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THE IMPORTANCE OF ROCKING IN BUILDING MOTION: AN EXPERIMENTAL EVIDENCE

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SUMMARY

The rocking motion of buildings is investigated in both the time and frequency domains on 7 CSMIP strong motion data sets corresponding to 4 buildings and 2 earthquakes. For the 3 stiffer buildings, a very high coherence is found between the rocking of the base and the transverse motion relative to the base, measured at any level, indicating a significant soil-structure interaction. For these 3 buildings, the measured rocking motion represents 25 to 50% of the total transverse motion of the roof relative to the base.

INTRODUCTION

The California Division of Mines and Geology, the organism in charge of the California Strong Motion Instrumentation Program, has instrumented several buildings with two or more vertical accelerometric sensors at the base level in order to obtain a measurement of their rocking motion. Four such buildings located south of the San Francisco Bay experienced a significant shaking during the April 24, 1984, Morgan Hill earthquake (M_L=6.2, noted earthquake # 1 below), and the corresponding records are presented in some detail in Ref. 1; moreover, three out these four buildings also experienced a comparable shaking during the March 31, 1986 Mt Lewis earthquake (M_L=5.8, earthquake # 2 below).

This paper presents the results of a careful analysis of these 7 strong motion data sets, leading to a reliable measurement of the respective parts of rocking and flexing in the total horizontal motion of these 4 buildings. After a short description of their main structural characteristics, the data analysis technique is detailed on the example of one particular building, and the results obtained for the whole data set are then presented and briefly discussed.

BUILDING CHARACTERISTICS

Building # 1: Town Park Towers Apartment Building (San Jose) *(Figure 1)*. This is a rectangular, 10 story apartment building, built on an alluvial soil with precast-prestressed concrete piles located under each walls. The vertical load carrying system consists of one-way post-tensioned flat slabs bearing on reinforced concrete (rc) walls; the lateral force resisting system consists of RC shear walls located at regular intervals for the transverse motion (the one we are interested in here), and RC shear walls along interior corridors in the longitudinal direction (stepped at 6th floor) (see Figure 1).

It is instrumented with 13 sensors connected to a central unit and having a common trigger; as we are interested in the transverse rocking motion, we only considered, in the present study, sensors # 1 and 2 (vertical sensors at the base level on south wall), and sensors 12, 9 and 6 (transverse sensors located, respectively, at the base, 6th floor and roof levels of this wall).

The fundamental transverse period of this building, measured from these strong motion data (Ref. 2), is 0.40 s; the corresponding mode shape exhibits a significant amount of torsion (30 to 50 %, depending on the earthquake, see Ref. 2).

Town Park Towers Apartment Building (San Jose) - CSMIP station # 57356

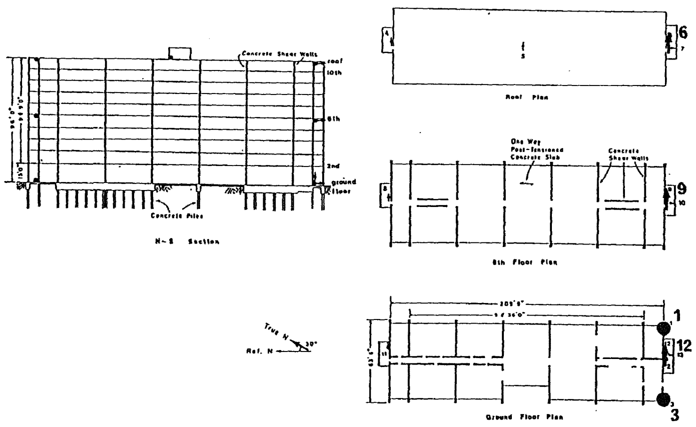

Figure 1 : Structural characteristics and sensor layout of building # 1.

Santa Clara County Office Building (San Jose) - CSMIP station # 57357

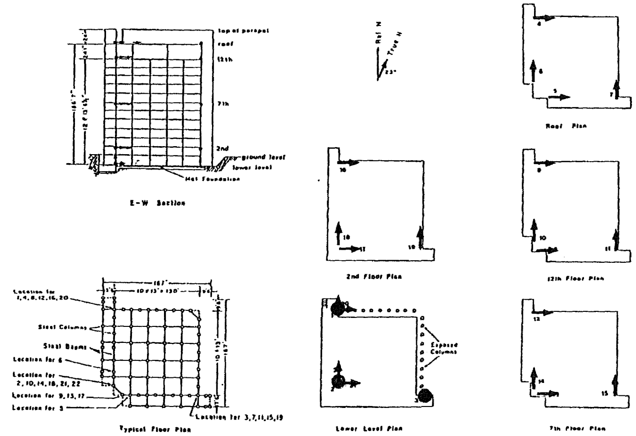

Figure 3 : Structural characteristics and sensor layout of building # 3.

Great Western Savings and Loan Building (San Jose) - CSMIP station # 57355

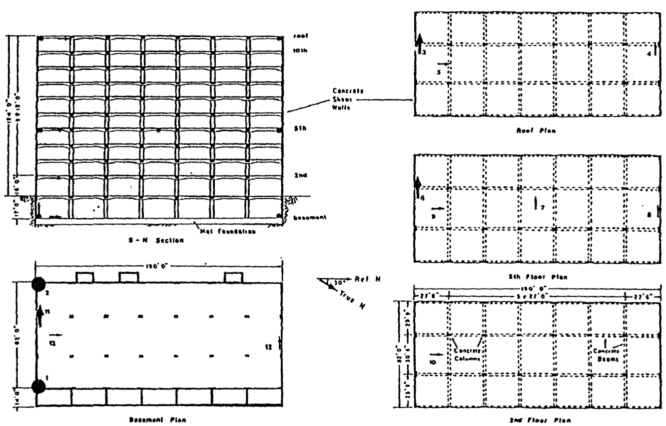

Figure 2 : Structural characteristics and sensor layout of building # 2.

Telephone Building (Watsonville) - CSMIP station # 47459

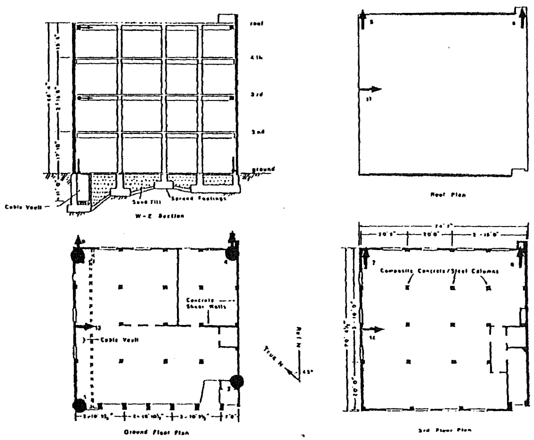

Figure 4 : Structural characteristics and sensor layout of building # 4.

Building # 2: Great Western Savings and Loan Building (San Jose) *(Figure 2)*. This is again a rectangular, 10 story office building, built on the same alluvial soil as building #1 with an embedded, rectangular mat foundation. The vertical load carrying system consists of RC floor slabs supported by rc pan joists supported by a RC frame; the lateral forces are resisted by the end shear walls in the transverse direction, and by a moment-resistant RC frame in the longitidunal direction.

It is instrumented with 13 sensors, but, as for building #1, we considered here only 5 of them (vertical sensors 1 and 2, and transverse sensors 11, 6 and 3).

The fundamental transverse period, measured from strong motion data (Ref. 2), is 0.60 s.

Building # 3: Santa Clara County Office Building (San Jose) *(Figure 3)*. This is a square, 12 story steel building, built on the same alluvial soil with a mat foundation. The vertical load carrying system consists of concrete slabs on metal decks and of steel columns, beams and joists; the lateral forces are resisted by a moment-resistant steel frame.

It is instrumented with 22 sensors, amongst which 3 vertical sensors at the base level allow to investigate the rocking motion in both horizontal directions, through the use of all sensors.

The fundamental period, measured from strong motion data (Ref. 2), is about 2.3 s in both directions (the two horizontal modes are obviously strongly coupled because of the structural and geometrical characteristics, which also result in a significant amount of torsion, about 25 %, see Ref. 2).

Building # 4:Watsonville Telephone Building *(Figure 4)*. This is a square, 4 story building founded on an alluvial soil with spread footings. The vertical loads are carried by concrete slabs and composite concrete/steel beams, girders and columns; the lateral forces are resisted concrete shear walls.

It is instrumented with 13 sensors, amongst which 4 vertical sensors located at each base level corner allow an estimation of the rocking motion in both horizontal directions, through the use of all sensors.

Its fundamental periods (Ref. 2) are 0.27 s in the EW direction and 0.19 s in the NS direction; the EW fundamental mode includes a large amount of torsion (about 50 %, Ref. 2).

The three San Jose buildings (# 1, 2 and 3) recorded the two strong motion events; building # 4 triggered only during the Morgan Hill earthquake. The epicentral distances and peak accelerations at base and roof levels in the directions of interest are summarized in Table 1, together with the building main structural characteristics, as indicated in Ref. 1.

Building #	Structural characteristics	Foundations Type	Foundations Soil	Number of stories above /below	Geometrical characteristics Height (m)	Geometrical characteristics Width (m)	Period T_0 (s)	Eq. #	epicentral distance (km)	peak acceleration (cm/s2) ground	peak acceleration (cm/s2) roof
1	RC shear-walls	Piles under the walls	Alluvium	10/0	29.3	19.4	0.40 (EW)	1	19	60.	135.
						(63.9)		2	23	35.	81.
2	RC shear-walls and frame	Mat foundation	Alluvium	10/1	37.8	25.0	0.60 (EW)	1	19.	60.	214.
						(57.9)		2	23	34.	76.
3	Steel frame	Mat foundation	Alluvium	13/0	49.5	54.0	2.3 (EW)	1	21	40.	170.
						54.0	2.3 (NS)	1		34.	166.
							(EW)	2	21.	28.	170.
							(EW)	2		43.	312.
4	Concrete Walls	Spread footings	Alluvium	4/0	20.2	21.5	0.19 (NS)	1	45.	60.	137.
						22.7	0.27 (EW)	1		107.	321.

Table 1 : Main characteristics of the data set.

DATA ANALYSIS TECHNIQUE

This section presents the data analysis technique on the particular example of building # 1.

Under the assumption of a rigid base, the rocking motion may be estimated from the difference between the respective vertical motions on each side of the end wall. This may be done indifferently on the acceleration, velocity or displacement records; the best readibility in the results has been obtained, however, with the acceleration records (processed, volume 2 records, as issued by CDMG, see Ref. 1), because they best sample the frequency range of interest:

$$\theta(t) = (v3(t) - v1(t)) / d$$

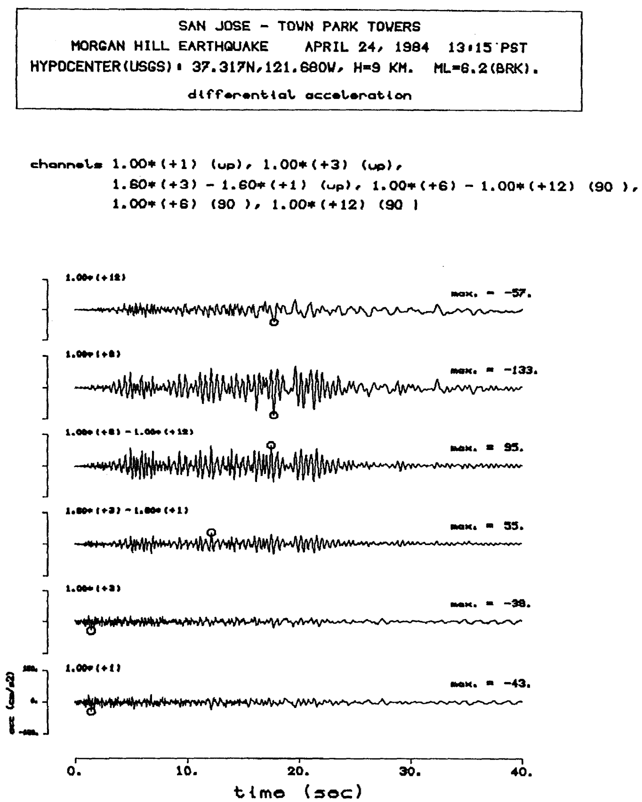

Figure 5: Original (processed) acceleration traces recorded by the vertical sensors (2 bottom traces) and the transverse sensors at base and roof levels (2 top traces), and comparison between the computed rocking motion at roof level (3rd trace from bottom), and the transverse motion of the roof relative to the base (4th from bottom).

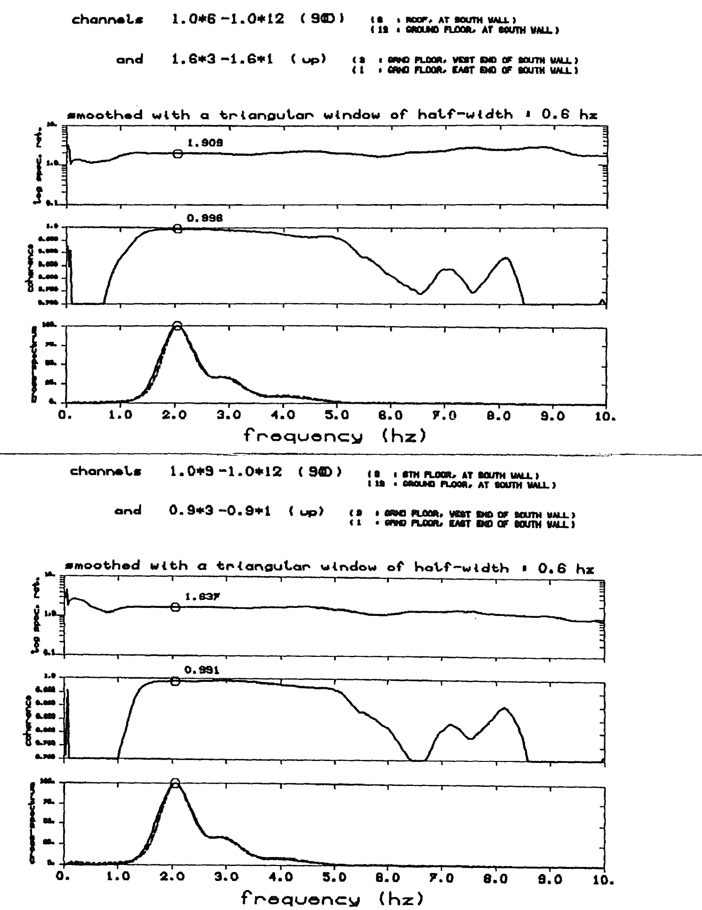

Figure 6: Normalized cross-spectrum, coherence and spectral ratios between the relative transverse motion at roof (top) and 6th floor (bottom) levels and the corresponding rocking motion. See text for further explanations.

where : $\theta(t)$ is the second derivative of the rocking angle, vi(t) is the acceleration history at sensor i, and d is the distance between the two vertical sensors.

If the base is flexible, the actual rocking motion of the building should be larger than $\theta(t)$, as was found in Ref. 3. In the remaining of this paper, we will assume that the base is rigid, and therefore obtain a lower bound for the rocking motion.

This rocking motion is to be compared with the total transverse motion at the same level relative to the base, which may be evaluated, on the present example, at the roof and 6th floor levels:

$$\mathrm{Troof}(t) = \mathrm{v6}(t) - \mathrm{v12}(t), \text{ and } \quad \mathrm{T6fl}(t) = \mathrm{v9}(t) - \mathrm{v12}(t)$$

Such a comparison is depicted on Figure 5 (for the roof level only), which shows very clearly that the rocking motion and the transverse motion have very similar signal shapes. This striking similarity is quantified in Figure 6 by the display of the coherence between both signals. This coherence has been computed from the Fourier auto- and cross- spectra of both signals, computed over their whole duration (40 s), and smoothed with a triangular running window having a 0.6 Hz half-width. Despite this broad smoothing window, the coherence reaches very high values (larger than 0.99 around 2 Hz), and remains very good between 1.5 and 4 Hz.

In this frequency band, the spectral ratio between both signals (also shown on Figure 6) is very flat, which allows a both very reliable and meaningful (because of the high coherence value) measurement of the percentage of rocking in the transverse relative motion: the so obtained values reach 52 % at roof level, and 62 % at 6th floor level, which means that the rocking motion is here more important than the flexing motion.

These both high coherence level and large amount of rocking prove the existence of a very strong soil-structure interaction, which, as well-known, induces a shift towards longer periods of the fundamental transverse mode compared with the fixed-base flexing mode. It would be of primary interest to derive from the present data a quantitative estimate of this shift, but this requires additional information on mass distribution within the structure and actual mode shapes and is beyond the scope of the present paper. It is possible, however, to obtain a very rough approximation of this period shift (neglecting the translational SSI) with the simple formula obtained for SDOF systems (presented in Ref. 4 for instance):

$$T_0/T_f = \sqrt{(1. + \mathrm{rock}/\mathrm{flex})}$$

In the present case, as the rocking is slightly larger than the flexing, we obtain a shift well above 40 %.

GLOBAL RESULTS: PRESENTATION AND DISCUSSION

The same analysis has been applied to the 6 other data sets, and the results are summarized in Table 2, which calls for several comments:

(i) for three out of these 4 buildings, the coherence level between the rocking motion and the horizontal motion is, despite the large width of the smoothing window, extremely high (well above 0.99) at any level (roof or intermediate floors). As for building # 1, the coherence remains very good in a 2 to 4 Hz wide frequency band centered on the fundamental mode. This result is a proof of the existence of a strong soil-structure interaction for these 3 buildings, which are "common" buildings built on "common" soils.

The only building for which the coherence is bad, and therefore the soil-structure interaction weak, is the steel building, as expected: it is the most flexible ($T_0 > 2$ s) and it has the largest base dimensions.

(ii) for these three concrete buildings(# 1, 2 and 4), the rocking motion shows up to be significant to large, since it accounts for 25 to 50 % of the total horizontal motion at roof level, and for 22 to 62 % at intermediate (approximately mid-height) levels. In other words, for these 3 buildings, the rocking motion represents at least half and at most 1.5 times, the purely flexing motion. The difference in rocking percentages at intermediate and top levels is to be related with the shape of the fixed-base flexing mode.

(iii) For the 3 buildings having experienced 2 earthquakes, the results are very consistent from one earthquake to another, since the rocking percentage remains the same (within 3 %). One may notice, however, that this percentage is slightly but systematically lower for the smaller earthquake (Eq. # 2), and the question then arises whether this slight difference might be due to either the non-linear behavior of the soil, or to a softening of the structure following Eq. # 1. Answering this question is still premature in our opinion.

(iv) Building # 4 has a square foundation, but different lateral force resisting systems in the horizontal directions: it is therefore worth noticing that, as expected, the rocking motion percentage is the larger in the "stiffer" direction. Moreover, we also performed a rough estimation of the "pure" rocking periods with simple SDOF systems formulas, and found exactly the same value (0.15 s) for the two directions, which is in good agreement with the square shape of the foundation.

Table 2 : Structural characteristics and rocking behavior for the 4 buildings investigated.
(The rocking behavior of the steel building at intermediate levels was not considered because of its too small amplitude).

Building #	Structural characteristics	Foundations Type	Foundations Soil	Number of stories (above /below)	Level	Height /width ratio h/d	Coherence level	Percentage Rocking /relative hz motion (%)	Natural period (s)	Remarks
1	RC shear-walls	Piles under the walls	Alluvium	10/0	Roof	1.60	0.996	52	0.40	Eq. # 1
					Roof		0.997	49	0.40	Eq. # 2
					6th fl.	0.85	0.993	62	0.40	Eq. # 1
					6th fl.		0.997	57	0.40	Eq. # 2
2	RC shear-walls and frame	Mat foundation	Alluvium	10/1	Roof	1.72	0.995	40	0.60	Eq. # 1
					Roof		0.994	37	0.60	Eq. # 2
					5th fl.	0.85	0.997	49	0.60	Eq. # 1
					5th fl.		0.997	44	0.60	Eq. # 2
3	Steel frame	Mat foundation	Alluvium	13/0	Roof	1.40	0.804	1.8	2.3	Eq. # 1, EW comp.
					Roof		0.811	2.4	2.3	Eq. # 2, EW comp.
					Roof	1.40	0.938	1.3	2.3	Eq. # 1, NS comp.
					Roof		0.975	1.1	2.3	Eq. # 2, NS comp.
4	Concrete Walls	Spread footings	Alluvium	4/0	Roof	0.95	0.998	51	0.19	NS comp.
					Roof	0.88	0.999	26	0.27	EW comp.
					3th fl.	0.49	0.997	50	0.19	NS comp.
					3th fl.	0.46	0.997	22	0.27	EW comp.

There still is, however, a lot of questions to be answered on this set of data, concerning the validity of the rigid base assumption, the actual period shift and the damping change between the fixed-base structure and the whole soil and structure interacting system, the significance (if any) of the slight differences between the response to the two earthquakes, and, of course, the comparison with the expected behavior of these buildings as predicted by either the simple techniques recommended in building codes, or state-of-the-art SSI codes.

CONCLUSION

The soil-structure interaction has thus been proved to be significant to large for several "common" concrete buidings, 4 to 10 story high, built on "common" alluvial soils. Though the shift between fixed-base period and the period of the unteracting system was not measured, it may be estimated from gross SDOF theories to significantly exceed 20% in 3 out of 5 cases. As SSI is generally beneficial for structures, such a result may have significant engineering consequences for the aseismic design of many "common" buildings for which the SSI is very often disregarded. It needs, however, to be confirmed by similar studies on other sets of data: we hope the present results will encourage the intrumentation of many more buildings with several vertical sensors at the base level.

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