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Demonstration of Torsional Coupling Caused by Closely Spaced Periods—1984 Morgan Hill Earthquake Response of the Santa Clara County Building

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The parameters of the dominant modes of vibration of the steel-framed Santa Clara County Office Building in San Jose, California, are determined using "the modal minimization method" for structural identification. The optimal estimates of the model parameters are determined by minimizing a selected measure-of-fit between the responses of the structure and the model. Two types of models are used: (1) A planar linear model with classical damping and (2) A three dimensional linear model consisting of rigid floor decks, where each floor is allowed three degrees of freedom - two orthogonal translations plus a rotation. The Santa Clara County Office Building continued vibrating in a free vibration manner with very low damping, long after the intense part of ground motion had ended. The records of its torsional motion exhibit a strong beating effect which is explained by the strong coupling of torsional and translational modes of vibration. Such a strong coupling of modes of vibration is attributed to the proximity of the value of torsional stiffness to that of translational stiffnesses.

INTRODUCTION

In recent years, numerous records of the dynamic response of structures to earthquake excitation, have been obtained and are available for analysis. Such data offer an opportunity to make a quantitative study of structural behavior at force and deflection levels directly relevant to earthquake-resistant design. A large group of structural response records were those obtained in about sixty buildings during the 1971 San Fernando earthquake (M_L=6.3) in California. These data provided the impetus for applying ideas from system identification theory in earthquake engineering. Although various procedures of structural identification have been employed in earthquake engineering since then, it is only recently that a fully systematized technique has been proposed. The technique, called "the modal minimization method", has been developed and implemented by Beck (1), and can be classified as an output-error method in the time domain. More specifically, the estimation of the optimum parameters of the mathematical model is achieved by using a suitable computer algorithm

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which minimizes some positive-definite measure-of-fit between the structural output and model output by systematically varying the model parameters. In a complementary study, McVerry (7) has applied an ouput-error approach in the frequency domain.

The San Fernando earthquake structural response data, although no doubt very valuable, can provide only limited information about the dynamic response of the structures which they were obtained from. The reason for this is the limited number of instruments that were deployed in each building (three three-component instruments usually located in the building's basement, near its midheight and roof). Very recently, as part of the California Strong Motion Instrumentation Program (CSMIP), the California Divison of Mines and Geology (CDMG) has instrumented various modern structures at various sites in the State of California. Of these heavily instrumented structures, the multi-story buildings are instrumented at several floors and at each floor more than one instrument are deployed so as to obtain a more complete picture of the dynamic structural response (including torsional motions) under earthquake excitation.

An opportunity to record the dynamic response of such well instrumented structures came about during the recent Morgan Hill earthquake which occurred on April 24, 1984 on the Calaveras fault southeast of San Jose, California. The earthquake triggered the strong motion accelerographs at nearly fifty stations. The stations include twenty-three extensively instrumented structures and twenty-five ground-response stations (2).

This paper deals specifically with the records obtained from a structure in San Jose, the Santa Clara County Office Building. The structural displacements and differential motions studied here are from the CSMIP processed-data report on Morgan Hill structural records (6). We extended Beck's "modal minimization method" to 3-D response analysis and we applied it in the analysis of the response data of the Santa Clara County Office Building in order to infer optimal parameters of a linear model of the structure (i.e., damping ratios, natural periods, modal shapes, etc.) and to better understand the dynamic response of this building.

SANTA CLARA COUNTY OFFICE BUILDING, SAN JOSE, CALIFORNIA

The 13-story Santa Clara County steel-framed building shown in Figure 1 was constructed in 1976. The structure has exterior cores at the south and west ends, a four-story atrium at the southwest corner and the lateral force-resisting system of the building is composed of moment-resisting steel frames. The vertical load-carrying system consists of concrete floor slabs overlying decks supported by steel framing. The foundation is a concrete mat (9,11). A total of twenty-two accelerometers were installed at locations throughout the building; the sensor locations are shown in Figure 1 (6). Pairs of sensors pointing east-west are placed at the lower level, 2nd, 7th and 12th floors. These pairs are complemented by pairs sensing north-south motion. This sensor layout is intended to record translational as well as torsional motions at these levels.

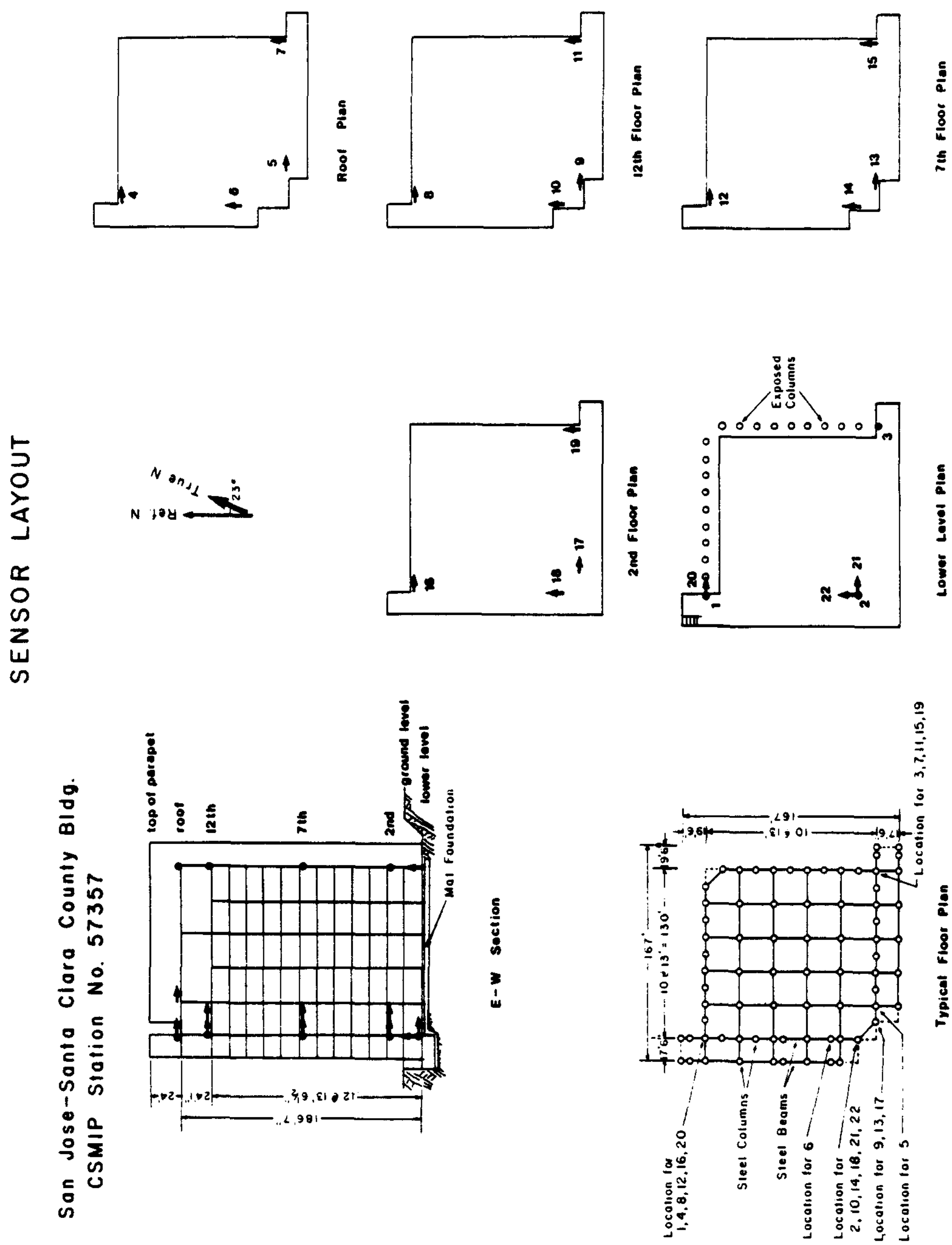

Figure 1 - Sensor layout in Santa Clara County Office Building (6).

During the Morgan Hill earthquake, the building oscillated for a long period of time (~80 seconds) (Figure 2). There was some nonstructural and content damage and very limited structural damage to this building. Visual inspection of the records shown in Figure 2 reveals that the predominant period of translational vibration of the building is approximately 2 seconds, while the torsional response has a period of vibration slightly shorter than 2 seconds. The building continued vibrating in a free vibration manner, apparently with low damping, long after the intense part of ground motion had ended. Particularly striking is the strong beating effect observed in the torsion records (Figure 2) which suggests the existence of two harmonics with frequencies which are very close to each other.

We performed two types of analysis for this building:
(1) One-dimensional (1-D) analysis considering the basic planar linear model with classical damping (see for example, p. 556, Clough and Penzien), and (2) Three-dimensional (3-D) analysis considering an idealized model consisting of rigid floor decks supported on massless axially inextensible columns and walls, where for each floor are allowed three degrees of freedom -two orthogonal translations plus a rotation.

Because the signals from the 22 sensors in this building were recorded with a common time base on two separate recorders (sensors 1-12 are connected to one recorder and 13-22 are connected to the second recorder) (6), we had to optimally synchronize the records first (8) and then proceed with the analysis.

In the following we present the results of these analyses.

ONE-DIMENSIONAL ANALYSIS:

For each of the two orthogonal directions, N-S and E-W, we estimated the dynamic parameters of the structure first by fitting a linear time-invariant model over the time segment 5-80 seconds, covering almost the entire duration of excitation and response. The parameter estimates are summarized on Table 1. For each direction there are two sets of estimates of the parameters. For the first set we used only one mode (out of the thirteen modes) to synthesize the response, while for the second set of parameters we used two modes. The quality of the fitting for the two-mode model is shown in Figure 3. Although the overall fitting is satisfactory, the matching of the model to the data deteriorates near the end of the response. This is particularly obvious in the N-S direction and less so in the E-W direction. This discrepancy can be easily explained by a reduction of the stiffness of the building and a consequent lengthening of the fundamental period of the structure as the excitation progresses. This change of the properties of the structure cannot be accomodated by the linear time-invariant model which we used, and this explains the missmatch observed in Figure 3.

To account for the lengthening of the fundamental period of the structure in N-S direction we segmented the excitation and response time

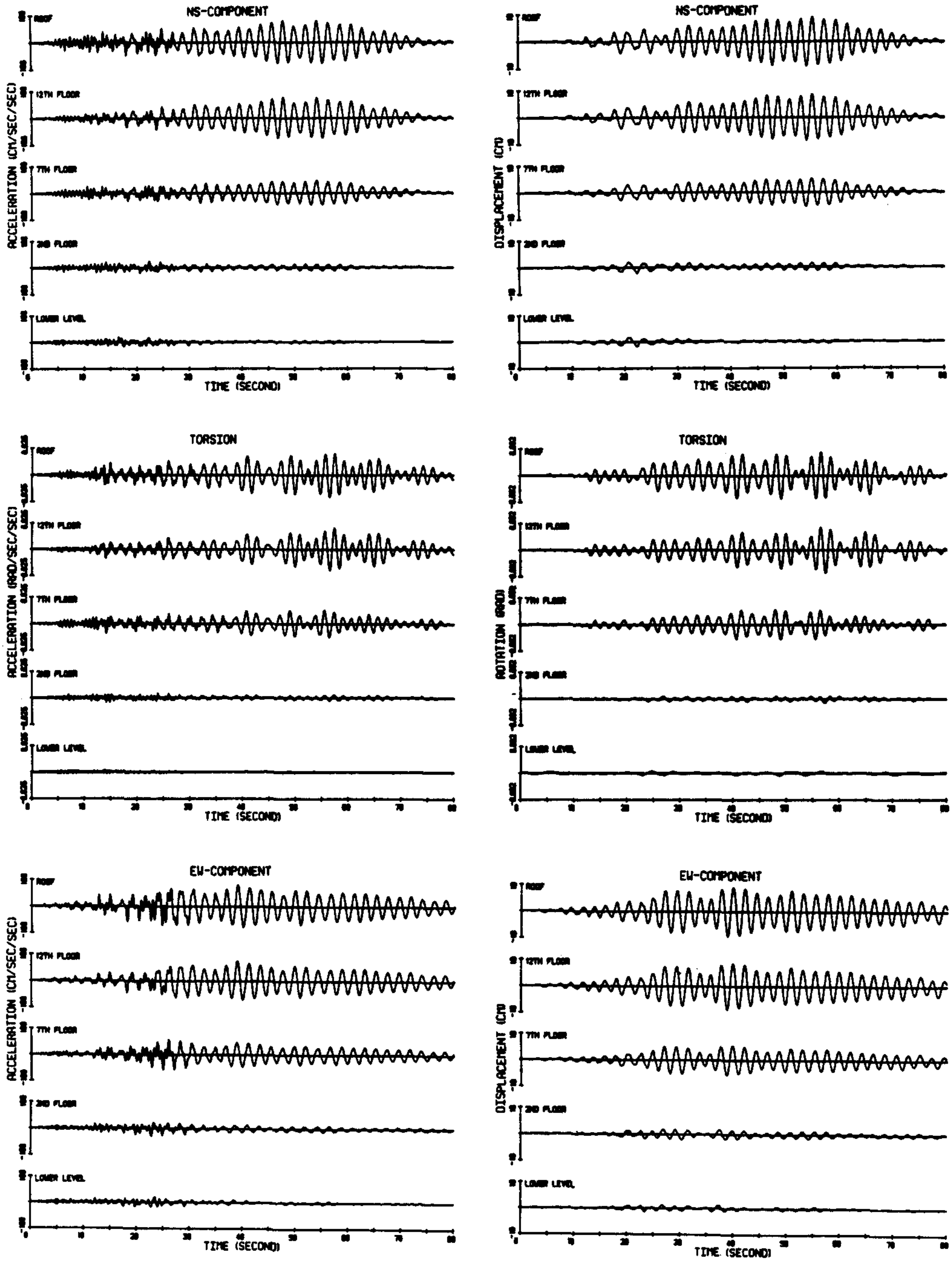

Figure 2 - Acceleration and displacement profiles of the translational and torsional motions of the Santa Clara County Office Building.

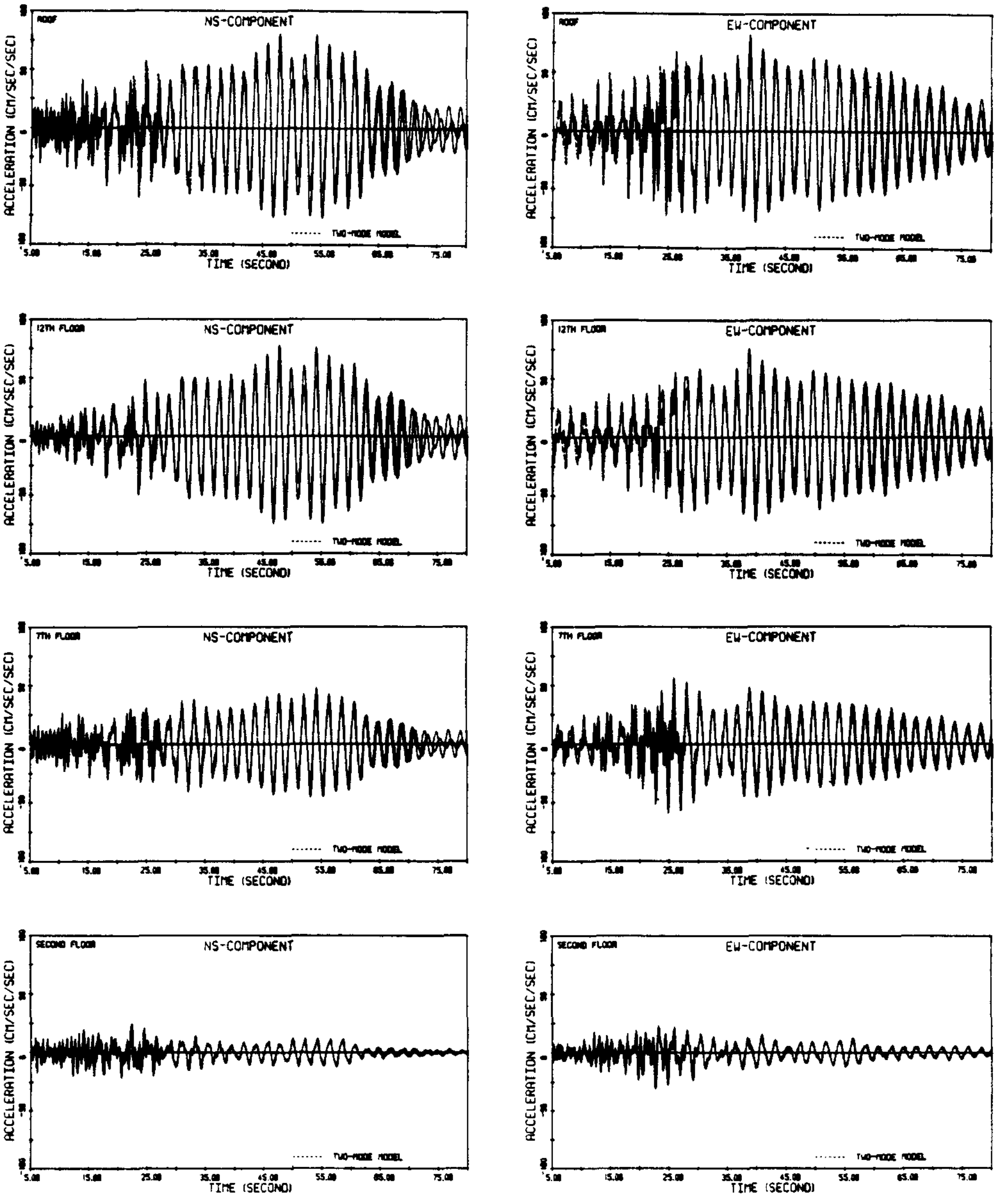

Figure 3 - Recorded response (——) and calculated response (---) of the optimal 1-D model with two modes determined by matching accelerations over the interval 5-80 seconds.

histories in that direction and fitted separate linear-time invariant models for each of segment. The results, summarized in Table 2, make evident the lengthening of the fundamental period of the building as the earthquake excitation progresses. Finally, in Figure 4 the modal shapes of the first two modes for each of the two orthogonal directions N-S and E-W, are compared with the modal shapes of a shear beam model with uniform mass and stiffness distribution.

TABLE 1
1-D Model Results for Time Interval 5-80 Seconds

	N-S		E-W	
T_1 (sec)	2.06	2.05	2.16	2.16
ζ_1 (%)	2.0	2.4	1.1	1.1
T_2 (sec)		0.69		0.71
ζ_2 (%)		3.8		3.6

THREE-DIMENSIONAL ANALYSIS:

Using the results of the 1-D analysis as a guide, we performed a three-dimensional analysis by segmenting the excitation and response time histories and fitting separate 3-D linear time-invariant models to each segment. The equations of motion of the mathematical model were formulated assuming that the centers of mass of all the floors lie on a vertical axis. The inferred parameters are summarized in Table 3 and typical samples of the quality of fitting of the model to the data are shown in Figures (5a,b,c). The modal shapes, as inferred from the analysis of the time segment 5-20 seconds, are shown in Figure 6. The modal shapes inferred from later time segments were ignored because, as pointed out by Beck (1), the determination of the effective participation factor (= the product of the participation factor times the element of the modal vector corresponding to the floor the motion of which is computed) is ill-conditioned for later portions of the records. This is because the basement acceleration is small for these time intervals and the structural motion is dominated by the free-vibration component which does not depend on the effective participation factor.

From Table 3 we detect again the lengthening of the fundamental period of the structure which we have observed in the 1-D analysis. From Figure 6 it becomes apparent that the 1st, 2nd, 4th and 5th modes have almost exclusively translational components, while the 3rd and 6th modes show a strong coupling of all three kinds of components (i.e., translational motions in the two orthogonal directions N-S and E-W as well as rotational motions) although the torsional component appears to be the

TABLE 2
I-D Model Results for Consecutive Time Intervals (N-S Direction)

	5-20 sec		20-40 sec		40-60 sec		60-80 sec	
T_1 (sec)	2.00	1.99	2.00	2.00	2.06	2.06	2.13	2.13
ζ_1 (%)	3.1	3.1	0.9	0.9	1.4	1.3	3.6	3.6
T_2 (sec)		0.68		0.69		0.73		0.69
ζ_2 (%)		2.6		5.4		0.0		2.0

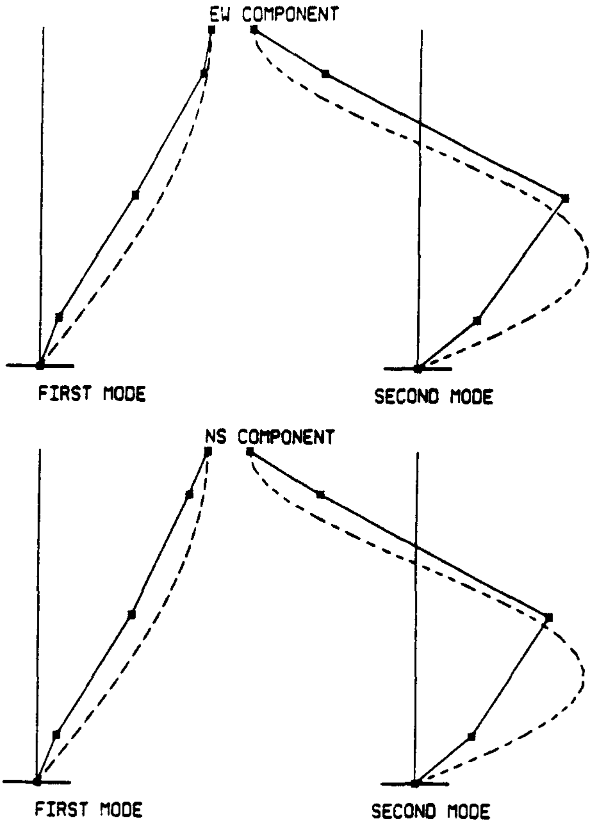

Figure 4 - The modal shapes which were determined by fitting 1-D models to the recorded response over the interval 5-80 seconds in the two orthogonal directions N-S and E-W. The modal shapes of a shear beam model are shown for reference.

TABLE 3
3-D Model Results for Consequtive Time Intervals

	1st mode	2nd mode	3rd mode	4th mode	5th mode	6th mode
		5-20 seconds				
T_r (sec)	2.07	2.04	1.65	0.70	0.61	0.58
ζ_r (%)	0.2	3.7	0.0	0.30	12.7	0.0
		20-40 seconds				
T_r (sec)	2.14	2.09	1.68	0.72	0.69	0.57
		40-60 seconds				
T_r (sec)	2.16	2.11	1.66	0.71	0.70	0.56
ζ_r (%)	12.1	1.0	1.9	10.6	0.6	4.3
		60-80 seconds				
T_r (sec)	2.10	2.15	1.69	0.72	0.70	0.58
ζ_r (%)	4.5	1.1	2.1	0.1	2.5	2.4

dominant. Stating this differently we could say that the 3rd and 6th modes are the first and second torsional modes of the structure. Examining the modal shapes which we inferred from the time segment 5-20 sec we see that the motion of the building, as it vibrates in the 1st and 4th modes, is along its diagonal which is oriented towards the NE-SW direction while in the 2nd and 5th modes the translational motion of the building is in the NW-SE direction. This behavior of the building can be explained by the presence of the two cores on the south and west side of the structure.

Finally, the strong beating effect which we observed in the torsional component of motion (Figure 2) is explained by the strong coupling of the first torsional mode (= 3rd mode in Figure 6) to the second translational mode (= 2nd mode in Figure 6) (5). More specifically, the translational mode has a period of ~2.10 sec while the torsional mode has a period of 1.66 sec. Two harmonic signals with these periods when combined produce a beating effect with a period equal to ~16 sec which is equal to the period of the beats we observe in Figure 3.

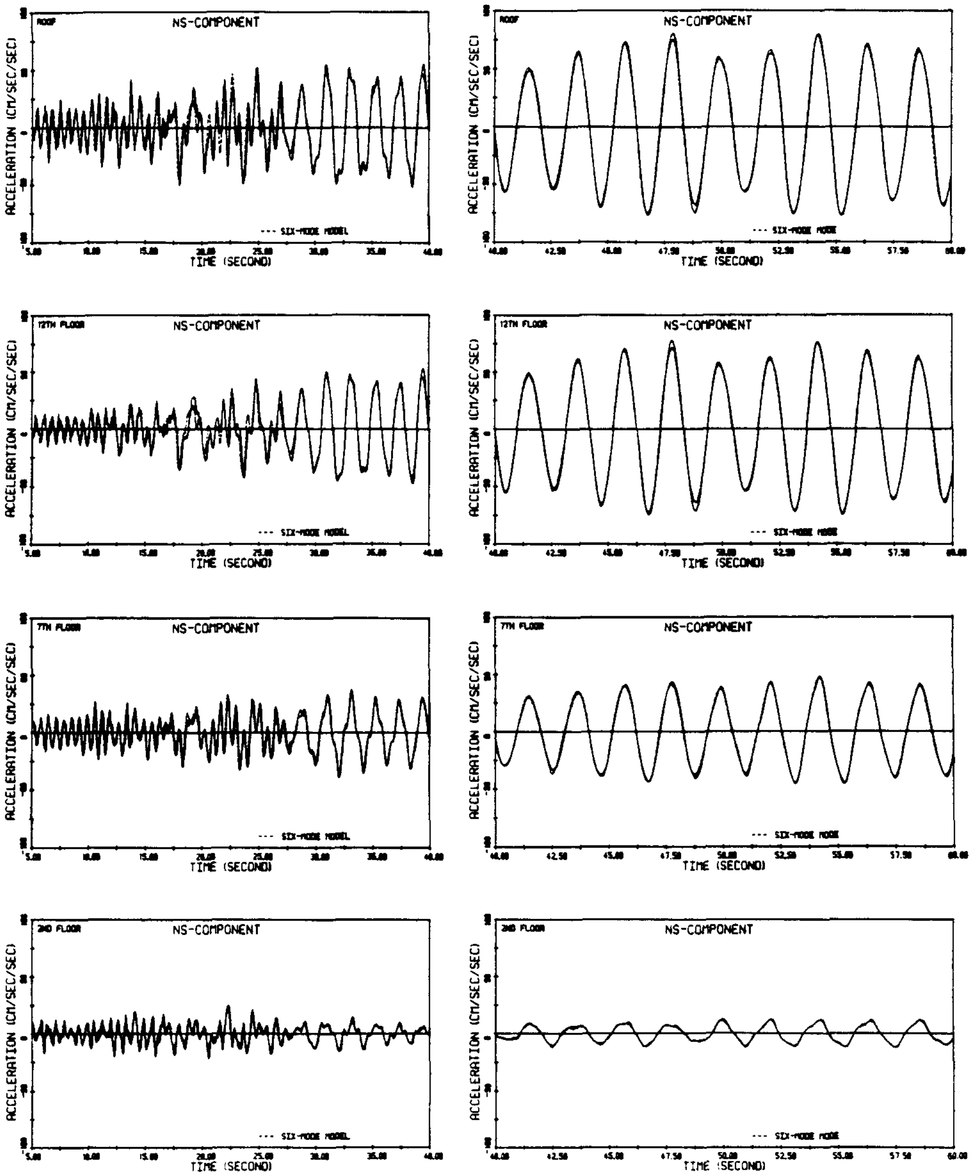

Figure 5a - Recorded response (——) and calculated response (---) of the optimal 3-D model with six modes determined by matching accelerations over the intervals 5-40 secs and 40-60 secs : NS component of response.

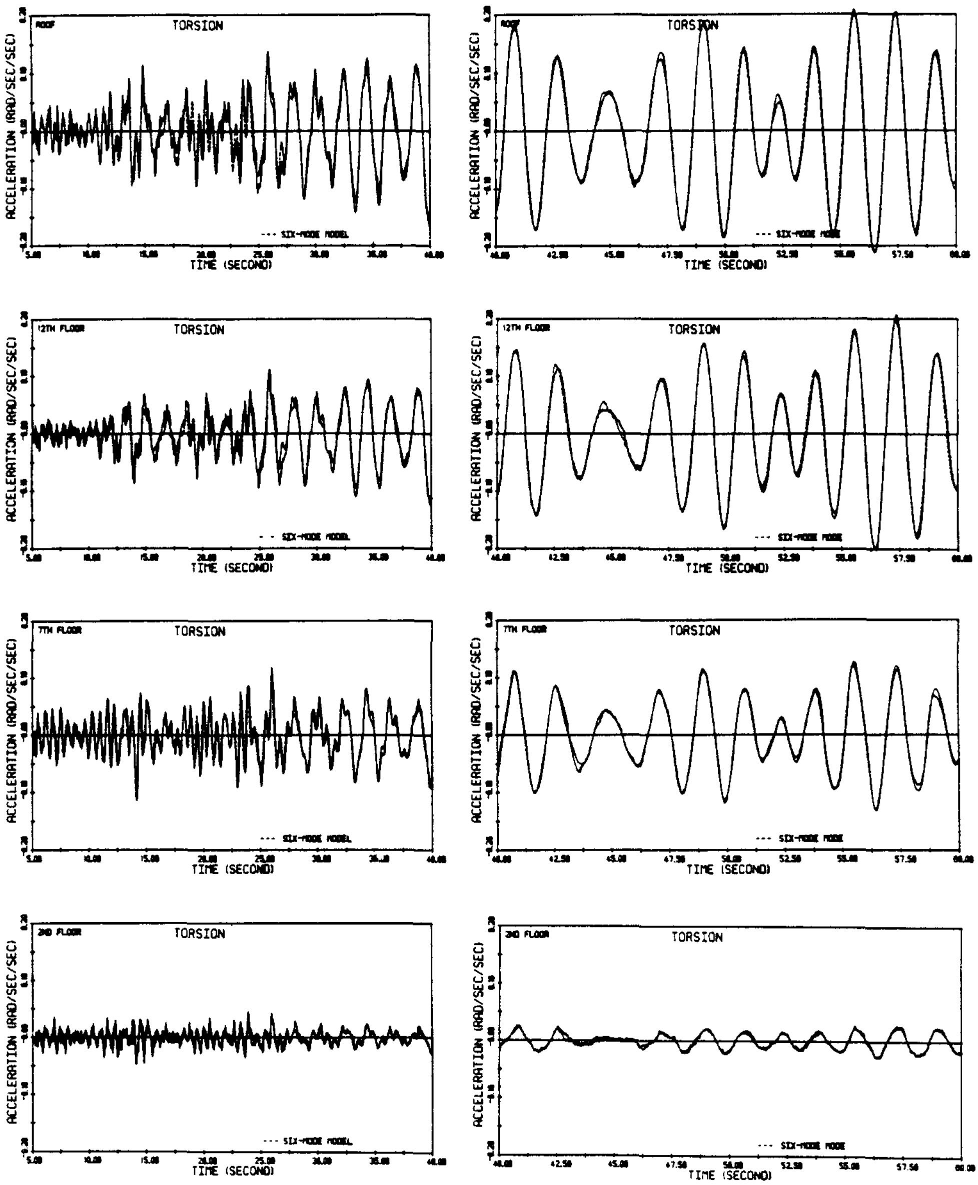

Figure 5b - Same as Fig. 5a : Torsional component of response.

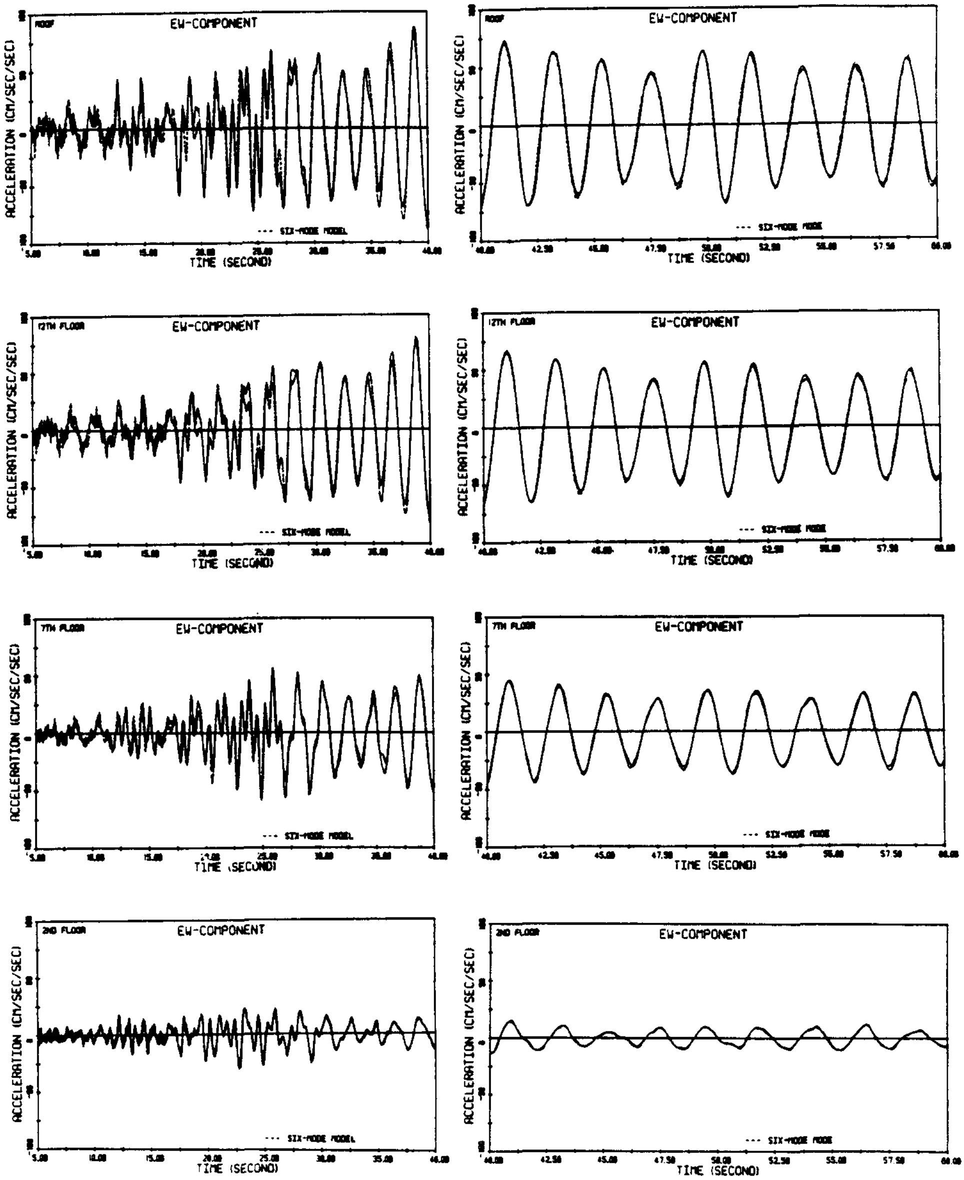

Figure 5c - Same as Fig. 5a : EW component of response.

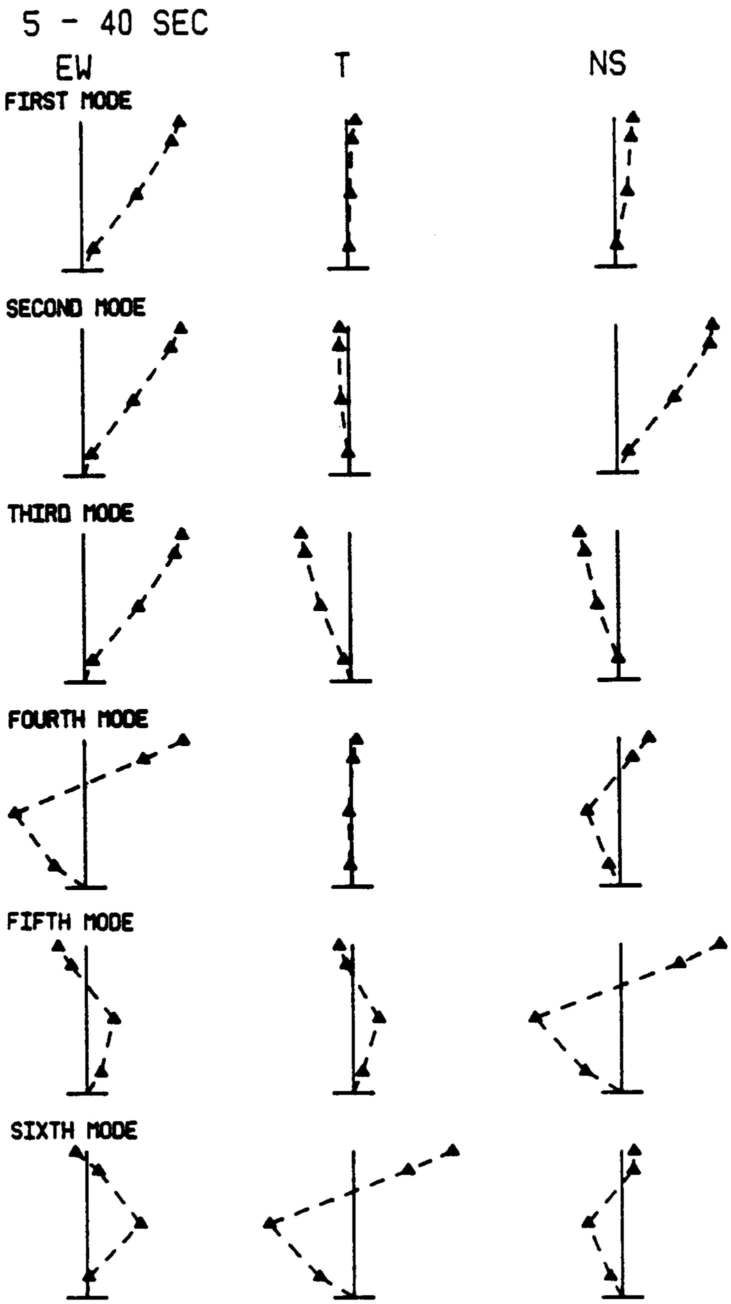

Figure 6 - The modal shapes which were inferred by fitting a 3-D model to the recorded response over the interval 5-20 seconds.

DISCUSSION

Of particular interest is the comparison of the response of the steel-framed Santa Clara County Building - the dynamic response of which we analyzed in the previous section - with the response of a well instrumented concrete structure, the Great Western Savings Building, which is located within 2 km from it. This building has a combination of concrete shear walls and moment resisting frames in two orthogonal directions (Figure 7). The acceleration and displacement profiles of the recorded translational and torsional motions of the building are shown in Figure 8. Shakal and Huang (10) point out that the strong torsional response (amplification ratio, root to base, was greater than a factor of five) of the Santa Clara County Office Building is in contrast to the insignificant torsional response of the Great Western Savings Building. Thus they conclude that the torsional motion of the steel-framed building is not simply due to compliance of the building with a torsional component of the incoming wave field, but rather represents a torsional response of the building to lateral motion at the base.

An analysis of the dynamic response of the Great Western Savings Building has been performed by Papageorgiou and Lin (1988). They found that the fundamental transverse (E-W), longitudinal (N-S) and torsional natural frequencies are 1.6 Hz, 1.10 Hz and 2.60 Hz. Clearly these frequencies are well separated, in contrast to the corresponding frequencies of the Santa Clara County Office building which were found to be very closely spaced to each other. This can be verified also by visually comparing the recorded responses of the two structures shown in Figures 2 and 8.

The dynamic behavior of the above mentioned two buildings could have been anticipated on the basis of the results of the analytical study presented by Hoerner (5). As he points out, rectangular buildings with an even dispersion of lateral force-resisting element (e.g., columns or walls) have translational and torsional stiffnesses (and corresponding natural frequencies) which are close to each other. Such buildings could have significant torsional response to translational excitation, as in an earthquake, and their response exhibits a strong beating effect which is explained by the strong coupling of torsional and translational modes of vibration. The Santa Clara County Office Building may be classified as belonging to the above category of buildings. On the contrary, the Great Western Savings Building is a nominally symmetric (i.e., has small accidental eccentricities) rectangular building with peripheral shear walls. As Hoerner (5) clearly states, such buildings have translational and torsional stiffnesses that are well separated, there is little modal coupling. Consequently no beating phenomena are present in the response and the torsional response to translational excitation is small.

Finally, even though consideration of the rocking of the foundation (which was ignored in the present analysis) may introduce small perturbations in the values of the parameter of the fundamental modes, the observations made above about the coupling of lateral and torsional responses are still valid and are not affected by the flexibility of the foundation medium [12].

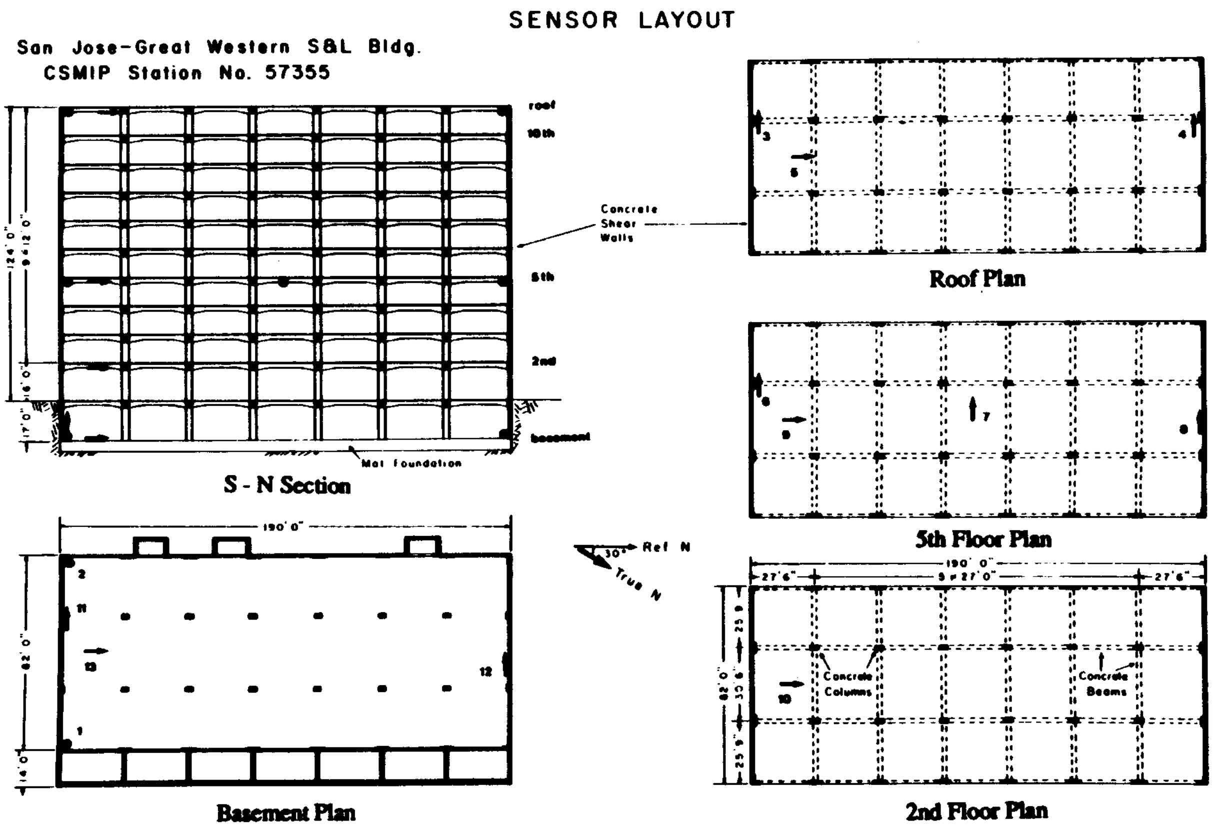

Figure 7 - Sensor layout in the Great Western Savings and Loan Building.

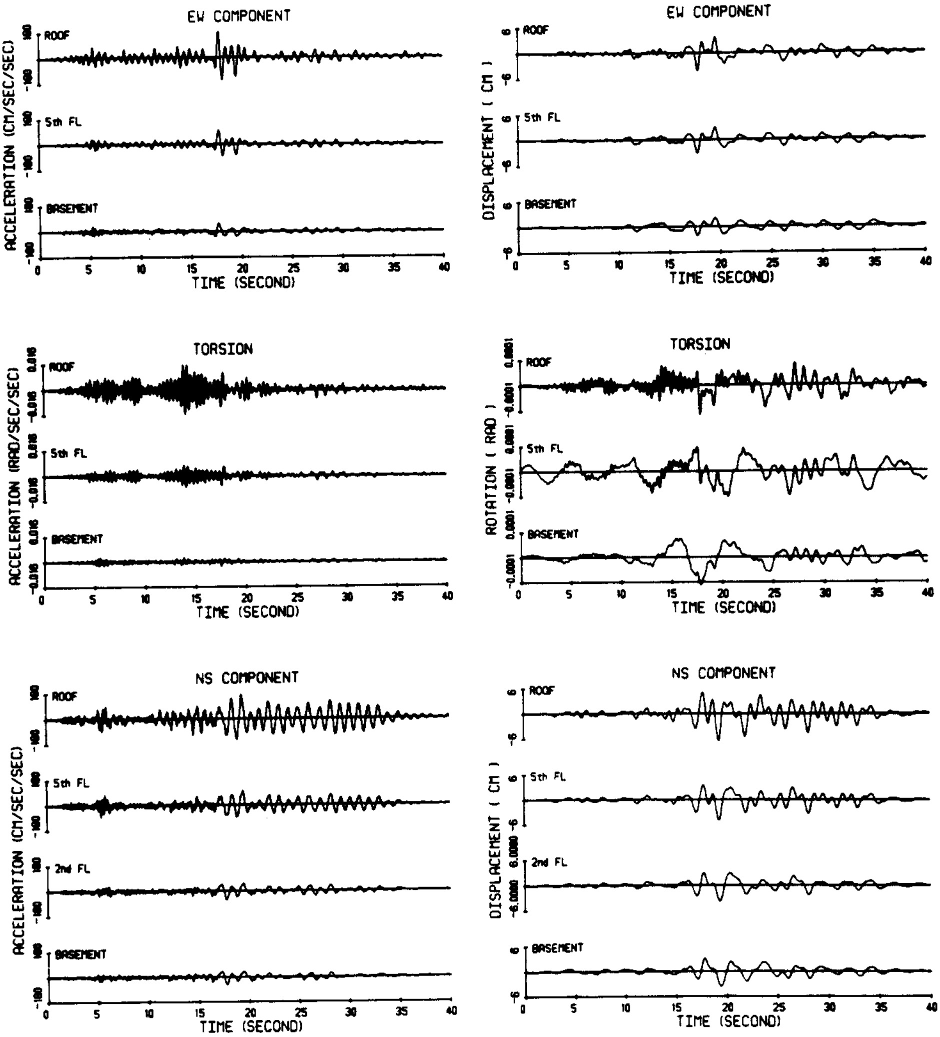

Figure 8 – Acceleration and displacement profile of the response of the Great Western Savings and Loan Building.

CONCLUSION

As has been pointed out above, structures - such as the Santa Clara County Office Building - which have closely spaced torsional and translational natural frequencies exhibit strong beating-type phenomena in their response. Such strong modal coupling makes earthquake response of structures different than that envisioned by codes. Thus, the traditional Square-Root-of-Sum-of-Squares (SRSS) rule for combining modal maxima - applicable to well-separated frequencies - may lead to significant errors. Other rules, such as the Complete-Quadratic-Combination (CQC) rule [13] (which degenerated into the SRSS rule for systems with well-spaced natural frequencies) have been recommended in the literature as better estimators of the true maximum response.

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