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SEISMIC RESPONSE AND ANALYTICAL MODELING OF THREE INSTRUMENTED BUILDINGS

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ABSTRACT

Earthquake response records obtained in three buildings located in San Jose, California are examined and interpreted in this paper. 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 employed different types of structural systems. Results of empirical and analytical models are compared with measured responses.

INTRODUCTION

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 the Strong Motion Instrumentation Program (SMIP)[4]. 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, Fig. 1.

BUILDING 1

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 slabs on reinforced concrete bearing walls. The lateral load resisting system consists of reinforced concrete shear walls. 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.

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TABLE 1.
Building data and gross response values

Building	1	2	3
Structural system	RC Shear Walls RC Moment Frames	RC Shear Walls	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
Max rocking (%)	50.0	45.0	-

(*) above/below ground

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 acceleration were 0.03 and 0.12 g, respectively. Amplification ratios indicated in Table 1 were obtained by dividing the peak acceleration at a location by the corresponding acceleration at the ground. Fourier amplitude acceleration spectra of vertical records obtained on the foundation for an EW oriented wall have relatively high amplitudes around the predominant period of the observed EW translational motion, indicating that the walls rotate at the foundation. Fourier amplitude acceleration spectra of properly scaled vertical records were used to estimate this rocking, Table 1 [2]. It's contribution was found to be close to 50% of the relative roof acceleration for both earthquakes in the transverse direction.

Drifts. Drifts obtained by subtracting horizontal displacement records from corresponding ground level displacements are small, Table 1. 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 [6]) and 0.10% for the NS direction (more than twice as much, but still less than 20% of the code permitted value). A strong effect on displacements due to the discontinuity of the shear wall at the sixth level in the NS direction was not observed.

The total and relative motions of the roof in two directions are plotted in Fig. 2. As seen in this figure in some cycles the maximum relative displacements in each direction occur at nearly the same time. Ground displacement contributes nearly half of the total displacement. Inspection of other records indicates that there was little torsion or bowing of the floor slabs [2].

Periods, Damping and Mode Shapes. Due to the low level of response, only the first mode could be reliably identified, Table 2. The periods include the effects of foundation flexibility. No significant differences in periods estimates were detected for the two earthquakes considered. Uniform Building Code estimates of period for the building are also shown in Table 2. The 1988 UBC estimates are improved over the 1985 values. The predominant mode shape for both directions is estimated to be 1.0, 0.4 and 0.0 for the roof, sixth floor and ground.

TABLE 2.
Periods (in seconds) and Damping for Building 1

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

(*) Using C_t =0.02.
(**) Using C_t computed using the effective area of the shear walls, according to the 1988 UBC [7].

Seismic Demands. Seismic demands for 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, disregarding any damping forces, and story shears and overturning moments were computed. 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 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.

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 1.5 m. (5 ft.) thick mat foundation (Fig 1).

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 g and 0.08 g, respectively. 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 response. Slab contributions to response are clearly visible in Fourier acceleration amplitude spectrum presented in Ref. [2] for frequencies between 4.0 and 5.0 Hertz. Analyses of the appropriately scaled vertical acceleration records at the base of the south shear wall indicate that, in the EW direction, between 35 to 45 % of the relative roof accelerations during the Morgan Hill earthquake and 35 to 40% during the Mt. Lewis earthquake are associated with rocking of the foundation.

Drifts. Drift indices in the EW direction do not exceed 0.07%, approximately fourteen percent of the value permitted by the 1985 UBC code at working stress levels. The NS deformations correspond to an average interstory drift index of around 0.1%. The structure displaces more in the NS direction, but there are major cycles where it develops nearly its maximum displacement in both directions simultaneously (Fig. 2). No significant torsion was detected from displacement records for this regular and symmetric building [4].

Periods, Damping and Mode Shapes. The periods and damping estimated for the building are summarized in Table 3. No significant differences in periods values were detected for the two earthquakes considered. 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.

TABLE 3.
Periods (in seconds) and Damping for Building 2

Direction	Mode	Measured Values	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	–	–	–	0.12	–
NS	1	0.91-0.96	1.0	1.1	–	0.74	3-5
	2	0.25-0.28	–	–	–	0.24	–
	3	0.14-0.18	–	–	–	0.13	–
Torsion	1	0.33-0.40	–	–	–	–	–

(*) Using C_t =0.02.
(**) Using C_t computed using the effective area of the shear walls, according to the 1988 UBC.

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 a 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.

BUILDING 3

This building (SMIP Station No. 57357) is a thirteen story 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. Lateral load resistance is provided by moment resisting frames. A mat foundation is used to support the building.

Acceleration response. The input motion to 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 for both events, and the maximum structural acceleration at the roof, obtained 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 (produced by beating associated with closely spaced modal periods and torsional coupling), soil-structure resonance and an unusual long duration, more than 80 seconds, Fig. 3.

Drifts. Maximum drift indices for the building during the Morgan Hill earthquake are on the order of 0.40% and 0.72% for the Mt. Lewis event. The 1988 UBC limited drifts under working stresses conditions to 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 nonfactored design loads. Damages occurred to nonsupported book shelves and to two members that braced a glass atrium at the the third floor. Figure 2 show that the roof displacements are bi-directional. Similar motion were obtained during the Mt. Lewis earthquake [2]. Significant torsion was observed in the building during both events. The relative displacement from one side of the building to the other was 12.32 cm. (4.85 inches) during the Mt. Lewis earthquake. This torsional displacement contributes roughly 19% to the relative roof displacement, Fig 3b.

The motion of the building exhibits 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. 3 where modulation of response amplitudes is strong. Inspection of the records, especially for the Mt. Lewis earthquake, indicates beating periods of about 100 and 16 seconds and an equivalent period of 2.2 seconds for the translational records and 1.85 seconds for the derived torsional displacements.

The in-plane flexibility of the floor diaphragms was investigated by comparing computed torsional motion as observed from EW and NS displacement records. The difference between maximum values of computed torsional motion were 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 contribute to this value as well.

Periods and Damping. Period and damping values observed are presented in Table 4. Due to the closeness of the periods damping is just a gross estimate. No significant differences in periods values were detected for the two earthquakes considered.

TABLE 4.
Periods (in seconds) and Damping 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	–	–	–

Demand versus UBC requirements. The calculated base shear coefficient required for 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. Inspection of derived hysteresis loops for the building [2] indicate, however, that it remained essentially elastic.

The response of the building is nonetheless very severe considering the intensity of the excitation. The long duration of the response and the high amplitude of the motion 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, and the resonance of the building due to the dynamic characteristics of the site. Foundation rocking was found not to have an important influence on the response.

Analytical Model. A three dimensional mathematical model was developed and analyses with bi-directional input of Building 3 were performed [3]. The model considered a spatial frame, incorporated the effect of beam-column joint flexibility and nonstructural element interaction. Nearly perfect match of the model and recorded response was found for most of the record length, Fig. 4. Analyses indicated that floor slab flexibility did not contribute significantly to the response. The analyses clearly indicate the profound effect of the three dimensional response on the behavior of this building.

CONCLUSIONS

The records of the three buildings studied herein provide significant insight into their dynamic characteristics and the accuracy of various code assumptions. Important effects of torsion, in-plane diaphragm deformations, bi-directional response, foundation flexibility and modal coupling have been observed. The importance of the presence of perimeter shear walls in reducing drifts and uncoupling modes is clearly observed, specially for service level earthquake loading. Period calculations using code empirical equations have recently 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 underestimation will result in equal or higher design shears; however, it may not give conservative design values if a specific site spectra is used. The use of the constant C_t in this equation assigned according the type of structural system generally gave results closer to the observed values.

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REFERENCES

1. ATC, "ATC-2, An Evaluation of a Response Spectrum Approach to Seismic Design of Buildings", 1974.
2. Boroschek, R., Mahin, S. and Zeris, C., "Engineering Interpretation of Responses of Three Instrumented Buildings in San Jose, California", SMIP Report in preparation.
3. Boroschek, R., Mahin, S. "Analytical Modeling and Corroboration of a Three Dimensional Structure", SMIP Report in Preparation.
4. Mahin, S., Boroschek, L., Zeris, C. "Engineering Interpretation of Responses of Three Instrumented Buildings in San Jose", Seminar on Seismological and Engineering Implications of Recent Strong Motion Data, 1989.
5. SMIP, "Processed Data from the Strong motion Records of the Morgan Hill Earthquake", Part II, OSMS-85-05, 1985.
6. Uniform Building Code (1985). International Conference of Building Officials.
7. Uniform Building Code (1988). International Conference of Building Officials.

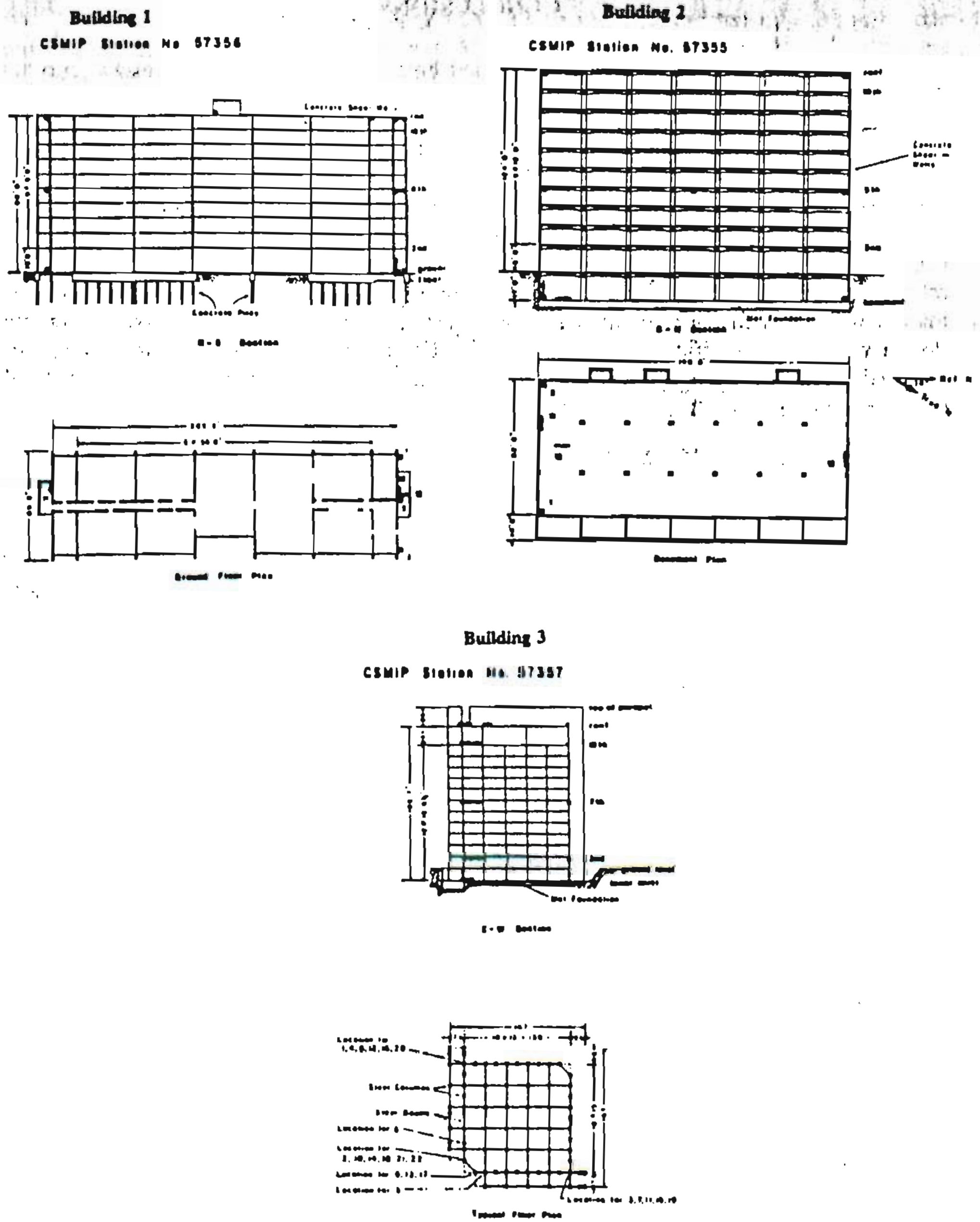

Figure 1 - Building plans.

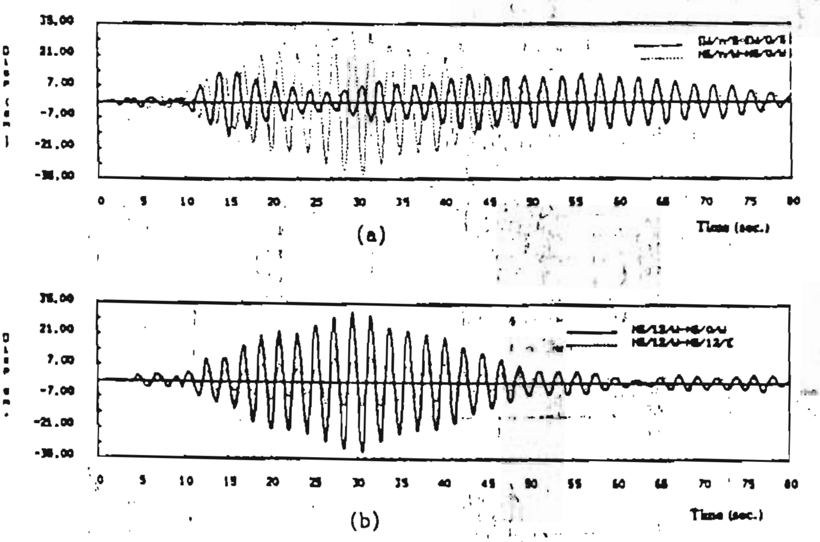

Figure 3 - Displacements for Building 3. Mt. Lewis Earthquake. a) Relative drift SW building corner. b) Twelfth floor relative drift and torsion, using NS records.

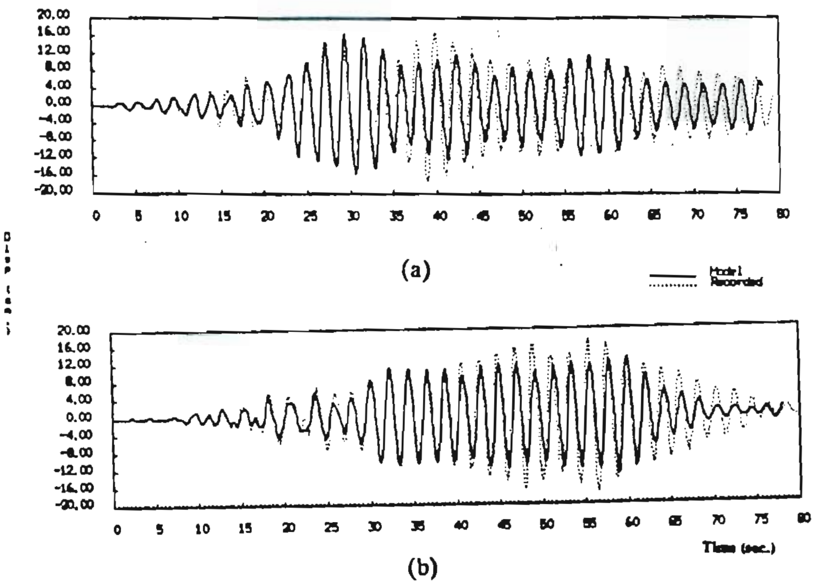

Figure 4 - Displacement for Building 3. Morgan Hill Earthquake. Relative drift twelfth floor: analytical model and recorded displacement. a) EW direction. b) NS direction.