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Publication Date

1990-09-01

REPORT NO.
UCB/SEMM-90/17

STRUCTURAL ENGINEERING
MECHANICS AND MATERIALS

SEISMIC RESPONSE OF THREE INSTRUMENTED BUILDINGS

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SEPTEMBER 1990

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Abstract

Earthquake response records obtained in three buildings located in San Jose, California are examined and interpreted in this report. The basic behavioral characteristics of these buildings are identified along with various engineering design parameters, such as period, damping, and mode shapes. The buildings all have about same number of stories, but employ different types of structural systems. Thus, observations related to the seismic response characteristics of different types of structural systems can be extracted from this data. Observed behavioral characteristics include torsion, in-plane floor slab deformation, shear wall rocking, modal coupling and soil-structure interaction effects.

Acknowledgments

Much appreciation is given to the Strong Motion Instrumentation Program staff, especially for reprocessing the records. The authors are also grateful to the assistance of the Strong Motion Instrumentation Program Building Sub-committee and to the Engineers of Record for the buildings studied. The financial support of the Department of Conservation is greatly appreciated. The findings of this study are, however, those of the authors alone.

Contents

CHAPTER 1

INTRODUCTION

Instrumentation of structures to obtain records of actual seismic response is a essential aspect of improving our understanding of the nature of seismic behavior. Ideally, structural response records would be used as part of an integrated investigation in which interpreted records are used along with analytical and experimental results to assess and improve current engineering practices. However, much valuable information can be directly derived from the response records themselves.

During the past decade the Strong Motion Instrumentation Program (SMIP) of the Division of Mines and Geology of the California Department of Conservation has installed strong motion instruments in over 100 buildings. In general, these instruments have been strategically distributed through a structure in order to obtain information on its dynamic characteristics, distribution of lateral forces, lateral displacements and drifts, diaphragm rigidities and soil-structure interaction. During the short time these instruments have been installed in structures, several recordings of significance have been obtained and processed.

In this study, the responses of three buildings subjected to the Morgan Hill earthquake of April 24, 1984 ($M_l = 6.2$) and the Mt. Lewis earthquake of March 31, 1986 ($M_l = 5.8$) are evaluated based on measured accelerograph records. These records were obtained and processed by SMIP. Peak ground motions ranged from 4 to 6% g. The buildings are located near one another in San Jose, California, between 19 and 23 km. (12 to 14 miles) from the epicenters. Each building employed a different type of structural system: reinforced concrete bearing walls, reinforced concrete frames and structural steel frames, Table 1 and Fig. 1. Additional information on the buildings and the records can be obtained in Refs. 1,7,9 and 10.

No structural analyses were performed as part of this investigation and none of the buildings studied suffered structural damages. None-the-less, important information regarding the basic behavioral characteristic of each structure and the different structural systems are developed from the records using simple non-parametric system identification techniques. These derived characteristics are compared with values suggested by codes and other guidelines.

CHAPTER 2

BUILDING 1

2.1 Introduction

This ten story residential building (SMIP Station No. 57356) was designed and constructed between 1971 and 1972, Fig. 1. The vertical load carrying system consists of one-way post-tensioned, lightweight concrete, flat slab on reinforced concrete bearing walls. The lateral load resisting system consists of reinforced concrete shear walls. In the transverse (EW) direction these are spaced at regular intervals, while in the longitudinal (NS) direction they are place along the center of the building. One of the major walls in the NS direction terminates at the sixth floor and additional irregularities occur at the ground level. A pile foundation provides support for this building.

Thirteen analog instruments were installed in the building. These were located to estimate such response features as modal characteristics, drift and torsional motions, rocking of the wall foundations, and in-plane diaphragm deformations. The records used to study this building (as well as all the other buildings considered herein) were reprocessed by SMIP to obtain a signal to noise ratio of approximately 10 to 1. This gives an reliability of about 1.5 cm/sec/sec (0.0015g) and 0.1 cm (0.04 inches) for absolute acceleration and displacements, respectively (10), and about twice these amounts for relative values. Records are identified herein by their recording orientation, followed by the floor level and, if appropriate, the side of the building where the instrument was installed. Thus, EW/r/S is the east-west record obtained on the south side of the roof.

2.2 Acceleration response

The maximum recorded ground acceleration (Table 1) was 0.06 g for the Morgan Hill earthquake and the maximum corresponding structural acceleration at the roof was 0.22 g. For the Mt. Lewis event these accelerations were 0.03 and 0.12 g, respectively.

Amplification ratios obtained by dividing the peak acceleration at a location by the corresponding acceleration at the ground were computed for various locations in the structure. They were found to be similar for both earthquakes studied, but from 22% to 100% larger in the NS direction than in the EW direction. Some of the processed records obtained for the Morgan Hill earthquake are shown in Fig. 2. This figure shows that the ground motion is characterized by relatively high frequency motions during the first 17 seconds and by much longer period motions during the latter portions of the record. Acceleration records from the upper parts of the building exhibit less frequency variations with time, and indicate that the structure is more flexible and suffers higher accelerations in the NS direction.

Fourier amplitude acceleration spectra (FAAS) of vertical records obtained on the foundations for an EW oriented wall have relatively high amplitudes around the predominant period of the observed EW translational motion, indicating that the walls rotate (rock) at the foundation. Vertical displacement records derived from the recorded accelerograms could not be used to evaluate this rocking because peak values were below the displacement confidence threshold. Consequently, FAAS of the vertical acceleration records at the base of the wall were computed near the predominant period of the system and scaled considering the wall to rock as a rigid body, to obtain an equivalent relative roof acceleration (also see Ref. 2). The effect of the vertical base accelerations on relative horizontal roof accelerations could then be assessed. This indicates that the rocking at the base of the walls contributes significantly to the first mode response at the roof in the transverse direction. This contribution is around 50% for the Morgan Hill earthquake and between 45 and 50 % for the Mt. Lewis earthquake. Moving window FAAS indicate that this percentage did not change considerably during the first 20 seconds of the records

2.3 Drifts

Drifts obtained by subtracting horizontal displacement records from corresponding ground level displacements are quite small, Table 1. In general, drift responses in the EW direction are of higher frequency and smaller amplitude than those for the NS direction. Average drifts between the roof and ground in the EW direction never exceeded 0.03% of the building height (less than 6% of the working stress level value permitted by the 1985 Uniform Building Code (11) and 0.10% for the NS direction (more than twice as much, but still less than 20% of the code permitted value).

Some deflected shapes for the building, including the rocking effect, are shown in Fig. 3 (seconds 26.88 to 28.52 [NS] and 18.00 to 21.50 [EW]). While these appear to indicate the presence of significant higher mode effects, especially in the EW direction, the low level of drift in comparison with the drift confidence level of the response, indicated as ϵ in the figures, makes such visual interpretation uncertain. A strong effect on displacements due to the discontinuity of the shear wall at the sixth level in the NS direction was not observed. However, deflected shapes in Fig. 3 are drawn linearly, interpolating between the three recording levels; thus, it may be difficult in any event to detect such effects from these plots.

The total and relative motions of the roof in two directions are plotted in Fig. 4. As seen in this figure in some cycles the maximum relative displacements in each direction occur at nearly the same time. The structure displaces about 2/3 less in the stiffer EW direction at times of peak bi-directional motion. The total displacement of the structure is about twice the relative displacement; thus, ground movement contributes about 50% to the total displacement.

Inspection of the records indicates that there was little torsion or bowing of the floor slab. Measured displacements values were near or below the confidence level for these derived records.

2.4 Periods and mode shapes

Due to the low level of response, only the first mode could be reliably identified for each principal direction of the building. Based on visual observation of the records as well as inspection of Fourier amplitude spectra and transfer functions, the periods were estimated as summarized in Table 2. These periods include the effects of foundation flexibility. From observation of running window FAAS, the EW direction period of the entire system (including soil-structure effects) is close to 0.5 seconds; the structure itself seems however to have a period lower than 0.4 seconds. No significant differences in period values were detected for the two earthquakes considered.

Uniform Building Code estimates of period for the building are also shown in Table 2. These values indicate that the 1985 UBC (11) incorrectly predicts the NS direction as being stiffer. The equations in the 1988 UBC (12) result in a value lying between the measured values for the two directions when the average period coefficient C_t is taken as 0.02 and under-estimate both periods by about 30% when the coefficient is computed according to 1988 UBC Eq. 12-4.

Based on the relative amplitudes of the records, the first mode shape for both directions is estimated to be 1.0, 0.4 and 0.0 for the roof, sixth and ground level, respectively. This mode shape includes the flexibility of the foundation.

2.5 Damping

Equivalent viscous damping coefficients were estimated from the records. Values obtained for the first mode are about 5% for the NS direction and between 11 and 14 % for the EW direction. The values for the EW direction are considered only approximate, due to the low level of response in this direction, the presence of signal noise and the probable influence of soil-structure interaction. The identification techniques used (half-power bandwidth method, peak of the total acceleration transfer function, logarithmic decrement of the free signal decay and equivalent spectral approach) were unable to provide definitive and consistent results.

2.6 Seismic demands

Story shears and overturning moments were estimated using accelerations linearly interpolated between values obtained at floors with recording stations. The inertia forces at each floor were then evaluated, and story shears and overturning moments computed disregarding any damping forces. During the Morgan Hill earthquake, Building 1 developed a base shear coefficient of 0.096 in the EW direction and 0.104 in the NS direction. Corresponding values for the Mt. Lewis earthquake were 0.048 and 0.045, respectively. The working stress base shear coefficients used in the design of the building were 0.08 and 0.10 for the EW and NS directions, respectively. Thus, the Morgan Hill earthquake corresponded roughly to a working stress level event for the design code employed. The 1988 UBC, however, requires design base shears coefficient nearly two times the original design values (0.18). Thus, for a similar building designed according to modern codes, this earthquake would have corresponded to a very minor event.

The shear capacity of the building can be estimated using the 1988 UBC. The estimated capacity corresponds to a base shear coefficient in the EW direction of 1.04 and 0.19 for the NS direction (evaluated at the second floor due to the complex framing at the ground level). The capacity of the walls in the NS direction is relatively small in comparison with the value for the EW direction, but appears consistent with code requirements.

CHAPTER 3

BUILDING 2

3.1 Introduction

This commercial/office building (SMIP Station No. 57355) is ten stories tall with one basement level. It was designed in 1964 and constructed in 1967. According to ATC-2 (1) this building could be the first reinforced concrete building explicitly designed for ductile behavior. The vertical load carrying system consists of light weight reinforced concrete joist floors supported on normal weight concrete frames. The lateral force resisting system consists of reinforced concrete shear walls at the ends of the building in the transverse (EW) direction and moment frames in the longitudinal (NS) direction. The building is supported on a 1.5 m. (5 ft.) thick mat foundation. The building is instrumented similar to Building 1 (Fig 1).

3.2 Acceleration response

As with Building 1, the maximum ground acceleration during the Morgan Hill earthquake was 0.06 g and the maximum structural acceleration was 0.22 g (Table 1). For the Mt. Lewis event these accelerations were 0.04 and 0.08 g, respectively. The EW direction develops slightly greater accelerations and larger amplification ratios than in the more flexible NS direction during the Morgan Hill earthquake and vice-versa for the Mt. Lewis earthquake. This behavior might be related to the different directions of incidence of the earthquakes. The maximum amplification ratio for the dual system (shear walls and moment frames) used in the EW direction was 3.6 and for the frame system used in the NS direction 3.1. The duration of intense motion in the NS direction was substantially longer than that in the EW direction (e.g., see Fig. 5).

Also, it is important to note that the accelerations at the center of the fifth floor diaphragm were about 20% larger than those at the ends for the Morgan Hill earthquake

(Fig. 5) and 100% larger for the Mt. Lewis earthquake, indicating that the diaphragm undergoes important in-plane response. In fact, the maximum structural acceleration and amplification ratios recorded anywhere in the building occurred at the center of the fifth floor diaphragm for the Mt. Lewis earthquake. Slab deformations contribute importantly to the response at the center of the diaphragm during the first 24 seconds of the earthquakes, though the most significant contribution occurred during a short burst, near second 5. Figure 6 presents Fourier amplitude acceleration spectra for the fifth floor EW acceleration records. Slab contributions to response are visible between 4.0 and 5.0 Hertz. The torsional frequency of the building can also be clearly observed in this figure, at 2.5 Hertz; this frequency does not contribute as strongly to the response at the center of the slab.

Analyses of appropriately scaled vertical acceleration records at the base of the south shear wall indicate that, in the EW direction, the contribution of rigid body rocking of the walls about their base to the first mode relative roof acceleration was between 35 and 45 % for the Morgan Hill and Mt. Lewis earthquake, Fig. 7.

3.3 Drifts

EW drifts are generally characterized by low level, but nearly constant amplitude cycles. However, some cycles between seconds 17 and 20 have more than double the amplitude of the other portions of the record, Fig. 8. Drift indices in the EW direction did not exceed 0.07%, approximately fourteen percent of the value permitted by the 1985 UBC code at working stress levels. In the more flexible NS direction, the response was larger, especially in the latter part of the record. The NS deformations correspond to an average inter story drift index of around 0.1%.

Figure 9 presents the deflected shape of the structure during its maximum response (seconds 14.4 to 21.5). The nearly linear deflected shape in the EW direction observed in this figure, is an indicator of the significant contribution of rigid body rocking of the structure on its foundation to the overall response. The significant effect of the embedment of the building on the deflected shape in the NS direction can also be observed in Fig. 9.

The structure displaced more in the NS direction, but there are several major cycles where it developed nearly its maximum displacement in both directions simultaneously (Fig. 4). The ratio of the maximum relative displacement in the EW and NS directions at the time of maximum response in the EW direction is nearly one, indicating a strong bi-directional effects. Again, more than 50% of the total displacement recorded in the building is contributed by the ground motion. No significant torsion was detected from displacement records for this regular and symmetric building.

3.4 Periods and mode shapes

The periods estimated for the building are summarized in Table 3. These periods include the effect of the foundation flexibility. It is interesting to note that the periods for the EW direction roughly obey the "rule of thumb" that the period of a shear wall building decreases more slowly than that of a cantilever beam (i.e.; the higher mode periods vary approximately as 1/6, 1/18, etc. of the fundamental period), while the periods in the NS direction obey the relationships for a frame (shear) building (i.e., 1/3, 1/5, etc. of the fundamental period). Periods estimated using the 1985 and 1988 UBC are somewhat higher than measured. The structure displaced more in the NS direction, but C_t results in a substantial under estimation of the period measured for the overall soil-structure system. Previously reported analytical results also underestimate these periods (1).

In the EW direction, the first and second mode shapes have the following relative amplitudes at the roof, fifth and basement levels: (1.0, 0.45, 0.0) and (1.0, -1.0, 0.0), respectively. In the NS direction the first, second and third mode shapes have the following ratios for the roof, fifth, second and basement levels: (1.0, 0.5, 0.1, 0.0), (1.0, -1.0, -0.36, 0.0) and (1.0, 0.6, 0.6, 0.0), respectively. These mode shapes include the flexibility of the foundation.

3.5 Damping

Viscous damping was estimated to be between 3 and 5% in the NS direction and, very approximately, between 5 and 10% in the stiffer EW direction.

3.6 Seismic demands

The building developed in the EW direction an estimated base shear coefficient of 0.14 during the Morgan Hill earthquake and 0.05 during the Mt. Lewis earthquake. In the NS direction, it developed base shear coefficients of 0.11 and 0.04, for the two earthquakes, respectively. The values achieved for the Morgan Hill earthquake are 83% larger than the non-factored values used in the original design in the EW direction and 25% larger in the NS direction. The 1988 UBC requires design forces 18% larger than used in the original design for the EW direction, and in the NS direction the base shear coefficient could be lowered by 32%, if a ductile frame were used.

The shear capacity of the two shear walls in the EW direction is estimated to be 4700 kips, 34% more than the demanded base shear and 153% more than required in the original design. No significant cracks were noted in the walls despite the relatively high intensity of the seismic response. Force-deformation relations computed for the building indicate the possibility of minor inelastic behavior, Fig. 10. This is believed to be associated mainly with soil-foundation conditions.

CHAPTER 4

BUILDING 3

4.1 Introduction

This building (SMIP Station No. 57357) is a thirteen story government office building located approximately 2 km. (1.3 miles) north of the other two buildings. It was designed in 1972 and construction was completed in 1976. The vertical load carrying system consists of a concrete slab on metal deck, supported by steel frames. The lateral force resisting system consists of a strong perimeter moment-resistant steel frame with tapered girders and four interior moment frames in each orthogonal direction. A mat foundation is used to support the building.

Twenty two analog instruments were installed and connected to two centralized recording units. Four accelerometers were located horizontally at four floors and three vertical and three horizontal accelerometers were located at the ground level (Fig. 1). A free field instrument had been installed, but the owner requested that it be removed shortly prior to the Morgan Hill event.

4.2 Acceleration response

The recorded peak ground acceleration for this building was lower than the other buildings, but the recorded structural motions were in general higher. The maximum ground acceleration observed (Table 1) was 0.04 g for both events, and the maximum structural acceleration, obtained at the roof during the Mt. Lewis earthquake, was 0.32 g. For the Morgan Hill earthquake the maximum acceleration was 0.17 g. Thus, the maximum amplification ratio for the Morgan Hill earthquake was nearly 5 and that for the Mt. Lewis event was greater than 7.

In general, structural response for both events is characterized by a relatively narrow banded periodic motion with strong amplitude modulation and an unusually long duration, more than 80 seconds. These features appear to be a consequence of closely spaced modal periods, lateral-torsional coupling, soil-structure interaction and low effective structural damping. As a result of this, maximum accelerations in the upper levels of the structure, especially in the NS direction, were developed for both earthquakes long after the maximum ground acceleration occurred (Fig. 11). FAAS and response spectra of the base records show an important contribution with predominant period close to the fundamental period of the structure, especially during the Mt. Lewis earthquake. It is believed that base records have been affected somewhat by the motion of the structure. Nevertheless, previous studies (3) indicate that predominant periods of the ground near the site could be close to those recorded in the structure.

Several analytical studies have indicated that buildings with small eccentricities and closely spaced translational and torsional uncoupled frequencies exhibit strong lateral torsional coupling (4,5,6,8). Analysis of simplified structures indicate that this lateral torsional coupling generally causes a decrease in base shear, base overturning moment and top floor lateral displacement at the "center of rigidity" in the direction of the applied earthquake, but an increase in the base torque relative to an associated uncoupled system (5). As shown by Newmark and Rosenblueth (8) buildings with similar uniform distributions of stiffness in their orthogonal principal directions, like Building 3, have closely spaced torsional and translational predominant frequencies and are thus susceptible to strong lateral torsional coupling.

4.3 Drifts

Maximum drift indices for the building are on the order of 0.40% and 0.72% for the Morgan Hill and the Mt. Lewis event, respectively. The 1985 UBC limits drifts under working stress conditions to 0.5% and the 1988 UBC uses a basic limit of 0.25 %, if an R_w factor of 12 is considered. Thus, the drifts experienced by the building were significantly larger than accepted by current design practices, for non-factored design loads. Damages occurred during the Morgan Hill event to nonsupported book shelves and to two members that braced a glass atrium at the the third floor.

Figures 4 and 12(a) show that the roof displacements are bi-directional, and that most of the total response is due to the structural deformations, and not the movement of the ground, as was the case with the other two buildings. Similar motions were obtained during the Morgan Hill earthquake.

Significant torsion was observed in the building during both earthquakes. The maximum total torsional rotation evaluated from the records was 0.003 radian (i.e. a relative displacement from one side of the building to the other of 12.32 cm. (4.85 inches)) during the Mt. Lewis earthquake. This torsional displacement contributed roughly 19% of the relative roof corner displacement, Fig. 12b. Figure 13 shows the twelfth floor slab total displacement motion for the Mt. Lewis earthquake. The torsional motions are evident, with the center of rotation lying at certain times far out side the building. This apparent "center of rotation" changes dramatically during the earthquake. The eccentricity that produces this torsion can be associated with irregular framing, an increase number of columns and nonstructural steel panels and walls on the west and south sides of the building and an irregular distribution of mass. The torsional motion of the building causes the response at the southwest corner of the building to be especially strong, with higher relative accelerations and displacement than in the other corners.

The building drifts and torsional motion can also be seen in Fig. 14 which presents the deflected shape of the building at various times during the Mt. Lewis earthquake. The pairs of lines in this figure represent the motion of the SW and SE corners in the NS direction and the motion of the SW and NW corners in the EW direction. A shear type deflected shape for the south side corners (north-south motion) is apparent and a strong torsional motion is observed during the strong response in the west side (east-west motion). At other times the motion in the EW direction exhibits shear type deformations or pure torsional response.

The motion of the building manifests the three dimensional interaction of more than three modes. This involves coupled translational and torsional motions. Interpretation of the response is complicated by the fact that the frequencies for several modes are similar leading to a beating or modal interference phenomenon. This phenomenon is clearly shown for the Mt. Lewis event in Fig. 12a where modulation of response amplitudes is strong. In the time domain, simple trigonometric series can be used to examine and simulate this behavior. Summing two trigonometric series will result in an equivalent natural period and a beating period. It can be shown that the sum of two trigonometric series have a beating period obtained by:

$$T_B = \frac{2T_1T_2}{T_1 - T_2}$$

and an equivalent natural period obtained by:

$$T_N = \frac{2T_1T_2}{T_1 + T_2}$$

where T_1 and T_2 are the periods of the trigonometric series and $T_1 > T_2$. This can be easily extended to three trigonometric signals. But it can also be used directly, as a further approximation, when two of the three interacting series have really close natural periods. This is the case for Building 3, which has similar structural characteristics in the orthogonal directions. Inspection of the records, especially for the Mt. Lewis earthquake, indicates two beating periods of about 100 and 16 seconds and an equivalent natural period of 2.2 seconds for the translational records and 1.85 seconds for the derived torsional displacements.

The use of the above formulae results in a system with periods of about 2.2, 2.1 and 1.7 seconds. As discuss latter, these same periods are also observed in the Fourier amplitude spectra of the records. A record simulated by this simple analogy is presented in Fig. 15. The striking similarity between this simple time series simulation and the recorded relative NS displacement drift, presented in Fig. 12.b, confirms the importance of modal coupling and beating on the response for this building.

The in-plane flexibility of the floor diaphragms was investigated by independently computing relative floor torsional displacement for the NS and EW directions. The difference between maximum values of these computed torsional motions provided an estimate of the in-plane floor flexibility equivalent to a shear strain of 0.0005 (2 cm.). However, the imprecise location of some of the instruments, noise effects, and the different time bases used for some of the recordings at the same level could affect this value.

4.4 Periods

The periods estimated for the building are presented in Table 4. These are substantially longer than estimated by either the 1985 or 1988 UBC. Nevertheless, the periods for the higher modes vary as one would expect for a shear structure.

4.5 Damping

Due to the interaction of the closely spaced modes, a clear identification of viscous damping was not possible by the approaches considered herein. A grossly approximate value was obtained by observing the free amplitude decay near the end of response records where the response was not significantly influenced by torsion (the amplitude modulation due to beating was still present, however) and also using the decay of the beating envelope. Estimated values are shown in Table 4.

4.6 Demand versus UBC requirements

The calculated base shear coefficient demanded by the Morgan Hill earthquake is 0.09 for both directions. For the Mt. Lewis earthquake these values are 0.16 and 0.07, for the NS and EW directions, respectively. The 1988 UBC would require a working stress design base shear coefficient of 0.043 in both directions, for a similar building having a moment resisting frame (R_w =12). Thus, the values demanded by the Morgan Hill earthquake are 2.1 times code recommended design forces. During the Mt. Lewis earthquake shear coefficients developed are 3.7 and 1.6 times the 1988 UBC code recommended values. Force-deformation plots are presented in Fig. 16. Because no significant yielding was observed in the structure, it can be assume that this building is significantly stronger than required by the 1988 UBC.

The response is nonetheless very severe considering the intensity for the excitation. The long duration of the response and the high amplitude of the motion is related to the long natural period of the structure (2.2 seconds), the three dimensional modes of the building constructively reinforcing one another during portions of the motion, the relative low damping, and the possible resonance on the building due to the dynamic characteristics of the site. Foundation rocking was found not to have an important influence on the response.

CHAPTER 5

CONCLUSIONS

The records of the three buildings studied herein have provided significant insight into their dynamic characteristics and the accuracy of various code assumptions.

1. Buildings 1 and 2 were substantially stiffer than Building 3 (all three building have similar total story masses). This increased stiffness contributed to a considerable reduction in the relative drifts, drift coefficients and amplification ratios observed in these buildings, Fig. 17.

2. The presence of shear walls in Buildings 1 and 2, in addition to their contribution to the translational stiffness, helps in reducing torsional response, and separates the periods of the translational and torsional modes of the structures, thereby diminishing the possibility of modal interaction.

3. A strong influence on relative response values from rocking of the building foundation was observed in Buildings 1 and 2. Rocking contributed 35 to 50% of relative roof acceleration values for the predominant structural mode in the direction of the shear walls in these buildings.

4. In-plane diaphragm flexibility was mainly observed in Buildings 2 and 3. Due to this flexibility in Building 2, maximum recorded center slab accelerations were nearly double those obtained at the ends of the diaphragms. For Building 3, the equivalent maximum slab deformation corresponded to a shear strain of 0.0005.

5. For Building 3 it is believed that the small irregularities in plan (mass and stiffness) create an eccentricity that together with the closeness of the modal frequencies produce the strong interaction observed between orthogonal translational and torsional motions. Also, Building 3 exhibits a rather large period for a thirteen story building

and, as reported by the building occupants, vibration due to small earthquakes produced unpleasant long duration motions of the structure. Damping as estimated for this building is believe to be low (less than 3%). The long duration of the response and the high amplitude of the motion recorded in the building is related with the long natural period of the structure (2.2 seconds), the three dimensional modes of the building constructively reinforcing one another during portions of the motion, the relative low damping, and the possible resonance of the building and site. Due to this special dynamic behavior, code type provisions based on equivalent static lateral loads and static eccentricities in plan will not be able to predict accurately the (linear) demands on the system. Building 3 is the only one studied herein that did not satisfy the drift coefficients limits established by the 1988 UBC. However, the base shear coefficient observed was substantially higher than that for the others buildings or required by the UBC. Torsional motion was strong and found to contribute up to 19% of the maximum relative displacement at the roof level.

6. Period calculation using code empirical equations have improved, but additional improvements are desirable. Building periods estimated using UBC 1988 Section 2312 Equation 12-4 generally were smaller than natural periods estimated from the records. For the base shear equation used in the code this under-estimation will result in equal or higher design shears; however, it may not give conservative design values, if a specific site spectra is used. Selection of the constant C_t employed in this equation according to the specific type of structural system used gave generally closer results than the more general value permitted.

7. Base shear coefficients given by the 1988 UBC code are 1.79, 0.55 and 0.27 times the maximum values demanded by the earthquake records studied for Building 1, 2 and 3, respectively. This indicates that these earthquakes could be considered to be lower than a service level earthquake for Building 1 and higher than this level for Building 2 and 3. Because strong nonlinear behavior was not observed from the records, it is believed that Buildings 2 and 3 are at least 1.82 and 3.72 stronger than required by the code at working stress level.

8. The response of the buildings is clearly bi-directional. Drifts near the maximum values occur nearly simultaneously in both directions for each of the buildings.

CHAPTER 6

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TABLES

TABLE 1 - BUILDING DATA AND GROSS RESPONSE VALUES

BUILDINGS	1	2	3
Structural System	RC Shear Walls	RC Shear Walls Moment Frames	Steel Moment Frames
No. Stories(*)	10/0	10/1	13/0
Height (m)	30	38	57
Pred. Period (sec.)	0.60-0.70	0.91-0.96	2.2
Max. Ground Accel. (g)	0.06	0.06	0.04
Max. Str. Accel. (g)	0.22	0.22	0.32
Max. Str. Disp. (cm)	2.06	3.25	33.19
Max. Str. Drift Coeff.	0.10	0.12	0.72
Max. Base Shear Coeff.	0.10	0.14	0.16
Max. Rel. Torsion (cm)	0.53	0.42	12.32
Max. Ampl. Ratio	4.06	3.59	7.05

(*) above/below ground

TABLE 2 - PERIODS (IN SECONDS) FOR BUILDING 1

Direction	Measured Values	1985 UBC	1988 UBC (*)	1988 UBC (**)	Damping (%)
EW	0.4-0.5	0.59	0.61	0.33	11-14 ***
NS	0.6-0.7	0.32	0.61	0.50	5

(*) $C_t = 0.02$.

(**) C_t computed using the effective area of the shear walls, according to 1988 UBC.

(***) Gross estimate.

TABLE 3 – PERIODS (IN SECONDS) FOR BUILDING 2

Direction	Mode	Measured Values	Quick Guess	1985 UBC	1988 UBC (*)	1988 UBC (**)	Anal. Model (1)	Damping (%)
EW	1	0.6-0.65	-	0.69	0.73	0.36	0.44	5-10
	2	0.2-0.25	0.10	–	–	–	0.12	–
NS	1	0.91-0.96	-	1.0	1.1	–	0.74	3-5
	2	0.25-0.28	0.31	–	–	–	0.24	–
	3	0.14-0.18	0.19	–	–	–	0.13	–
Torsion	1	0.33-0.40	–	–	–	–	–	–

(*) C_t= 0.02 for transverse direction and C_t = 0.03 for the longitudinal direction.

(**) C_t computed using the effective area of the shear walls, according to 1988 UBC.

TABLE 4 – PERIODS (IN SECONDS) FOR BUILDING 3

Direction	Mode	Measured Values	Quick Guess	1985 UBC	1988 UBC	Damping (%)*
EW	1	2.15-2.20	-	1.3	1.77	< 2-3
NS	2	2.05-2.10	-	1.3	1.77	< 2-4
Torsion	3	1.70	-	-	-	-
EW	4	0.65-0.75	0.72	-	-	-
NS	5	0.60-0.70	0.69	-	-	-

(*) Gross estimate

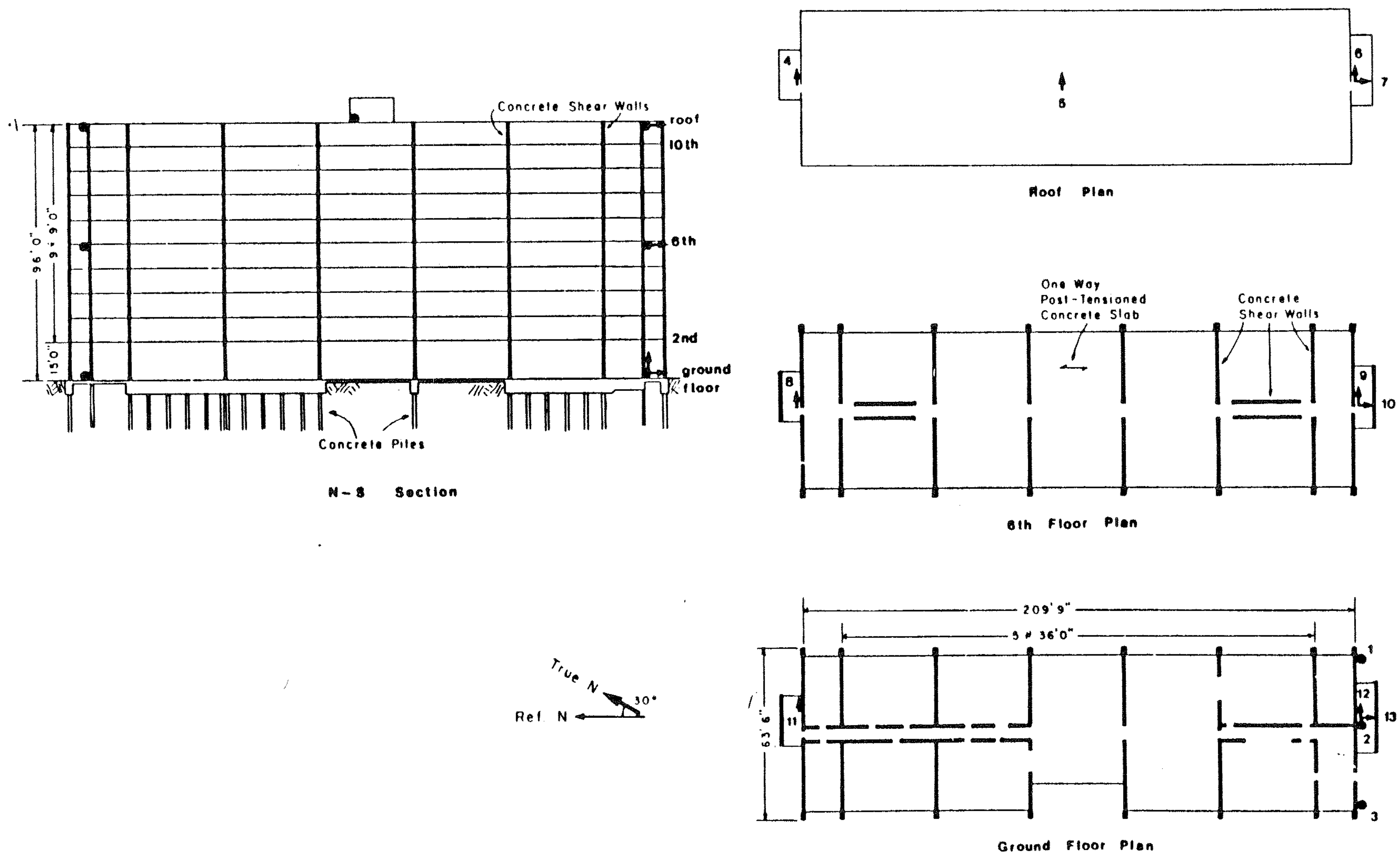

Figure 1a - Building 1. Plan and sensor layout (10).

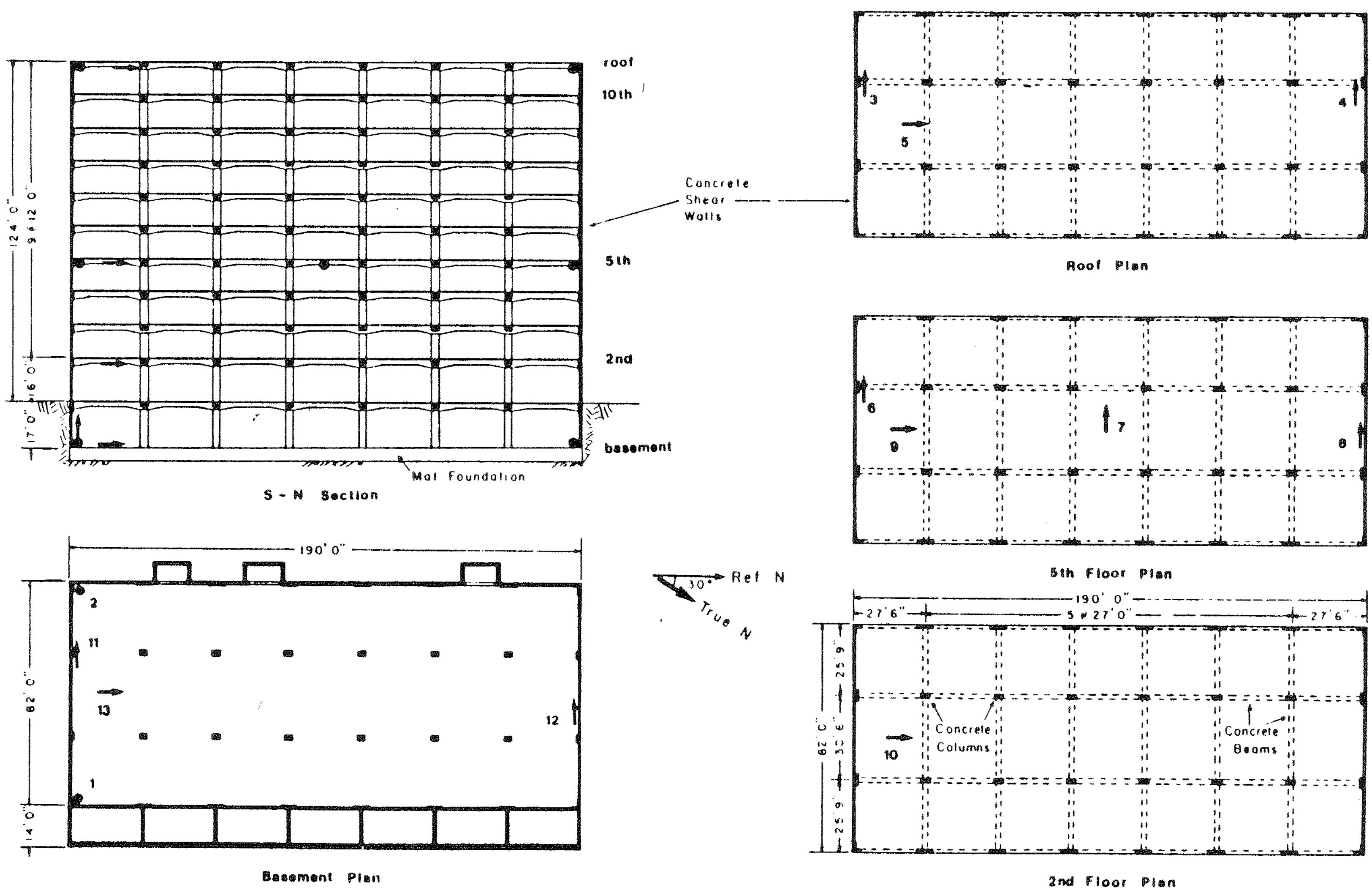

Figure 1b - Building 2. Plan and sensor layout (10).

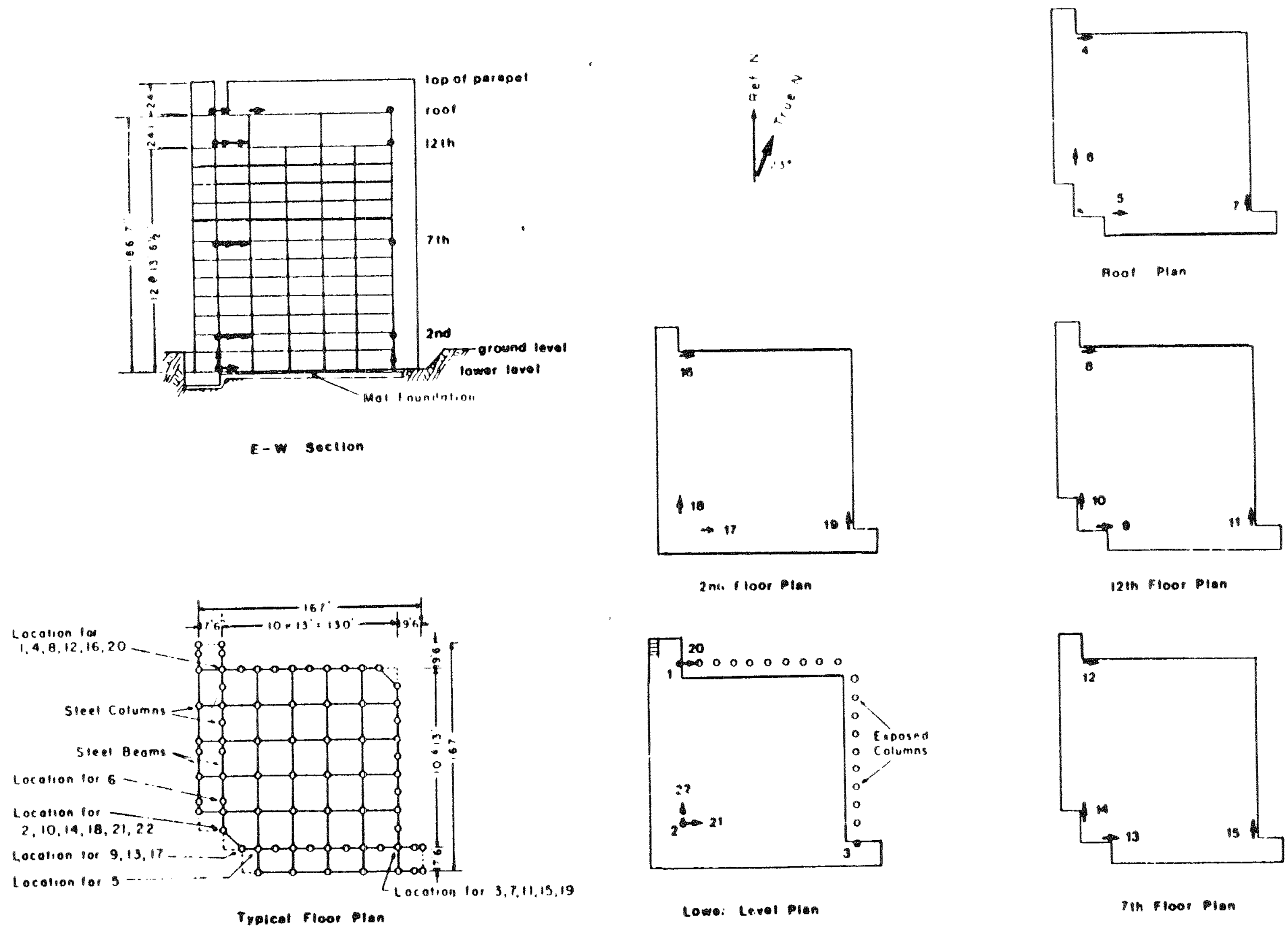

Figure 1c - Building 3. Plan and sensor layout (10).

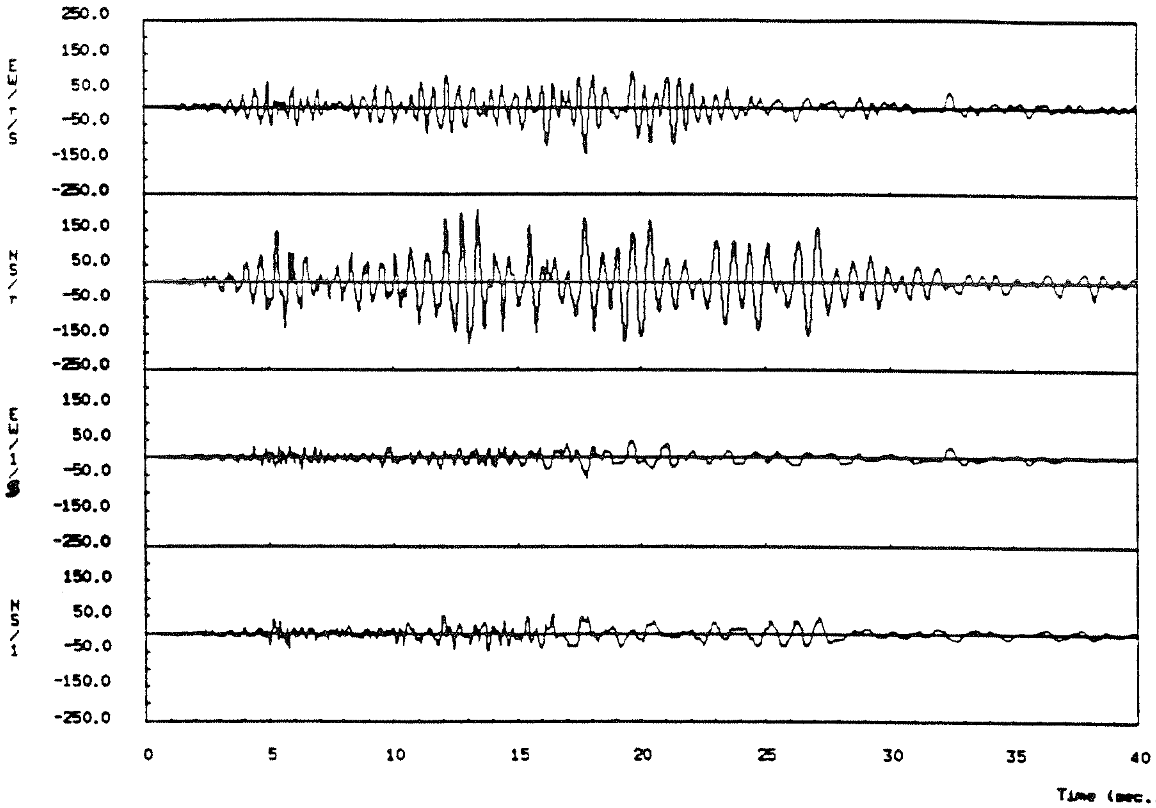

Figure 2 - Accelerations Records for Building 1. Morgan Hill Earthquake.

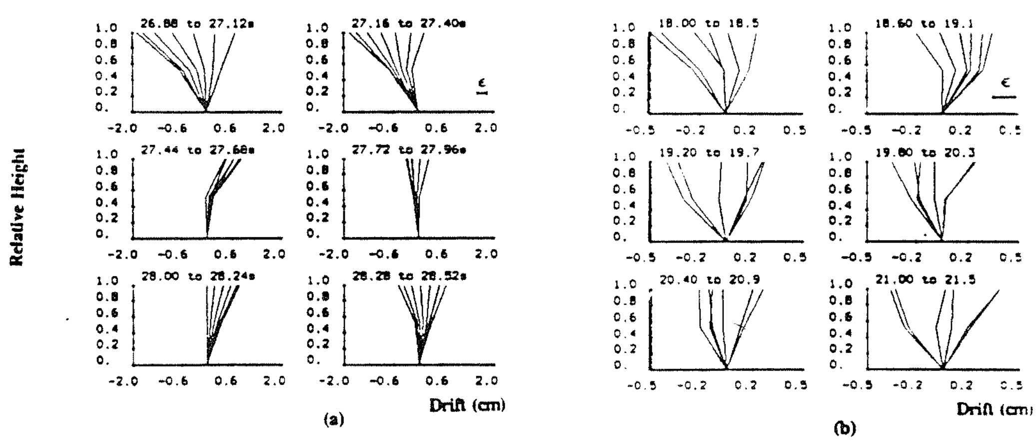

Figure 3 - Relative deflected shapes for Buiding 1. Morgan Hill Earthquake.
a) NS direction. b) EW direction.

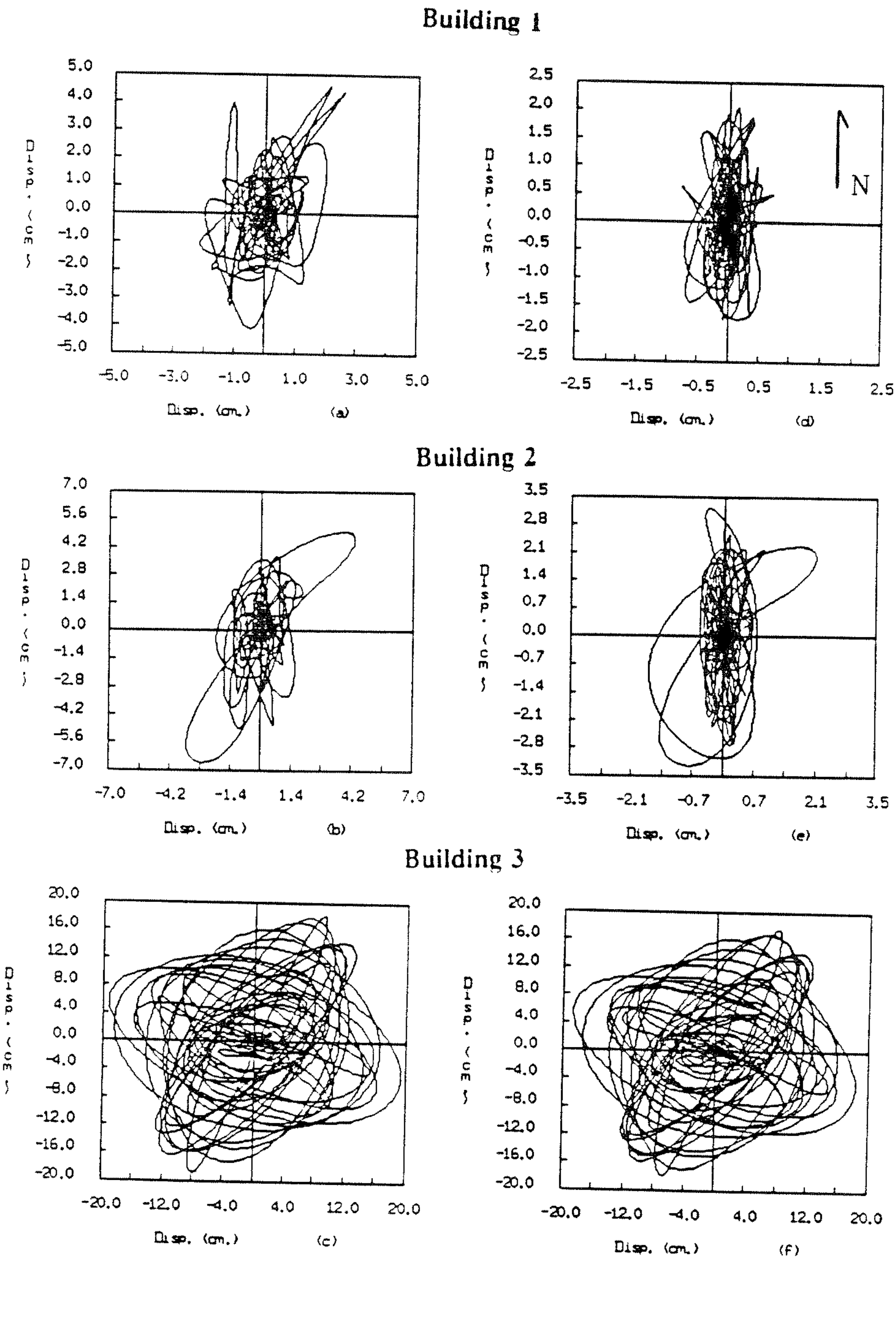

Figure 4 - Roof Orbital Displacement Histories for the Morgan Hill Earthquake: Total Displacement a, b, c; Relative displacement d, e, f.

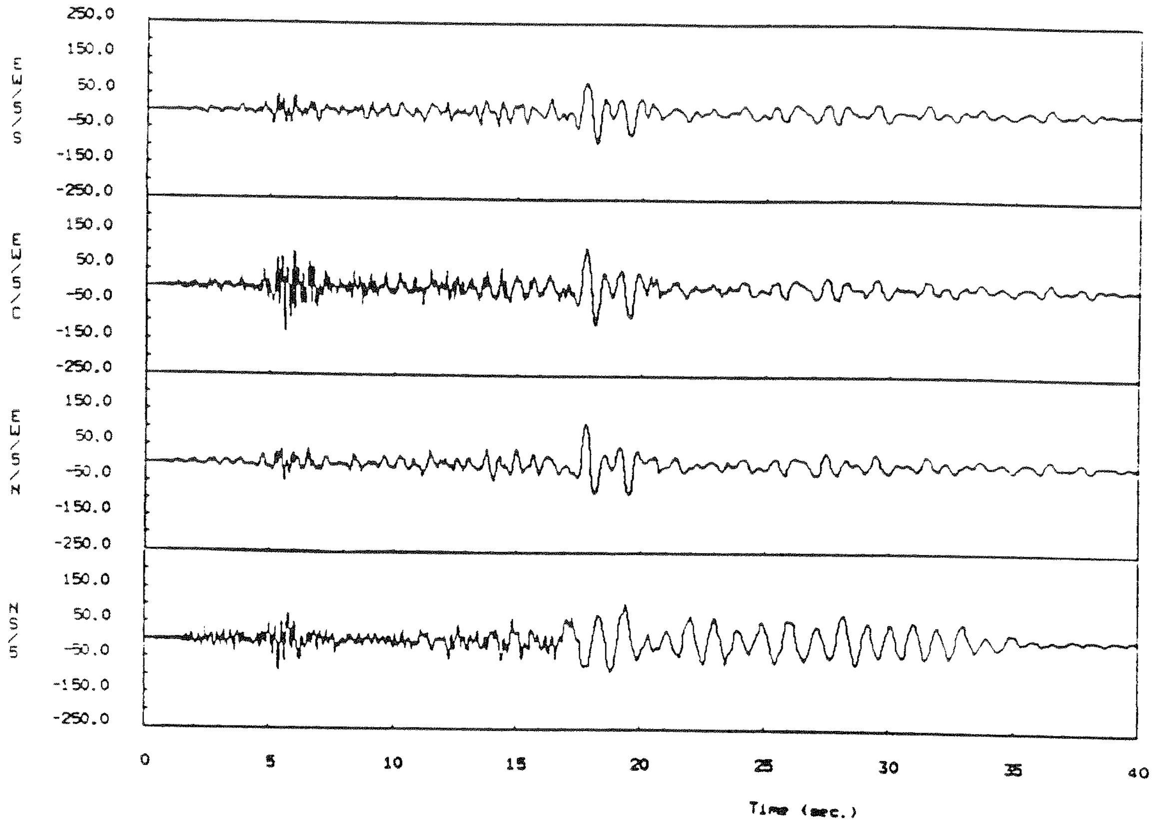

Figure 5 - Acceleration records for Building 2. Morgan Hill Earthquake. Fifth floor.

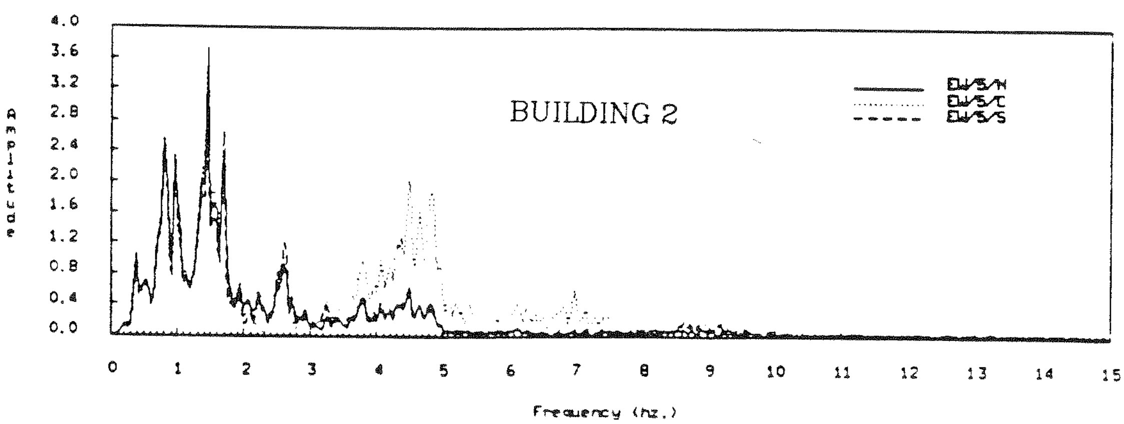

Figure 6 - Fourier Amplitude Acceleration Spectra. Fifth floor records. Dotted line presents center slab record. Morgan Hill earthquake.

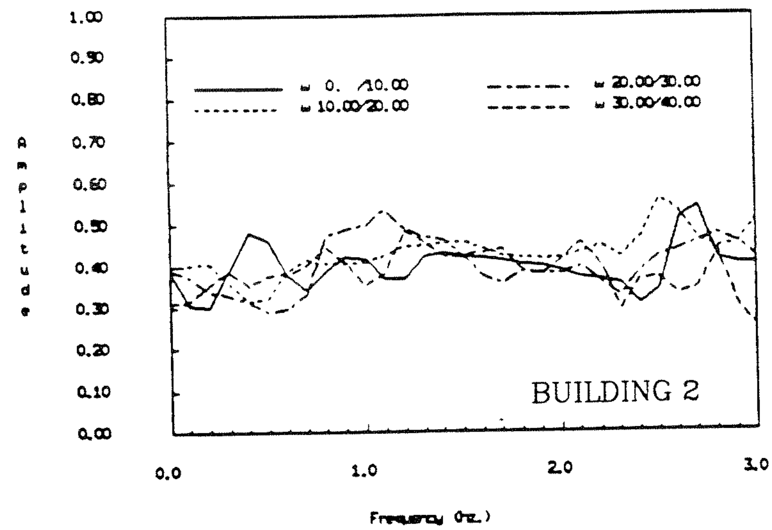

Figure 7 - Spectral ratio between roof rigid body acceleration rocking base on vertical records and relative roof acceleration for different time windows. Morgan Hill earthquake.

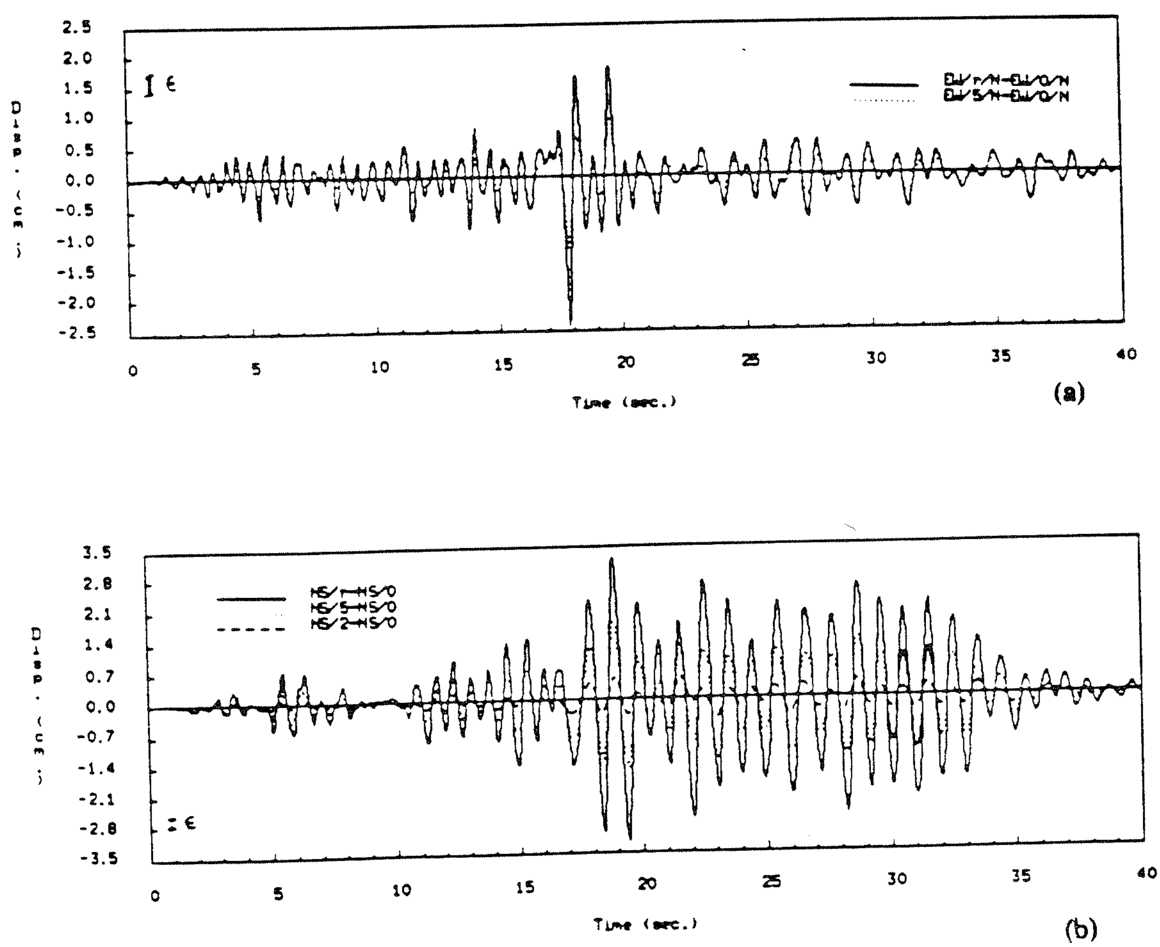

Figure 8 - Drifts for Building 2. Morgan Hill Earthquake.

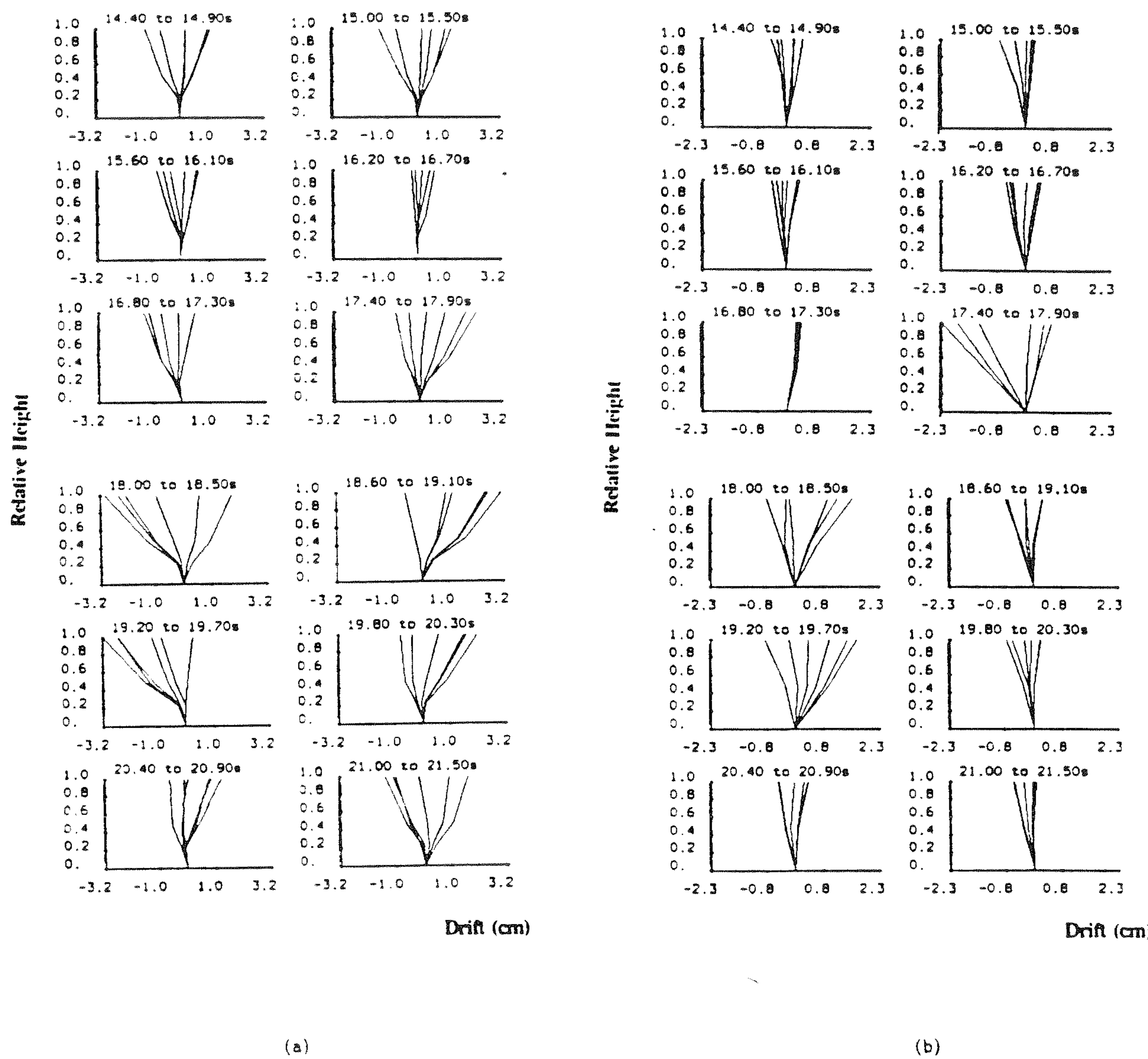

Figure 9 - Relative deflected shapes for Building 2. Morgan Hill Earthquake.
a) NS direction. b) EW direction.

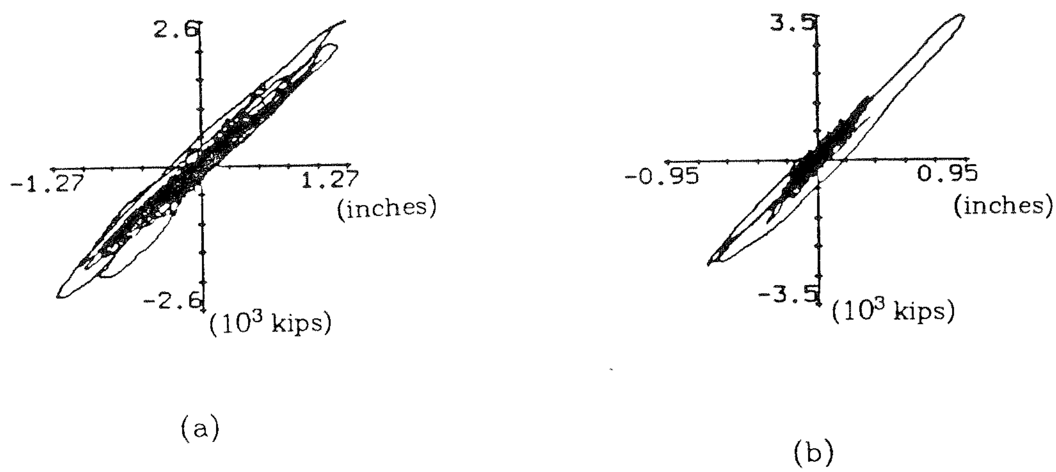

Figure 10 - Force-Deformation plot for Building 2. Morgan Hill Earthquake.
a) NS direction. b) EW direction.

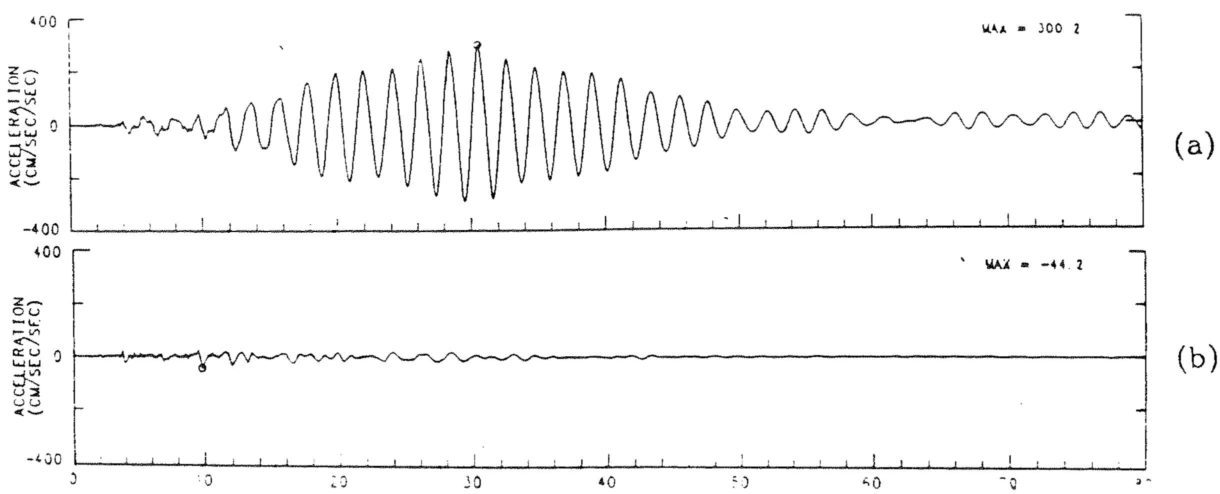

Figure 11 - Acceleration records for Building 3. Mt. Lewis Earthquake.
a) Twelfth floor (NS/12/W) and b) Ground level (NS/0/W) (10).

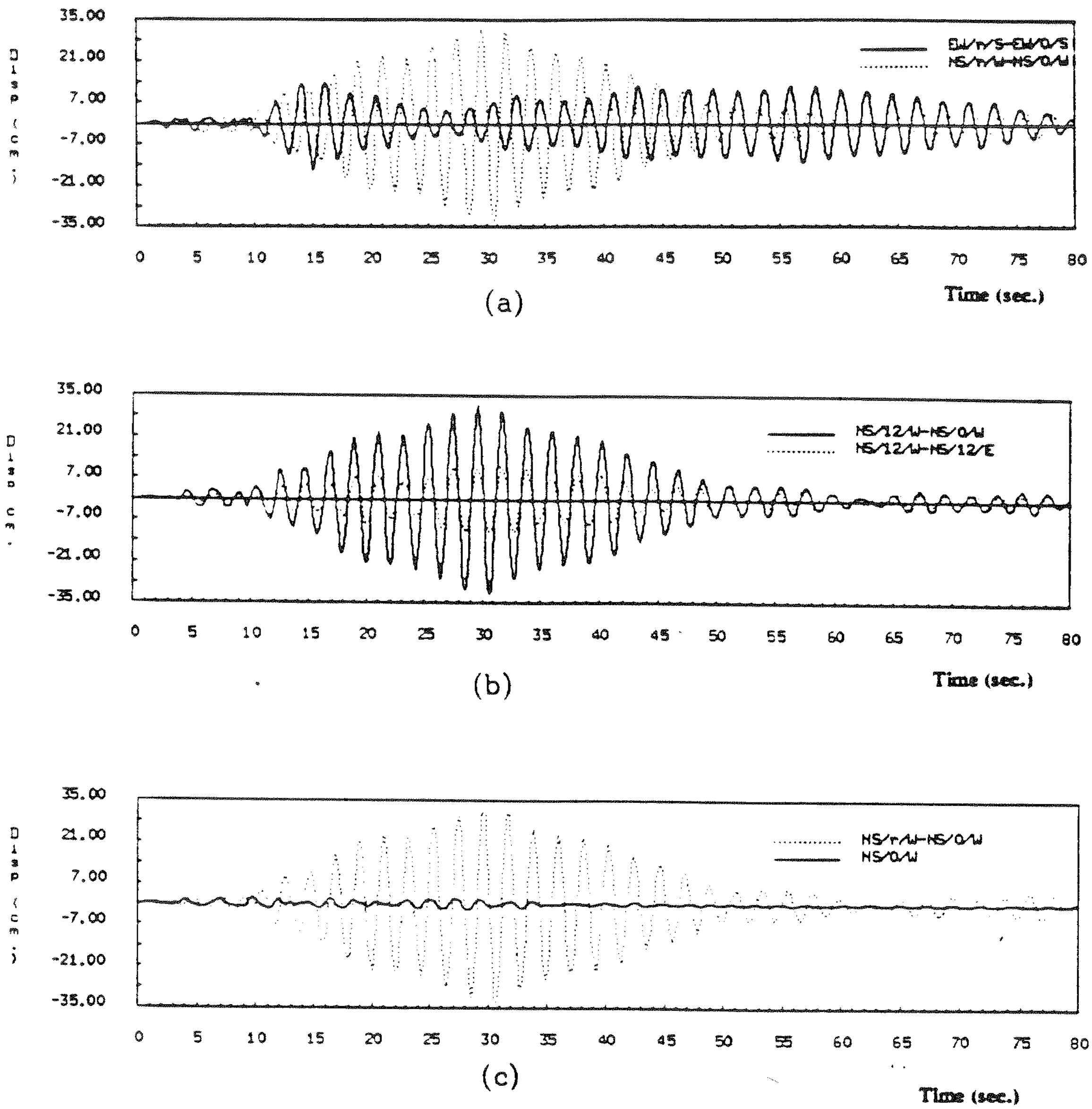

Figure 12 - Displacements for Building 3. Mt. Lewis Earthquake. a) Relative drift SW building corner. b) Twelfth floor relative drift and torsion, using NS records. c) Roof relative drift and base motion, NS direction.

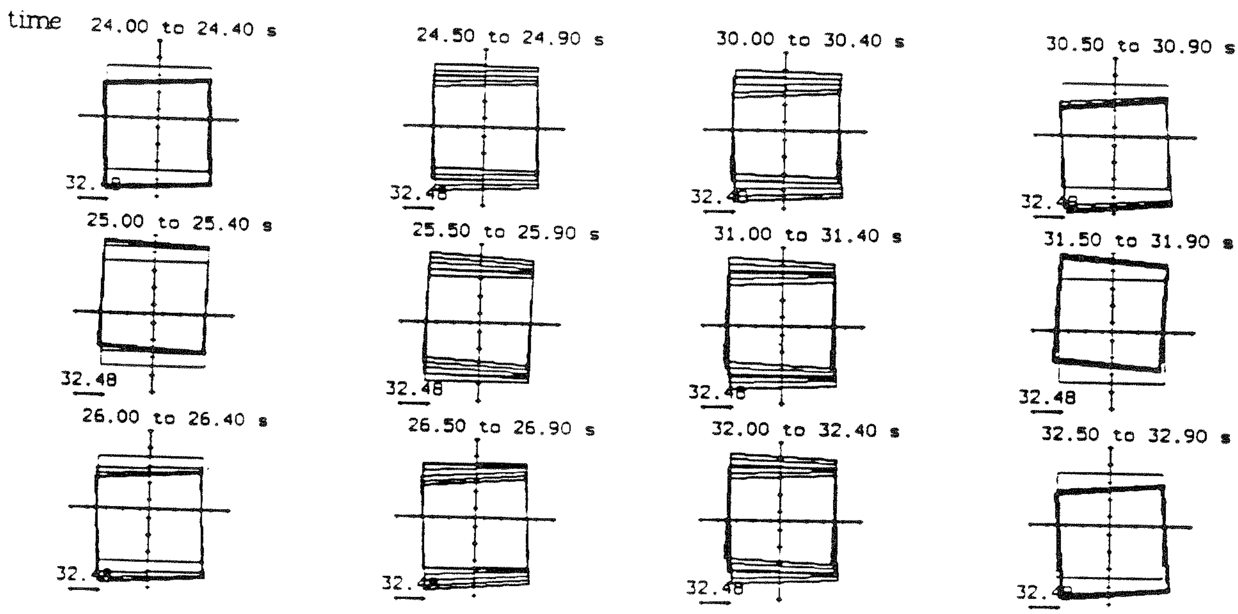

Figure 13 - Twelfth floor motion for Building 3. Mt. Lewis Earthquake.

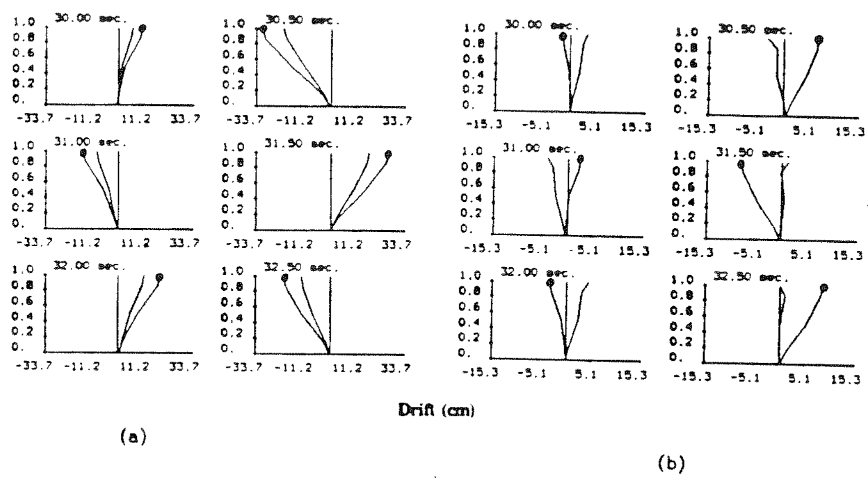

Figure 14 - Relative deflected shapes for Building 3 at different times for the Mt. Lewis Earthquake. a) NS direction. b) EW direction.[● South-West corner].

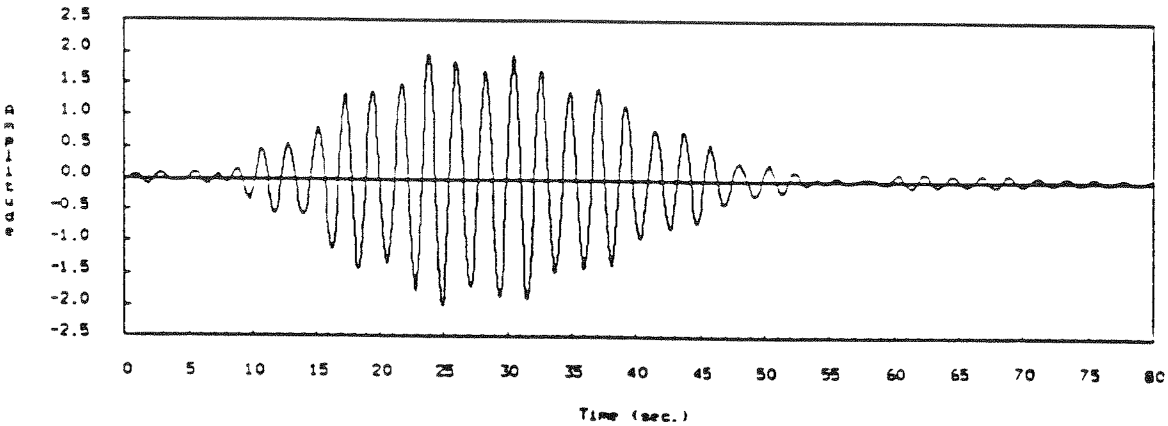

Figure 15 - Simple trigonometric series use to model Building 3 NS roof drifts. Periods of function are 2.25, 2.15 and 1.66 and the corresponding amplitudes are 1.0,1.0,0.2. Signal have been modified by a Bogdanoff envelope.

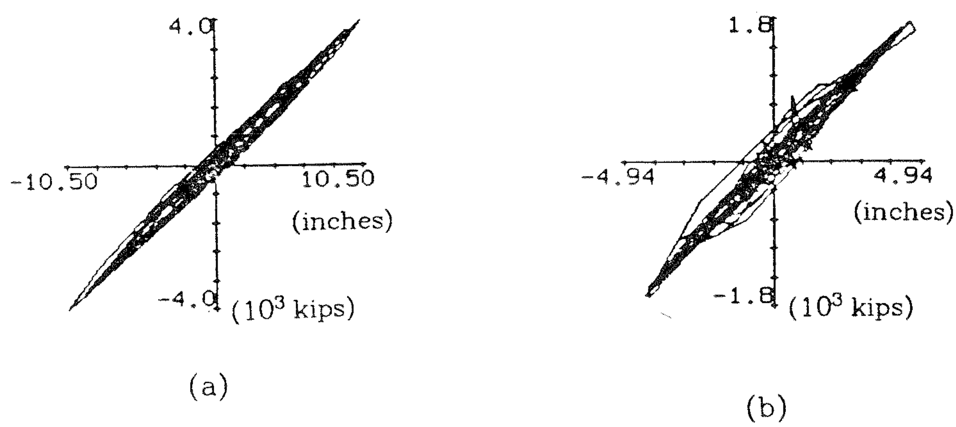

Figure 16 - Force-Deformation plot for Building 3. Mt. Lewis Earthquake. a) NS direction. b) EW direction.

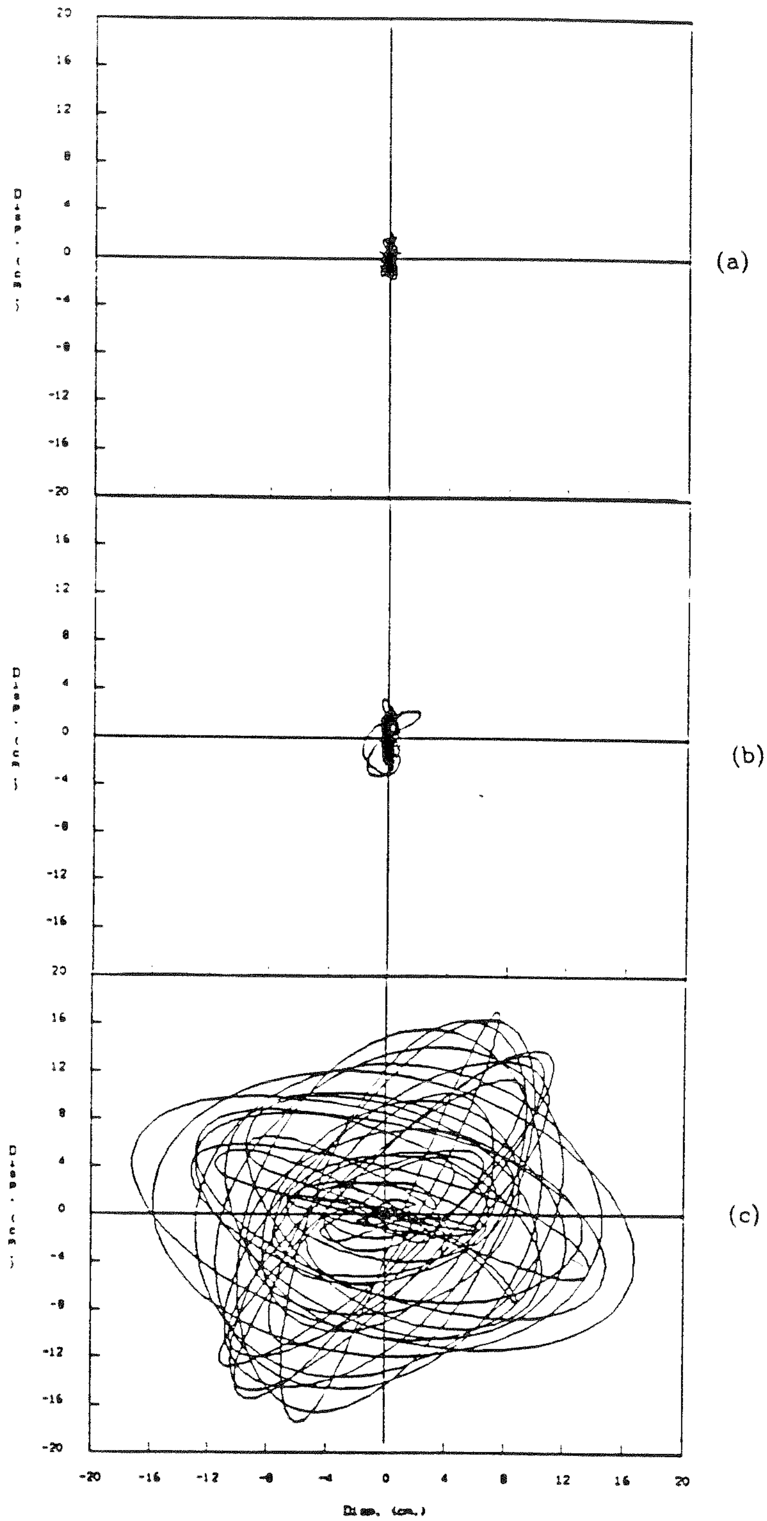

Figure 17 - Comparison of Roof Orbital Relative Displacement Histories. Morgan Hill earthquake. a) Building 1. b) Building 2. c) Building 3.