AN EVALUATION OF UNREINFORCED MASONRY WALL PERFORMANCE

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ABSTRACT

The objective of this study is to evaluate the assumption that there is no amplification of ground motion at the top of unreinforced masonry walls. Strong-motion records at the top and bottom of shear walls for one unreinforced masonry building, and four reinforced concrete buildings were analyzed. Accelerations at the top of reinforced concrete shear walls typically did not exceed 1.31 times ground accelerations. The unreinforced masonry building was damaged during the motions analyzed making results unreliable for predicting future motions.

OBJECTIVE

The objective of the ATC-27 project is to review and analyze existing strong-motion data from instrumented buildings to evaluate the assumption that there is no amplification of ground motion at the top of unreinforced masonry walls (in the direction parallel to the wall). The assumption is of concern because it may not be appropriate for multi-story buildings, especially those with large window openings in the wall.

SELECTION OF INSTRUMENTED BUILDINGS AND EARTHQUAKES TO ANALYZE

The principal sources of strong-motion records from buildings are California Strong-Motion Instrumentation Program (CSMIP): Seismic engineering Branch, USGS; and the California Institute of Technology (CIT). The buildings instrumented by the USGS or CIT typically were high-rise steel or concrete frame structures. Furthermore, the instrumentation was designed (for the USGS and CIT studies) primarily to obtain gross building movement, not relative movement within a building. Included in the CSMIP data base are a sufficient number of low-rise and moderate-rise shear wall buildings that are of value to this study. Furthermore, those buildings are sufficiently well instrumented so that relative motions between parts of the buildings (for example, between the top and bottom of shear walls) can be determined. Consequently, all building motions analyzed were from the CSMIP data base of strong-motion records.

The first, and obvious, building selected was the Old Gilroy Firehouse, the only unreinforced masonry building for which detailed strong-motion records are available. The building is shown with instrument location, in Figure 4.1. In-plane instrumentation does exist, at the top and bottom of the north wall, for this structure. Strong-motion records are available for this building from the Loma Prieta Earthquake 10/17/89.

No other strong-motion records are available, at this time, for this or other unreinforced masonry structures. Therefore, strong-motion records from other buildings having similar characteristics as typical unreinforced masonry buildings were sought.

For this study, unreinforced masonry buildings typically 1) are rectangular in plan, 2) are 4-stories or less tall, 3) have perimeter shear walls with or without openings, and 4) have flexible wood frame horizontal diaphragms at the floor levels and at the roof. Instrumented buildings having similar characteristics, for which strong-motion records exist, were sought. In particular, records were sought only from those buildings that had in-plane shear wall motions recorded.

Other buildings studied included the following: Second, the four-story Pacific Telephone Building in Watsonville, again, for the Loma Prieta Earthquake, 10/17/89, was studied. By its proportions, the building is a bit too "high rise" but it does have perimeter shear walls (of reinforced concrete) with many openings. It also has, which is not great for this study, rigid concrete floor diaphragms. It is hoped something can be learned about in-plane deformations of shear walls with openings. See Figure 4.2.

Third, the Marshall Electronics Group Building in Milpitas was included. Strong-motion records exist for this building for the Loma Prieta earthquake, 10/17/89, as well, and were analyzed. This building is a two-story reinforced concrete tilt-up structure with large openings around the entire perimeter. It has a plywood, on wood frame, roof diaphragm. The second floor structure consists of concrete fill on metal deck on open-web steel joists. This second floor diaphragm is perhaps stiffer than desired compared to a wood diaphragm, but it is still far more flexible than a cast-in-place concrete diaphragm. Again, this structure was selected because the analysis of in-plane deformations of shear walls with openings would hopefully be rewarding. See Figure 4.3 for a simple description of the building and the instrumentation plan.

Fourth, the Glorietta K Warehouse in Hollister was selected to be studied. Strong-motion records for the Loma Prieta earthquake 10/17/89, the Morgan Hill Earthquake 4/24/84, and the Hollister earthquake 1/26/86 were studied. The Glorietta K Warehouse is a typical, one-story concrete tilt-up warehouse with a wooden panelized roof structure. There are very few openings in the entire building. See Figure 4.4 for a simple description of the building and the instrumentation plan.

And fifth, the Interstate Van Lines building in Redlands was studied. Records for this building from the Palm Springs Earthquake 7/8/86 were studied. This is another warehouse, very similar to the Glorietta K Warehouse. See Figure 4.5 for a simple description of the building and the instrumentation plan.

PRELIMINARY ANALYSIS PROCEDURE USED

It is to be remembered that the objective of this study is to evaluate the assumption that there is no in-plane amplification of motion from top to bottom of unreinforced masonry walls. With this objective in mind, the following bits of information were obtained for the analysis of each input (ground motion)/output (structure motion) pair of time histories: peak input acceleration, peak output acceleration, peak input displacement, peak output displacement, peak and RMS relative displacement, and best-fit single-degree-of-freedom dynamic response characteristics. Those dynamic response characteristics are those coefficients C1, C2, and C3 such that C1*A(t) + C2*V(t) + C3*D(t) fits AG(t), the input (ground) acceleration, in a least-squares sense

where

A(t)	output acceleration relative to input acceleration;
V(t)	output velocity relative to input velocity; and
D(t)	output displacement relative to input displacement.

All functions are a function of time, t. The natural frequencies, damping ratios, and participation factors can be readily obtained from the best-fit coefficients C1, C2, and C3. The procedure used is similar to that which was described by Raggett J. and Rojahn C. in "Use and Interpretation of Strong-Motion Records From Highway Bridges", FHWA-RD-78-158, Fed Highway Admin., Wash DC, October 1978.

Although the specific objective of the study was to evaluate in-plane motions at the top of walls relative to motions at the base of walls, all response motions were analyzed relative to the ground motions, for the instrumented buildings and earthquakes identified.

DISCUSSION OF RESULTS

Discussed are preliminary findings of the analyses of in-plane motions recorded for the five buildings described earlier. First will be a summary of in-plane motion analyses for the four reinforced concrete shear wall buildings that have similar expected dynamic response characteristics to unreinforced masonry buildings. If it is assumed that the modulus of elasticity for unreinforced brick is 1,2000,000 psi, and that for reinforced concrete is 3,000,000 psi, then reinforced concrete building relative displacements could be multiplied by 2.5 to be approximations of similarly proportioned unreinforced masonry buildings. Included in this summary are a series of ratios:

Accel Ratio = (peak response acceleration)/(peak ground acceleration)

Disp Ratio = (peak response displacement)/(peak ground acceleration)

Rel Disp Ratio=(peak relative disp)/(peak ground displacement)

All three ratios give a sense of the degree to which the wall is following the ground motions as a rigid body. See the summary on Table 4.1 Shown on Figures 4.6 to 4.11 are typical displacement time histories, one for each direction, for each of the buildings sampled, starting one second before the maximum relative displacement was recorded.

Some observations from the tabulated results and sampled time histories follow:

1. For all time histories presented, the response displacement is very similar in shape and magnitude to the ground displacement.

2. The peak roof acceleration for the Pacific Telephone building is substantially greater than the peak ground acceleration. This building is much more of a "high-rise" building and is probably the least representative of typical unreinforced masonry buildings.

3. Most buildings are on flexible soil material. Note the very long ground motion periods for all cases.

4. Base rocking components have not been identified yet in the Pacific Telephone Building response and the Marshal Electronics group building response. Base rocking cannot be identified in the other responses.

For all cases, best-fit dynamic response characteristics made little sense. Best-fit coefficients were identified, but natural frequencies, damping ratios, and participation factors derived from the best-fit coefficients made little physical sense. This exists for two reasons. First, it was attempted to identify dynamic response characteristics from small differences between two time histories (very similar input and output time histories). In such cases huge variations in dynamic response characteristics cause minor variations in the response. Therefore, response analyses, relative to the input, produce unreliable results. Second, a single-degree-of-freedom linear elastic model does not appear to be a very good approximation to the observed behavior. A linear elastic model is a model of convenience (it is easy to formulate and solve) and it is very appropriate for wall behaved flexible structures (such as high-rise building steel frames). It simply is not a particularly good model of low-rise, shear-wall, building behavior.

This leads to the potentially most substantive but most frustrating analysis, that of the Old Gilroy Firehouse (the only true unreinforced building analyzed).

One pair of records exists that is capable of being analyzed to determine in-plane wall flexibility; the east-west motion (transverse) motion of the north wall of the building. It should be noted that this is not the north wall of the main building proper, but is the north wall of the 20-foot unreinforced masonry addition to the main building proper. The addition is loosely attached to the main building (the full height joint connecting the two is a grouted joint, but one that is cracked throughout). Nonetheless, it is still an unreinforced masonry wall with in-plane strong motion instrumentation top and bottom. See Figure 4.12 for the input, output, and relative displacement time histories and pertinent peak values for this input/output pair of time histories.

The peak value ratios are rather startling

Accel Ratio = 1.453
Disp Ratio = 1.107
Rel Disp Ratio = 1.328

From the record it is obvious that there is significant flexibility in the wall (and an undetermined amount of base rocking). Furthermore, it is obvious that the top of the wall is not following, even remotely, the wall base motion. Note that the peak relative displacement of the top of the wall to the base is a whopping 12.904 cm! It is hard to imagine an unreinforced masonry building distorting this much over a height of about 30 feet without damage, and herein lies the frustration. There is, in fact, a large crack (about 1/2 inch permanent opening) extending down from the parapet to a second floor window on the north wall. Obviously, there was significant cracking on this wall; and therefore, a large relative roof-to-ground displacement could be expected. This does taint the results, however, and makes the records of little value for use in predicting unreinforced masonry wall performance in anything less than damaging levels. Note that the peak ground acceleration was 0.285 g.

PRELIMINARY CONCLUSIONS

Based upon the preliminary analyses made, in-plane distortions of concrete (and presumably unreinforced masonry) shear walls with and without modest openings, are not expected to be large relative to absolute ground displacements. Peak-roof-to-peak ground acceleration ratios of 1.31 or less have been observed for one and two story shear walls (the 1.31 was for a tilt-up wall building with large openings). For a 4-story "high-rise" concrete shear wall type building this acceleration ratio was as high as 3.118. Typically, however, for a shear wall type walls, for low-rise buildings, an acceleration ratio of 1.31 or less was observed.

Unreinforced masonry "walls" run the full range from a solid one-story shear wall, to a frame, typical of many store fronts. An unreinforced masonry frame (or wall with such large openings that it behaves like a frame) will of course be very flexible. It also will be very weak, and should not be relied upon to resist seismic loads. The dynamic response characteristics of the structural <u>reinforcements</u> would then govern predicted and future responses.

A preliminary conclusion of this study so far is that the assumption that there is no amplification of ground motion at the top of unreinforced masonry walls (in the direction parallel to the wall) is a reasonable assumption for normally proportioned unreinforced masonry walls. This does not mean, however, that building motions in general parallel to shear walls are not amplified. Quite to the contrary, building motions in general are greatly amplified, between shear walls because the horizontal diaphragms are so flexible, and at the same time, are so heavily loaded by those walls perpendicular to the isolated motion.

As an example, consider the transverse motions of the Marshall Electronics Group building. Shown on Figure 4.13 are the building peak accelerations, conservatively assumed here to act simultaneously. For this preliminary analysis, consider this to be a reasonable (but conservative) approximation to the true overall peak response. Assuming the modal response shown, the overall building base shear is 1.73 times the overall building base shear that would have occurred had the building moved with the ground without amplification. For this case, the overall building base shear based upon in-plane wall distortions alone would have been 1.23 times the overall base shear that would have occurred had the building moved with the ground without amplification. This too is far below the actual, expected, peak base shear. Obviously, the overall building response amplification has little to do with end wall flexibility. This behavior is expected to be similar for most unreinforced masonry buildings with flexible wooden, horizontal diaphragms.

BUILDING	PEAK GROUND ACCEL(g)	DIRECT	ACCEL RATIO	DISP RATIO	REL DISP RATIO
INTERSTATE	.037	LONG	1.036	1.025	0.185
	.037	LONG	1.038	.994	0.170
	.042	TRANS	1.101	1.046	0.267
	.042	TRANS	.914	.998	0.309
GLORIETTA	.360	LONG	.965	1.029	0.053
	.360	LONG	1.057	1.064	0.084
	.238	TRANS	1.4120	.999	0.070
	.252	TRANS	.993	.990	0.042
MARSHALL	.090	TRANS	1.252	.986	0.038
	.100	TRANS	1.310	.974	0.030
PACIFIC BELL	.253	TRANS	3.118	1.042	0.237
	.272	TRANS	1.524	1.068	0.162

TABLE 4-1

PEAK ROOF/GROUND RATIOS

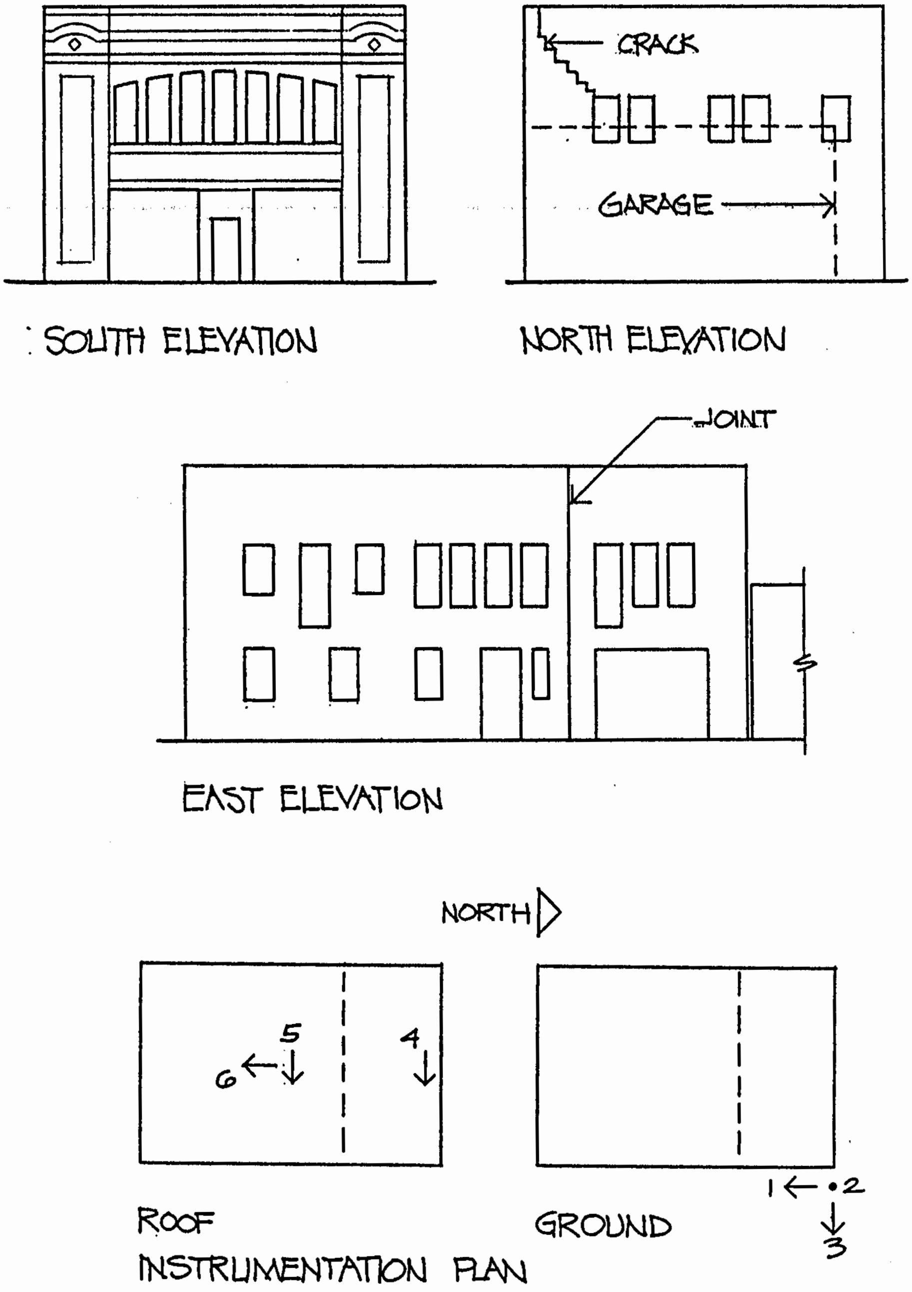


FIGURE 4.1
ELEVATIONS AND INSTRUMENTATION PLANS - OLD GILROY FIREHOUSE

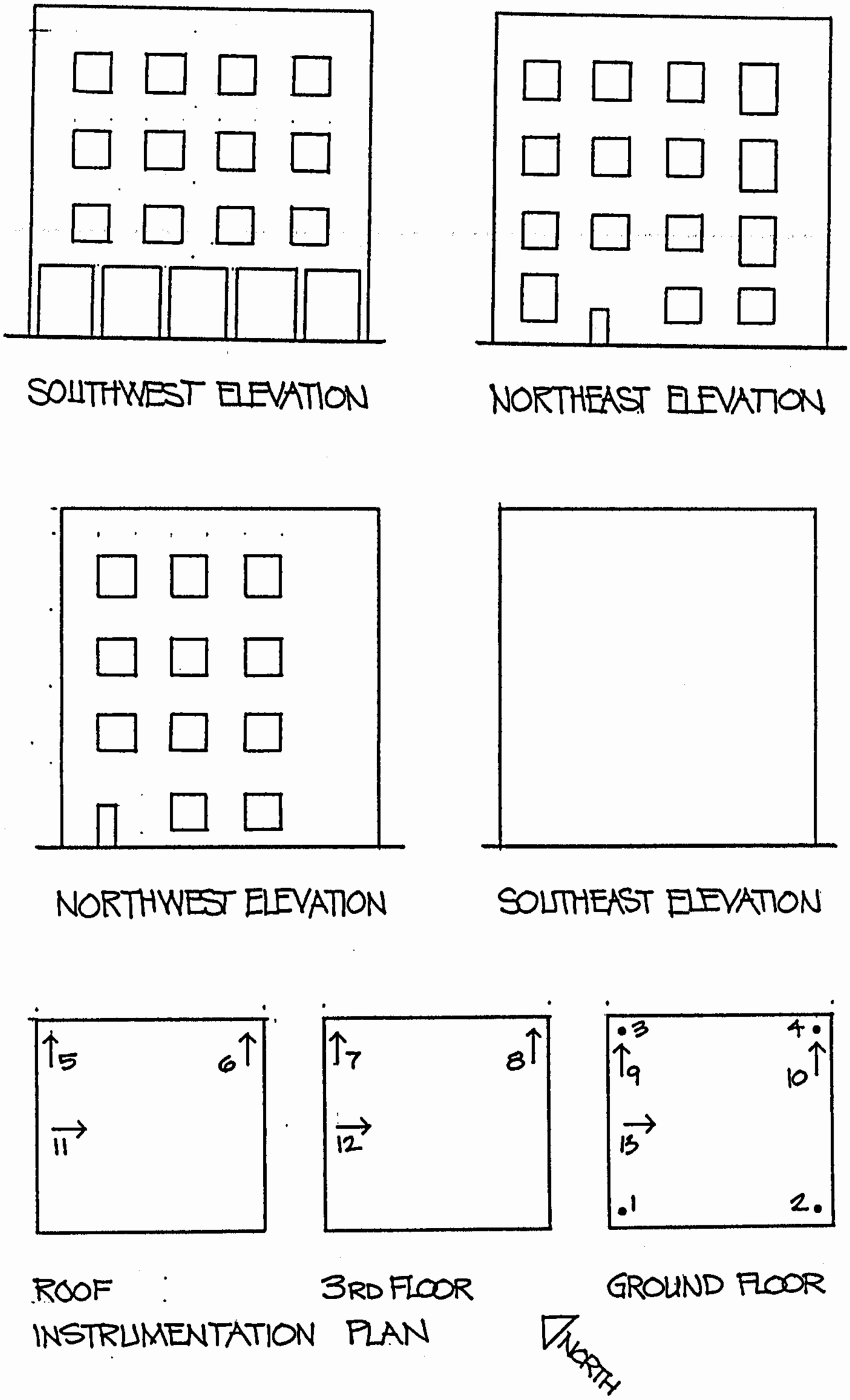


FIGURE 4.2 ELEVATIONS AND INSTRUMENTATION PLANS PACIFIC TELEPHONE BUILDING

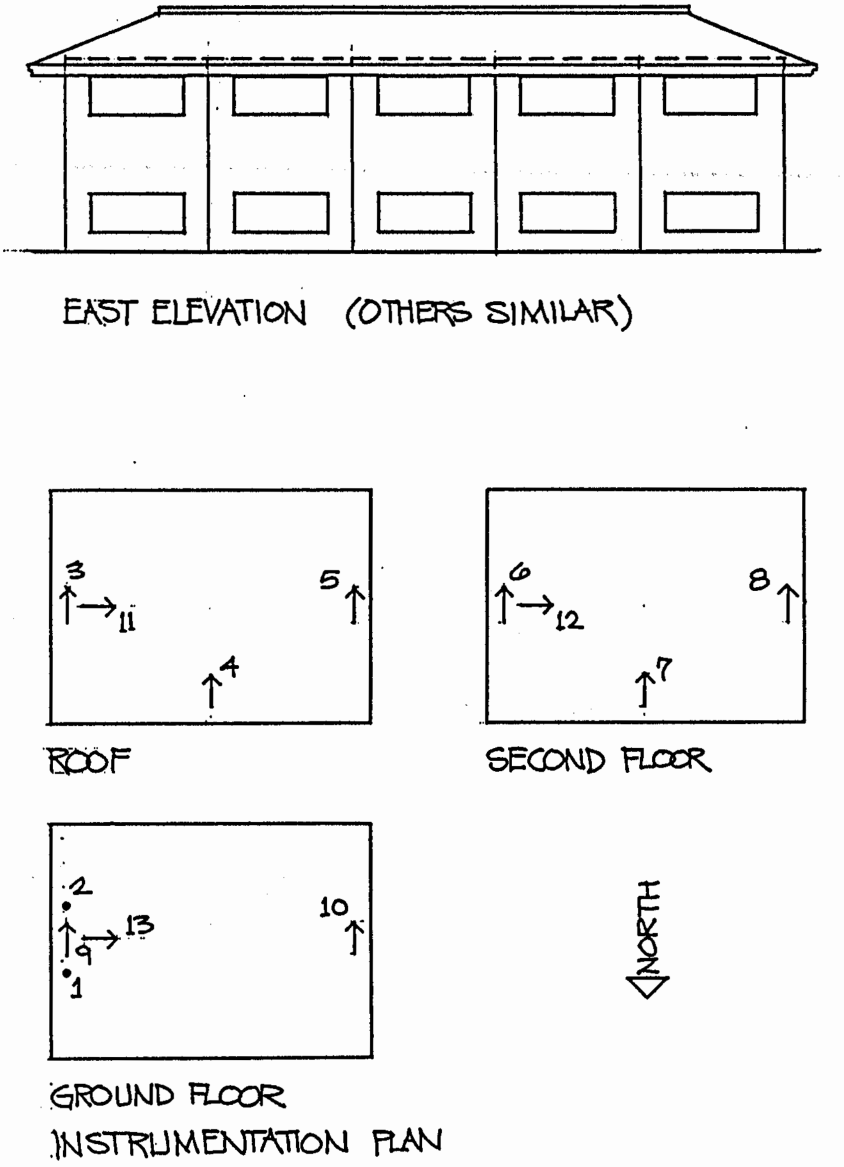


FIGURE 4.3
ELEVATIONS AND
INSTRUMENTATION PLANS
MARSHALL ELECTRONICS
GROUP BUILDING

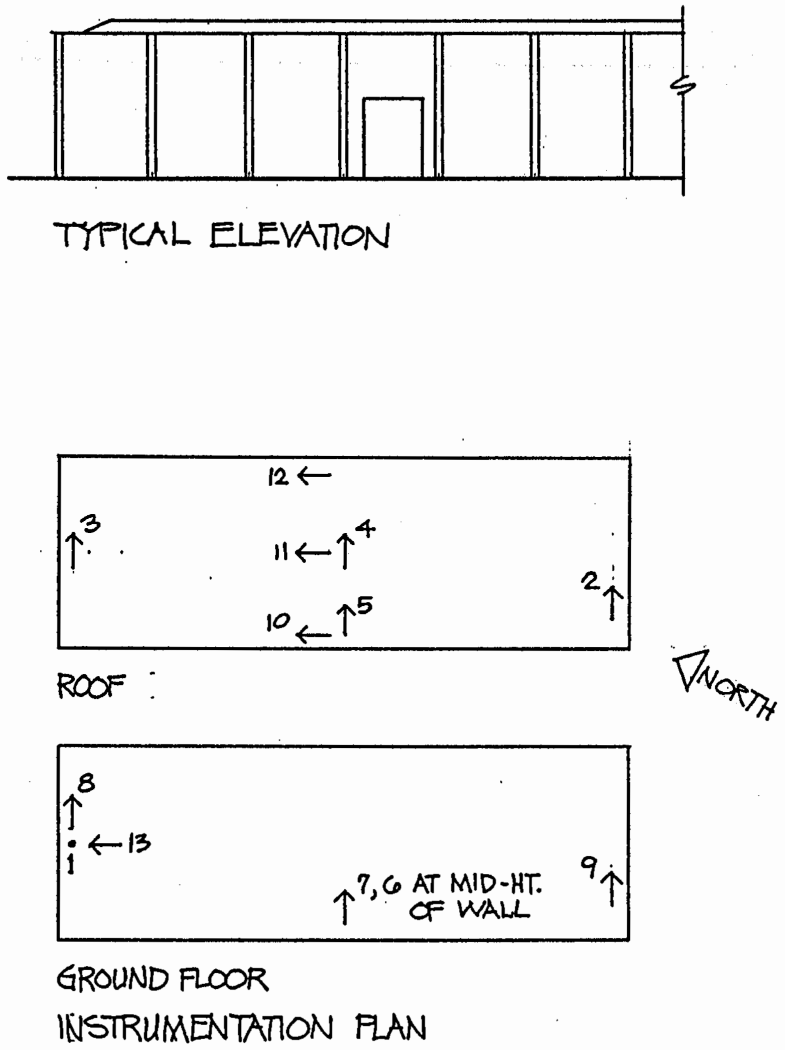
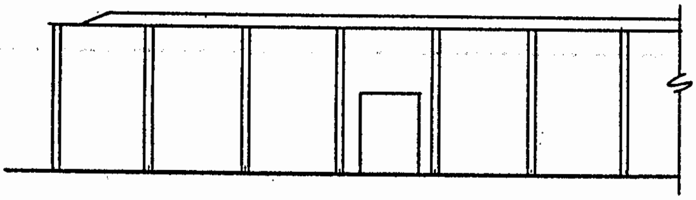
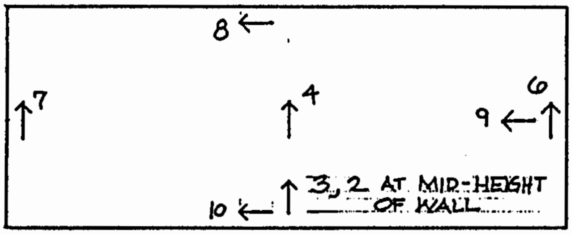


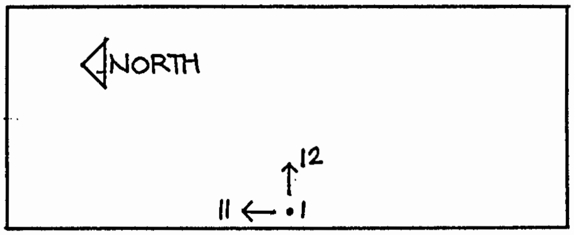
FIGURE 4.4
ELEVATIONS AND
INSTRUMENTATION PLANS
GLORIETTA K WAREHOUSE



TYPICAL ELEVATION

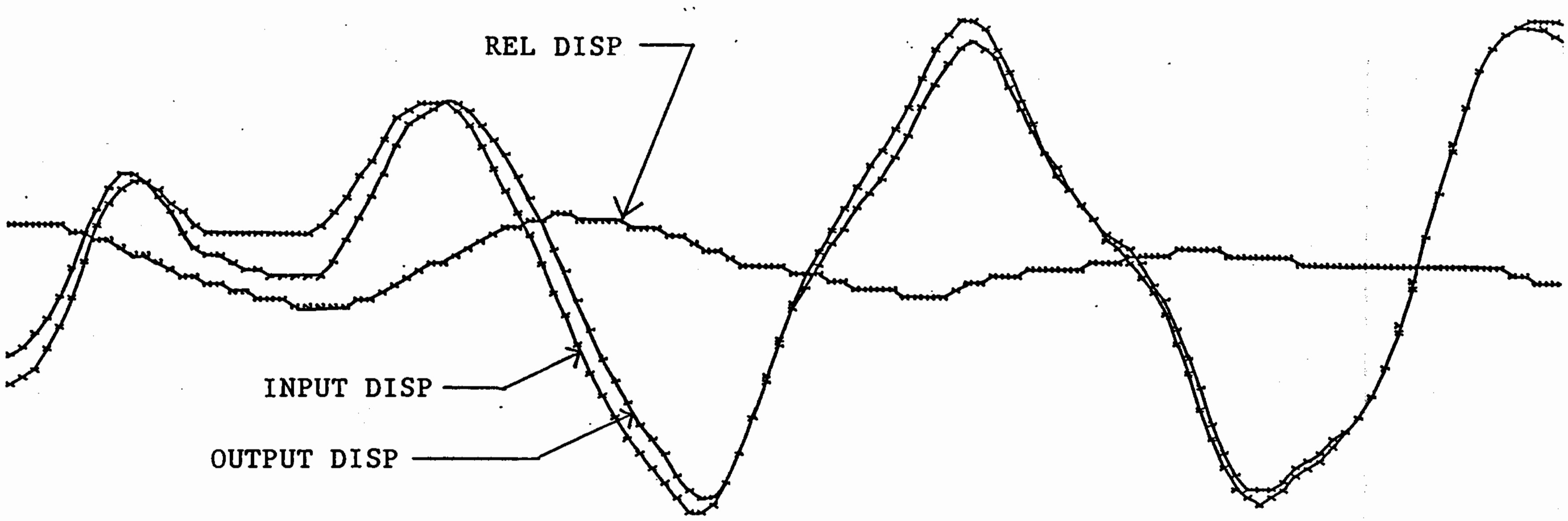


ROOF



GROUND FLOOR
INSTRUMENTATION PLAN

FIGURE 4.5
ELEVATIONS AND
INSTRUMENTATION PLANS
INTERSTATE VAN LINES
BUILDING



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INTERSTATE VAN LINES WAREHOUSE
PALM SPRINGS EARTHQUAKE 7/8/86
INPUT CHANNEL NUMBER 11
OUTPUT CHANNEL NUMBER 10
MAX Y ACCEL (CM/SEC/SEC)=    38.021
MAX X ACCEL (CM/SEC/SEC)=    36.646
MAX Y DISP (CM)=       .322
MAX X DISP (CM)=       .324
MAX REL DISP(CM)=       .055
RMS REL DISP (CM)=       .017
MAX X, Y,OR Z DISP (CM) =       .324
START TIME (SEC)=    21.360
FINISH TIME (SEC)=    26.360
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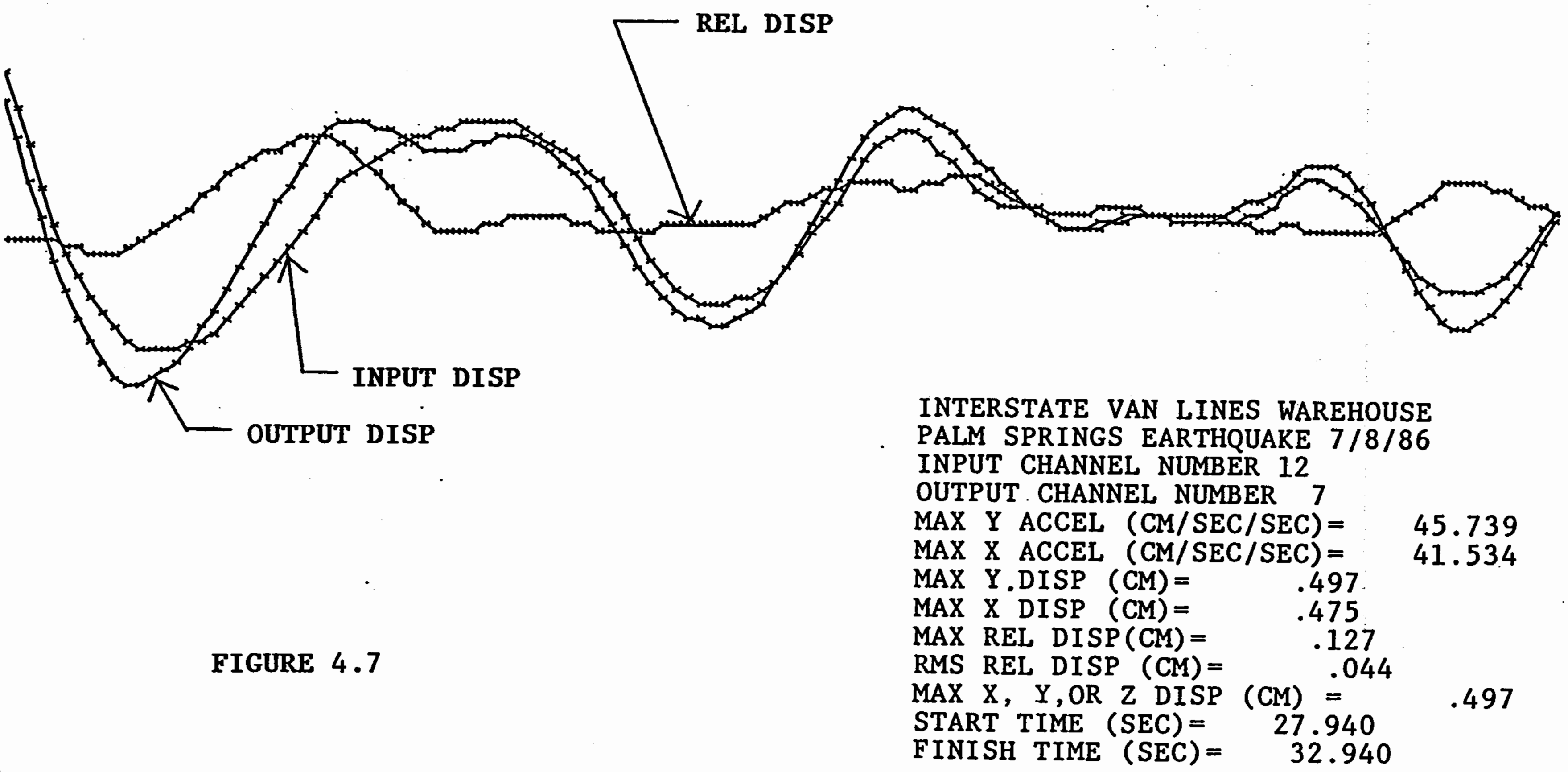
FIGURE 4.6



INTERSTATE VAN LINES WAREHOUSE
PALM SPRINGS EARTHQUAKE 7/8/86
INPUT CHANNEL NUMBER 12
OUTPUT CHANNEL NUMBER 7
MAX Y ACCEL (CM/SEC/SEC)= 45.739
MAX X ACCEL (CM/SEC/SEC)= 41.534
MAX Y.DISP (CM)= .497
MAX X DISP (CM)= .475
MAX REL DISP(CM)= .127
RMS REL DISP (CM)= .044
MAX X, Y,OR Z DISP (CM) = .497
START TIME (SEC)= 27.940
FINISH TIME (SEC)= 32.940

FIGURE 4.7

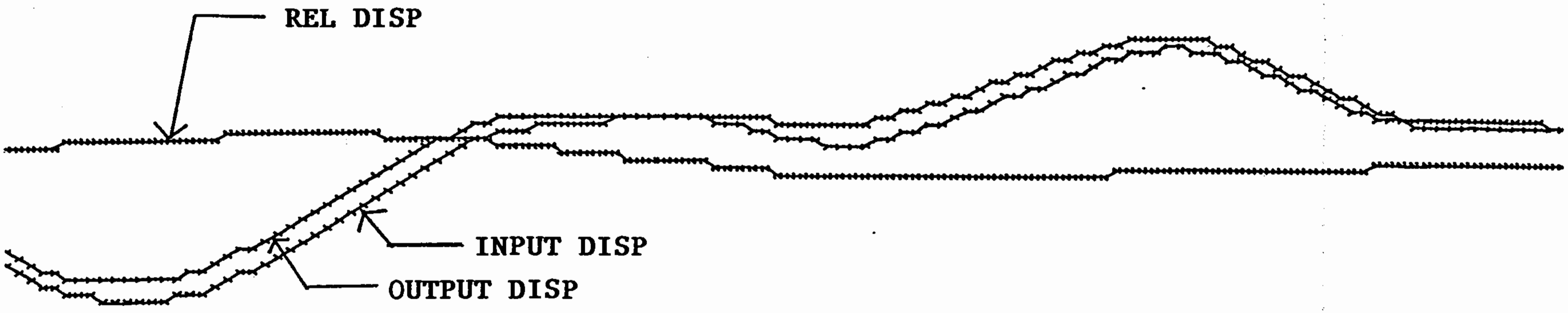


FIGURE 4.8

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GLORIETA WAREHOUSE
LOMA PRIETA EARTHQUAKE 10/17/89
INPUT CHANNEL NUMBER 13
OUTPUT CHANNEL NUMBER 10
MAX Y ACCEL (CM/SEC/SEC)=    372.841
MAX X ACCEL (CM/SEC/SEC)=    352.852
MAX Y DISP (CM)=    18.366
MAX X DISP (CM)=    17.260
MAX REL DISP(CM)=     1.456
RMS REL DISP (CM)=      .499
MAX X, Y,OR Z DISP (CM) =    18.366
START TIME (SEC)=    20.060
FINISH TIME (SEC)=    25.060
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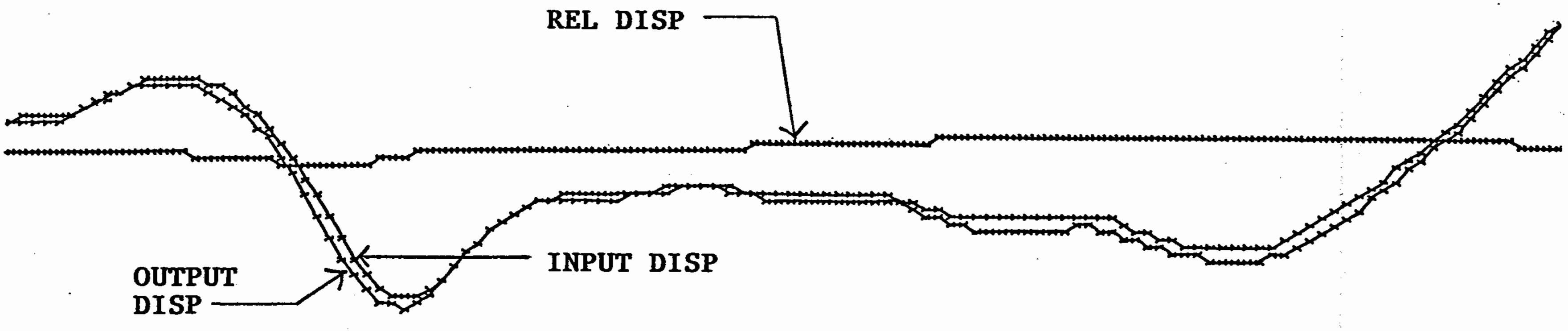

FIGURE 4.9

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GLORIETTA WAREHOUSE
LOMA PRIETA EARTHQUAKE 10/17/89
INPUT CHANNEL NUMBER  9
OUTPUT CHANNEL NUMBER  2
MAX Y ACCEL (CM/SEC/SEC)=   261.539
MAX X ACCEL (CM/SEC/SEC)=   233.504
MAX Y DISP (CM)=    21.597
MAX X DISP (CM)=    21.626
MAX REL DISP(CM)=     1.519
RMS REL DISP (CM)=      .372
MAX X, Y,OR Z DISP (CM) =    21.626
START TIME (SEC)=    11.060
FINISH TIME (SEC)=    16.060
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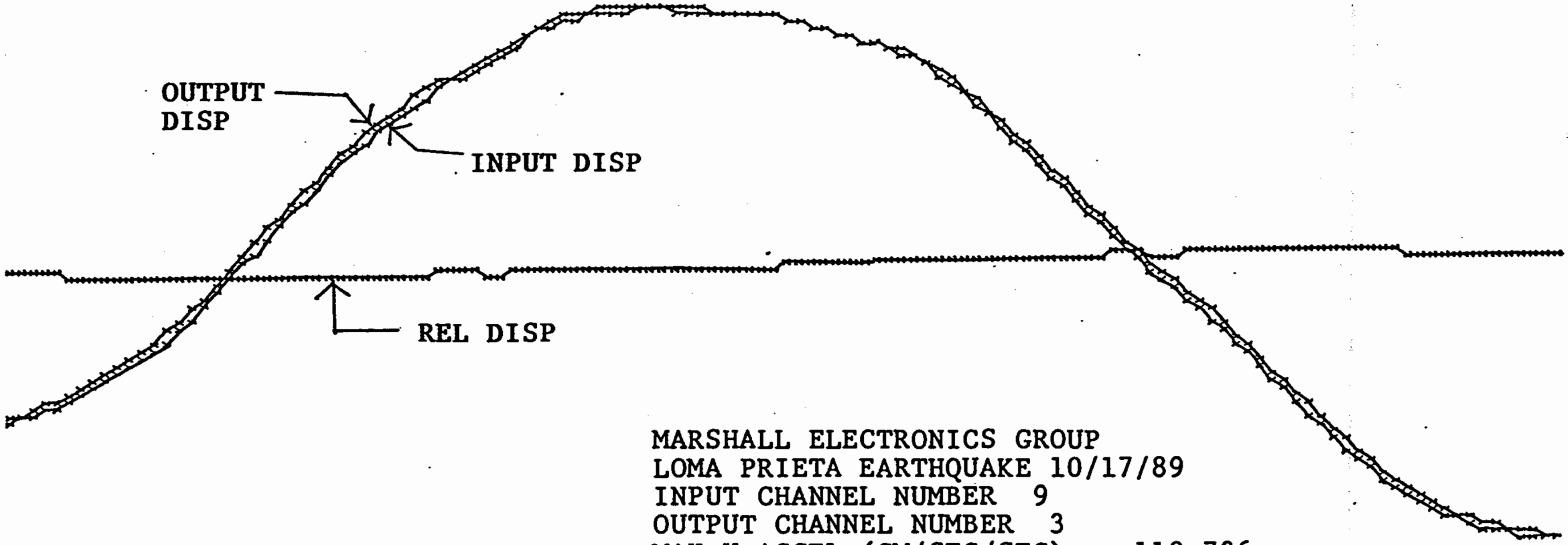

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MARSHALL ELECTRONICS GROUP
LOMA PRIETA EARTHQUAKE 10/17/89
INPUT CHANNEL NUMBER  9
OUTPUT CHANNEL NUMBER  3
MAX Y ACCEL (CM/SEC/SEC)=   110.736
MAX X ACCEL (CM/SEC/SEC)=    88.421
MAX Y DISP (CM)=    23.814
MAX X DISP (CM)=    24.161
MAX REL DISP(CM)=      .926
RMS REL DISP (CM)=      .301
MAX X, Y,OR Z DISP (CM) =    24.161
START TIME (SEC)=    12.560
FINISH TIME (SEC)=    17.560
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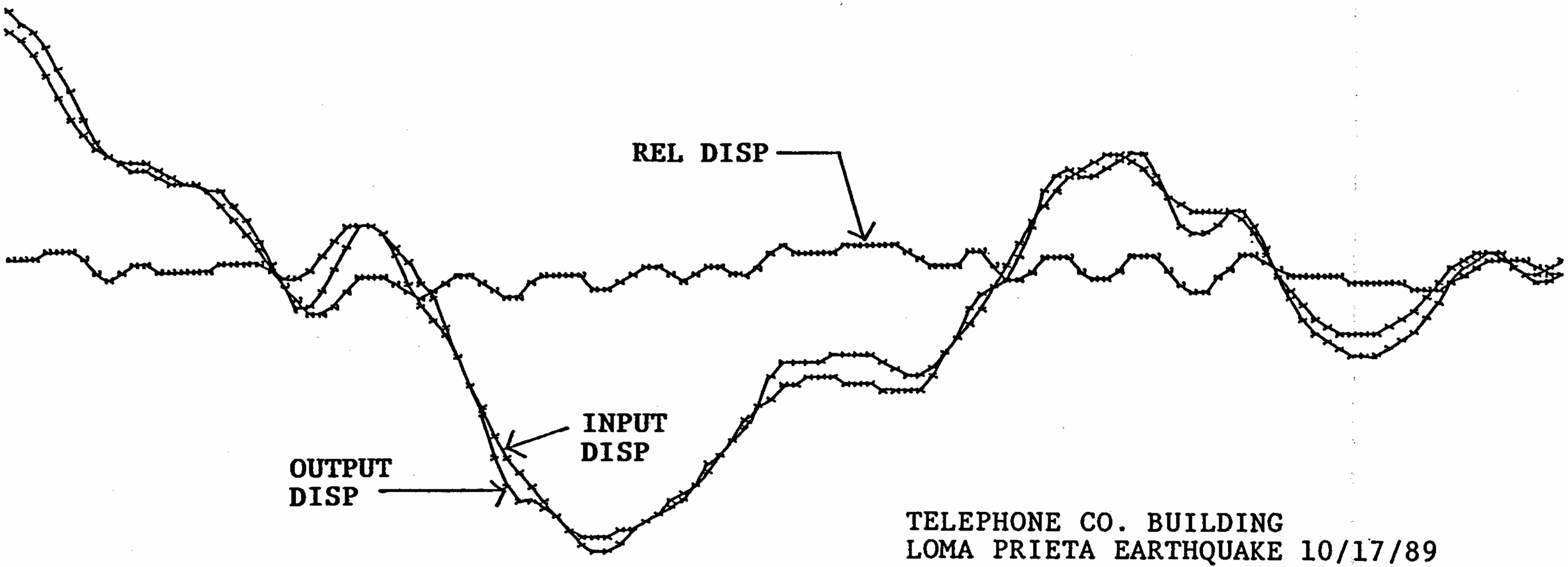
FIGURE 4.10



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TELEPHONE CO. BUILDING
LOMA PRIETA EARTHQUAKE 10/17/89
INPUT CHANNEL NUMBER  9
OUTPUT CHANNEL NUMBER  5
MAX Y ACCEL (CM/SEC/SEC)=   406.604
MAX X ACCEL (CM/SEC/SEC)=   266.776
MAX Y DISP (CM)=     9.562
MAX X DISP (CM)=     8.953
MAX REL DISP(CM)=     1.447
RMS REL DISP (CM)=      .305
MAX X, Y,OR Z DISP (CM) =      9.562
START TIME (SEC)=    4.720
FINISH TIME (SEC)=     9.720
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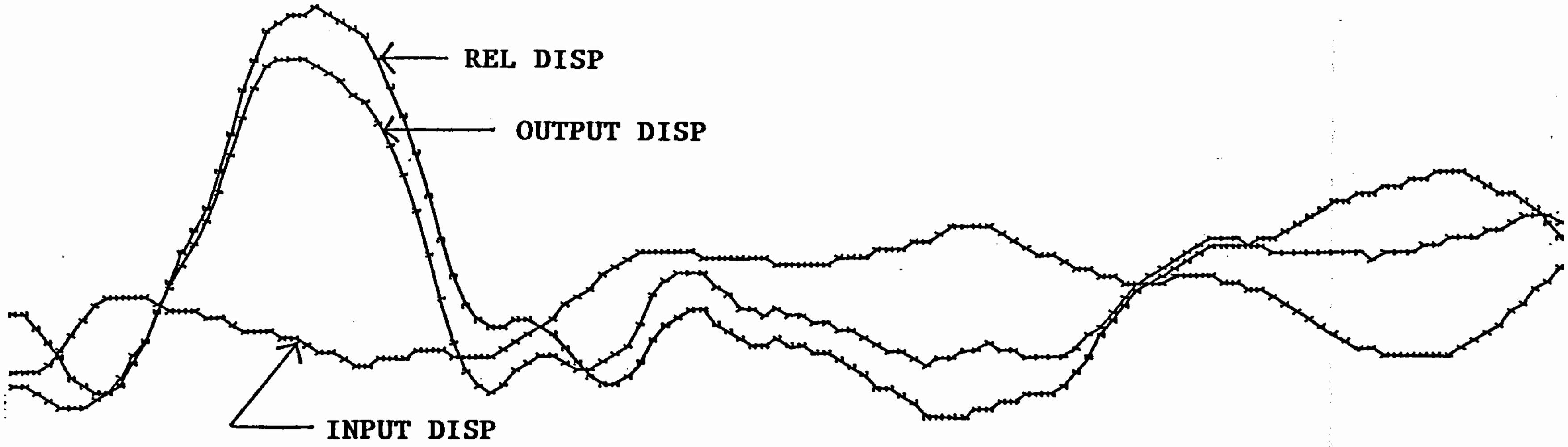
FIGURE 4.11



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OLD GILROY FIREHOUSE
LOMA PRIETA EARTHQUAKE 10/17/89
INPUT CHANNEL NUMBER  3
OUTPUT CHANNEL NUMBER  4
MAX Y ACCEL (CM/SEC/SEC)=   406.464
MAX X ACCEL (CM/SEC/SEC)=   279.742
MAX Y DISP (CM)=    10.753
MAX X DISP (CM)=     9.716
MAX REL DISP(CM)=    12.904
RMS REL DISP (CM)=     2.966
MAX X, Y,OR Z DISP (CM) =    12.904
START TIME (SEC)=     4.860
FINISH TIME (SEC)=     9.860
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FIGURE 4.12

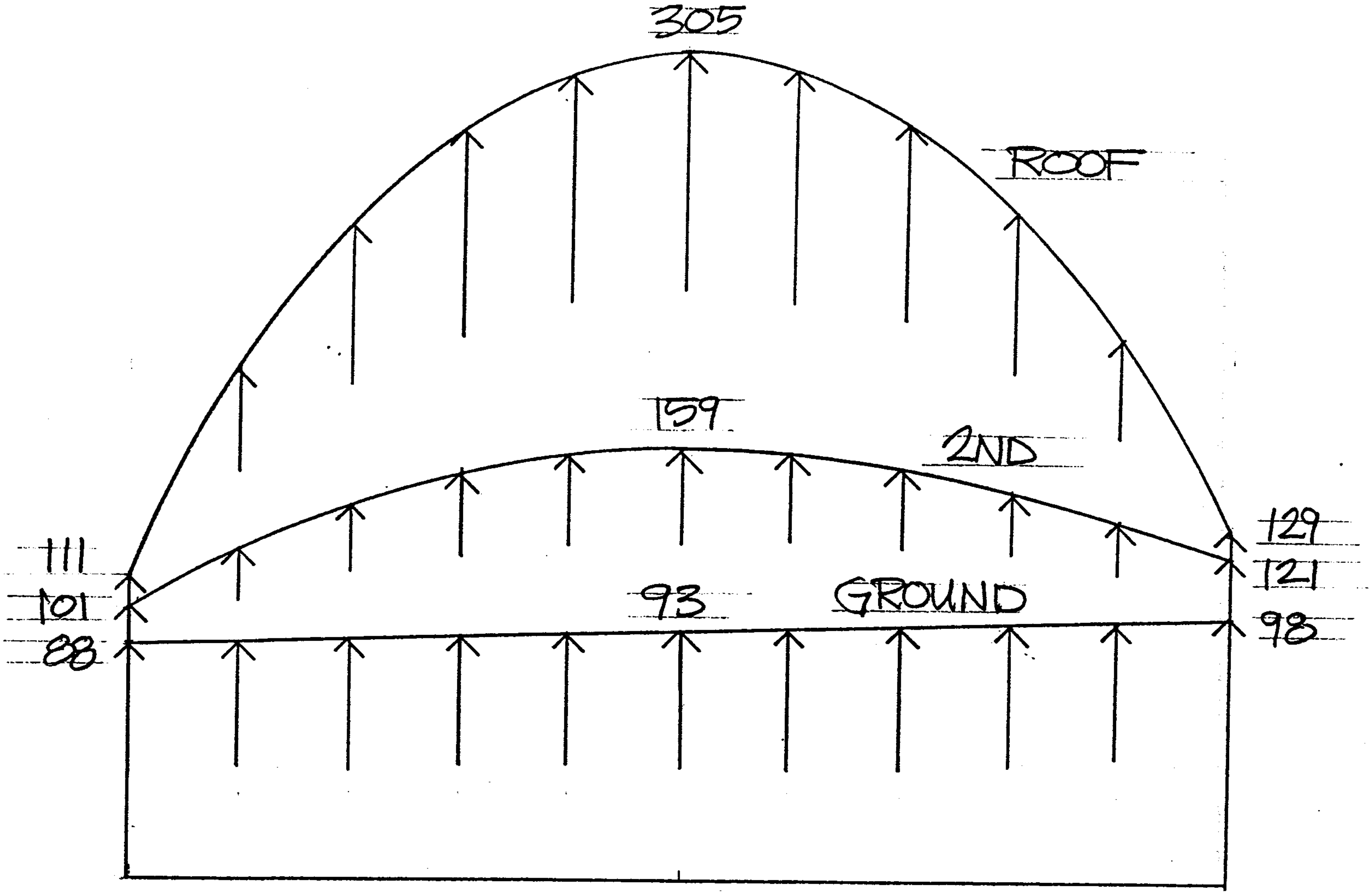

FIGURE 4.13
ENVELOPE OF PEAK ACCELERATIONS (CM/SEC^2)
MARSHALL ELECTRONICS GROUP BLDG.