ENGINEERING INTERPRETATION OF THE RESPONSES OF THREE INSTRUMENTED BUILDINGS IN SAN JOSE

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ABSTRACT

The response records obtained in three buildings located in San Jose, California are examined and interpreted in this paper to identify their basic behavioral characteristics and engineering design parameters such as period, damping, and mode shapes. The buildings range in height from 10 to 13 stories. Observations related to the seismic response of different types of structural systems are extracted from this data.

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 and increasing the reliability of our design and analysis methods. Ideally, structural response records would be used as part of a integrated investigation in which interpreted records are used along with analytical results and, if damage is observed, experimental data to assess and improve current engineering practices. However, the recorded responses can by themselves provide much valuable information.

In this study, the responses of three buildings subjected to the Morgan Hill earthquake of 24 April, 1984 and the Mt. Lewis earthquake of March 31, 1986 are evaluated based on measured accelerograph records obtained by the Strong Motion Instrumentation Program (SMIP) of the Division of Mines and Geology of the California Department of Conservation. These buildings are located near one another in San Jose, California, about 20 km (12 miles) from the epicenter. These buildings are described in Table 1 and Fig. 1. Additional information on the buildings and the records can be obtained in Refs. 1-3.

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 characteristics of the structures are developed from the records and information regarding the differences in seismic responses as affected by structural system and earthquake ground motion characteristics is obtained.

BUILDING 1

This ten story residential building (SMIP Station No. 57356) was designed and constructed between 1971 and 1972. The vertical load carrying system consists of a 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 one of the major walls is terminated at the sixth floor and additional irregularities occur at the ground level. A pile foundation provides support for the building.

TABLE 1 -- BUILDING DATA

Name	Structural System	No. of Stories	Height (ft)	Peak Horiz. Accel. (g) Ground	Peak Horiz. Accel. (g) Building	Max.Amp. Ratio
Building 1	RC Shear Wall	10	96	0.06(0.03)	0.22(0.12)	4.0 (4.1)
Building 2	RC Shear Wall RC Moment Frame	10	124	0.06(0.04)	0.22(0.08)	3.6 (2.7)
Building 3	Steel Moment Frame	13	187	0.04(0.04)	0.17(0.32)	4.9 (7.1)

Note: Numbers in (..) correspond to the Mt. Lewis earthquake

Thirteen analog instruments were installed in the building. These were located to estimate such response features as torsional motions, wall rocking, 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 displacements (and about twice this amount for relative drifts).

Acceleration Response. -- The maximum ground acceleration (Table 1) was 0.06g and the maximum structural acceleration at the roof was 0.22g. The amplification ratios for the various locations in the structure were computed. They were found to be (Table 1) 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 during the Morgan Hill earthquake are shown in Fig. 2. These show that the ground motion is characterized by relatively high frequency motions during the first 17 seconds of motion and by much longer period motions during the latter portions of the motion. The acceleration response records indicate that the building is more flexible in the NS direction and suffers higher accelerations in that direction.

Drifts. -- Drifts obtained by subtracting horizontal displacement records from corresponding ground level displacements are quite small, on the same order as the accuracy of the measurements. In general, the drifts in the EW direction are of higher frequency and smaller in amplitude than 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 value permitted by the 1985 UBC) and 0.07% for the NS direction (more than twice as much, but still less than 17% of the code value). Some deflected shapes for the building are shown in Fig. 3. The motion of the roof in two directions is plotted in Fig. 4. The total displacement of the structure is nearly twice the relative displacement. It should be noted that the structure is somewhat stiffer in the EW direction, but it achieves several cycles where the maximum displacements are nearly reached simultaneously in both directions.

Inspection of the records indicates that there was little torsion or bowing of the floor slabs. Measured values were near or below the confidence level for the records. Rocking of the walls was estimated using the derived vertical displacement records at the base of the walls. Unfortunately, these values were below the confidence level of the records. Analysis of transfer functions obtained from Fourier amplitude spectra of the corresponding vertical acceleration records indicates that the rocking of the walls contributes to the response in the transverse direction.

Periods and Mode Shape. -- Due to the low level of response, only the first mode could be 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 estimated periods are summarized in Table 2. No significant differences were detected for the two earthquakes. Uniform Building Code estimates for the building are also shown in the table. These indicate that the 1985 UBC equation incorrectly identifies the NS direction as being stiffer. The equations in the 1988 UBC 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 conservatively under-estimates both periods by about 30% when the coefficient is computed according to the code.

TABLE 2 -- PERIODS (IN SECONDS) FOR BUILDING 1

Direction	Measured Values	1985 UBC	1988 UBC C_t = 0.02	1988 UBC C_t computed
EW	0.4-0.5	0.59	0.61	0.33
NS	0.6-0.7	0.32	0.61	0.50

Based on the records, the first mode shape for both directions is estimated to be 1.0, 0.4 and 0.0 for the roof, sixth floor and ground. There does appear to be some higher mode influence especially for the NS direction. However, the response is too small to correctly identify this without using system identification procedures.

Damping. -- Equivalent viscous damping coefficients were estimated. Values obtained for the first mode are about 5% for the NS direction and 11 to 14 % for the EW direction. This latter value is considered unreliable due to the low level of response in this direction and soil-structure interaction effects.

Seismic Demands. -- During the Morgan Hill earthquake the building developed a base shear coefficient of 0.096 in the EW direction and 0.1 in the NS direction. Corresponding values for the Mt. Lewis earthquake were 0.05 and 0.04, respectively. The working stress design base shear coefficient was 0.08 and 0.097 for the EW and NS directions, respectively. Thus, the Morgan Hill earthquake corresponded roughly to working stress level event for the design code. The 1985 UBC, however, requires design base shears nearly 2.6 times larger than the original design values. Thus, for a similar building designed according to modern codes, this earthquake corresponds to a very minor earthquake. The shear capacity of the building can be estimated easily using the 1988 UBC. This capacity corresponds to a base shear coefficient in the EW direction of 1.04 and 0.14 for the NS direction. The capacity of the walls in the NS direction are relatively small in comparison with the values for the EW direction, but appear consistent with code requirements. Dynamic analyses of the building would be desirable to better characterize soil structure interaction effects, the effects of discontinuities of the walls in the NS direction and to better estimate the capacity of the building.

BUILDING 2

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. 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 5 ft. (1.5 m) thick mat foundation. The building was instrumented similar to Building 1.

Acceleration Response. -- As with Building 1 the maximum ground acceleration during the Morgan Hill earthquake was 0.06g and the maximum structural acceleration was 0.22g. The EW direction develops slightly greater accelerations and larger amplification ratios than in the more flexible NS direction. More significant is the fact that the duration of intense motion in the NS direction is substantially longer. Also, it is important to note that the accelerations at the center of the fifth floor diaphragm are about 20% larger than those at the ends for the Morgan Hill earthquake and 100% larger for the Mt. Lewis earthquake, indicating that the diaphragm undergoes important in-plane deformations.

Drifts. -- EW drifts are characterized by low, but nearly constant amplitude, cycles of drift with several cycles between 17 and 20 sec. having more than double the amplitude of the other portions of the record (Fig 5). Drift indices in the EW direction do not exceed 0.06%, approximately one-tenth the value permitted by the code at working stress levels. Analyses of the corrected vertical acceleration and displacement records under the shear wall indicate that nearly 35% of the lateral displacement in the EW direction is associated with rigid body rocking about the base of the wall. In the more flexible NS direction, the response is largest during the latter portions of the record. The deformations correspond to average interstory drift index of only around 0.1%. The structure displaces more in the NS direction, but as with Building 1 there are several major cycles where it develops its maximum displacement in both directions simultaneously (Fig. 4). No significant torsion was detected in the records for this regular symmetric building.

Periods and Mode Shapes. -- The periods estimated for the building are summarized in Table 3. It is interesting to note that the periods for the EW direction roughly obey the rule of thumb for cantilever structures that the higher mode periods vary as 1/6, 1/18, etc. of the fundamental period, while the periods in the NS direction obey the relationships for a shear (frame) building (i.e., 1/3, 1/5, etc. of the fundamental period). Values obtained using the 1985 and 1988 UBC code give values similar to the measured values. However, it is very significant to note that the more refined computation of C_t results overly conservative prediction, in comparison with Building 1.

TABLE 3 -- PERIODS (IN SECONDS) FOR BUILDING 2

Direction	Mode	Measured Values	Quick Guess	1985 UBC	1988 UBC C_t=0.02	1988 UBC Calc. C_t	Computed Ref. 4
EW	1	0.6-0.65	0.63	0.69	0.73	0.36	0.44
	2	0.2-0.25	0.28	--	--	--	0.12
NS	1	0.91-0.96	0.94	1.0	1.1	--	0.74
	2	0.25-0.28	0.31	--	--	--	0.24
	3	0.14-0.18	0.19	--	--	--	0.13
Torsion	1	0.33-0.40	--	--	--	--	--

In the EW direction, the first and second mode shape 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.

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

Seismic Demands. -- The building developed in the EW direction a 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, respectively, for the two earthquakes. The values for the Morgan Hill earthquake are 88% larger than the non-factored values used in the original design in the EW direction and 27% 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. Additional analyses are desired to assess the influence of soil-structure interaction, bidirectional motion and damping on the response and to better estimate the frame capacity.

BUILDING 3

This building (SMIP Station No. 57357) is a thirteen story office building located approximately 1.5 miles (2 km) 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. Lateral load resistance is provided by moment resisting frames. Due to special framing around the elevators and nonuniform placement of cladding, the structure is not symmetric and torsional response is expected. 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. A free field instrument had been installed, but was removed shortly prior to the Morgan Hill event.

Acceleration Response. -- The input motion to this building was less severe than for the other buildings, but the amplification ratio was higher so the recorded motions were similar or higher than those measured in the other structures. The amplification for the Morgan Hill earthquake was in excess of 5 and that for the Mt. Lewis event was greater than 7. Even more significantly, one can see from Fig. 6 that the structure and ground records have an unusually long duration, more than 80 seconds. This is also true for the Mt. Lewis earthquake. The first thirty seconds of the response exhibits growing response and relatively higher frequencies than in the later portions which are characterized relatively narrow band periodic motion with strong amplitude modulation. This amplitude modulation is similar to mechanical beating of dynamic systems with closely spaced natural frequencies. It is significant to note that the maximum accelerations in the upper levels of the structure are developed long after the strong motion portion of the ground shaking has ended.

Drifts. -- Maximum drift indices for the building for the Morgan Hill earthquake are on the order of 0.33% and 0.65% for the Mt. Lewis event. The 1985 UBC limited drifts under working stresses to 0.5% and the 1988 UBC uses a limit of 0.4% or, if an R_w factor of 12 were used, of 0.25%. Thus, the drifts experienced by the building were larger than accepted by current design practices for nonfactored design loads. Damages to nonsupported book shelves occurred in the upper levels and some nonstructural damages to partitions were observed in the lower levels. Figure 4 shows that the roof displacements are bidirectional, and that most of the total response is due to the structure, and not the ground as with the other two buildings.

The motion of the building shows the three dimensional interaction of more than three modes. This involves coupled translational and torsional motions. It is complicated by the fact that the modal frequencies are similar leading to a beating or modal interference phenomenon. This is clearly shown for the Mt. Lewis event in Fig. 7 where there is a clear transfer of energy from the NS and EW directions. Simple trigonometric time series can be used to examine this behavior. Summing three series will result in a equivalent natural period and two beating periods. Inspection of the records indicates beating periods of about 100 and 16 seconds and an equivalent period of 2.2 seconds. This results in a system with periods about 2.2, 2.1 and 1.7 seconds. The long duration of the response and the high amplitude of the motion may in significant part be associated with the energy in the system transferring slowly back and forth between the first three modes, and the modes constructively reinforcing one another during portions of the motion. The response also appears to be amplified due to resonance of the building to the dynamic characteristics of the site. The contribution of these factors to the severe motion of the building requires more detailed analysis. Foundation rocking is not important.

The flexibility of the floor diaphragms was investigated by using three of the records obtained at a level to predict the fourth, assuming rigid diaphragm action. Errors between 12% and 24% were detected indicating significant diaphragm flexibility. 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 contribute to this error as well.

Periods and Mode Shapes. -- Period values shown in Table 4 have been estimated. The periods are substantially longer than estimated by either the 1985 or 1988 UBC.

TABLE 4 -- PERIODS (IN SECONDS) FOR BUILDING 3

Direction	Mode	Measured Values	1985 UBC	1988 UBC	Damping %
EW	1	2.15-2.2	1.3	1.77	2-3
NS	2	2.05-2.1	1.3	1.77	3-4
Torsion	3	1.70	--	--	--
EW	4	0.65-0.75	--	--	--
NS	5	0.60-0.70	--	--	--

Damping. -- Due to the interaction of the closely spaced modes, a clear identification of viscous damping was not possible. An approximate value was obtained by observing the free amplitude decay near the end of response records not significantly influenced by torsion. Estimated values are shown in Table 4.

Comments. -- In view of the complex nature of the three dimensional response, influenced by beating of closely spaced coupled modes, possible soil-structure interaction effects, site amplification, the resistance of cladding and other nonstructural components, it is desirable that a dynamic analysis of this building be carried out. System identification procedures should be utilized to better characterize modal characteristics. A free field instrument should be installed to assist in identifying the relative importance of soil-structure interaction and site effects.

CONCLUSIONS AND RECOMMENDATIONS

The records of the three buildings studied herein have provided significant insight into their dynamic characteristics and the accuracy of various code assumptions. The studies show that the response of buildings is clearly bidirectional and directionally oriented in accordance with their stiffnesses. The behavior of the first two buildings was dominated by the Morgan Hill earthquake while that of the third building was controlled by the less sever Mt. Lewis event. Period calculations using code empirical equations have improved, but additional improvements are desirable. SMIP program data may be indispensable in this effort. The buildings were subjected to levels of excitation consistent with their working stress design basis. No structural damage was thus expected or observed. While much information was obtained in these studies, structural analyses of the buildings are required. Studies are also needed to better assess the confidence that can be place in relative drift and deformation values obtained by manipulation of displacement records derived from processed acceleration records.

ACKNOWLEDGMENTS

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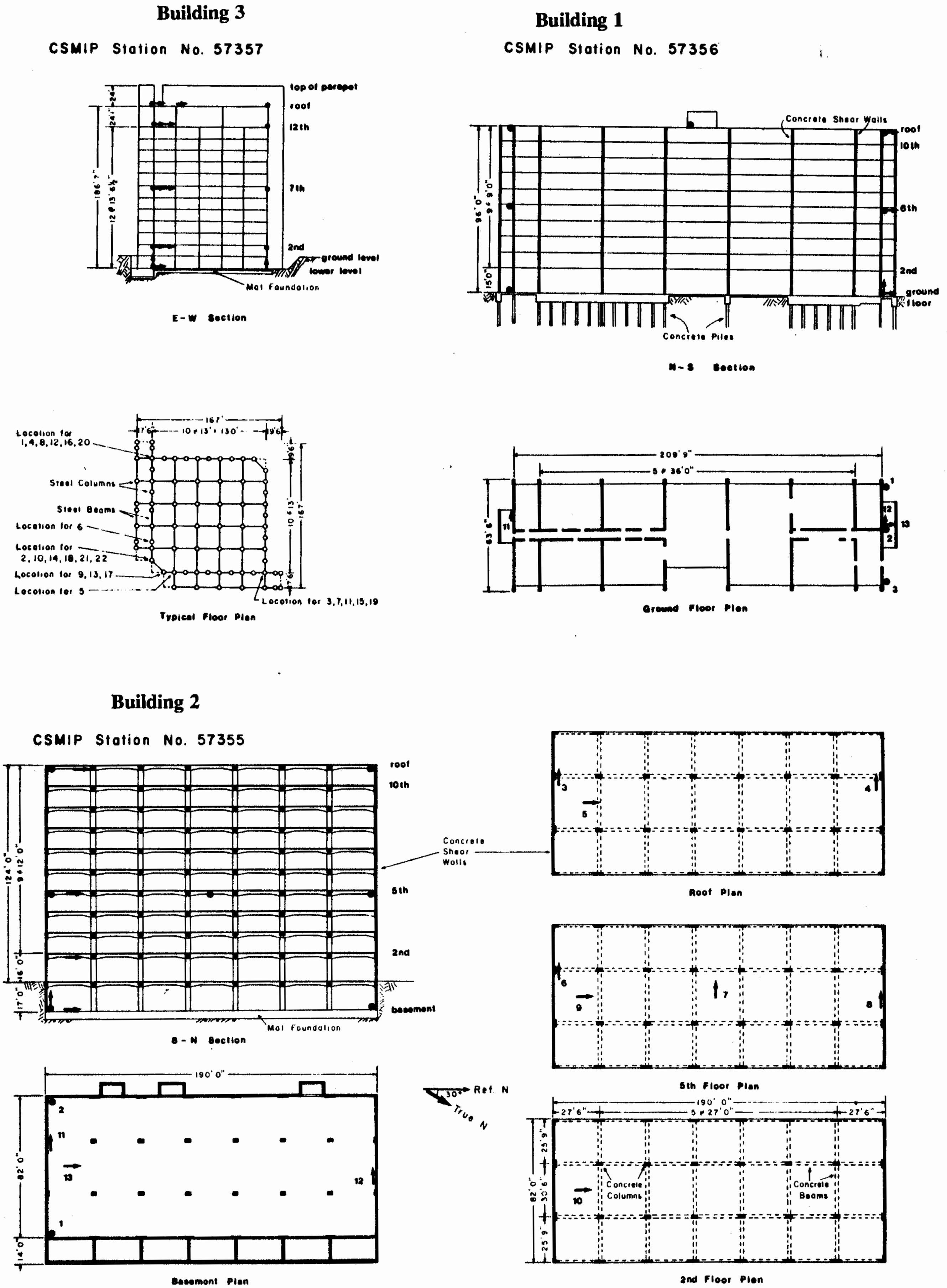


Figure 1- Buildings Studied.

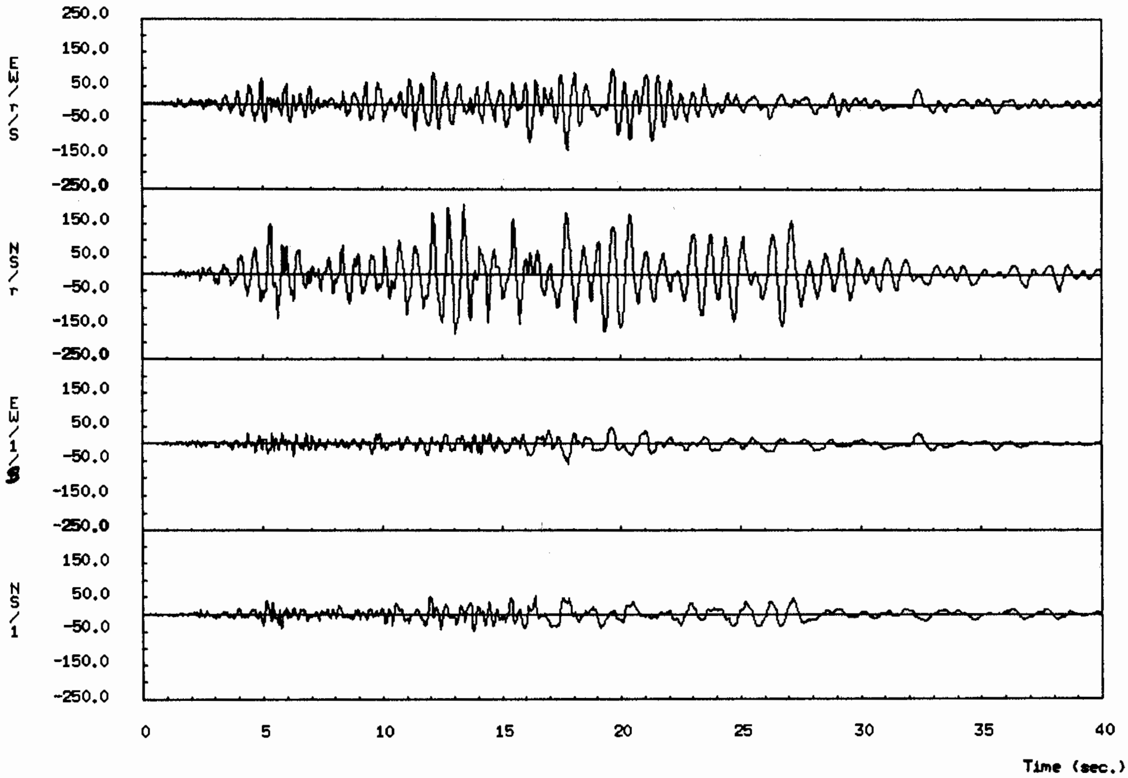


Figure 2 - Acceleration Records for Building 1. Morgan Hill Earthquake.

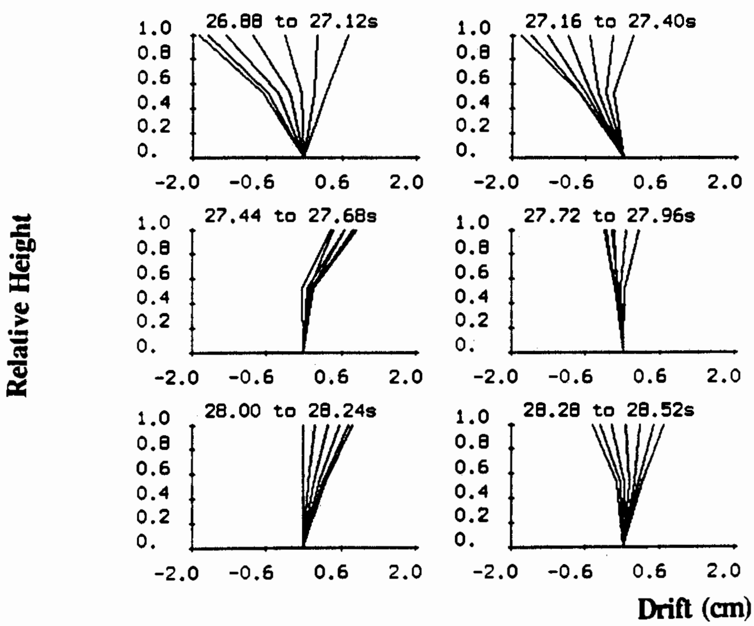
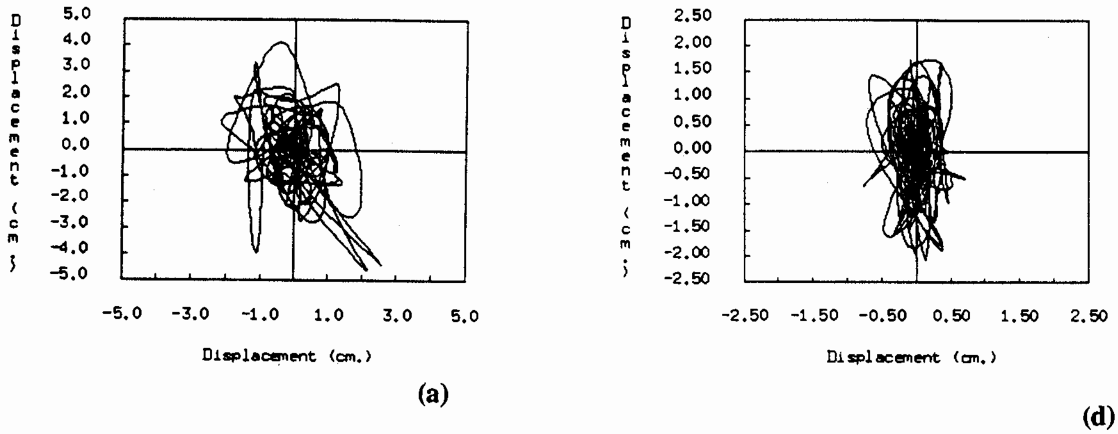
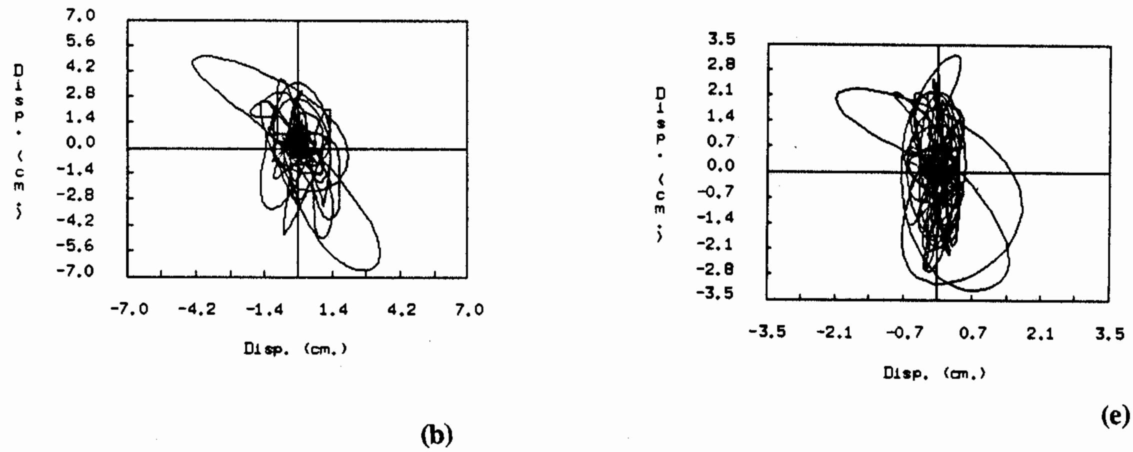


Figure 3 - Relative NS drift. Morgan Hill Eart'iquake.

Building 1



Building 2



Building 3

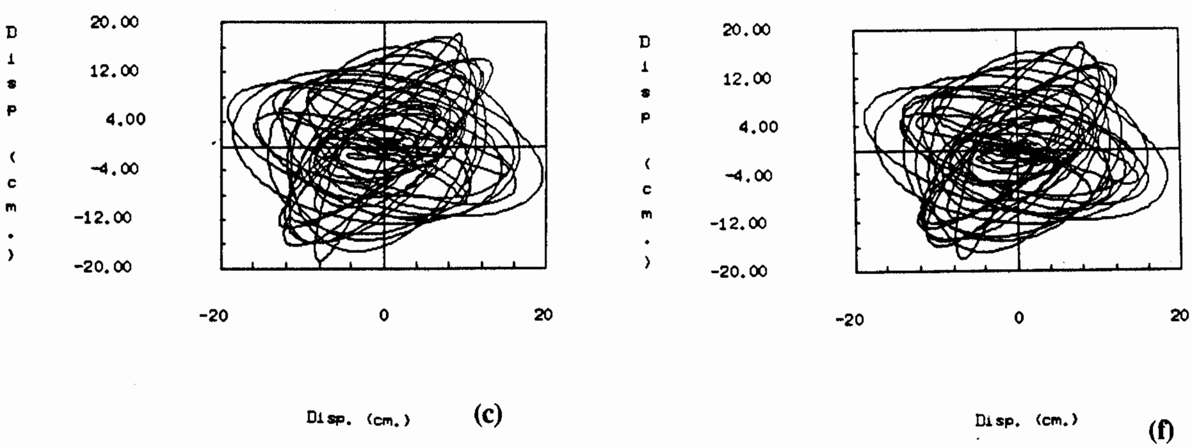


Figure 4 - Roof Orbital Displacement Histories: Total Displacement a, b, c; Relative Displacement d, e, f.

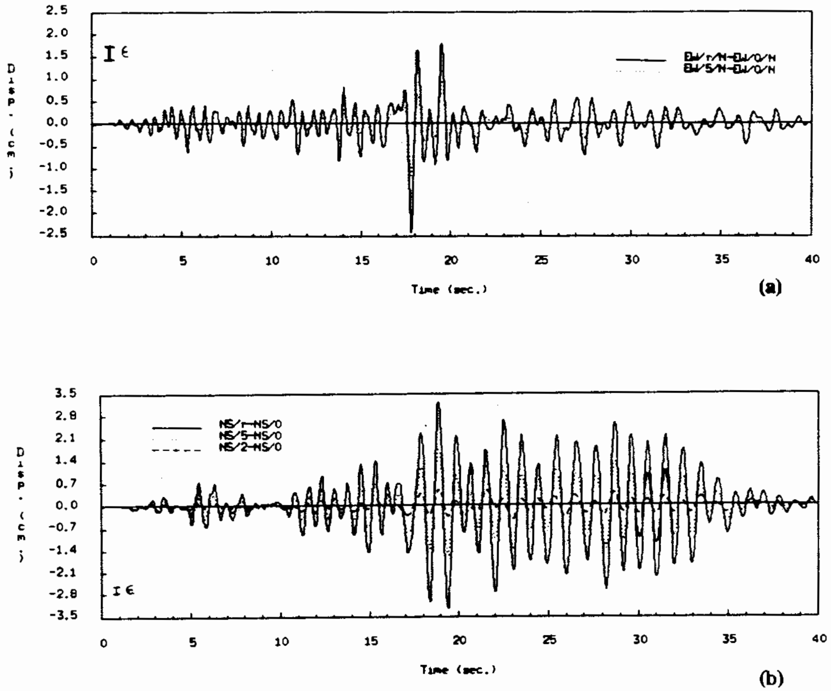


Figure 5 - Drifts for Building 2. a) EW direction; b) NS Direction.

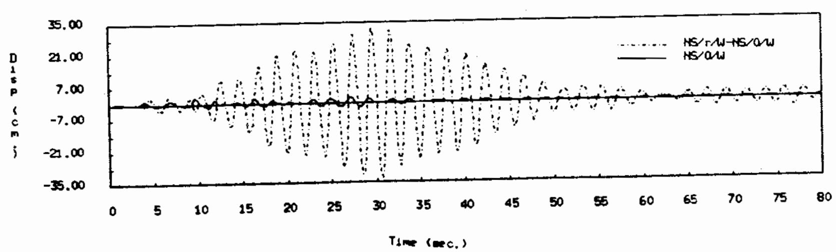


Figure 6 - Displacement Record Building 3. Roof and Base

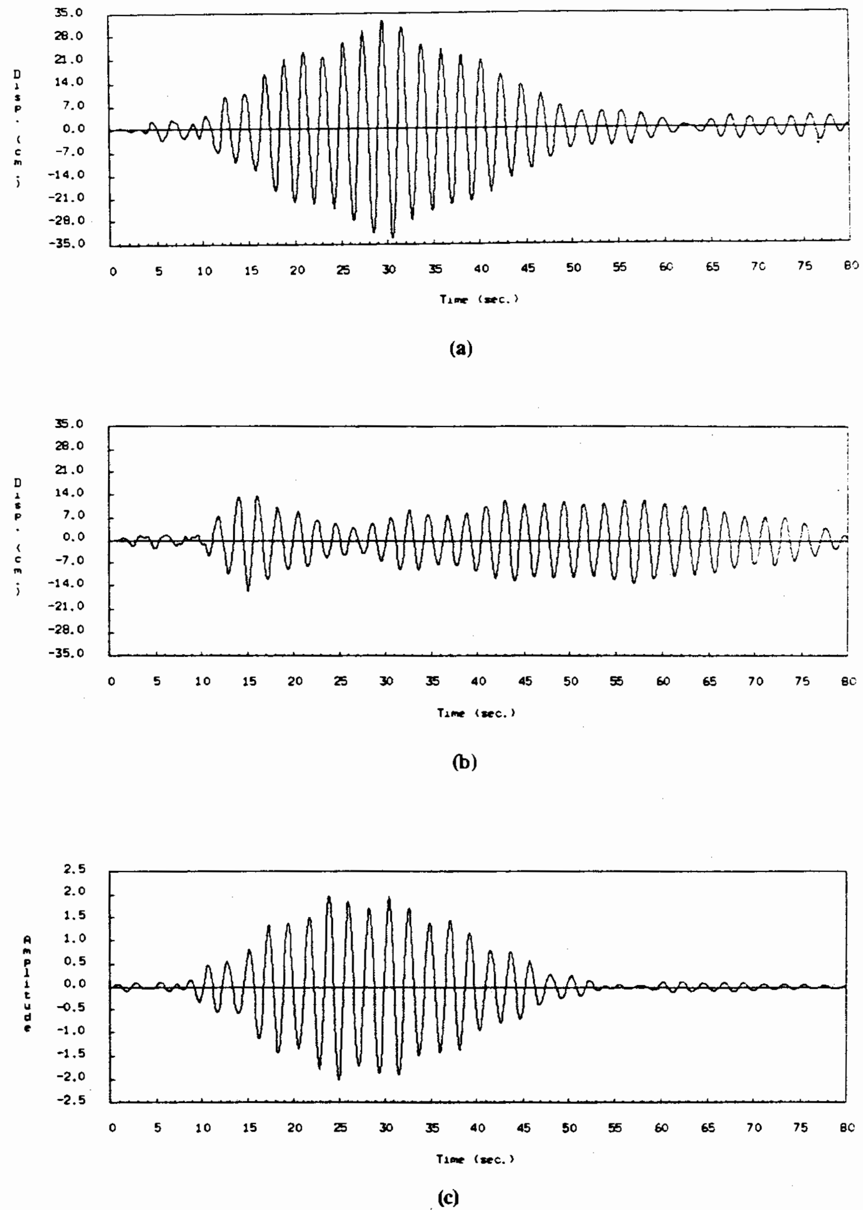


Figure 7 - Beating Behavior Building 3, Mt. Lewis Earthquake. a) Roof displacement NS direction. b) Roof displacement EW direction. c) Simple trigonometric series.