at 2.46 Hz., two other translational modes at 3.86 and 4.93 Hz., and finally the torsional mode at 5.11 Hz.

TABLE 4
Frequencies from Modal Analyses (Hz)

Mode Number	Model				
	(a)	(b)	(c)	(d)	(e)
1	1.83	1.57	0.92	0.88	0.88
2	1.83	1.57	0.92	1.12	0.88
3	10.31	3.90	6.57	2.46	2.46
4	10.31	3.90	6.57	3.86	3.86
5	14.08	7.64	7.64	4.93	3.86
6	27.83	13.26	16.09	5.11	5.11

(a) Fixed Base Model (No Soil Springs)

(b) Only Linear Springs (K_x, K_y, K_z)

(c) Only Rotational Springs ($K_{\phi_x}, K_{\phi_y}, K_{\phi_z}$)

(d) All Springs ($K_x^1 = 2K_{x_1}, K_y, K_z, K_{\phi_x}, K_{\phi_y}^1 = 2K_{\phi_y}, K_{\phi_z}$)

(e) All Springs ($K_x, K_y, K_z, K_{\phi_x}, K_{\phi_y}, K_{\phi_z}$)

TIME-HISTORY ANALYSES

Using one of the three-component records obtained at basement stations (TRA, TRB and TRC) as input to the mathematical model developed for the computer program SAP, time-history analyses were performed to compare the calculated responses at the top floor (node 23 in Figure 13) with those from recorded responses. The input data originally recorded at the N-S and E-W orientations were not rotated in the directions of the principal axes of the building. The analyses were performed only for two cases: Model Case A (using model d of Table 4) and Model Case B (using model e of Table 4). Only 5% structural damping was used for these analyses. For soil, no damping was provided because the computer program SAP does not have that particular capability. The displacement responses and corresponding Fourier spectra at the top floor from recorded motions (Figures 14 a and b) are compared with those at node 23 (top floor) from mathematical Model Case A and Model Case B (Figures 14 c and d and Figures 14 e and f, respectively). Considering the variety of assumptions made in modelling the structure (stiffnesses of the stick model and associated soil springs), and use of input data not rotated in the directions of the principal axes of the building, these figures show only reasonable agreement of displacements and dominant frequencies. The result to be emphasized here is that only the mathematical models incorporating soil-structure interaction and particularly rocking provide reasonably good agreement with the recorded responses.

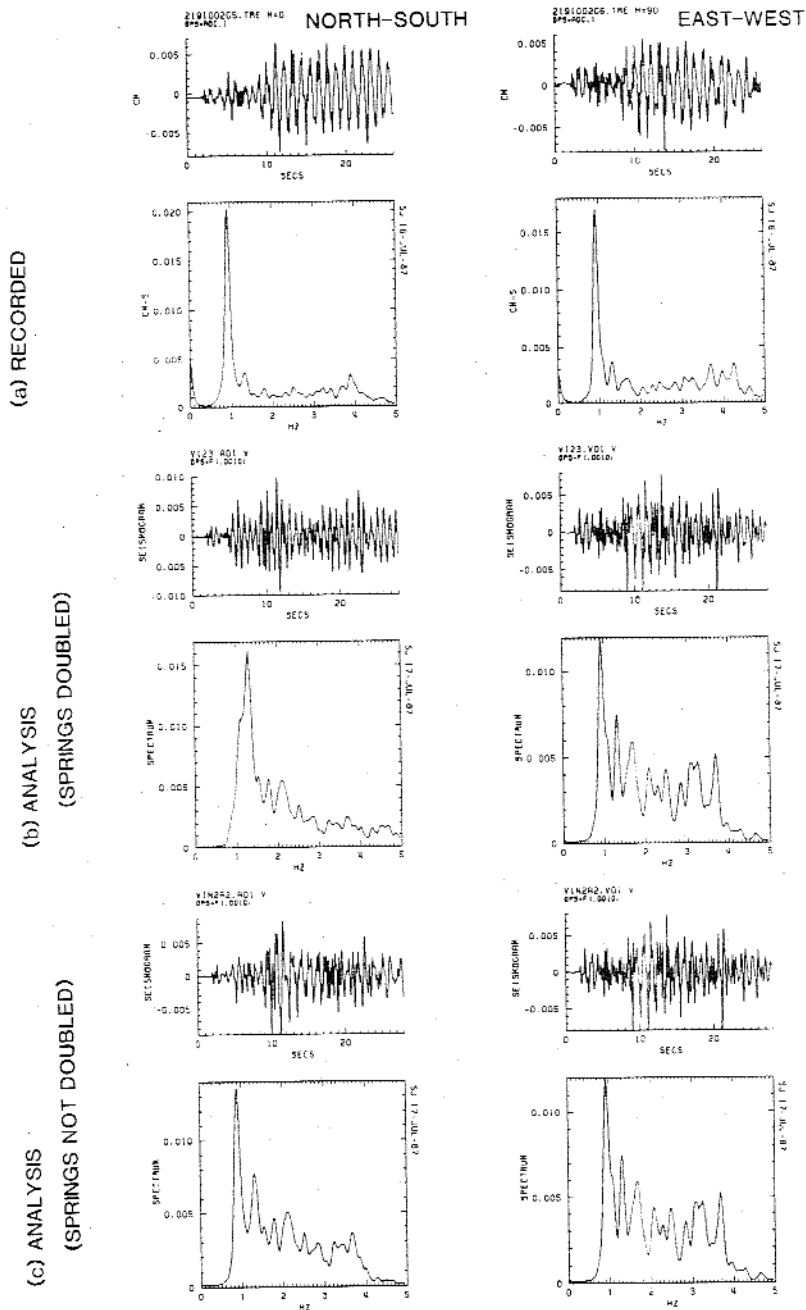


Figure 14.

Comparison of recorded displacements (integrated from velocity records) and Fourier spectra at the top floor.

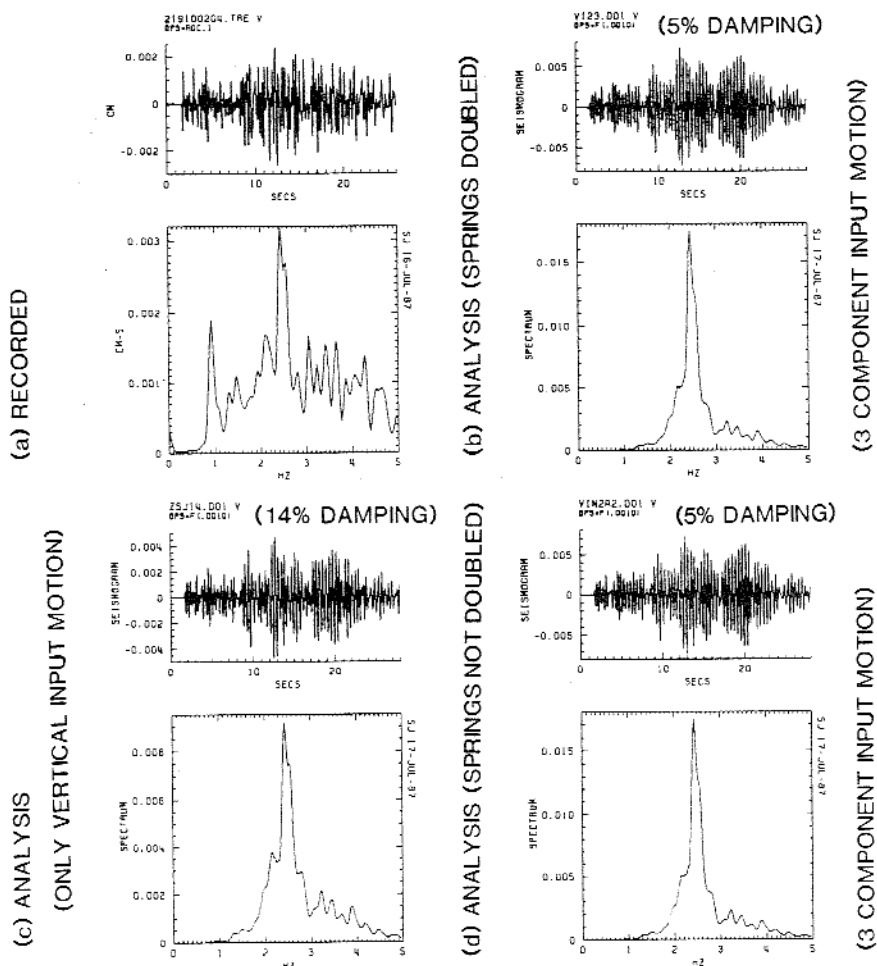


Figure 15.

Comparison of vertical displacements and corresponding Fourier spectra: a) recorded, b) from analysis with 5.0% damping and three component input, c) from analysis with 14.0% damping and only vertical input, and d) from analysis with 5.0% damping and three component input.

DISCUSSION AND CONCLUSIONS

The free vibration test results show that the structure has a dominant mode at approximately 1 Hz and damping of approximately 1.5%. This mode essentially is a translational mode coupled with torsion. Fourier spectra from these records also show that the building has another significant mode at approximately 4 Hz. Both of these frequencies are confirmed from Fourier spectra plots determined from aftershock records at the top floor of the building.

Furthermore, the vertical motions caused rocking and significant damping. The vertical motions experienced by the structure are significantly amplified at approximately 2.25-2.5 Hz. On the other hand, there is substantial damping of approximately 14% associated with the vertical actions. It is possible that the structure dissipates the incoming energy through soil-structure interaction—this may be the reason for its good performance during the $M_s = 7.8$, 3 March 1985 earthquake.

Mathematical models of the building including soil-structure interaction prove that rocking plays an important role in the vibrational behavior and associated characteristics of the building. Reasonable agreement has been attained in obtaining frequencies comparable to those determined from recorded motions by using the stick model with frequency-independent soil springs. To evaluate the effect of torsion, it is essential to develop mathematical models that are more detailed than the current stick model.

It is important to note that better results would have been derived with full synchronization and additional instrumentation. However, even with this limited data, we are able to identify for this unique structure a mode at 1 Hz that cannot otherwise be estimated with rule of thumb formulas, low structural damping and rocking soil-structure interaction that is prevalent in its behavior.

ACKNOWLEDGEMENTS

The field work portion of the study presented herein was carried out under the supervision of the second author during July–August of 1985, as an addendum to a project related to site response studies of the 3 March 1985 earthquake. Subsequent analyses that constitutes the scope of this paper was supervised by the second author, also. Much of this work has been compiled in USGS Open-File Report 86–90. In preparation of the plots a program developed by C. Mueller was used. R. Eis and L. Hollis drafted the majority of the figures. C. Sullivan typed the final manuscript.

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