DYNAMIC AMPLIFICATION OF GROUND MOTIONS BY LOW-RISE, STIFF SHEAR WALL BUILDINGS

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ABSTRACT

Dynamic amplification was defined as the ratio of actual peak base shear to an equivalent rigid-body base shear. Peak base shear was determined by summing the product of mass-times-acceleration for every element of mass in the building. An acceleration distribution over the entire building was assumed in terms of recorded acceleration time histories at several locations. Time histories from four buildings were studied. Due to diaphragm flexibility primarily, the dynamic response of these buildings did not differ significantly from that which would have occurred from a five-to seven-story steel frame. Actual peak base shears for this class of buildings was as high as 1.76 times equivalent rigid body base shears.

OBJECTIVES OF STUDY

The original objective of this study was to analyze existing strong motion data of low-rise, unreinforced masonry shear wall buildings with flexible diaphragms, to determine the extent to which ground motions are amplified by wall. Since there is only one unreinforced masonry building heavily instrumented, and since that instrumentation is not as complete as desired, the principal objective was expanded to include all low-rise buildings with stiff (masonry or concrete) shear walls and to include the affects of wall and diaphragm flexibility. A secondary objective of this study was to determine if any improvements to exiting strong motion instrumentation plans were warranted.

DESCRIPTION OF THE EVALUATION PROCEDURE

The principal objective of this study requires that amplification of ground motion be defined. Amplification of ground motion can be defined in a number of ways. It can be an instantaneous ratio of building motion to corresponding ground motion, it can be some averaged ratio, it can be a distortion reference to a corresponding absolute motion, and so forth. The best measure of amplification of ground motion should relate to the quantities used to design this class of buildings.

Typically, low-rise, masonry and concrete shear wall buildings with flexible diaphragms are designed using a distribution of static-equivalent forces. The most commonly referenced single measure of this set of static-equivalent forces is the design base shear, V. Amplification of ground motion will therefore be defined as the ratio of peak base shear allowing wall and diaphragm flexibility to that which would occur if the building were a rigid body.

Specifically for this study, peak base shear is defined as the maximum, total, seismically induced force transmitted to the foundation by all shear walls and frames parallel to the motion being considered. Out-of-plane wall forces at the ground level are not included. Three peak base shears are defined (one real and two hypothetical). The first is the actual, measured, peak base shear (V3) including all of the contributions due to actual wall and diaphragm flexibilities. The second is a hypothetical, "what if", peak base shear (V2) that would have occurred if diaphragms were infinitely rigid. In this analysis, diaphragm motions are assumed to equal the average of the motions of the walls to which they are attached. The third is another hypothetical, "what if", peak base shear (V1) that would have occurred if all diaphragms and walls were infinitely rigid.

Peak base shears (for each building, for each earthquake, for each principal direction) were computed by numerically summing the product of each element of mass and its corresponding acceleration, over the entire building, for each increment of time, for the actual and the two hypothetical cases. It is relatively easy to determine the mass of every element. If the building has been instrumented well, it is also reasonably easy to approximate the acceleration of every element in terms of the measured accelerations. A building (whether high-rise or low-rise) is instrumented well if all significant motions excited by an earthquake, for all parts of the building, can be approximated with accuracy from the recorded motions. For the class of structures considered in this study, the building is instrumented very well if, for each principal direction (typically transverse and longitudinal) the horizontal motions at a) the mid-points of every diaphragm are recorded, b) the intersections of every shear wall line and every diaphragm are recorded, and c) the bases of shear walls are recorded. If base rocking motion is expected to be significant, then at least a pair of vertical motions must be recorded for each shear wall, as well. Certainly, some approximations can be made with fewer instruments (such as approximating the base motions of several parallel shear walls with the base motion of a single shear wall).

For example, consider the assumed acceleration distribution for a simple, one-story, rectangular warehouse building. It is instrumented well if, for each direction, the base motion of each end wall is recorded, the in-plane motion at the top of each end wall is recorded, and the mid-span horizontal diaphragm motion is recorded. End wall accelerations are assumed to vary linearly between the base and top motions. The horizontal motion of the roof diaphragm (and tributary mass of the out-of-plane walls) is assumed to vary as a half-sine-wave between end wall top motions. Acceleration distributions are constructed similarly for multi-bay and multistory buildings.

For each building, for each principal direction, for each earthquake, the three base shears defined previously were computed from the building properties and the recorded accelerations. For each time history, extreme values of V1, V2, V3 were identified (from the time histories V1(t), V2(t), V3(t) respectively).

DESCRIPTION OF BUILDINGS STUDIED

The basic criteria used to select buildings for this study were a) that the building have masonry or concrete shear walls and flexible diaphragms, b) that the building was sufficiently well instrumented so the acceleration distribution over the entire building could be determined with accuracy, and c) that strong-motion records for the building exist. Few buildings, other than those instrumented under the California Strong-Motion Instrumentation Program (CSMIP) are instrumented well enough to meet the above criteria. Therefore, those buildings in the CSMIP constitute the primary data base.

Only one unreinforced masonry building (for which strong-motion earthquake records exist) is instrumented in CSMIP. It is a 2-story rectangular building located in Gilroy, California. The strong-motion instrumentation, in both principal directions, does not meet the minimum requirements for this analysis. This building was included in this study, however, because it is the only building of its type, and some upper/lower bound analyses of its motions can be made that will yield at least some information regarding unreinforced masonry building behavior.

The second building is a rectangular reinforced concrete tilt-up warehouse building in Hollister, California with plan dimensions of 100 feet by 300 feet. The building has a wood, penalized roof system. The building is instrumented well. See Figure 7 for its elevations and Figure 8 for its instrumentation plan.

The third building is also a rectangular reinforced concrete tilt-up warehouse building in Redlands, California with plan dimensions of 90 feet by 235 feet. The building has a wood, penalized roof system. The building is instrumented well.

The fourth building is a 2-story rectangular office building in Milpitas, California with plan dimensions of 125 feet by 168 feet. It has precast, reinforced concrete tilt-up walls with large and uniformly spaced openings on all elevations. The second floor diaphragm has metal decking over open-web joists with a concrete topping. The roof diaphragm is plywood over wood framing. The building is instrumented well.

EARTHQUAKE STRONG MOTION RECORDS ANALYZED

The earthquake motions analyzed for this study are summarized in the following table:

		PEAK GROUND ACCELERATION		PEAK ROOF ACCELERATION		
BUILDING	EARTHQUAKE	LONG	TRANS	LONG	TRANS	EQ MAG
Gilroy URM	Loma Prieta (10/17/89)	.24	.28	.55	.98	7.1
Hollister Warehouse	Loma Prieta (10/17/89)	.36	.25	.45	.82	7.1
	Hollister (1/26/86)	.14	.12	.25	.30	5.5
	Morgan Hill (4/24/84)	.07	.08	.12	.25	6.2
Redlands Warehouse	Palm Springs (7/8/86)	.04	.05	.11	.13	5.9
Milpitas Building	Loma Prieta (10/17/89)	.14	.10	.59	.33	7.1

DISCUSSION OF RESULTS

For this paper the motions of only one building, the Hollister Warehouse, will be discussed. The complete results of this study will be presented by the Applied Technology Council in ATC-27.

Shown in Figure 1 are schematic elevations of the Hollister Warehouse. Shown in Figure 2 are the strong-motion instrument numbers and locations.

Results are presented in two ways. Shown on Figure 3 is a sample of the three inertial force time histories (V1(t), V2(t), and V3(t)) defined previously that were obtained, for each building, for each earthquake studied, and for each direction of motion considered (transverse or longitudinal). The inertial total force time histories do not equal precisely base shear time histories. The base shear time history equals the inertial time history plus a velocity dependent (damping force) time history. However, at the peaks of a total inertial force time history, the velocity is zero. Therefore, the PEAK inertial forces do equal the PEAK base shears. On the figure, the peak values of V1, V2, and V3 are identified. Shown on Figure 4 are the peak base shears (equal to the peak inertial forces) for the Hollister Warehouse normalized with respect to the peak rigid body base shear, V1 (the measure of amplification in this study due to wall, or wall-plus-diaphragm, flexibility).

Amplification due to wall flexibility only is evaluated with peak base shear, V2. For both directions of motion, for all earthquakes, V2 maximum observed was 1.14. Generally, for this class of buildings, there is insignificant amplification due to wall amplification alone. The 14% amplification observed for transverse motions of the Hollister warehouse for the Morgan Hill earthquake may be due to base rocking more than due to wall flexibility, but it was not observed in the other two earthquakes studied for this warehouse. It comes as no surprise that amplification due to wall flexibility alone for this class of buildings, is insignificant.

Amplification due to wall and diaphragm flexibility (the actual behavior) for warehouse structures can be very significant, depending upon the aspect ratio of the diaphragm. For longitudinal motions (a width to depth aspect ratio of 0.33) actual peak base shears are only marginally greater than the equivalent rigid body base shear (the maximum amplification is 7%). This again is to be expected; it is intuitively obvious. For transverse motions (aspect ratio of 3.00) actual peak base shears were found to be 76%, 55%, and 15% greater than their rigid body equivalents. These amplifications can be very significant, but obviously are not uniformly large for all earthquakes. This behavior is expected if a long, narrow warehouse is modeled as an equivalent 2-story shear wall building. The "first story" shear walls are the actual, very stiff concrete, end shear walls of the warehouse. The "second story" shear walls are the relatively flexible, plywood, horizontal half-diaphragms to mid-span. The "second story" lumped mass consists of approximately two-thirds of the entire roof and out-of-plane wall mass. This "second story" lumped mass can be very large, particularly when the out-of-plane walls are concrete, are 300 feet long, and are 31 feet high (as they are for the Hollister warehouse). This two-mass system with large masses and one soft spring (the plywood roof diaphragm) can have a fairly long fundamental period: approximately 0.65 seconds for the Hollister warehouse. This period is apparent from the V3(t) time history (shown in Figure 3 for the Hollister Warehouse). This fundamental period is typical for a five- to seven-story steel frame building, and is not generally associated with a one-story building having solid concrete shear walls. Once it is understood that a long, narrow warehouse is dynamically similar to a five- to seven-story steel frame building, then the observed amplification of motion is not at all surprising. The behavior of the other three buildings studied was similar.

Thus far, effects of wall and diaphragm flexibility on overall building base shear have been studied . The effects of diaphragm flexibility on the diaphragm itself is even more dramatic.

Three plywood shear time histories were computed for transverse motions of the Hollister warehouse for the Loma Prieta earthquake. The first was the actual plywood time history observed. The actual peak plywood shear observed was 3103 plf. The plywood is 3/4 inch CDX with 10d nailing at 1-3/4" on centers. Allowable shear is approximately 930 plf. The peak value was 3.34 times the design value, with apparently no damage. If the roof diaphragm motions were assumed to equal the wall-top motions, the peak plywood shear would have been 1510 plf; and, if the roof diaphragm motions were assumed to equal the ground motion (assuming the building responded rigidly to the earthquake), the peak plywood shear would have been 1430 plf. Amplification of plywood shear due to wall and diaphragm flexibility (over rigid body behavior) was 117%.

Amplification of wall anchorage due to wall and diaphragm flexibility was even greater. Out-of-plane wall anchorage time histories for transverse motions of the Hollister warehouse to the Loma Prieta earthquake were also computed. The plywood diaphragm is nailed directly to a 4x plate, which is bolted directly to the top of the wall; there are no joist or purlin anchors. Actual peak roof-to-wall anchorage load at diaphragm mid-span was 1081 plf. The actual peak roof-to-wall anchorage load adjacent to an end wall was 345 plf. The peak roof-to-wall anchorage assuming rigid body behavior would have been 327 plf. The amplification of wall-to-roof anchorage due to wall flexibility was 221%. The allowable load on the anchor bolt-to-wood nailer is equivalent to 306 plf. The peak value measured was 3.53 times this allowable value, with apparently no damage.

CONCLUSIONS

Low-rise, shear wall buildings are generally expected to be stiff, and are not expected to have dynamic amplification factors much greater than one. If the low-rise buildings have heavy, stiff walls and flexible horizontal diaphragms their behavior in earthquakes can be quite different.

The strong-motion records from four low-rise buildings (with stiff shear walls and flexible diaphragms) were analyzed. Peak base shears were amplified as much as 82% over what the peak base shear would have been if the building was assumed to behave as a rigid body.

The amplification of elements of such buildings obviously was greater than the amplification of the peak base shear for the entire building. For one building, the Hollister warehouse, the actual roof plywood shear was found to be (for transverse motions) 117% greater than what it would have been had the building behaved as a rigid body; and wall-to-roof anchorage, 221% greater.

Transverse motions of buildings of this class will behave very similarly to five- to seven-story steel frame buildings. Similar amplifications of motion can be expected. The longitudinal motions of such buildings show little amplification of motion because the primary contributor to that amplification, diaphragm flexibility, is expected to be small.

Although no complete set of strong-motion records were available for a unreinforced building, because their floor diaphragms typically are very flexible, it can be expected that their peak base shears can be greatly amplified. The upper and lower bound analyses made in this study for one unreinforced masonry building, with a diaphragm aspect ratio of only 1.06, showed this to be the case.

For this class of buildings, deformations that vary horizontally are just as important as deformations that vary with height. Consequently, the number of strong motion-instruments required to describe the complete acceleration distribution with accuracy can be large. In general, in order that building motions be described unambiguously, the motion of every horizontal diaphragm, the motion of every wall-to-diaphragm connection, and the motion of each shear-wall-to-ground connection should be recorded.

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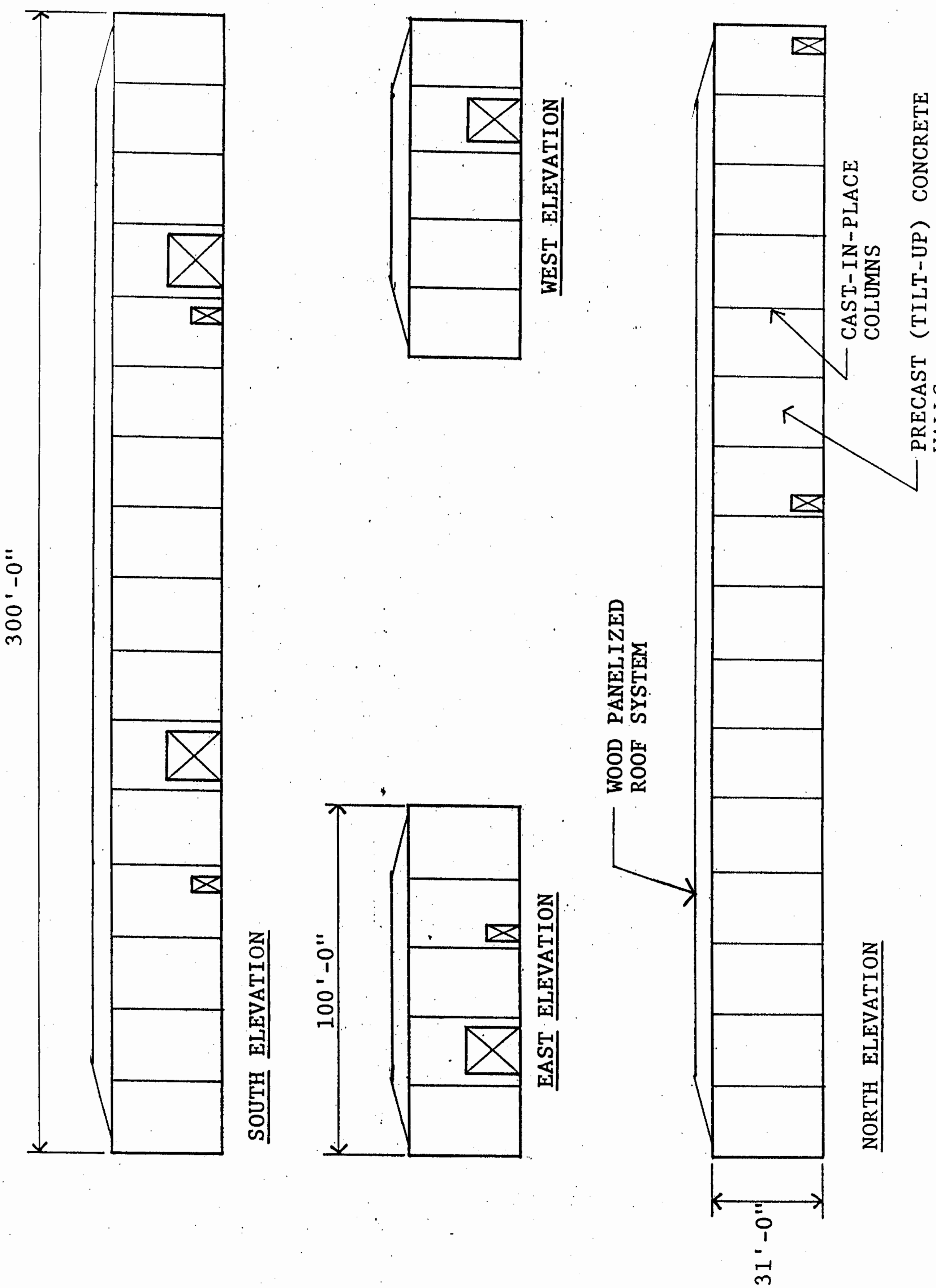


FIGURE 1 - 1-STORY WAREHOUSE BUILDING HOLLISTER, CALIFORNIA

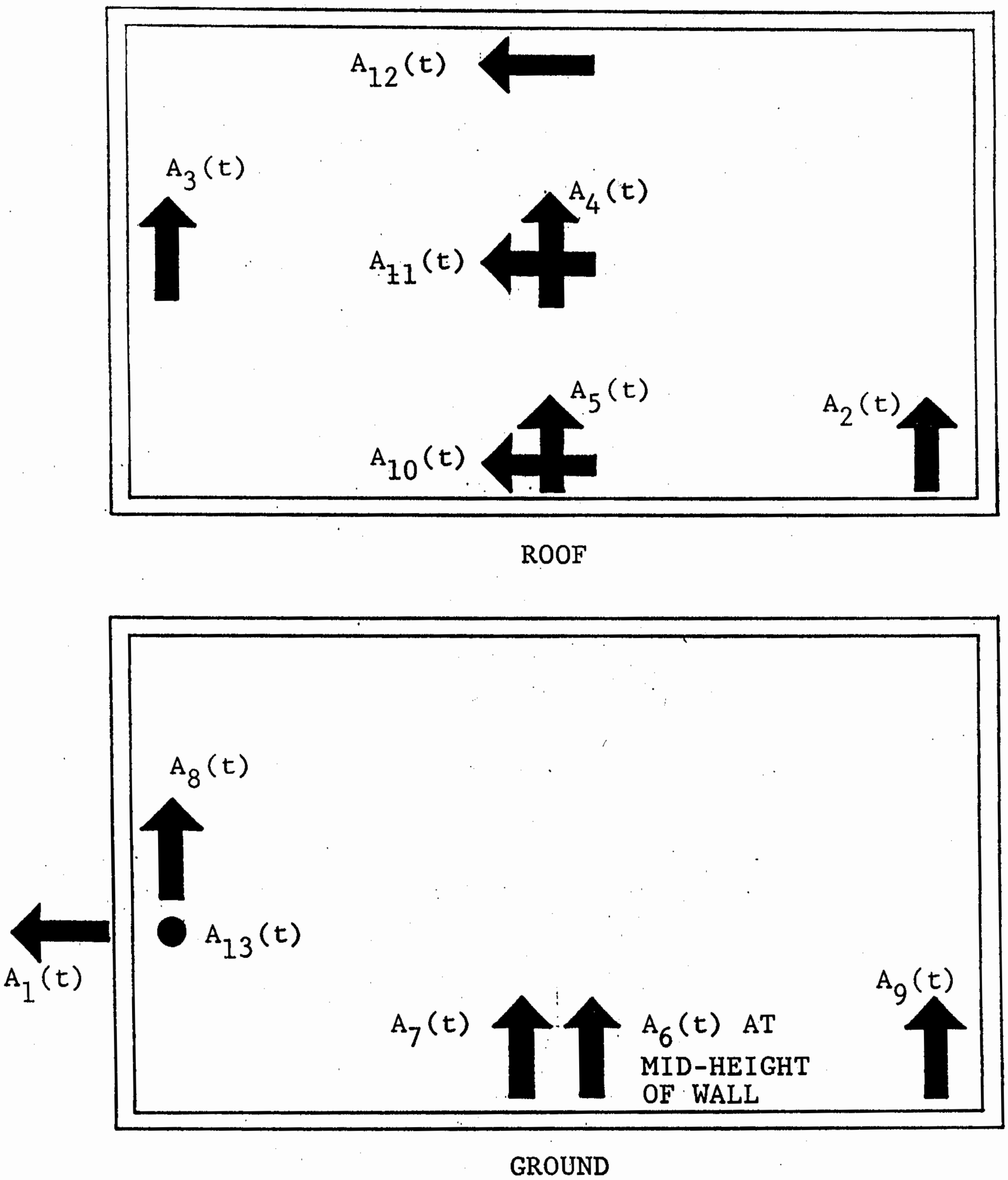
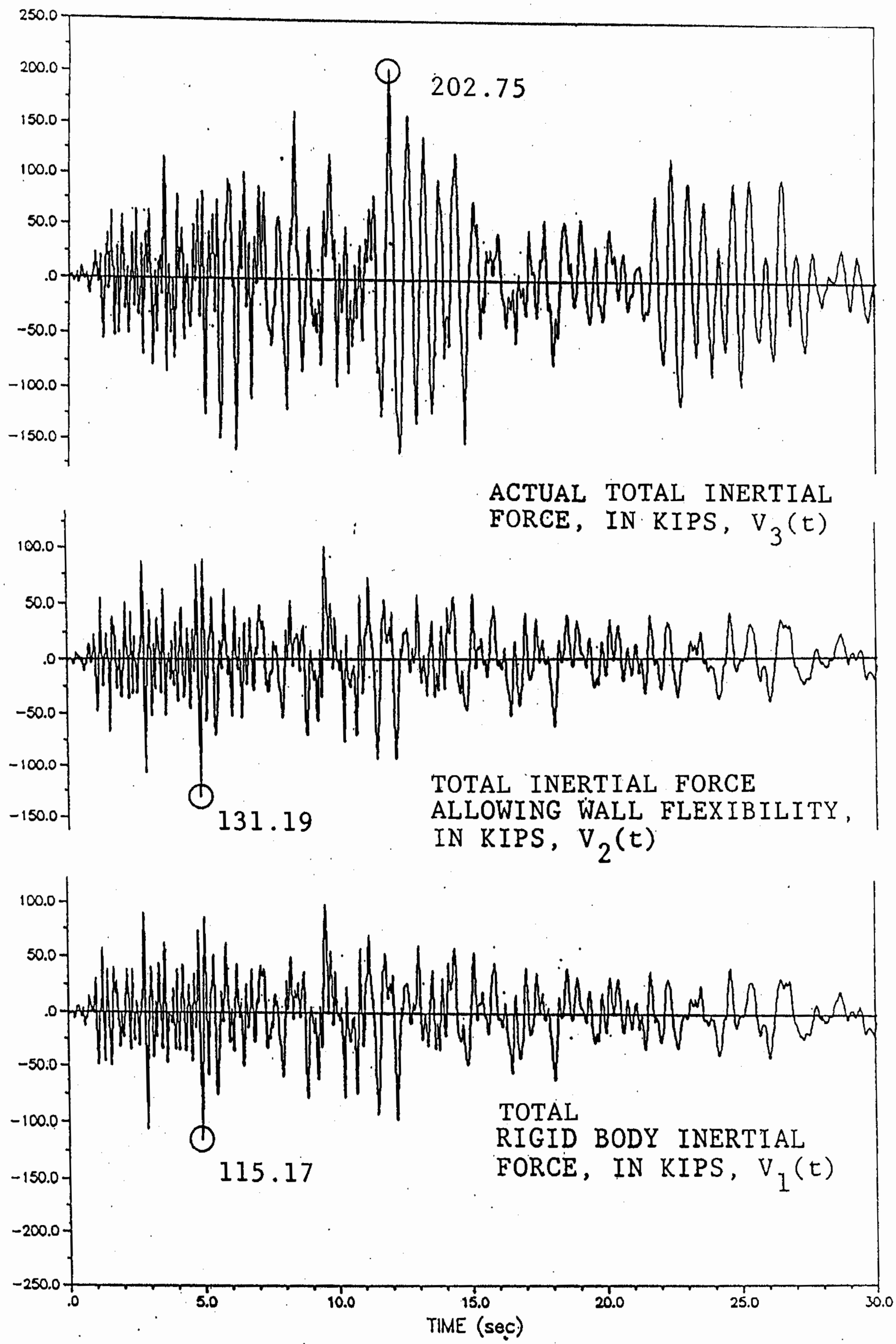
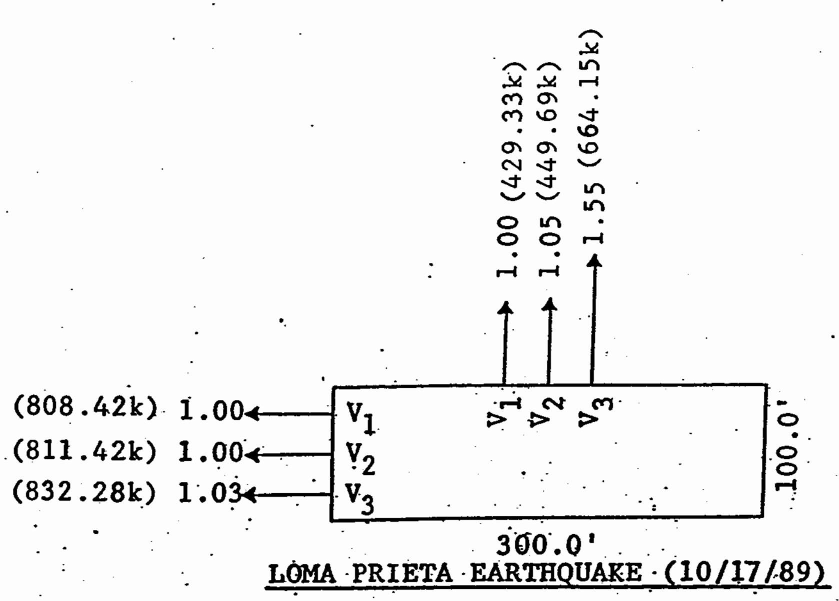


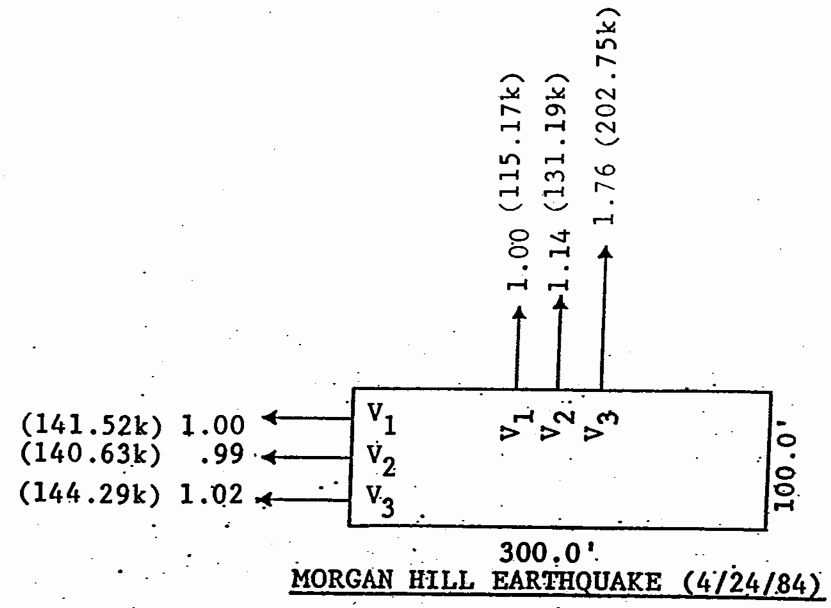
FIGURE 2 - INSTRUMENTATION PLAN
1-STORY WAREHOUSE BUILDING
HOLLISTER, CALIFORNIA

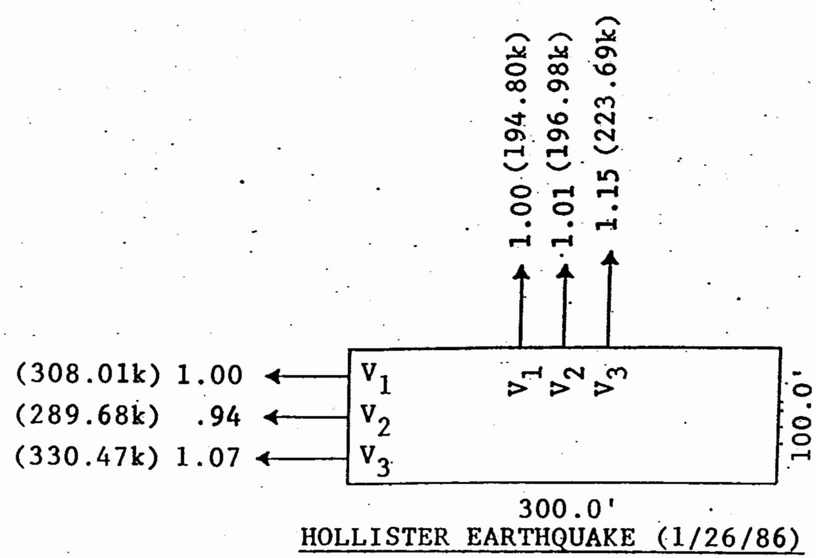


TRANSVERSE MOTION

FIGURE 3 1-STORY HOLLISTER WAREHOUSE
MORGAN HILL EARTHQUAKE (4/24/84)







V_1 - PEAK RIGID BODY BASE SHEAR

V_2 - PEAK BASE SHEAR ALLOWING WALL FLEXIBILITY

V_3 - PEAK BASE SHEAR ALLOWING WALL AND DIAPHRAGM FLEXIBILTY (ACTUAL CASE)

FIGURE 4
PEAK BASE SHEARS
1-STORY HOLLISTER WAREHOUSE