CALIFORNIA INSTITUTE OF TECHNOLOGY

EARTHQUAKE ENGINEERING RESEARCH LABORATORY

ACCELEROGRAMS RECORDED AT CALTECH DURING THE WHITTIER NARROWS EARTHQUAKES OF OCTOBER 1 AND 4, 1987: A PRELIMINARY REPORT

By

M. B. Levine, J. L. Beck, W. D. Iwan,
P. C. Jennings and R. Relles

Report No. EERL 88-01

A Report on Research Supported by a Grant
from the National Science Foundation

Pasadena, California

1988

ACCELEROGRAMS RECORDED AT CALTECH DURING THE WHITTIER NARROWS EARTHQUAKES OF OCTOBER 1 AND 4, 1987:

A PRELIMINARY REPORT

M. B. Levine, J. L. Beck, W. D. Iwan, P. C. Jennings and R. Relles

EERL 88-01, August 1988

TABLE OF CONTENTS

ABSTRACT

Accelerogram records were recovered from 12 stations of the California Institute of Technology strong-motion array following the Whittier Narrows earthquake of October 1, 1987 and its major aftershock on October 4, 1987. This report presents the unprocessed accelerograms as well as the seismological characteristics of these two events. The locations of the Caltech strong-motion stations are described in detail, and some preliminary results are deduced from the accelerograms. The damage produced in the Los Angeles area is also briefly described.

ACCELEROGRAMS RECORDED AT CALTECH DURING THE WHITTIER NARROWS EARTHQUAKES OF OCTOBER 1 AND 4, 1987: A PRELIMINARY REPORT

1. INTRODUCTION

1.1 Earthquake Description

The Whittier Narrows earthquake of October 1, 1987 occurred 5 to 10 km northwest of the northwestern end of the Whittier fault, below the Puente Hills. This area is located approximately 15 km east of downtown Los Angeles and 10 km southeast of Pasadena, where the campus of the California Institute of Technology is situated (Figure 1). The mainshock which occurred at 7:42 (P.D.T.) has an estimated magnitude $M_L = 5.9$. Preliminary investigations suggest that the causative fault is a gently north-dipping thrust fault with an east-west strike, located at a depth of 11 to 16 kms [3]. This fault mechanism is somewhat different from that of the larger Whittier fault, which is a steeply dipping strike-slip fault with a small reverse component. In a preliminary analysis, Caltech seismologists suggest that the main shock was a double event which was separated by a 1 to 2 second interval. The first source mechanism corresponds to an east-west striking and gently dipping thrust fault, and the shallower second source location corresponds to a more steeply dipping fault further to the south [3].

The main aftershock occurred on October 4, 1987 at 3:59 (P.D.T.). Its estimated magnitude is $M_L = 5.3$. Its epicenter is located 2.5 km northwest of the main event, so it occurred closer to Pasadena. Also, the fault mechanism of this aftershock is very different from that of the main event, since it is predominantly that of a north-northwest right-lateral strike-slip vertical fault, at an estimated depth of 12 km. The focal mechanisms of both the main event and the major aftershock are illustrated in Figure 2.

Because of the proximity of these earthquakes to the Caltech campus, the recorded accelerograms were able to show the different focal mechanisms. As will be seen later, the main event records indicate relatively high vertical accelerations, whereas the major aftershock records show very important horizontal accelerations.

The California Division of Mines and Geology (CDMG) [2] reports that the maximum ground acceleration for the main shock from their network was 63%g, recorded at the Tarzana station, 2 miles south of Reseda in the San Fernando Valley (Figure 1). This value is surprisingly high compared to peak accelerations recorded at CDMG stations close to the epicenter which were less than 45%g, and may be due to anomalous site effects [2]. The USGS network recorded a peak horizontal ground acceleration of 63%g in the basement of the Whittier Tower on Bright Avenue, which is located in Whittier close to the causative fault. However, all the other records nearby indicate that ground motions were less than 50%g [11]. The maximum ground response measured by the Caltech strong-motion network during the mainshock is 36%g. These records also indicate that the duration of significant shaking was about 4 to 5 seconds.

In general, the October 4 aftershock records are characterized by smaller peak accelerations, 35%g maximum ground response was recorded by CDMG instruments at Obregon Park in Los Angeles, and shorter durations (1–2 seconds). However, initial estimates from the Caltech array in Pasadena indicate that peak ground accelerations for this aftershock may have reached as much as 59%g on campus. Because the recording site lies along the path defined by the strike of the fault at about a 10 km distance, such a high peak acceleration may be due to focusing of seismic waves in the strike direction.

1.2 Structural Damage

Because of the events' moderate accelerations and short durations, the Whittier Narrows earthquakes caused relatively little damage to engineered structures in the Los Angeles area. Most of the damage was concentrated around the cities of Whittier and Rosemead during the main shock of October 1, 1987. Losses due to this sequence of earthquakes are estimated to exceed $360 million.

The types of buildings most affected by the temblors were old unreinforced masonry buildings that had not been retrofitted, and single family houses whose foundations were not properly attached to the structure or whose chimneys cracked or collapsed during the earthquakes. Most retrofitted masonry buildings behaved properly during the earthquakes, except for a few which suffered minor damage such as cracking of wall piers and of unbraced exterior brick walls.

Several tilt-up type buildings experienced partial roof collapse due to inappropriate wall anchors for out-of-plane loads. Much damage was also witnessed on the campus of the California State University at Los Angeles, located several miles away from the epicenter of the October 1 event. The damage at that site included a fallen panel along a two-story parking structure, which was poorly anchored to its reinforced-concrete spandrel beam, shear cracks on the interior walls of a reinforced concrete building, and shear cracks on the support columns of a walkway which connected two wings of the library. Other major structural damage included an unoccupied, reinforced-concrete frame parking structure in Whittier, which was L-shaped in plan and which failed mainly because of shorter columns around the perimeter than within the interior, causing an imbalance in stiffness which produced shear failures in many of the exterior columns; a large four-story steel frame building in Rosemead, designed in 1982, where the third and fourth story brace members buckled; and a skewed two-span reinforced-concrete highway overpass at the junction of the 5 and 605 freeways, several miles south of Whittier, that suffered shear cracking in the shorter columns along one of the bents. References [6], [7] and [20] give more details about the failures mentioned above, and both conclude that, despite the strong ground motions in the Los Angeles area, the total amount of damage was relatively small and generally limited to structures that were not seismically sound.

None of the buildings on the Caltech campus in Pasadena suffered structural damage. However, Spalding administrative building, a three story reinforced-concrete structure, located on the southwest side of campus (Figure 3), suffered significant nonstructural damage.

1.3 Caltech Strong-Motion Network

Twelve of the fourteen stations of the California Institute of Technology strong-motion array recorded the October 1, 1987 Whittier Narrows earthquake and its major aftershock on October 4, 1987, for a total of 87 channels of recorded data. Nine of these stations are located on the Caltech campus, as illustrated in Figure 3, two are on the JPL campus, about 10 km north-west of Caltech, and one is located on a hillside about 5 km west of the Caltech campus. The four force-balance accelerometers comprising the ten channel CR1 recording system located in Millikan Library monitor the structural response of this nine-story reinforced-concrete building at the basement, the sixth floor and at two locations on the roof in two horizontal directions (east-west and north-south) and at the basement and on the roof in the vertical direction. All other accelerographs on the Caltech campus measure nominally ground response in the east-west, north-south and vertical directions. An inner ring of 4 instruments is located at less than 300 feet around Millikan Library, in the basements of Mudd, Bridge, Keck and Noyes Laboratories (Figure 3). These buildings are all two-story reinforced-concrete structures built on mat-type foundations, so the ground motions recorded at these sites will also include soil-structure interaction effects due to the stiff foundations. The accelerograph in Noyes malfunctioned and no records were recovered from the Whittier Narrows earthquake or its major aftershock. However, these earthquakes were properly recorded on all the other instruments within this inner ring. Millikan Library is equipped with a CR1 recording system, whereas the other buildings were monitored with SMA-1 accelerographs.

An outer ring of five instruments is located at more than a 500 feet radius around Millikan Library. These sites are at the Brown Gymnasium and a house on California Boulevard on the south side of campus, the Athenaeum Faculty Club and the Industrial Relations Center (IRC) on the east side of campus, and a house on Lura street on the north side of campus (Figure 3). For the houses on Lura Street and California Boulevard, and the two-story wood structure of the IRC, the instruments were positoned at ground level in the garage. Because of the type of structure, little soil-structure interaction is expected at these sites. The other instruments within the outer ring are also located at ground level, but because these foundations are more rigid, the records of the "ground" accelerations will also reflect some soil-structure interaction. All instruments within the outer ring functioned properly, and recorded the main earthquake as well as its major aftershock. All the instruments in the outer perimeter are SMA-1 accelerographs, which record all three components of shaking (north-south, east-west and vertical).

In addition to studying soil-structure interaction, a purpose of having these two rings of instruments on campus is to study the variations in ground motion within an urban (as opposed to a free-field) environment.

The three other locations of the Caltech strong-motion array are situated on the campus of the Jet Propulsion Laboratory (JPL) in La Cañada, which is 10 km to the northwest of Pasadena (Figure 1). Base and structural responses were monitored in two multistory buildings. The first one is Building 180, which is a nine-story, steel-frame structure, instrumented with two RFT-250 accelerographs located at the basement and roof levels. This building has been extensively studied in past earthquakes and ambient vibrations tests [10]. The other is Building 183, which is a nine-story, steel-frame structure, instrumented with three SMA-1 accelerographs, located at the first floor, fifth floor and roof levels. Unfortunately, the Building 183 instruments did not turn off at the end of the main shock and so did not record the major aftershock because the recording film was depleted. Also, a CR1 system in Building 238 jammed and failed to record any of the Whittier Narrows events.

A SMA-1 accelerograph was also installed to measure ground motions next to the Kresge seismological laboratory located off campus, approximately 5 km to the west of Pasadena. Because Kresge laboratory is situated on a hilltop, it is expected that the ground response will have a smaller amplitude and a higher frequency content than the accelerograms recorded at the nearby campus. Recently, a digital accelerograph has been installed next to the SMA-1, and, hence, the next time these instruments are triggered it will be possible to make a direct comparison between recording and processing results for both analog and digital records. Unfortunately, the digital instrument was not installed at the time the Whittier Narrows sequence occurred in October 1987.

As a cooperative project with Caltech, six digital, three-component, GEOS instruments were installed in Millikan Library by the USGS on October 5, to monitor the response of roof levels at the north-east, south-east and north-west corners. These sensors recorded motions due to two aftershocks, one on October 16 ($M_L = 2.8$) and another on October 21 ($M_L = 2.2$), which demonstrated significant rocking motions of the Library's foundation in phase with the north-south fundamental mode, as discussed later. These data provide useful background for the planned study of the digitized versions of the Millikan Library records described in this report.

2. ACCELEROGRAM ANALYSES

The following analyses of the Whittier earthquake records is based on direct interpretation of the unprocessed 70-mm film traces, and is thus only preliminary. Final results will be published once the records have been digitized and processed.

Ground and structural response film records for both the main event and the major aftershock are reproduced in Appendix A. These records are given in the same order as they appear in Table 1 and Table 2. The station locations, trace components, as well as the maximum accelerations for the ground response during both events, and the time they occurred after triggering, are summarized in Table 1. Similar information relating to the structural response obtained from the Caltech strong-motion network is summarized in Table 2. Peak motions were directly measured off the film traces, and multiplied by each of the instruments' sensitivity obtained in previous calibrations, to come up with the corresponding values of the acceleration in units of g. Because there is some uncertainty involved in the way these peaks were measured, and because no instrument correction or filtering has been applied to the records, the values given for the peak accelerations (Tables 1 and 2) are approximate. Final values for these maximum accelerations will be published once the records are digitized and corrected. The Kresge Seismological Laboratory mainshock and major aftershock records are not given in Appendix A because the developed film trace is very dark and could not be properly photocopied. The peak horizontal accelerations in the October 1 event at this site were 8%g to the north and 11%g to the east.

2.1 Ground and Basement Response

2.1.1 Whittier Narrows Earthquake of October 1, 1987 ($M_L = 5.9$)

The October 1, 1987 Whittier Narrows "ground" response accelerograms recorded on the Caltech campus are mainly characterized by relatively high vertical accelerations and motions in the horizontal east-west and north-south directions of comparable amplitudes. The duration of significant shaking lasted approximately 4 to 5 seconds. The responses were the highest in the accelerograms recorded in the garages of the houses on California Boulevard and Lura Street in which soil-structure interaction effects should be small and, hence, the records should nearly reproduce free-field motion. Peak vertical accelerations were 0.26g in the Lura Street house, with maximum accelerations of 0.36g in the east-west direction and 0.35g in the north-south direction; these were also the highest horizontal ground accelerations recorded on campus. For the house on California Boulevard, peak accelerations were 0.20g (up), 0.26g (N), and 0.17g (E), respectively, and in general maximum ground responses were usually lower for all components for records obtained from the basements of the larger campus structures. It is also important to note that sites where nearly free-field conditions are expected, such as the Lura Street and California Boulevard houses and the I.R.C. building, recorded overall higher response amplitudes over a longer period of time, than stations located at the foundation level of reinforced concrete structures such as the Brown Athletic Center and Keck Laboratories. The frequency content of the accelerograms for both of these types of recording sites is also significantly different. Comparison of the records obtained at the Lura Street house and at Keck Laboratory, which are less than 200 feet apart, shows that, for all three components, higher frequencies have been filtered out by the stiff foundation system of Keck Laboratory, which also considerably attenuated the amplitudes of the vertical trace. These differences constitute direct evidence of how soil-structure interaction can substantially alter the frequency content of the recorded ground response. Hence, when earthquake records are used as base excitation for dynamic response analyses of structures, consideration should be given as to where these ground accelerations have been originally recorded.

The fact that the October 1, 1987 Whittier Narrows earthquake records exhibited relatively high vertical accelerations, especially at the "free-field" stations (Lura Street house, California Boulevard house and the I.R.C. building), is consistent with the source mechanism of the causative thrust fault. Also, as has been mentioned earlier, preliminary seismological analyses suggest that this temblor might have been a double event along the same thrust fault. This phenomenon is substantiated by a double train of P-waves arriving, separated by a 1.4 sec to 1.8 sec interval, and is particularly visible in some of the vertical accelerogram traces, such as the I.R.C., Athenaeum Faculty Club, and Lura Street house ground responses.

Once the traces are digitized and processed, more detailed analyses will be performed on these accelerograms. For instance, spectral analysis would reveal the predominant frequency content of each of the records, and comparisons between the free-field traces and basement responses will uncover information on the changes in frequency content due to soil-structure interaction. In particular, it will be interesting to see how much the response of a tall structure, such as Millikan Library, can contribute to the surrounding ground shaking. Also, cross-correlation techniques can be used to study wave propagation phenomenon within an extensively-monitored small urban area.

Since the fault geometry is fairly well known and the accelerograph stations are close to the epicenter, the digitized records should provide useful "high frequency" information about the source mechanism.

2.1.2 Whittier Narrows Aftershock of October 4, 1987 ($M_L = 5.3$)

Similar analyses can be performed on the accelerograms obtained for the major Whittier Narrows aftershock on October 4, 1987, measuring 5.3 on the Richter scale. The records are mainly characterized by a sharp horizontal double pulse, usually stronger in the north-south components than in the east-west components, and by smaller vertical motion. The duration of significant shaking lasted approximately 1–2 seconds, hence, the aftershock was significantly shorter than the main event. As was noticed in the analysis of the October 1 earthquake, the responses were the highest at the sites which best simulated free-field conditions in that soil-structure interaction was not expected to be substantial. The largest accelerations were recorded in the north-south trace of the Lura Street house accelerogram and measured 0.59g, the highest reported ground acceleration from the aftershock in the Los Angeles basin. On that same accelerogram, the east-west trace peaked at 0.28g. For the California Boulevard house, maximum accelerations were 0.52g in the north-south direction, and 0.32g in the east-west component, whereas for the I.R.C. building, these reached 0.28g and 0.23g, respectively. The other measured peak accelerations recorded on campus are listed in Table 1, and are consistently larger in the north-south components than in the east-west ones.

The fact that these Whittier Narrows aftershock records exhibit large horizontal motions, especially at the "free-field" stations, is consistent with the source mechanism of the causative right-lateral strike-slip fault along a north-northwest axis, which is almost in perfect alignment with the Caltech campus in Pasadena. From Table 1, it can also be seen that the peak ground responses are all much larger in the north-south than in the east-west component, which agrees with the fault strike orientation relative to the recording sites. There is a strong double pulse in the initial accelerations of all horizontal components. The waveform corresponding to the first half-second of this pulse should produce a half-second "hump" to the north in the displacement, which is consistent with the far-field displacement pattern of a strike-slip fault. Also, it suggests that buildings of five or so stories may have been hit relatively severely during the major aftershock because the predominant ground frequency coincided with the fundamental frequency of these structures.

Analyses similar to those proposed for the October 1 earthquake can be performed on the October 4 aftershock, such as identification of the waves emitted by the source, wave propagation effects, and soil-structure interaction. Since the two events are very different, this should prove to be a good way of testing the theories used to perform the above analyses.

2.2 Structural Response

2.2.1 Millikan Library

The Millikan Library building is a nine-story, reinforced concrete structure built in 1966. The library is 69 by 75 feet in plan and is 158 feet high above the basement level (Figure 4). Reinforced concrete shear walls located at the east and west end of the structure resist lateral loads in the north-south direction. The reinforced concrete central core, which houses the elevator and the staircase, provides partial resistance to the east-west loads. The foundation system consists of a 32 feet wide concrete pad below the central core, extending through the east-west length of the building, and by 10 feet wide beams positioned below the columns on the north and south sides of the structure. These beams are connected to the central pad by stepped beams. The foundation is built on alluvium composed of medium to dense sands, 900 feet above the bedrock. It did not suffer any structural damage during the Whittier earthquake or its major aftershock.

The location of the accelerometers in the basement, sixth floor and on the roof is illustrated in Figure 4. The recorded accelerograms are reproduced in Appendix A, Figures A.1 and A.2. During the main event, maximum accelerations in the NS (north-south) direction of 0.60g and 0.57g were recorded at the east and west sides of the roof, respectively. The approximate fundamental period of vibration in that direction is 0.75 sec. For the EW (east-west) components, the maximum accelerations recorded at the roof are 0.30g and 0.27g, with a fundamental period of about 1.00 sec. Table 3 summarizes the natural periods of Millikan Library, determined from both small amplitude, ambient vibration tests and strong-motion earthquake records. It can be noticed that there is a definite lengthening of the periods, especially in the NS direction. The results obtained during the October 1 Whittier event can be compared to those results from the stronger, but more distant, San Fernando earthquake of February 9, 1971 ($M_L = 6.5$), for which the recorded maximum ground accelerations of 20%g in the NS direction is similar, but yet the maximum acceleration of 31%g at the roof was almost half that of this last event. Clearly, the amplitudes around the NS fundamental frequency for the San Fernando earthquake were much lower than for the recent earthquake and, hence, did not excite the building's NS fundamental mode as much.

At the time of the San Fernando event, the response of Millikan Library had been extensively studied, and many post-earthquake forced vibration tests had been performed, such as those reported in [4], [5], [8] and [9]. The temporary 17% lengthening during the earthquake of the fundamental period in the NS direction, from 0.53 sec to 0.62 sec, can possibly be attributed to micro-cracking of the east and west end shear walls, loosening of connections between the non-structural components and structural system, and foundation softening. During the main Whittier event, the same fundamental period lengthened by 39%, from 0.54 sec prior to the earthquake to

0.75 sec, suggesting a temporary loss of stiffness in the NS direction of almost 50%. This, again, may be due to the same reasons suggested for the San Fernando earthquake stiffness reduction. Two separate studies have been published which analyze Millikan Library and its response to earthquake motion and forced excitation. Luco, et al. [8] concluded that the observed 8% change after the San Fernando earthquake in the NS fundamental period was due to a permanent loss of about 17% of the NS structural stiffnesses during the earthquake, rather than by loss of soil-foundation stiffness in that direction, as inferred by Foutch and Jennings [17]. As will be discussed later, the GEOS instruments installed after the Whittier Narrows earthquakes may give additional information on this point.

For the EW fundamental mode, the Millikan Library system experienced a temporary increase in the period of 40% during the San Fernando earthquake, and a permanent increase of 14% after the event. According to Luco, et al. [8], this may be attributed to both a permanent decrease of the structural stiffness of about 25%, and a smaller loss in the foundation stiffness. During the Whittier Narrows earthquakes, the structure-soil-foundation system suffered an increase in its EW fundamental period of 25%, and again the GEOS instrument records may help determine the share of structural versus soil-foundation yielding. However, based on past experience, it is possible to assume that part of the loss of structural stiffness can be attributed to micro-cracking of the central core, which is meant to resist the EW motions, and to loosening of nonstructural components within the building.

Once the accelerograms are digitized and processed, it will be possible to separate the translational and torsional motions at the roof by respectively adding and subtracting the motions at both east and west ends. Using system identification techniques [12], the torsional fundamental mode of the structure during the earthquake can be identified, and could be compared to the results obtained during post-San Fernando earthquake forced-vibration tests. From ambient vibration tests performed on Millikan Library in 1968, Udwadia and Trifunac have reported that the fundamental torsional period is about 0.35 sec [16].

During the major Whittier Narrows aftershock on October 4, 1987 ($M_L = 5.3$), the records of the Millikan Library show that the maximum accelerations at the roof were 0.40g in the NS direction and 0.33g in the EW direction, versus 0.25g and 0.17g, respectively, at the base. Hence, even though the NS ground peak measured during the aftershock was greater than that recorded during the main event by 30%, the peak roof response was smaller by 31%, which may be attributed to the different frequency content of both earthquakes around the fundamental NS frequency. The increase in the NS fundamental period was not as severe, increasing to 0.75 sec during the main shock, but only 0.70 sec during the aftershock. In the EW direction, for comparable ground excitation levels in the two events, the peak response recorded at the roof was greater by 14%

during the aftershock, with a 2% increase in the fundamental EW period from 1.00 sec to 1.02 sec between the two events. It will be necessary to perform more extensive studies of the digitized records in order to better understand the behavior of this structure during both events.

Soil-structure interaction and the loss of stiffness of the structure-foundation-soil system of Millikan Library can also be studied in the records obtained with the six U.S.G.S. GEOS instruments installed after the major Whittier Narrows shocks. These instruments successfully recorded three components of motions at the NW, NE and SE corners of the basement and the roof during two aftershocks on October 16 ($M_L = 2.8$) and October 21 ($M_L = 2.2$). The results obtained for the October 16 aftershock by Hanks et al. [13] are reproduced in Figures 5 through 11. Figures 5, 6 and 7 are the recorded velocity traces at the basement SE, NE and NW corners, and the roof SE, NE and west center for the vertical, NS and EW components, respectively. Hanks et al. also studied the frequency content of each of these traces as indicated by the Fourier spectra of Figure 8 for the roof velocities, and of Figure 9 for the basement velocities. These instruments were located in such a way that is is possible to separate the translational motions from the torsional motions by respectively adding and subtracting the records. Hanks et al. [13] concluded that the NS fundamental mode was accompanied by rocking of the foundation and its period had decreased from about 0.75 sec during the Whittier Narrows earthquake to 0.59 sec. Since the level of shaking was very small during the October 16 aftershock (approximately 0.06%g maximum NS ground acceleration versus 19%g for the main shock), it can be argued, from the change in periods in Table 3, that the soil-structure system suffered a permanent loss of stiffness of about 16% in the NS direction (9% increase in period since the ambient tests of May 1976) as a result of the October 1 and 4 earthquakes. Similarly, in the EW direction, the fundamental structural period during the October 16 aftershock showed a 5% increase from the 1976 ambient tests, and reflects a 9% permanent loss of stiffness arising from the Whittier Narrows events. Hanks et al. were also able to detect the structure's torsional mode in the spectra at 0.43 sec.

Because the GEOS instruments are digital recorders, it is easy to exactly align the traces in time. Comparison of the vertical basement and roof traces (Figure 10) clearly shows the propagation of the first arriving P-wave through the structure. This set of traces, recorded during the October 16 aftershock, displays a 0.02 sec delay between the basement and the roof, which are about 35 m apart, hence, indicating that the approximate velocity of the propagating wave was about 1.75 km/sec. Careful analysis of these traces should also reveal the existence of the initial propagating wave being reflected down from the roof.

By filtering the vertical basement traces and the NS roof traces around the 0.59 sec fundamental mode of the system, and by then aligning these traces in time (Figure 11), it is possible to witness clear evidence of base rocking. Along line AA′, the roof motions at all three locations are perfectly in phase moving to the north, and the records at the base show that at the same time, the south side of the foundation is going up, while the north side is going down. Similarly, along line BB′, the roof motions remain in phase moving to the south, while the south side of the base is going down and the north side is going up. Hence, the NS fundamental mode of the structure-foundation-soil system of Millikan Library is a combination of near-rigid foundation slab rocking about the EW axis, and the structure deforming in the NS direction.

In March 1988 a very low-level forced shaking test was performed on Millikan Library. This test showed the fundamental NS mode to be at 0.59 sec, and the fundamental EW mode at 0.85 sec, as indicated in Table 4. The forced shaking test also measured the building's fundamental torsional mode at 0.41 sec. These figures are very close to those obtained by Hanks, et al. with the GEOS instruments during the October 16 aftershock ($M_L = 2.8$). This validates the fact that Millikan Library suffered a permanent loss of stiffness during the Whittier Narrows events as discussed above.

The extensive amount of data recorded throughout Millikan Library during the main shock of October 1, during the aftershocks of October 4, 16 and 21, and through forced-shaking tests will enable researchers to perform a complete study of the structure's linear and nonlinear dynamic behavior, which should include the rocking motions at the base, as well as the torsional response of the building.

2.2.2 JPL Building 180

Building 180 is a nine-story steel-frame structure built in 1961. The building is 40 by 220 feet in plan and is 146 feet high above the basement level (Figure 12). The lateral loads in the NS direction are resisted by welded steel spandrel trusses and by steel columns partially encased in concrete. The longitudinal loads are carried by a frame consisting of steel girders and columns. The foundation system consists of continuous strip footing running longitudinally. This structure was strengthened after the 1971 San Fernando earthquake and did not suffer any structural damage during the Whittier Narrows main event or major aftershock.

The locations of the accelerometers in the basement and on the roof are illustrated in Figure 12. The recorded accelerograms for the October 1 and October 4 events are reproduced in Appendix A (Figure A.11 and A.12). Building 180 has been extensively studied in the past [10,14], and the summary of previous dynamic analyses as well as the current results are listed in Table 4.

The ground response recorded at JPL for both the main event and major aftershock are lower than those obtained on the Caltech campus. This is expected, since JPL is located about 10 km away from Pasadena and the propagating waves have attenuated in amplitude. However, the predominant characteristics of the two events are still apparent in both sets of traces. The main shock, which was mainly a thrust-fault mechanism, has a relatively large vertical component and high NS response. The major aftershock, produced by a north-northwest trending strike-slip fault, exhibits a periodic waveform in the NS component, with a period of about 0.2 sec, and very little motion in either the vertical or EW traces. The periodic appearance of the NS basement response recorded at JPL is visually very different from the ground motion recorded at Caltech for the same event. These differences may be attributed to the combination of wave propagation and dispersion phenomena, local site effects and structure-foundation interaction.

The structural responses of Building 180 to both the October 1 and October 4 events are shown in Figures A.11a and A.12a of the appendix. These records are characterized by relatively high vertical motions; for the main event, the maximum vertical ground acceleration is 18%g, compared to the peak vertical roof response of 29%g, and for the major aftershock, these figures are 11%g and 32%g, respectively. This is consistent with the behavior of the structure during the San Fernando earthquake in 1971, for which the maximum vertical accelerations were 13%g at the ground and 25%g at the roof. As suggested by J. Wood [14], the high vertical roof response may in part be due to the floor system's flexible steel trusses. However, a 3-D modal analysis of the structure should be performed to determine the contribution to these vertical motions of possible north-south roof slab rotation about the east-west axis.

The building's response along its longitudinal axis to both major Whittier events was relatively small, since peak E-W accelerations at the roof were only 12%g for the October 1 earthquake and 11%g for the October 4 earthquake, compared to maximum ground accelerations of 10%g and 8%g, respectively. However, the records show that the structure's motions were much larger along its transverse axis, since peak roof accelerations reached 40%g versus 17%g at the ground for the October 1 event, and 26%g versus 18%g, respectively, for the October 4 event. These values are summarized in Table 2 and again in Table 4 where they can be compared to the ones obtained during the San Fernando earthquake, for which the structure seems to have behaved quite differently, since peak accelerations were relatively much higher in the EW than in the NS records. Changes in the building's dynamic behavior may be due in part to the strengthening performed in 1974.

The approximate values of Building 180's EW fundamental period during the two major Whittier events are listed in Table 4, as well as results previously obtained from small-amplitude ambient vibration tests, the Lytle Creek earthquake (1970) and the San Fernando earthquake (1971) for both the EW and NS directions. Because the building was strengthened, its original dynamic properties were altered, and the results obtained during the Whittier Narrows earthquakes should be compared to the ambient vibration test results of July 1975 and June 1976, for which the fundamental period of the structure was about 0.95 sec along the longitudinal EW axis and

1.05 sec along the transverse NS axis. During the Whittier Narrows event of October 1, 1987, the building's fundamental period was approximately 1.3 sec longitudinally, which, compared to the post-strengthening results, reflects a period increase of 37%. It is difficult to identify the NS fundamental modal period of the structure from the undigitized film traces. The records will have to be processed before any conclusion about the building's earthquake response in its fundamental transverse mode can be made. However, these records show that Building 180 responded strongly to the October 1 event in its second transverse mode, for which the period is approximately 0.45 sec. McVerry [5] had identified the second transverse modal period to be 0.33 sec for the 1970 Lytle Creek earthquake and 0.40 sec for the 1971 San Fernando event. Hence, the period of the second NS mode increased by an extra 12% during the main Whittier Narrows earthquake. During the major aftershock, the longitudinal EW modal period appears to be 1.3 sec, which is the same as the structure's fundamental period during the main event of October 1. These values are close to those obtained through system identification techniques for the 1971 San Fernando earthquake [5,10]. Even though the building was strengthened in 1975, this may suggest that it behaved similarly to all three events in its longitudinal fundamental mode. As for the main Whittier Narrows earthquake, it is difficult to observe the building's fundamental transverse modal period directly from the film traces obtained during the major aftershock. However, the structure seems to have responded primarily in its second transverse mode, with a period of about 0.40 sec, with strong contribution from the third mode estimated to be at around 0.25 sec. This value for the third transverse modal period agrees with the results obtained by McVerry and Beck using system identification techniques on the synchronized San Fernando records [19].

Previous analyses of Building 180 from ambient vibration tests had identified the fundamental torsional mode period to be very close to that of the fundamental EW modal period. However, when time-invariant identification techniques were applied in the time domain by Beck [10] and in the frequency domain by McVerry [5] to the EW components of the base and roof records obtained during the San Fernando earthquake, this torsional mode could not be detected. Spectral analysis of the roof response revealed the existence of a double peak around the second longitudinal mode frequency, making it difficult to properly identify the modal properties of the structure. Both Beck and McVerry concluded that this double peak was caused by the time variation of the second longitudinal mode parameters, as the structure was behaving nonlinearly, and not by torsional response. However, since these analyses were made, modal identification techniques have improved in that they can now use at once all three components of shaking recorded at all the instrumented locations [12]. Applying this multiple input-multiple output modal identification method to both sets of records obtained at Building 180 during the two major Whittier Narrows events should locate the torsional modes, if they were excited.

The preliminary analyses of the Building 180 records obtained during the Whittier Narrows sequence show that it sustained strong levels of shaking, comparable to those measured during the San Fernando earthquake. Initial estimates of the fundamental periods indicate that the structure behaved nonlinearly during the temblors, and that it possibly suffered permanent loss of stiffness. Further investigations are required to determine the building's dynamic behavior during the two events, and to determine the effects of the post San Fernando earthquake structural strengthening.

2.2.3 JPL Building 183

JPL 183 is an asymmetric steel frame building comprised of a nine-story structure, 140 feet by 65 feet in plan, and an adjoining three-story structure at the east side, 113 feet by 65 feet in plan. Building 183 is partially embedded into the hillside along its north side up to the third floor, and measures 126 feet at its highest from the first floor to the roof level (Figure 13). The floor system consists of a concrete slab supported by I-beams. In the main structure, the transverse NS load is carried by the I-beams and by steel columns encased in concrete. The longitudinal loads are carried by the concrete slabs and steel columns. The foundation system consists of two strips of stepped concrete footing pads running longitudinally below each side of the structure. The location of the accelerometers is illustrated in Figure 13, and this is the first time that this structure has provided earthquake response records. Witnesses of the Whittier Narrows earthquake on October 1 and the major aftershock on October 4 have reported unusually high motions in the building, which is supported by the large accelerations in the NS direction observed in the accelerograms (Figure A.13).

Unfortunately, the accelerometer located at the fifth floor malfunctioned approximately 18 seconds after triggering by the main event, jamming the start-up and cut-off mechanism of the recorder. Hence, the response of JPL 183 to the major aftershock could not be recorded. The main shock traces are reproduced in Figure A-13 of the appendix; the roof record is much darker than the other traces because of photographic processing reasons.

As shown in Table 2, the peak accelerations in the NS components of Building 183 obtained at the fifth floor and roof levels are relatively large (37%g and 54%g, respectively) compared to the recorded ground peak acceleration of 20%g. The structural records show that Building 183 responded primarily in its two first transverse modes, with very little contribution from longitudinal motions. These two transverse modes are estimated to have periods of about 0.75 sec and 0.25 sec, making the ratio of the two first frequencies equal to 3. This ratio is consistent with theoretical results obtained for structures modeled as shear beams on fixed foundations. Fortunately, even though the instruments malfunctioned, they have recorded enough of the response of Building

183 to the Whittier Narrows main event to make it possible to properly investigate its dynamic behavior and properties.

3. CONCLUSION

The Whittier Narrows earthquake of October 1, 1987 ($M_L = 5.9$) and its major aftershock on October 4, 1987 ($M_L = 5.3$), which had epicenters only 10 km away from Pasadena, have been the most extensively monitored events in the history of the Caltech local strong-motion array. Preliminary analyses, based on the raw film traces, have suggested that the digitized and processed records will provide a wealth of information about ground motion variations and structural response mechanisms.

REFERENCES

1. Report of the Los Angeles County Earthquake Commission, "San Fernando Earthquake, February 9, 1971," November 1971.
2. A. F. Shakal, et al., "CSMIP Strong-Motion Records from the Whittier, California Earthquake of October 1, 1987," California Divisions of Mines and Geology, Office of Strong Motion Studies, Report OSMS 87-05, November 1987.
3. L. Jones, D. Given and K. Hutton, "Seismicity of Southern California, October 1987," USGS-CIT Memo, November 1987.
4. D. Foutch, G. Housner and P. Jennings, "Dynamic Responses of Six Multistory Buildings During the San Fernando Earthquake," Report No. EERL 75-02, Earthquake Engineering Research Laboratory, Caltech, Pasadena, California, October 1975.
5. G. H. McVerry, "Frequency Domain Identification of Structural Models from Earthquake Records," Report No. EERL 79-02, Earthquake Engineering Research Laboratory, Caltech, Pasadena, California, October 1979.
6. "Summary of the October 1, 1987 Whittier, California Earthquake: An EQE Quick Look Report," EQE Incorporated, Newport Beach, California, October 1987.
7. M. Celebi, G. Brady and H. Krawinkler, "Preliminary Evaluations of Structures: Whittier Narrows Earthquake of October 1, 1987," USGS Open-file report 87-621, October 1987.
8. J. E. Luco, M. D. Trifunac and H. L. Wong, "On the Apparent Change in Dynamic Behavior of a Nine-Story Reinforced Concrete Building," *Bulletin of the Seismological Society of America*, **77**, 6, pp. 1961–1983, December 1987.
9. H. Iemura and P. C. Jennings, "Hysteretic Response of a Nine-Story Reinforced-Concrete Building During the San Fernando Earthquake," Report No. EERL 73-07, Earthquake Engineering Research Laboratory, Caltech, Pasadena, California, 1973.
10. J. L. Beck, "Determining Models of Structures from Earthquake Records," Report No. EERL 78-01, Earthquake Engineering Research Laboratory, Caltech, Pasadena, California, June 1978.
11. E. Etheredge and R. Porcella, "Strong-Motion Data from the October 1, 1987 Whittier Narrows Earthquake," USGS Open-file report 87-616, Menlo Park, California, October 1987.
12. S. D. Werner, J. L. Beck and M. B. Levine, "Seismic Response Evaluation of Meloland Road Overpass Using 1979 Imperial Valley Earthquake Records," *Earthquake Engineering and Structural Dynamics*, **15**, pp. 249–274, 1987.
13. T. C. Hanks, R. D. Borcherdt, M. Celebi, C. S. Mueller, and J. L. Beck, "The Motion of Millikan Library at Very Small Amplitudes," Fall Meeting American Geophysical Union, San Francisco, December 1987.
14. J. H. Wood, "Analysis of the Earthquake Response of a Nine-Story Steel Frame Building During the San Fernando Earthquake," Report No. EERL 72-04, Earthquake Engineering Research Laboratory, Caltech, Pasadena, California, October 1972.

15. M. D. Trifunac and F. E. Udwadia, "Time and Amplitude Dependent Response of Structures," *International Journal of Earthquake Engineering and Structural Dynamics*, **2**, pp. 359–378, 1974.

16. F. E. Udwadia and M. D. Trifunac, "Ambient Vibration Tests of Full Scale Structures," *Proceedings Fifth World Conference on Earthquake Engineering*, Rome, 1974.

17. D. A. Foutch and P. C. Jennings, "A Study of the Apparent Change in the Foundation Response of a Nine-Story Reinforced Concrete Building," *Bulletin Seismological Society of America*, **68**, 1, pp. 219–229, February 1978.

18. E. Etheredge and R. Porcella, "Strong-Motion Data from the Whittier Narrows Aftershock of October 4, 1987," USGS, Open-file report 88-38, Menlo Park, California, January 1988.

19. G. McVerry and J. L. Beck, "Structural Identification of JPL Building 180 Using Optimally Synchronized Earthquake Records," Report No. EERL 83-01, California Institute of Technology, Pasadena, August 1983.

20. H. J. Degenkolb, et al., "The Whittier Narrows Earthquake, October 1, 1987," H. J. Degenkolb Associates, Engineers, San Francisco, California, 1987.

Table 1. Ground and basement response to the Whittier Narrows earthquakes, October 1987 recorded on the Caltech strong-motion network. (The time given is relative to the triggering time.)

Station			Max Accelerations			
		Component	Oct. 1, 1987 ($M_L = 5.9$)		Oct. 4, 1987 ($M_L = 5.3$)	
			%g	time (sec)	%g	time (sec)
Millikan Library		N	-19	5.5	-25	2.6
		Up	+18	1.25	-11	2.7
		E	-17	3.65	-17	2.45
Inner Ring	Mudd Laboratory	N	+19	5.05	-29	2.35
		Up	-16	0.9	-17	2.65
		E	-19	3.45	+17	2.25
	Bridge Laboratory	N	-17	5.35	+18	2.4
		Up	-14	0.85	+10	0.6
		E	+19	3.2	-17	2.5
	Keck Laboratory	N	-20	3.25	-20	2.2
		Up	-10	3.95	-8	2.9
		E	+19	4.75	+15	2.25
Outer Ring	Brown Athletic Center (Gym)	N	-19	2.7	-36	2.5
		Up	+16	3.8	-13	0.45
		E	+18	3.3	-19	2.40
	California Blvd. House	N	-26	3.3	-52	2.5
		Up	-20	2.5	+17	2.2
		E	-17	3.4	-32	2.7
	Athenaeum Club	N	-21	3.3	-26	2.4
		Up	-15	3.3	+17	2.05
		E	+12	3.9	+16	2.25
	Industrial Relations Center (IRC)	N	+25	4.75	-28	2.4
		Up	-21	3.2	+13	2.1
		E	+27	4.7	+23	2.2
	Lura St. House	N	+35	3.7	-59	2.5
		Up	-26	2.5	-17	0.35
		E	+36	4.45	-28	2.4

Table 2. Structural response to the Whittier Narrows earthquakes, October 1987, recorded on the Caltech strong-motion network.

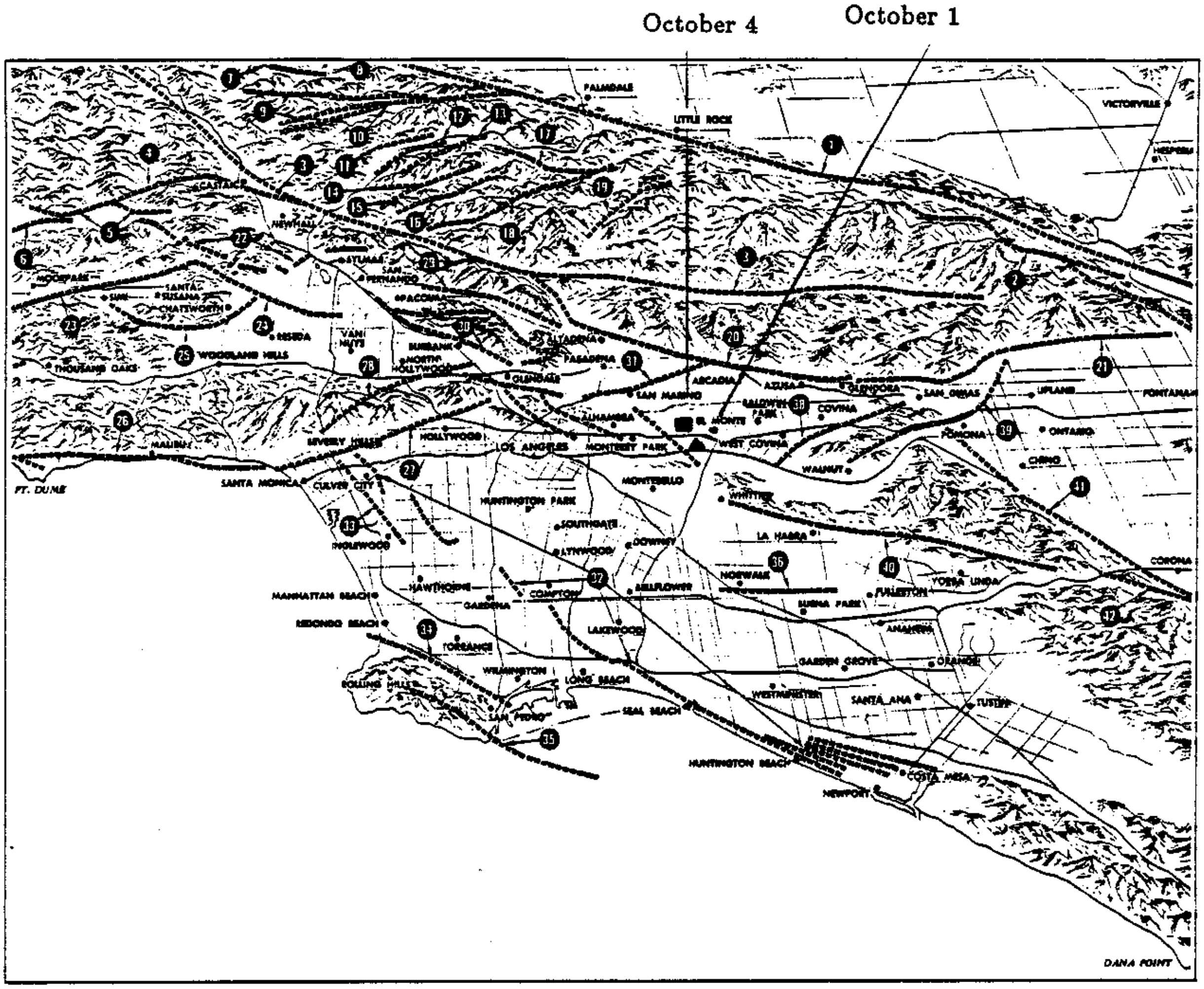
Station	Level	Max Accelerations (%g)		
		Component	Oct. 1, 1987 ($M_L = 5.9$)	Oct. 4, 1987 ($M_L = 5.3$)
Millikan Library	Basement	S	+19	+25
		Up	+18	-11
		E	-17	-17
	Sixth Floor	S	+28	---
		E	+22	---
	West Roof	S	+57	+43
		Up	-27	+21
		E	-27	+33
	East Roof	S	+60	+36
		E	-30	+33
JPL Building 180	Basement	E	+10	+8
		Up	-18	-11
		N	-17	+18
	Roof	E	-12	+11
		Up	+29	+32
		N	-40	-26
JPL Building 183	First Floor	N	-20	---
		Up	+11	---
		W	-9	---
	Fifth Floor	N	+37	---
		Up	+11	---
		W	-10	---
	Roof	N	+54	---
		Up	-19	---
		W	-10	---

Table 3. Fundamental periods for Millikan Library determined from small-amplitude ambient vibrations and strong-motion earthquake response. The Whittier earthquake results are from interpretation of the 70-mm film records and are preliminary. The table was adapted from [5].

Date	North-South			East-West		
A=Ambient E=Earthquake F=Forced	Ground Peak (%g)	Roof Peak (%g)	Natural Period (s)	Ground Peak (%g)	Roof Peak (%g)	Natural Period (s)
July 1969 (A)			0.53			0.69
Sept. 12, 1970 (E) (Lytle Creek)	1.9	5.4	0.52	1.9	3.5	0.71
Feb. 9, 1971 (E) (San Fernando)	20	31	0.62	18	34	0.98
March 1971 (A)			0.56			0.80
May 1976 (A)			0.54			0.79
Oct. 1, 1987 (E) (Whittier, $M_L = 5.9$)	19	58	0.75	17	29	1.00
Oct. 4, 1987 (E) (Whittier, $M_L = 5.3$)	25	40	0.70	17	33	1.02
Oct. 16, 1987 (E) (Whittier, $M_L = 2.8$)			0.59			0.83
March 1988 (F)			0.59			0.85

Table 4. Fundamental periods for JPL Building 180 determined from small-amplitude ambient vibrations and strong-motion earthquake response. The Whittier earthquake results are from interpretation of the 70-mm film records and are preliminary. All other results are from [5].

Date	**S82E (Longitudinal) E**			**S08W (Transverse) N**		
A=Ambient E=Earthquake	Ground Peak (%g)	Roof Peak (%g)	Natural Period (s)	Ground Peak (%g)	Roof Peak (%g)	Natural Period (s)
1963 (A)			0.91			0.88
Sept. 12, 1970 (E) (Lytle Creek)	1.5	2.5	1.02	2.4	3.7	1.13
Feb. 9, 1971 (E) (San Fernando)	21	38	1.25	14	21	1.42
July 1971 (A)			1.05			1.11
Feb. 1972 (A)			1.00			1.15
July 1975 (A) (After Strengthening)			0.96			1.04
June 1976 (A)			0.95			1.05
Oct. 1, 1987 (E) (Whittier $M_L = 5.9$)	10	12	1.30	17	40	?
Oct. 4, 1987 (E) (Whittier $M_L = 5.3$)	8	11	1.30	18	26	?



1 – SAN ANDREAS FAULT
2 – SAN JACINTO FAULT
3 – SAN GABRIEL FAULT
4 – HOLSER FAULT
5 – SAN GAYETANO FAULT
6 – OAK RIDGE FAULT
7 – LIEBRE FAULT ZONE
8 – CLEARWATER FAULT
9 – BEE CANYON FAULT
10 – SAN FRANCISQUITO FAULT
11 – VASQUES CANYON FAULT
12 – MINT CANYON FAULT
13 – GREEN RANCH FAULT
14 – SOLEDAD FAULT
15 – POLE CANYON FAULT
16 – MAGIC MOUNTAIN FAULT
17 – ACTON FAULT
18 – TRANSMISSION LINE FAULT
19 – PACIFIC MOUNTAIN FAULT
20 – SIERRA MADRE FAULT ZONE
21 – CUCAMONGA FAULT ZONE
22 – SANTA SUSANA THRUST
23 – SANTA ROSA FAULT
24 – NORTHRIDGE HILLS FAULT
25 – CHATSWORTH FAULT
26 – MALIBU COAST FAULT
27 – SANTA MONICA FAULT
28 – SEPULVEDA FAULT
29 – TUJUNGA FAULT
30 – VERDUGO FAULTS
31 – RAYMOND HILL FAULT
32 – NEWPORT INGLEWOOD FAULT ZONE
33 – CHARNOCK FAULT
34 – PALOS VERDES FAULT ZONE
35 – CABRILLO FAULT
36 – NORWALK FAULT
37 – WORKMAN MILL FAULT EXTENSION
38 – WALNUT CREEK FAULT
39 – SAN JOSE FAULT
40 – WHITTIER FAULT ZONE
41 – CHINO FAULT ZONE
42 – ELSINORE FAULT

Figure 1 Los Angeles area fault map [1] (October 1, 1987 epicenter, October 4, 1987 epicenter).

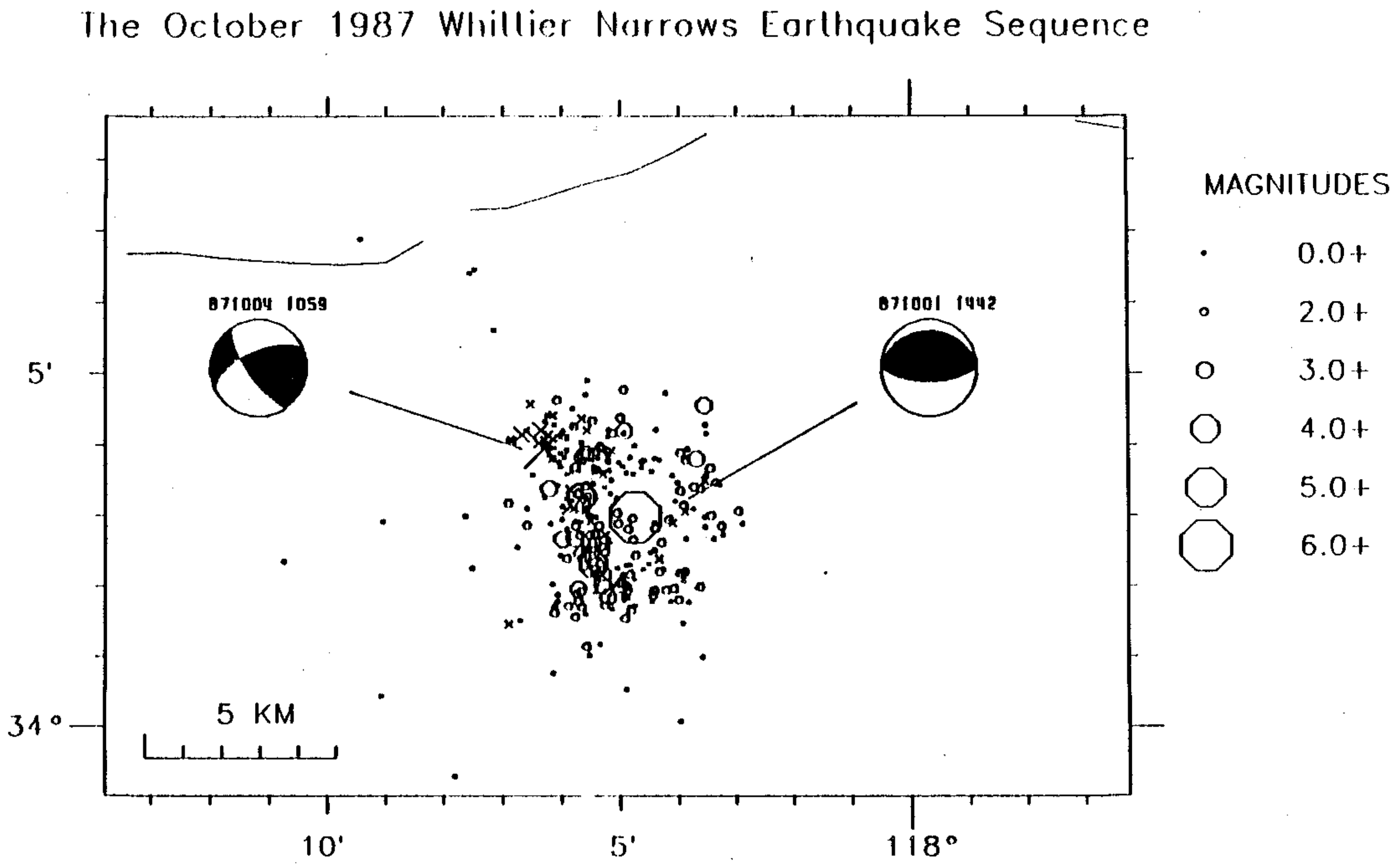


Figure 2 Focal mechanisms of the main event (right) and major aftershock (left) [3].

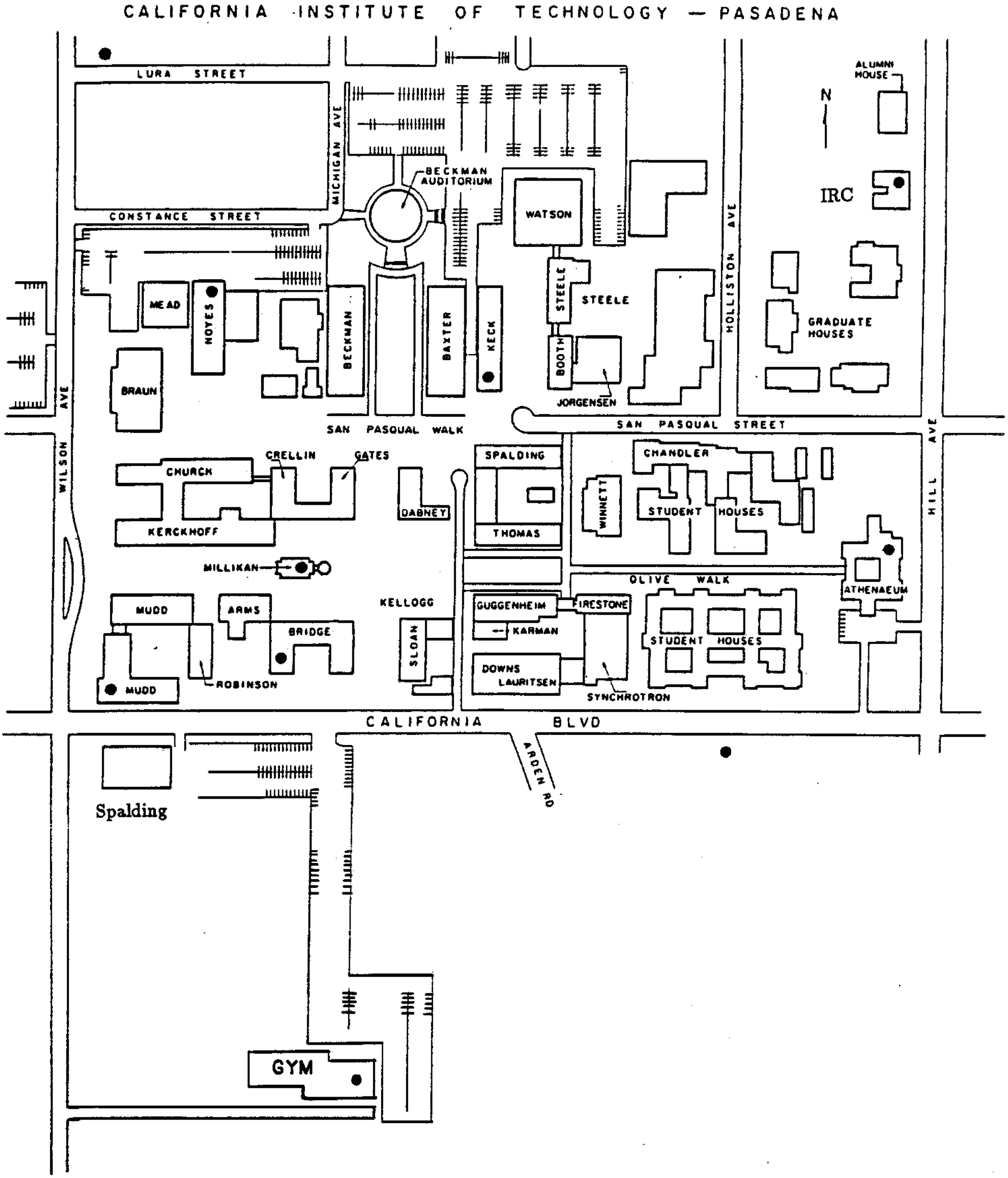
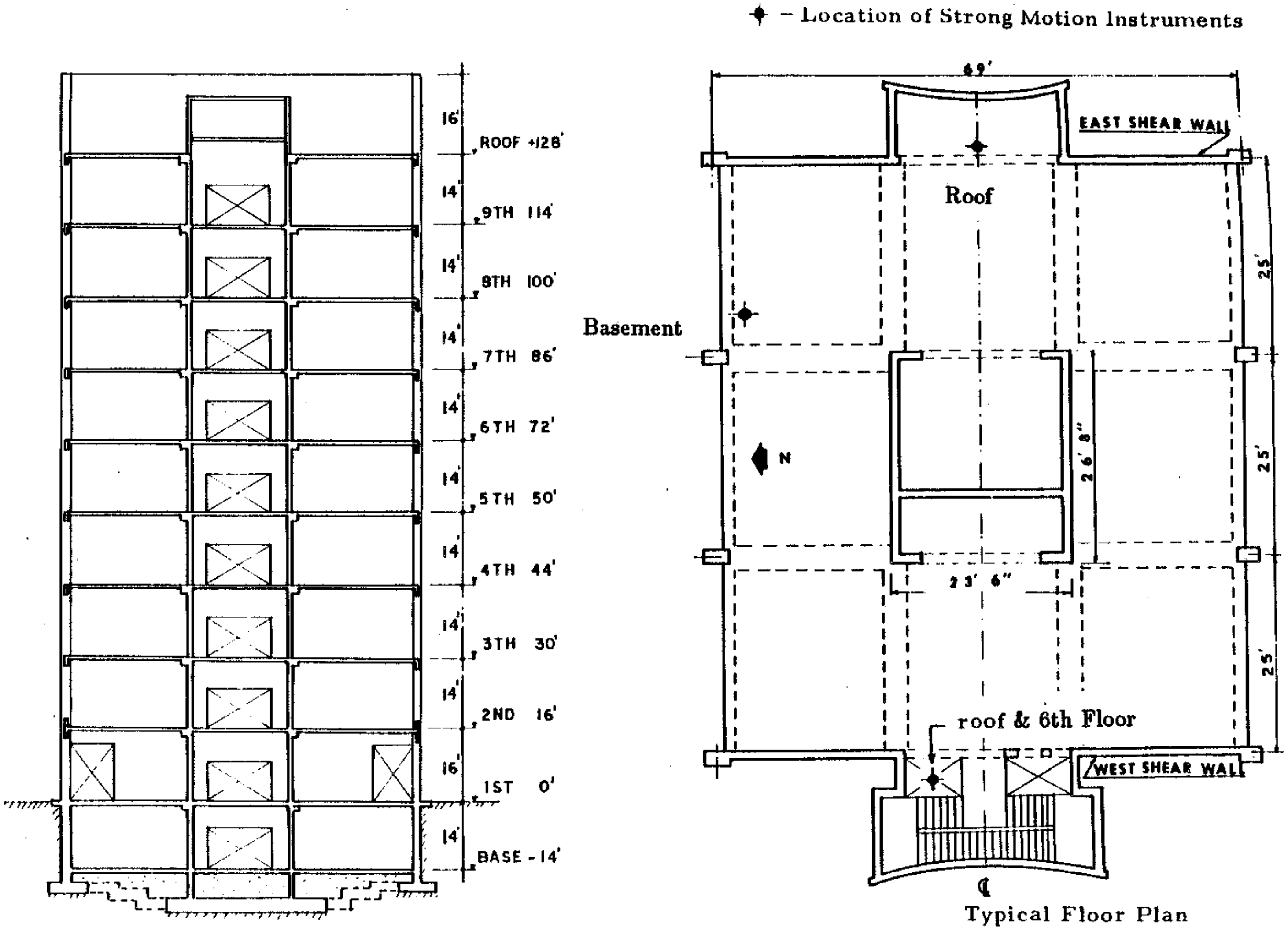


Figure 3 Locations of the strong-motion accelerographs on the Caltech campus (see Table 1).



Transverse Section

Typical Floor Plan

Figure 4 Schematic of Millikan Structural System [4].

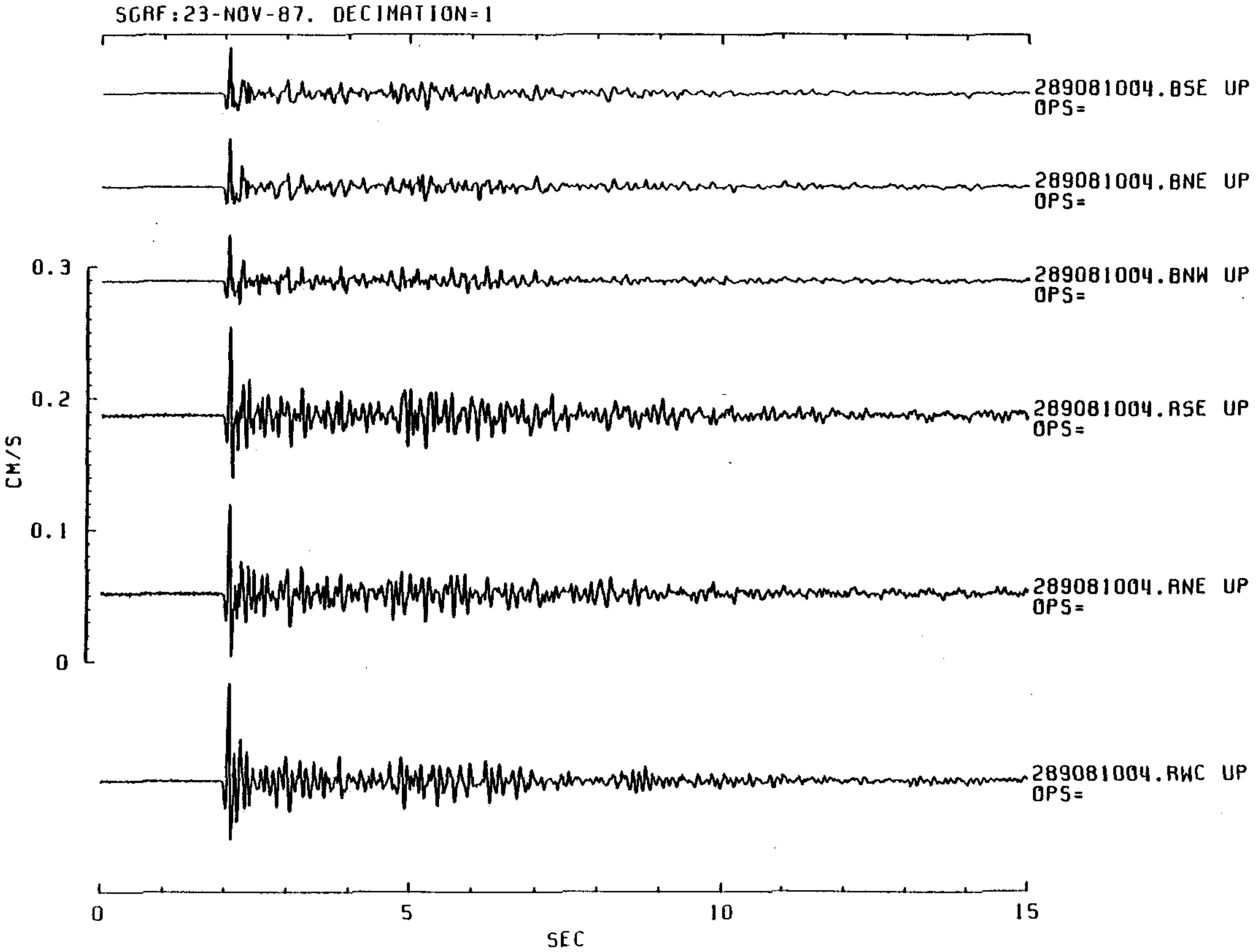


Figure 5 GEOS vertical component velocity traces recorded October 16, 1987 at the basement SE, NE and NW, and roof SE, NE and west-center locations.

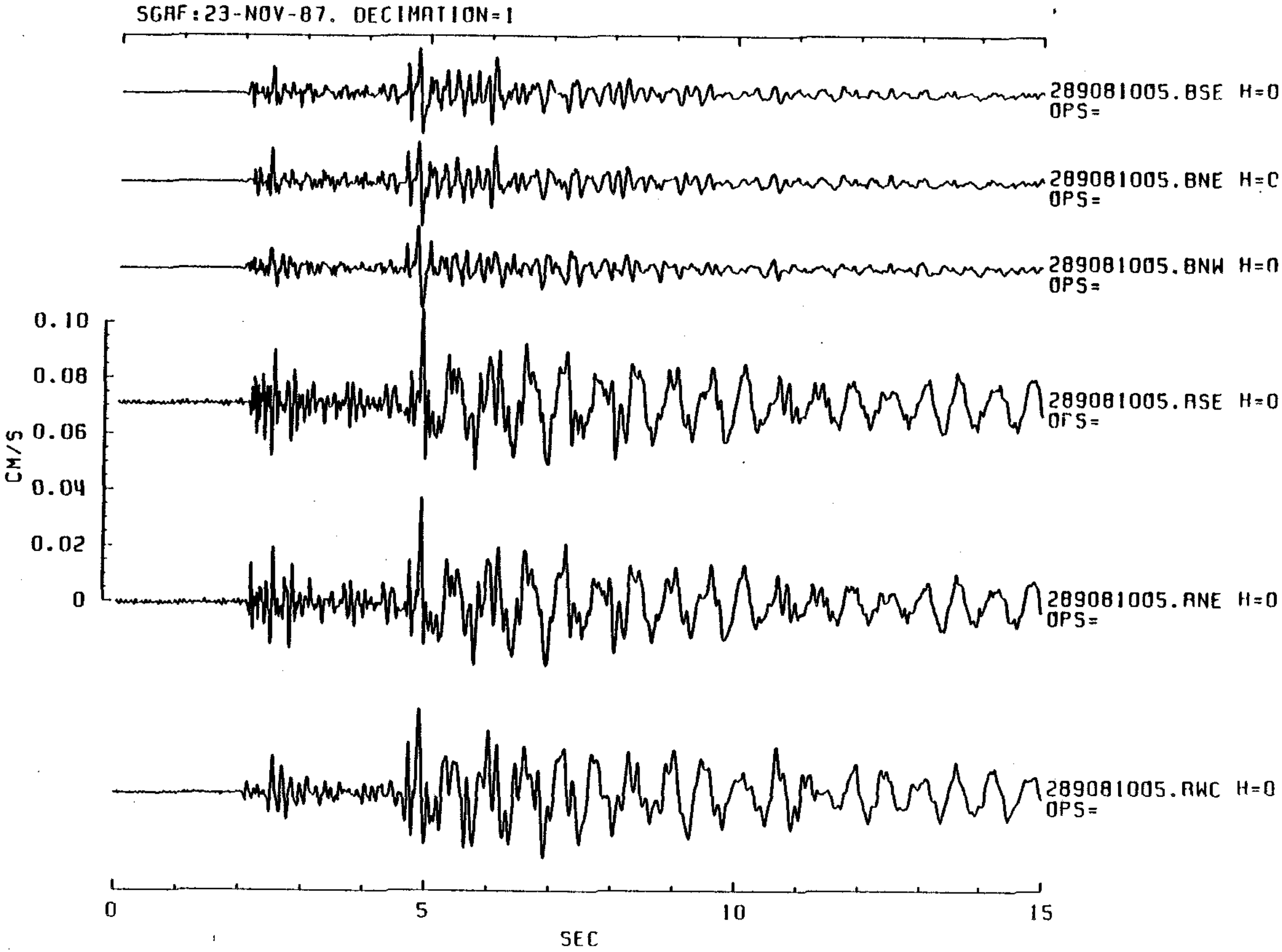


Figure 6 GEOS NS component velocity traces recorded October 16, 1987 at the basement SE, NE and NW, and the roof SE, NE and west-center locations.

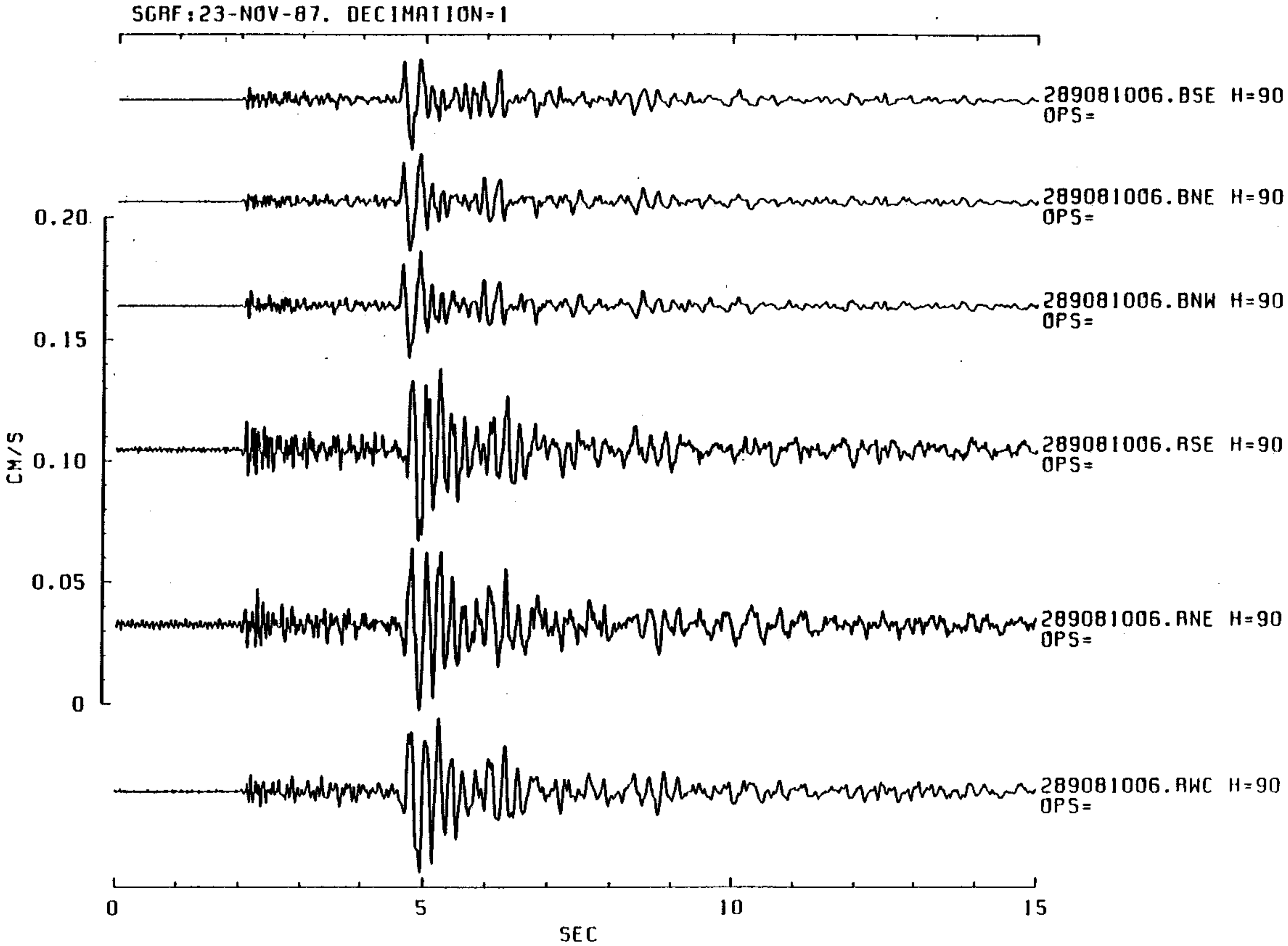


Figure 7 GEOS EW component velocity traces recorded October 16, 1987 at the basement SE, NE and NW and roof SE, NE and west-center locations.

MILLIKAN LIBRARY
AFTERSHOCK OF OCTOBER 16, 1987

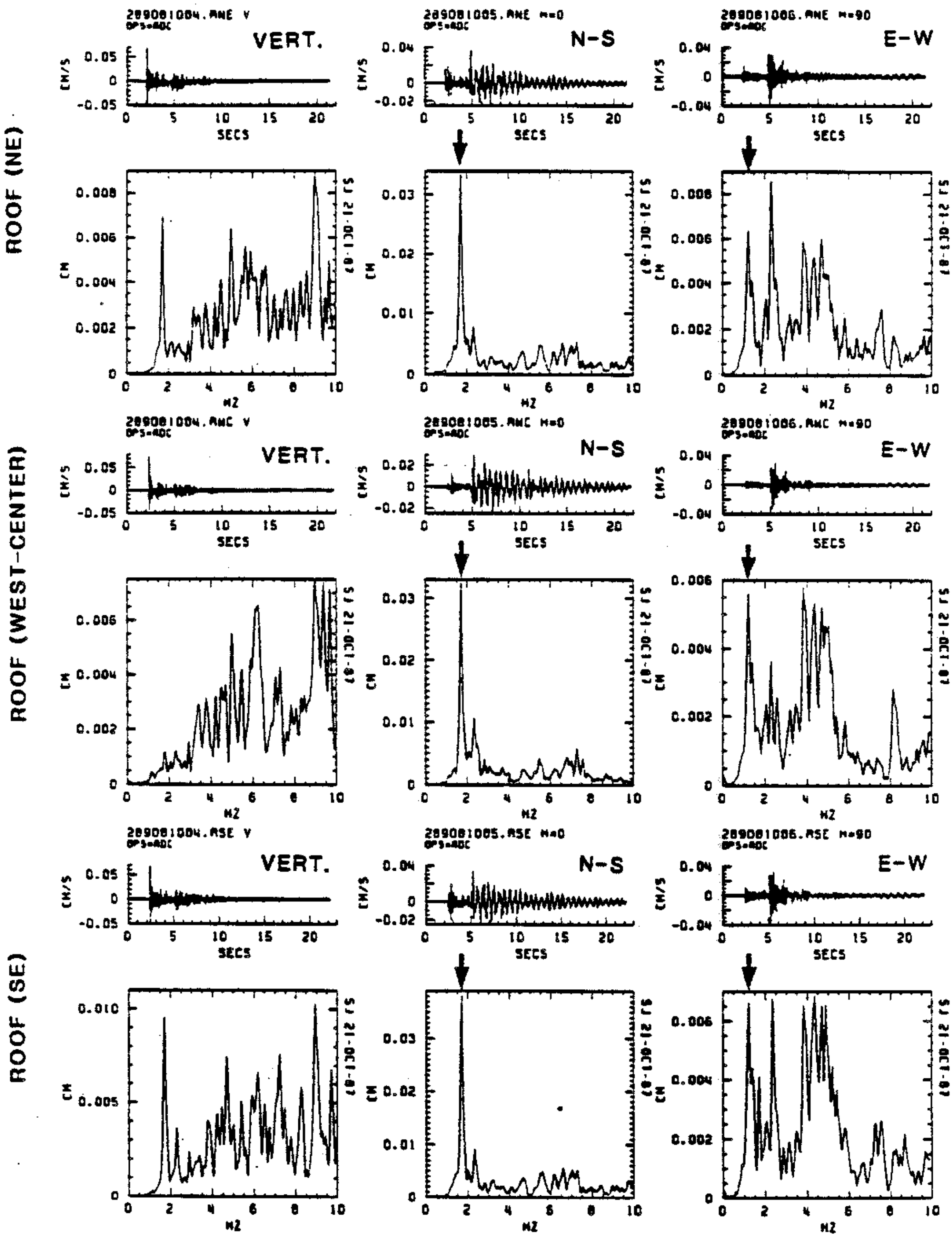


Figure 8 Velocity spectra of the GEOS, NE, west-center and SE roof locations for the three measured components.

MILLIKAN LIBRARY

AFTERSHOCK OF OCTOBER 16, 1987

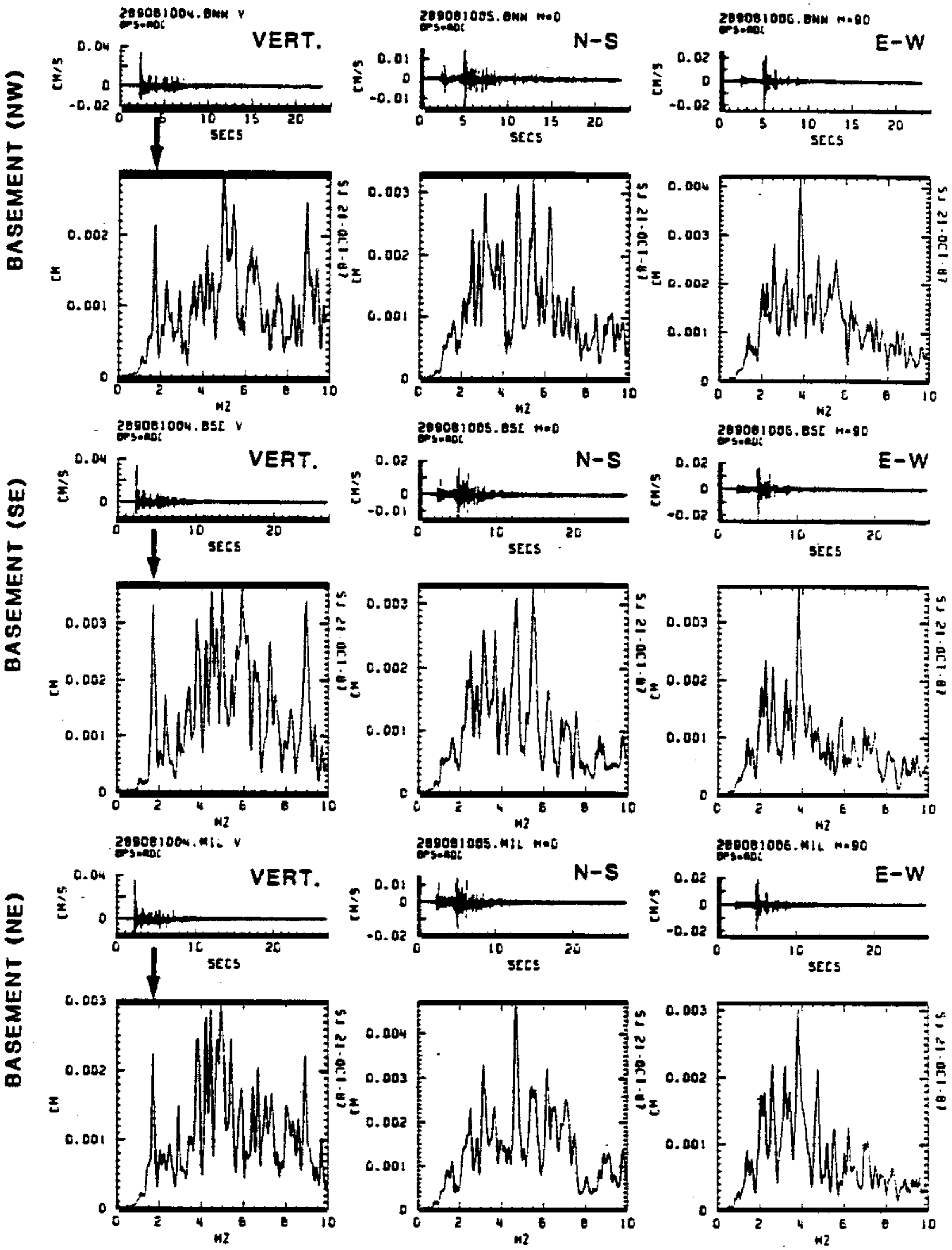


Figure 9 Velocity spectra of the GEOS NW, SE, NE basement locations for the three measured components.

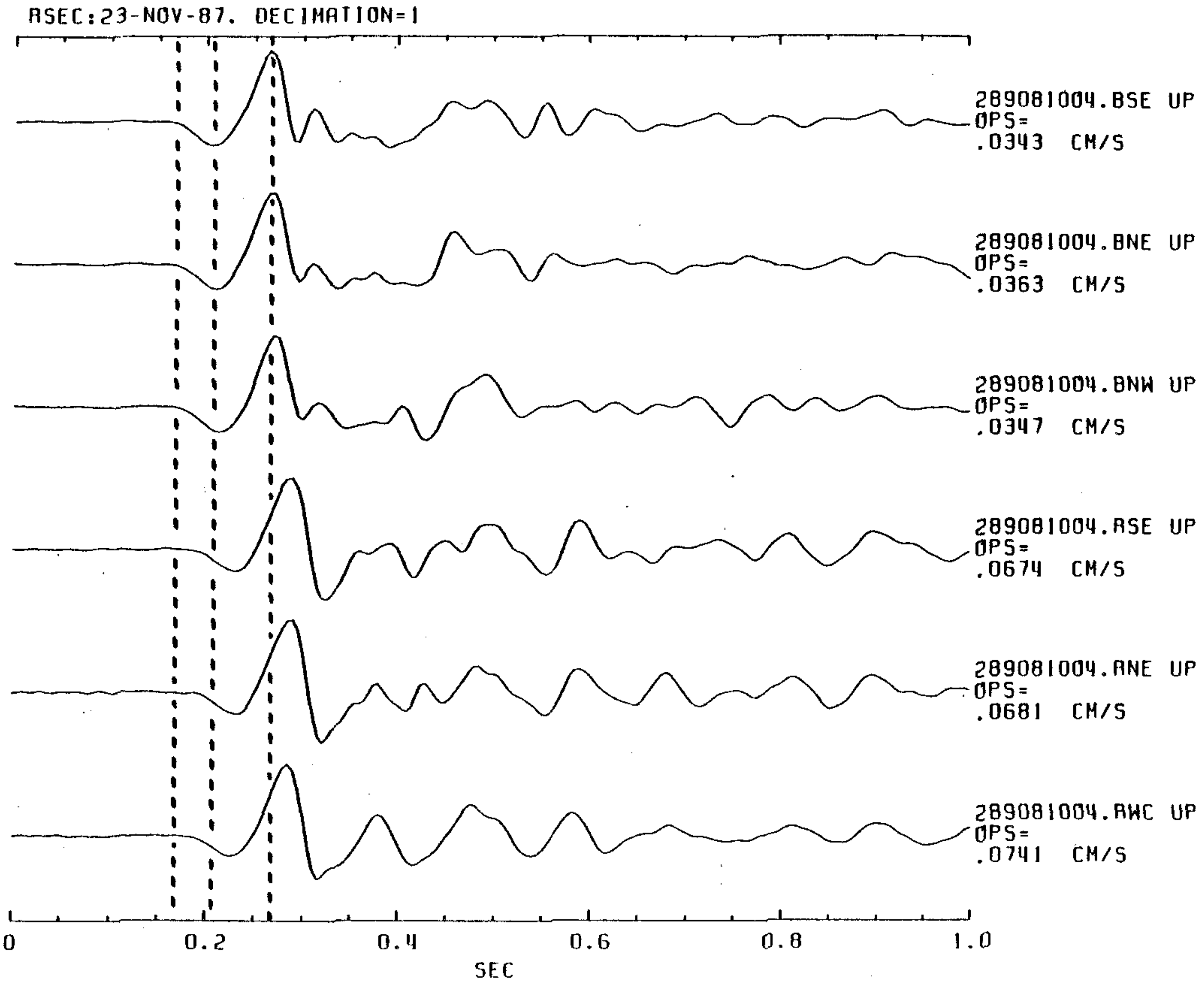


Figure 10 P-wave propagation delay along Millikan Library as recorded by the GEOS instruments during the October 16, 1987 Whittier Narrows sequence.

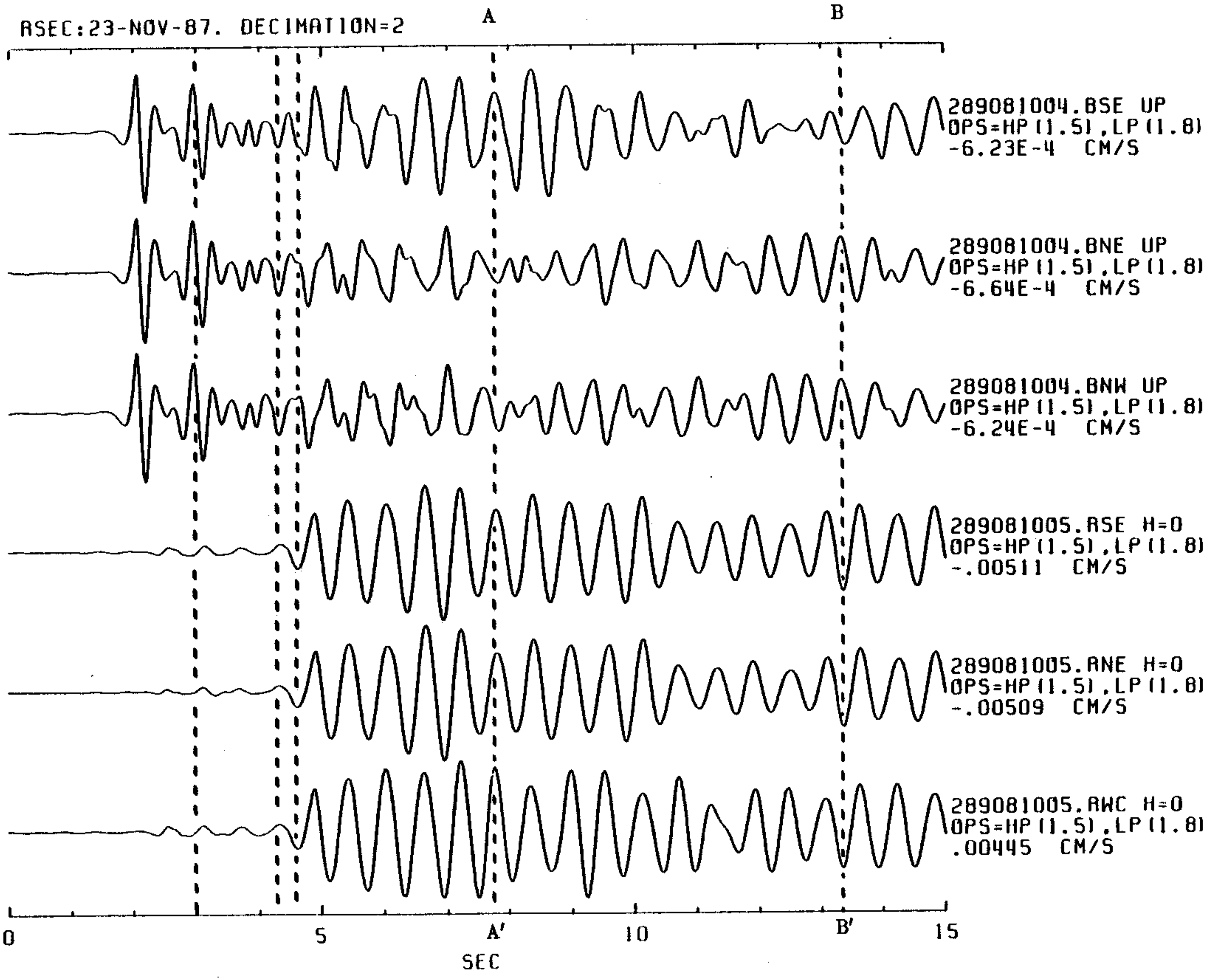


Figure 11 GEOS instrument vertical basement traces and NS roof traces filtered around the NS fundamental period (0.59 sec), which clearly indicate rocking of the base during the October 16, 1987 aftershock.

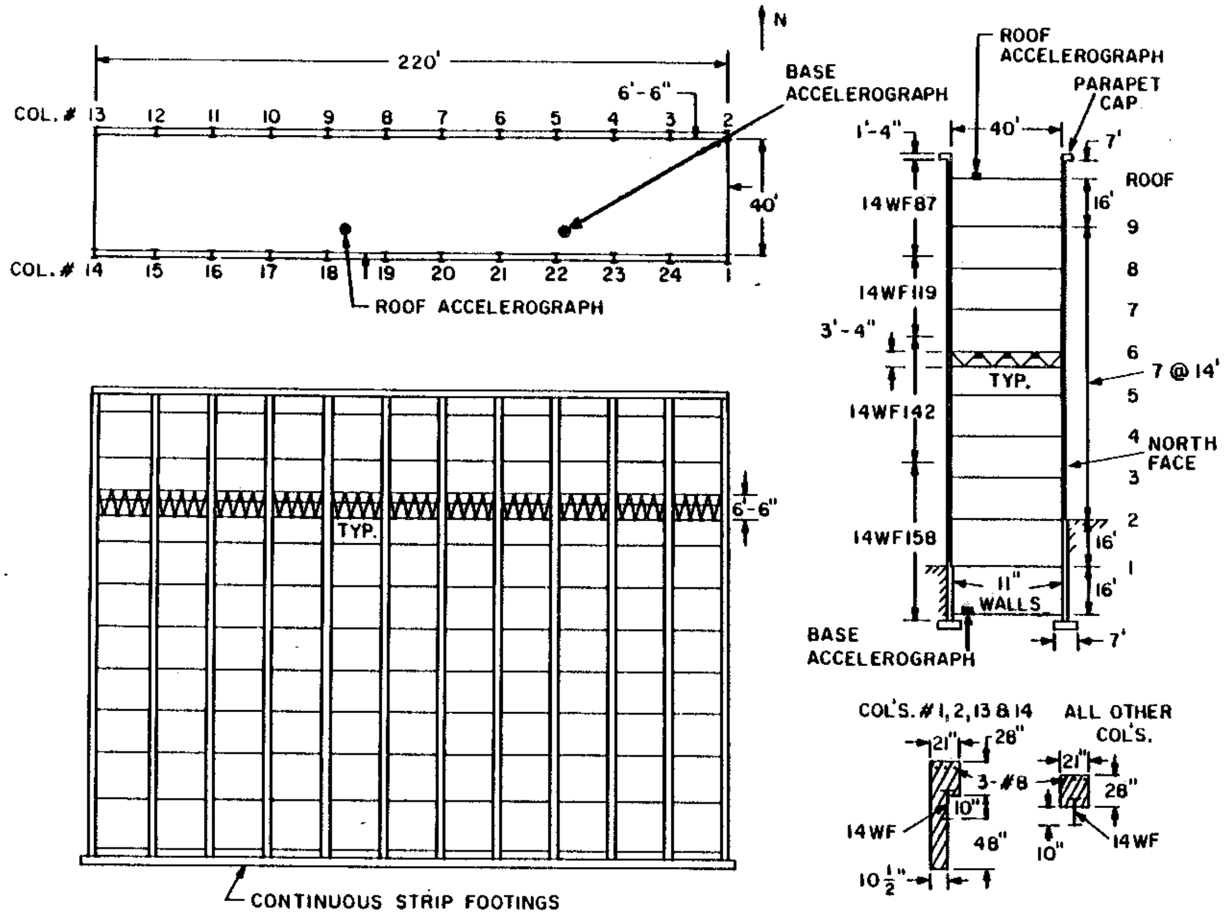


Figure 12 Longitudinal and transverse sections and floor plan of JPL Building 180 [4].

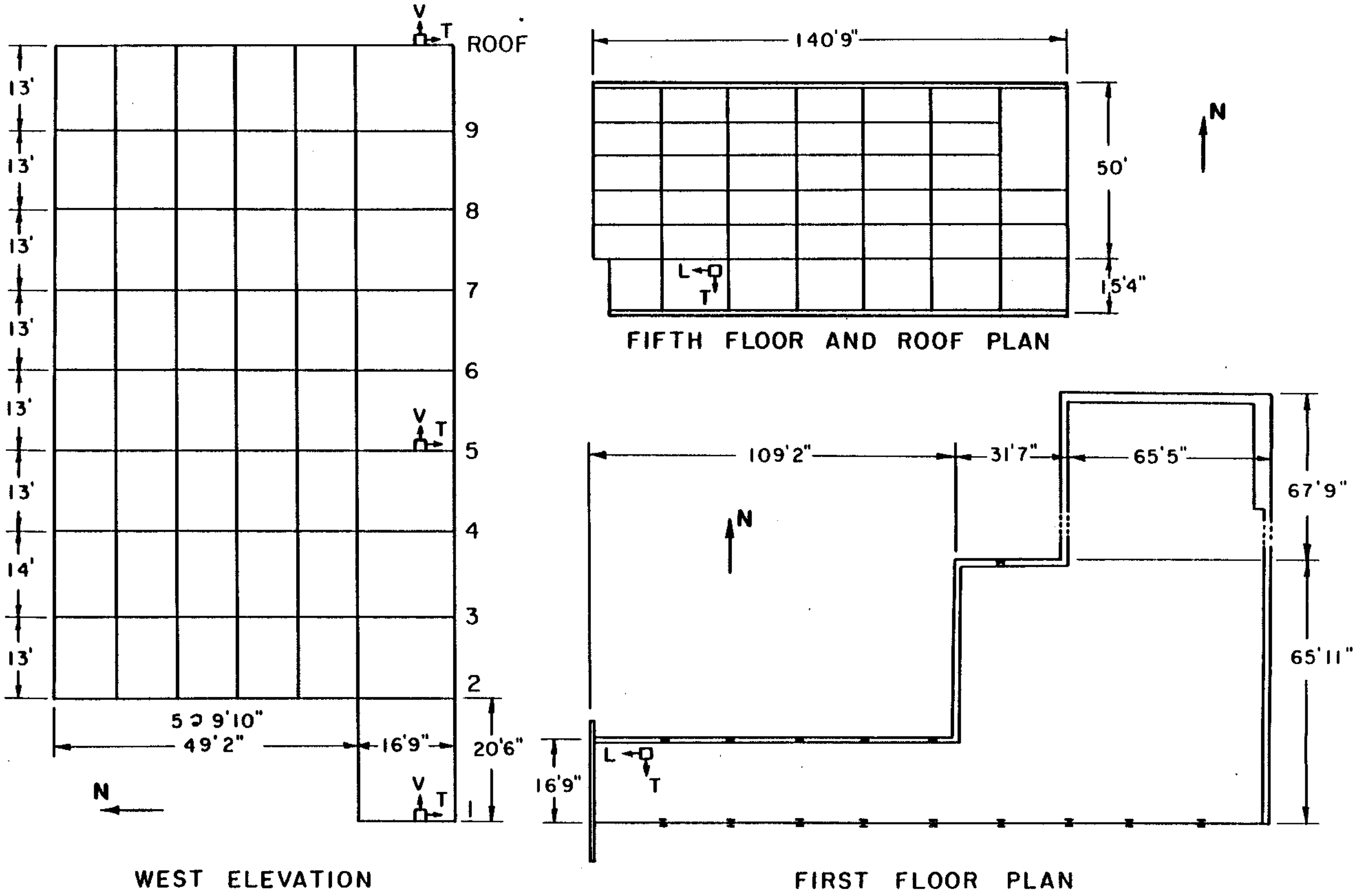


Figure 13 Schematic of JPL Building 183 structural system.

APPENDIX

PHOTOCOPIED ACCELEROGRAMS

FROM THE CALTECH NETWORK

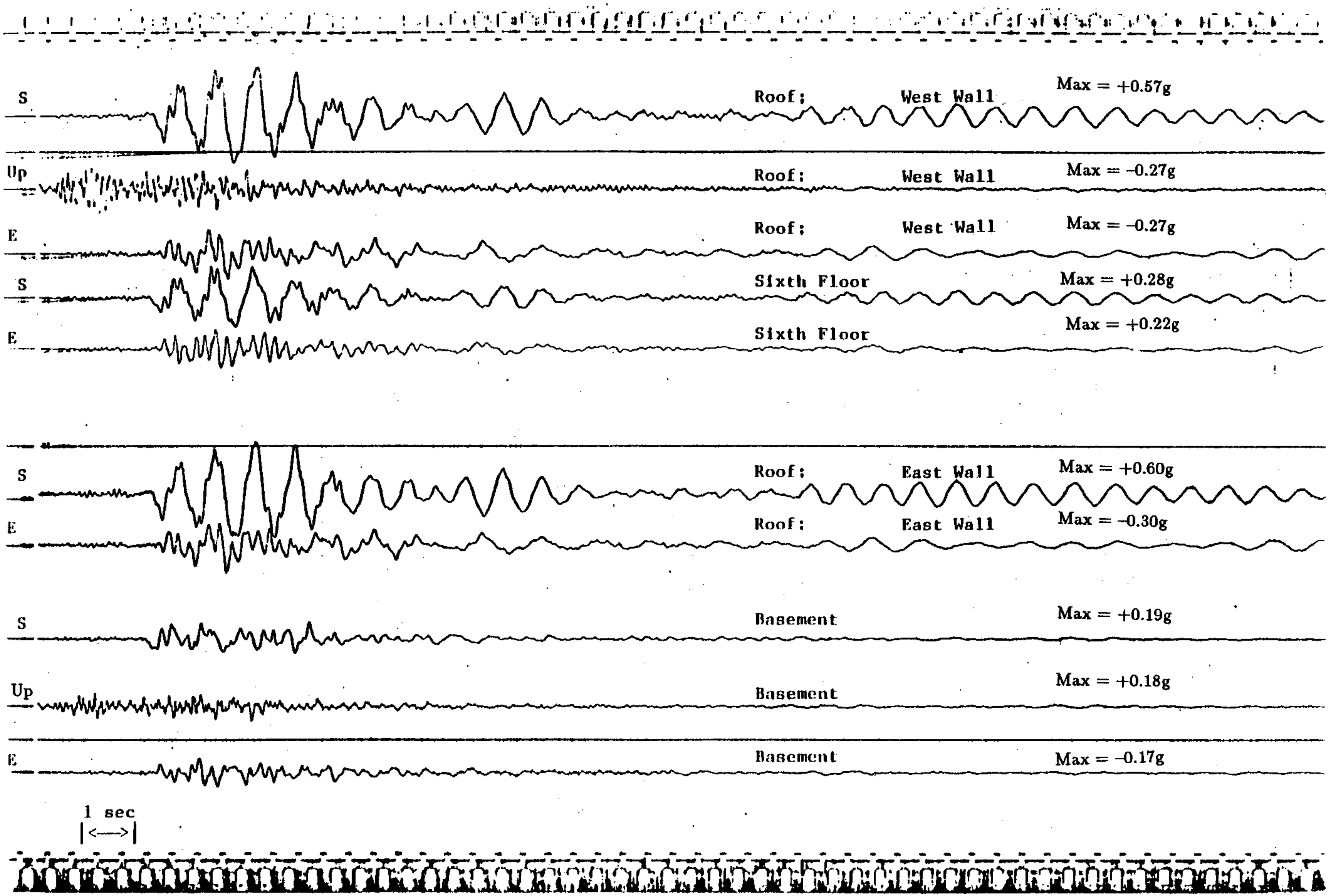


Figure A.1 Millikan Library, Whittier earthquake, October 1, 1987 ($M_L = 5.9$).

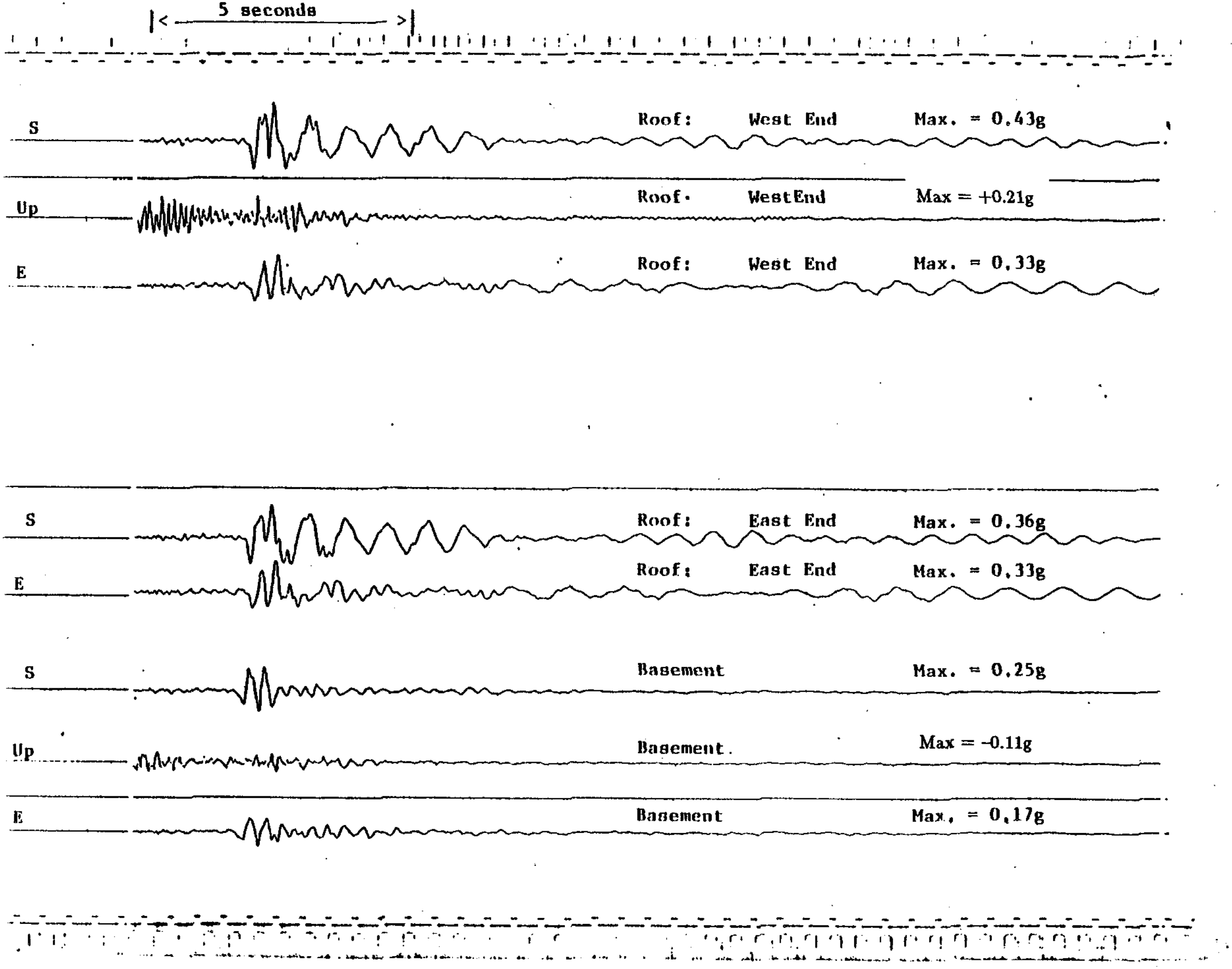


Figure A.2 Millikan Library, Whittier aftershock, October 4, 1987 (M_L = 5.3)

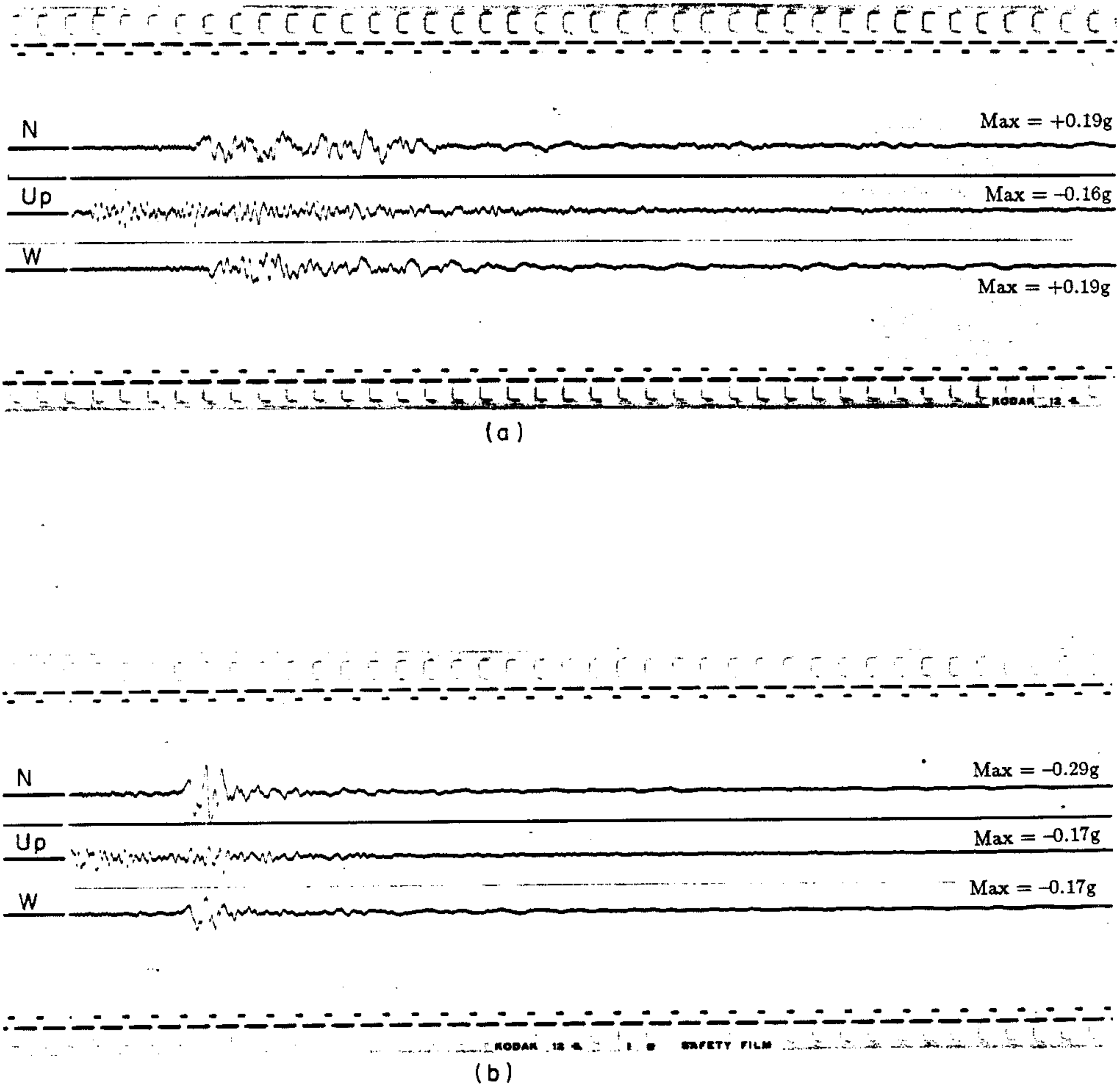


Figure A.3 Mudd Laboratory
(a) Whittier earthquake, October 1, 1987 ($M_L = 5.9$).
(b) Main aftershock, October 4, 1987 ($M_L = 5.3$).

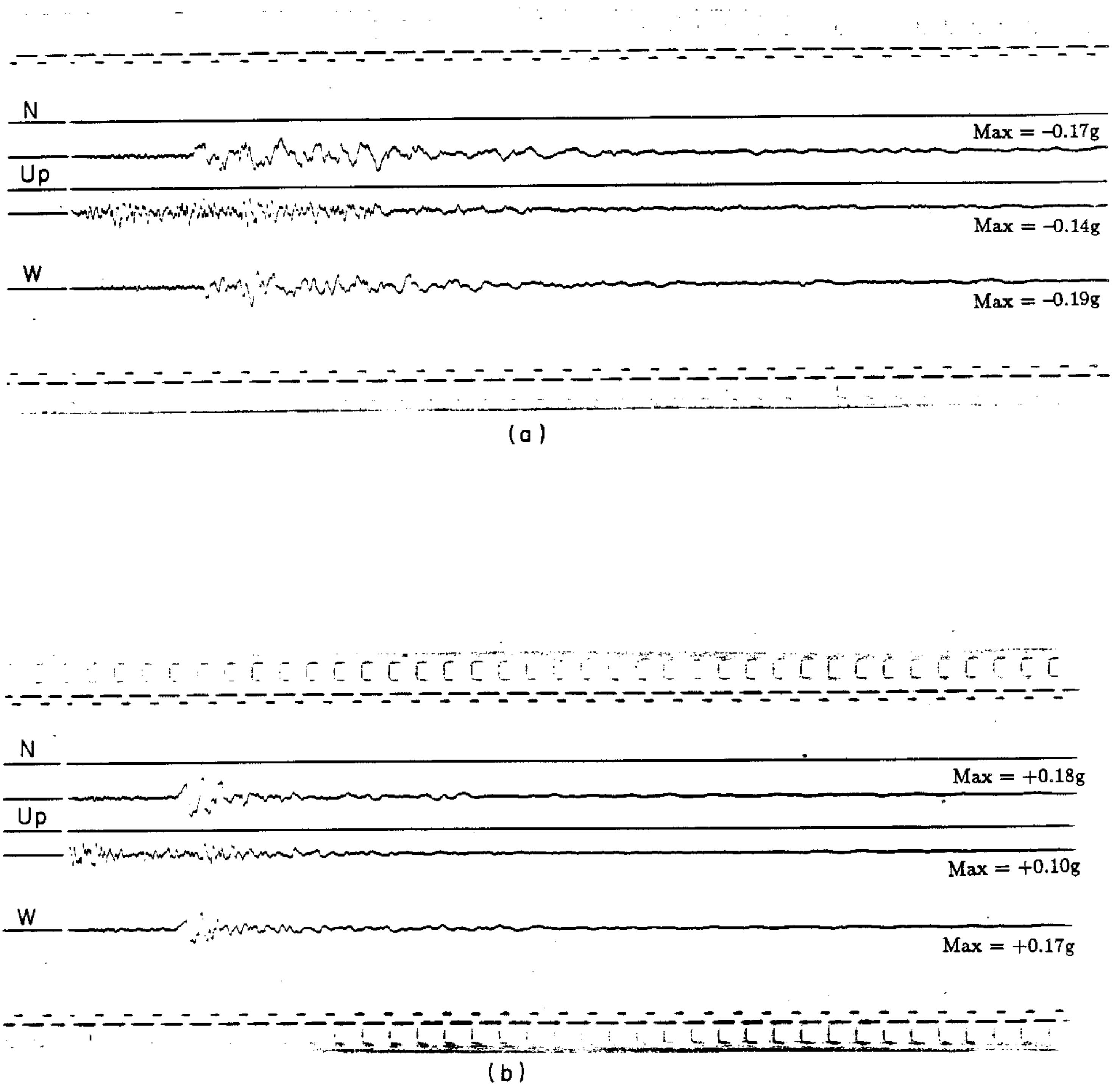


Figure A.4 Bridge Laboratory
(a) Whittier earthquake, October 1, 1987 ($M_L = 5.9$).
(b) Main aftershock, October 4, 1987 ($M_L = 5.3$).

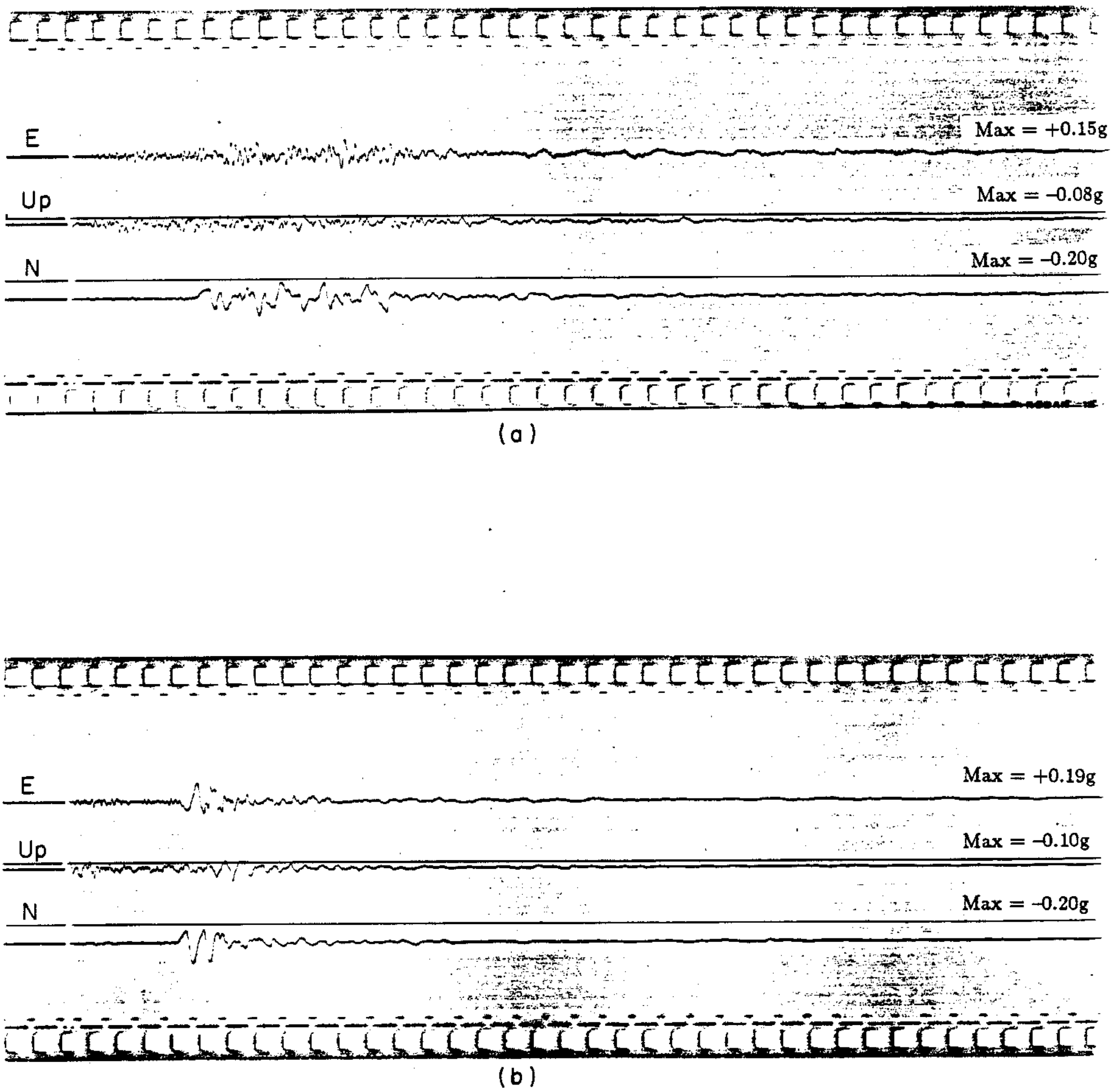


Figure A.5 Keck Laboratory
(a) Whittier earthquake, October 1, 1987 ($M_L = 5.9$).
(b) Main aftershock, October 4, 1987 ($M_L = 5.3$).

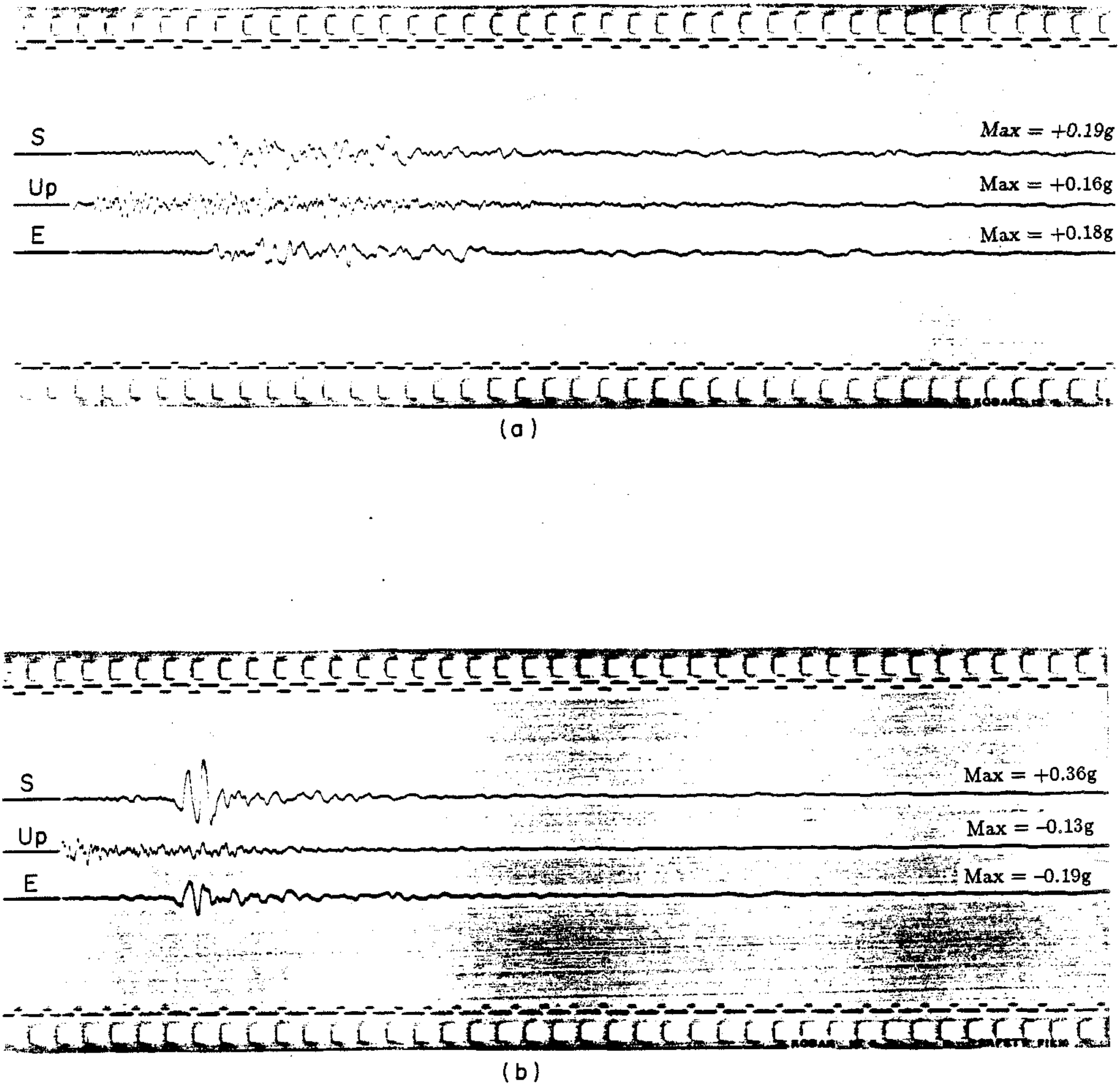


Figure A.6 Brown Athletic Center
(a) Whittier earthquake, October 1, 1987 ($M_L = 5.9$).
(b) Main aftershock, October 4, 1987 ($M_L = 5.3$).

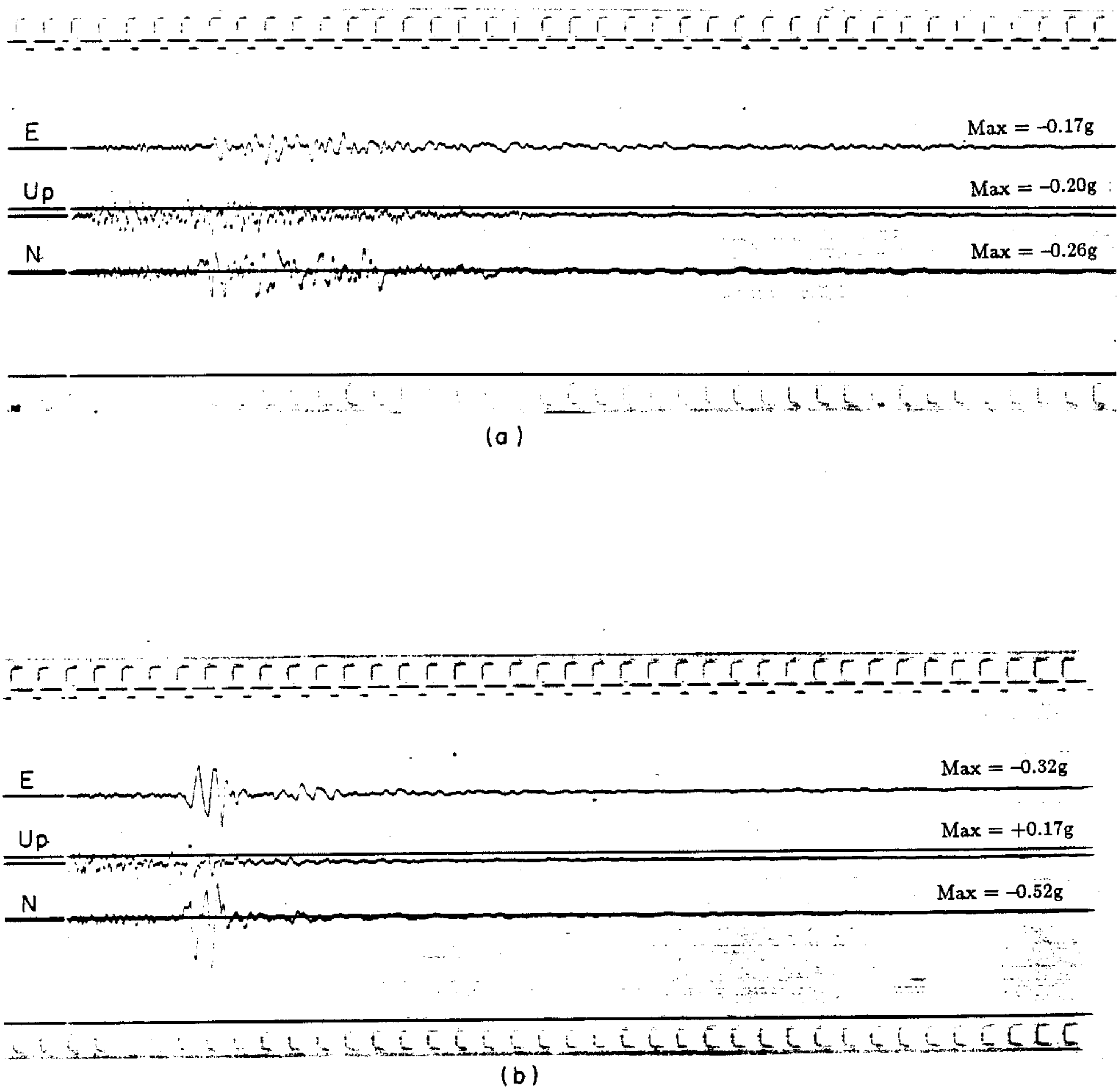


Figure A.7 California Boulevard house
(a) Whittier earthquake, October 1, 1987 ($M_L = 5.9$).
(b) Main aftershock, October 4, 1987 ($M_L = 5.3$).

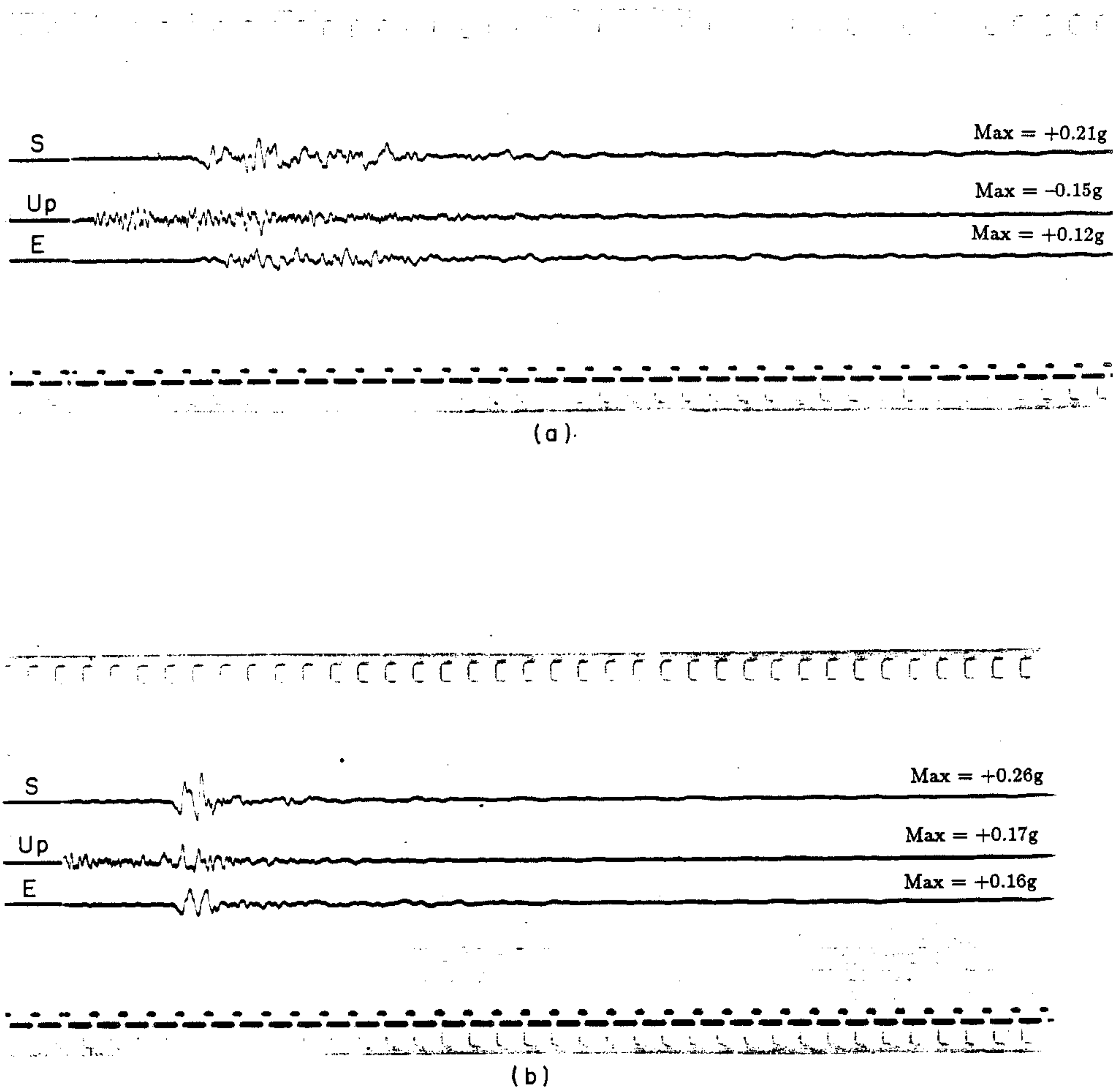


Figure A.8 Athenaeum Faculty Club
(a) Whittier earthquake, October 1, 1987 ($M_L = 5.9$).
(b) Main aftershock, October 4, 1987 ($M_L = 5.3$).

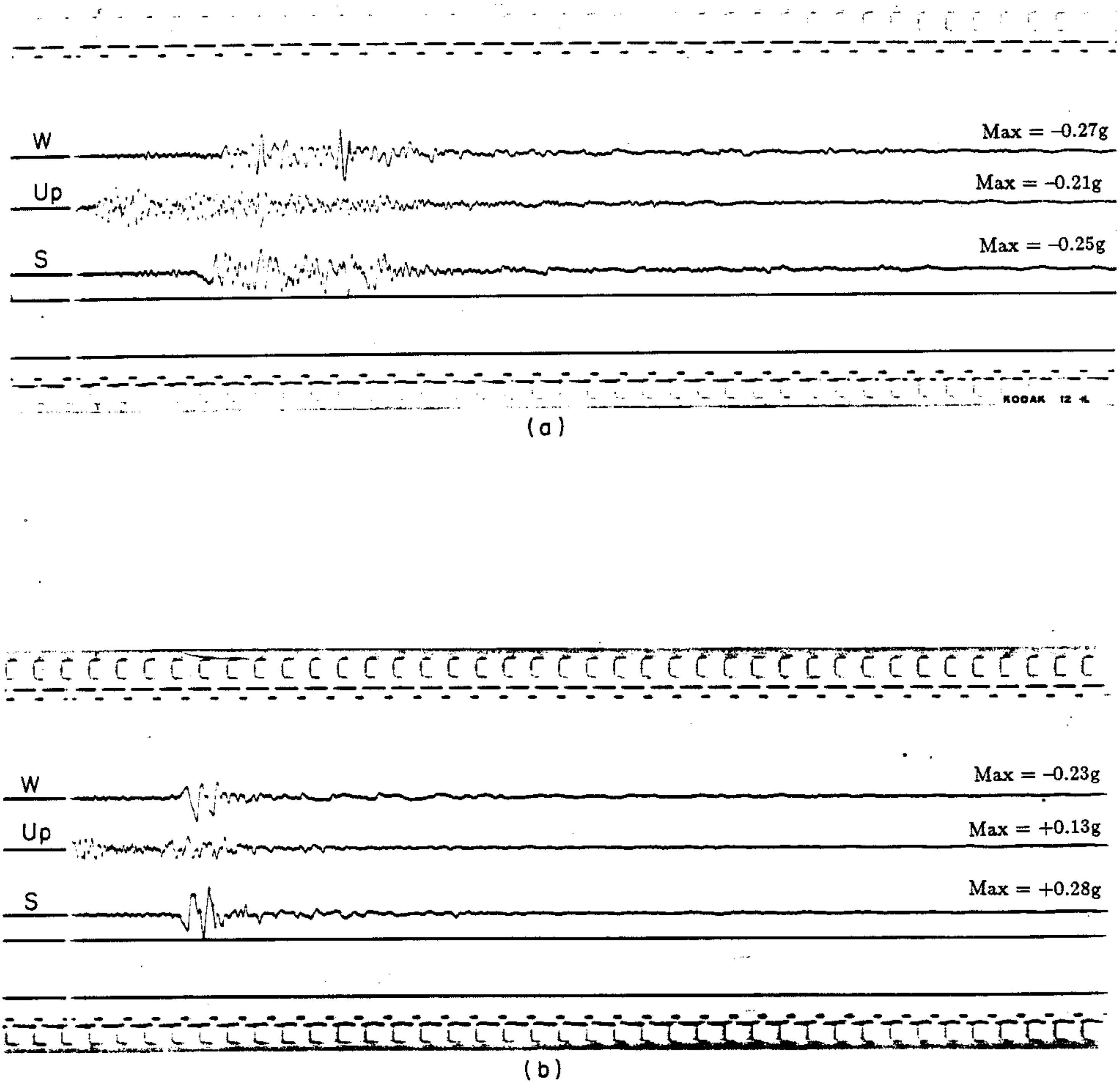
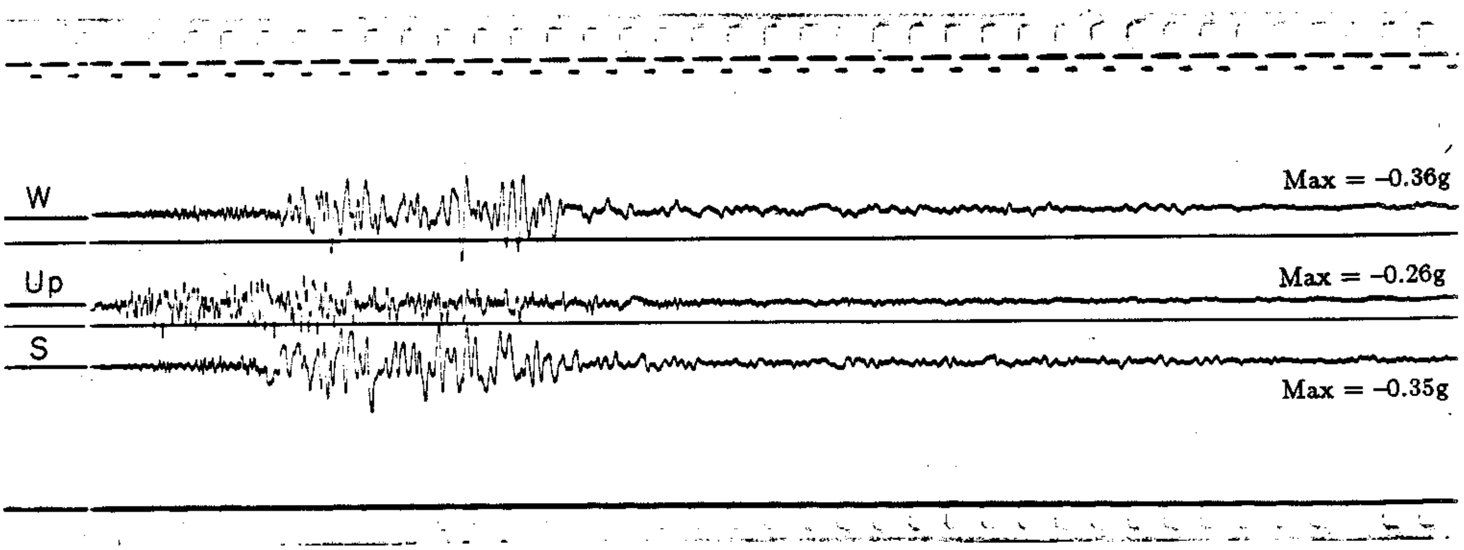
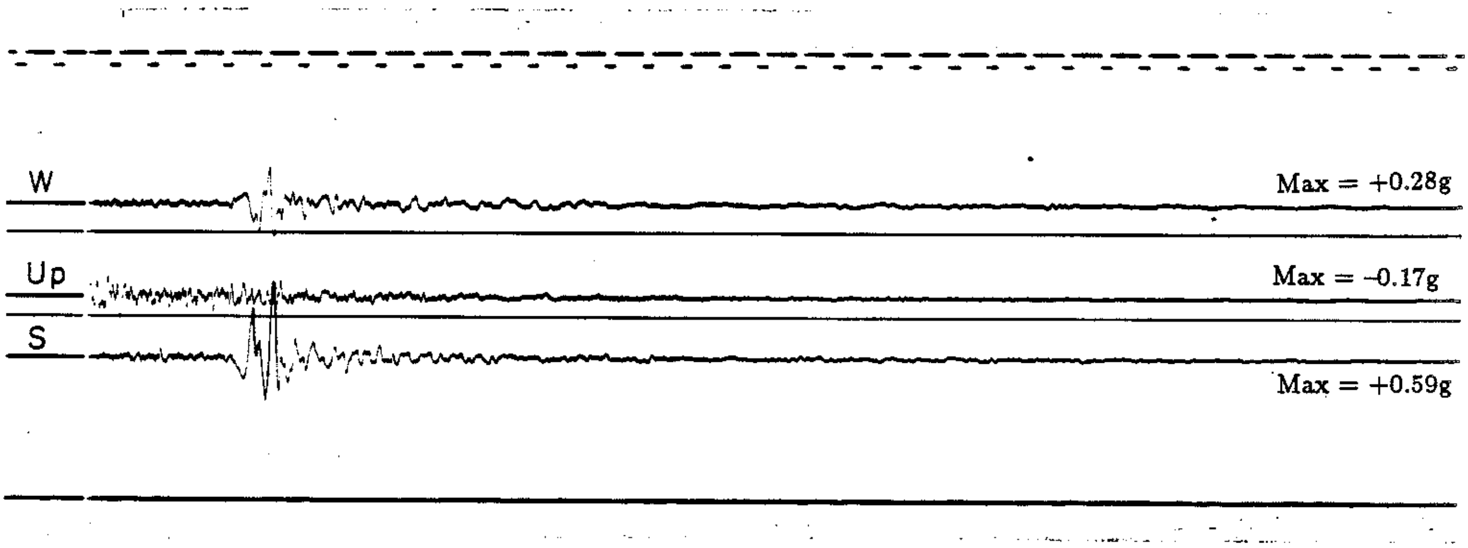


Figure A.9 Industrial Relations Center
(a) Whittier earthquake, October 1, 1987 ($M_L = 5.9$).
(b) Main aftershock, October 4, 1987 ($M_L = 5.3$).



(a)



(b)

Figure A.10 Lura Street house
(a) Whittier earthquake, October 1, 1987 ($M_L = 5.9$).
(b) Main aftershock, October 4, 1987 ($M_L = 5.3$).

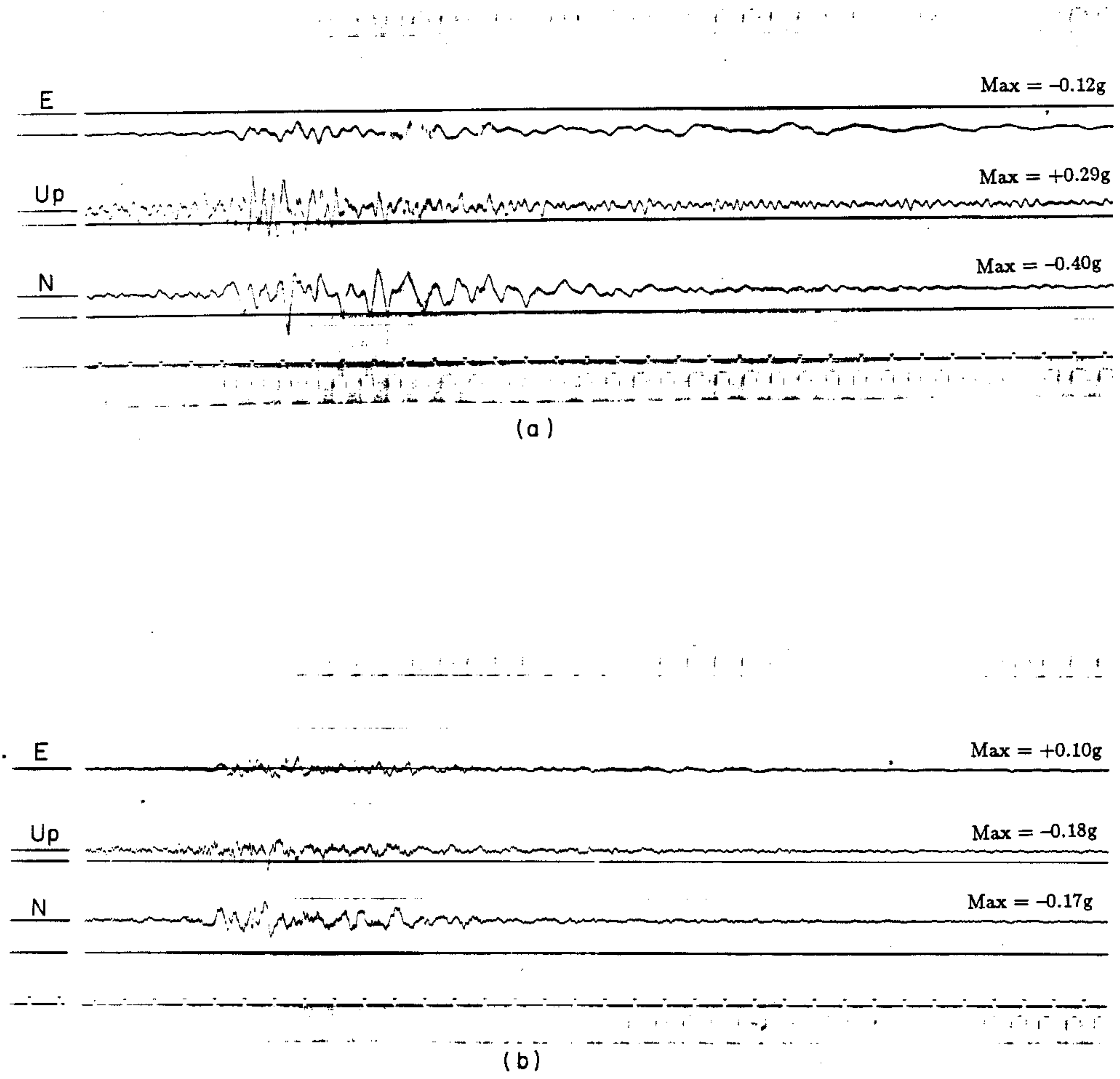


Figure A.11 JPL Building 180, Whittier earthquake, October 1, 1987 ($M_L = 5.9$)
(a) Roof
(b) Basement

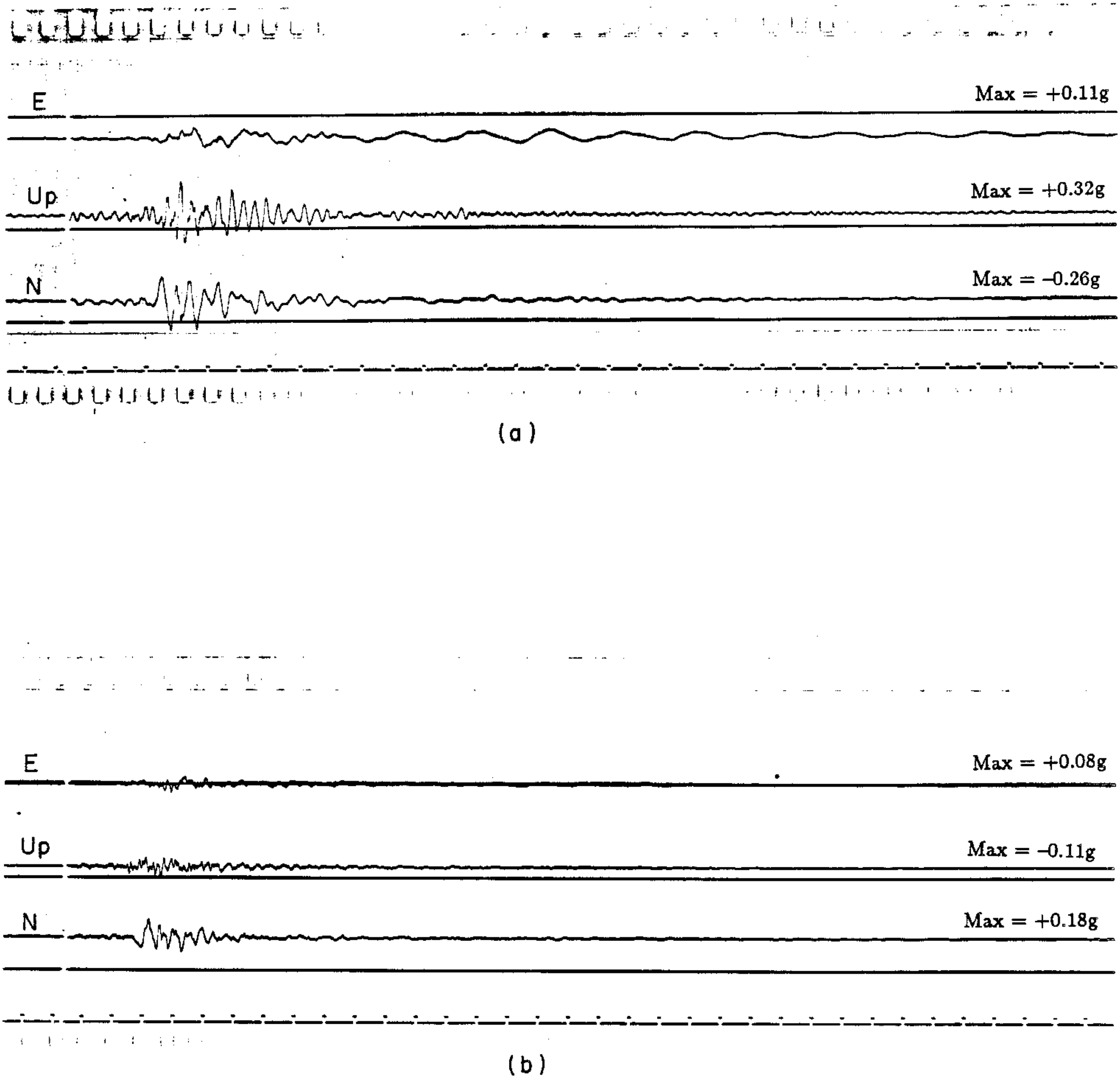


Figure A.12 JPL Building 180, Main Aftershock, October 4, 1987 ($M_L = 5.3$)
(a) Roof
(b) Basement

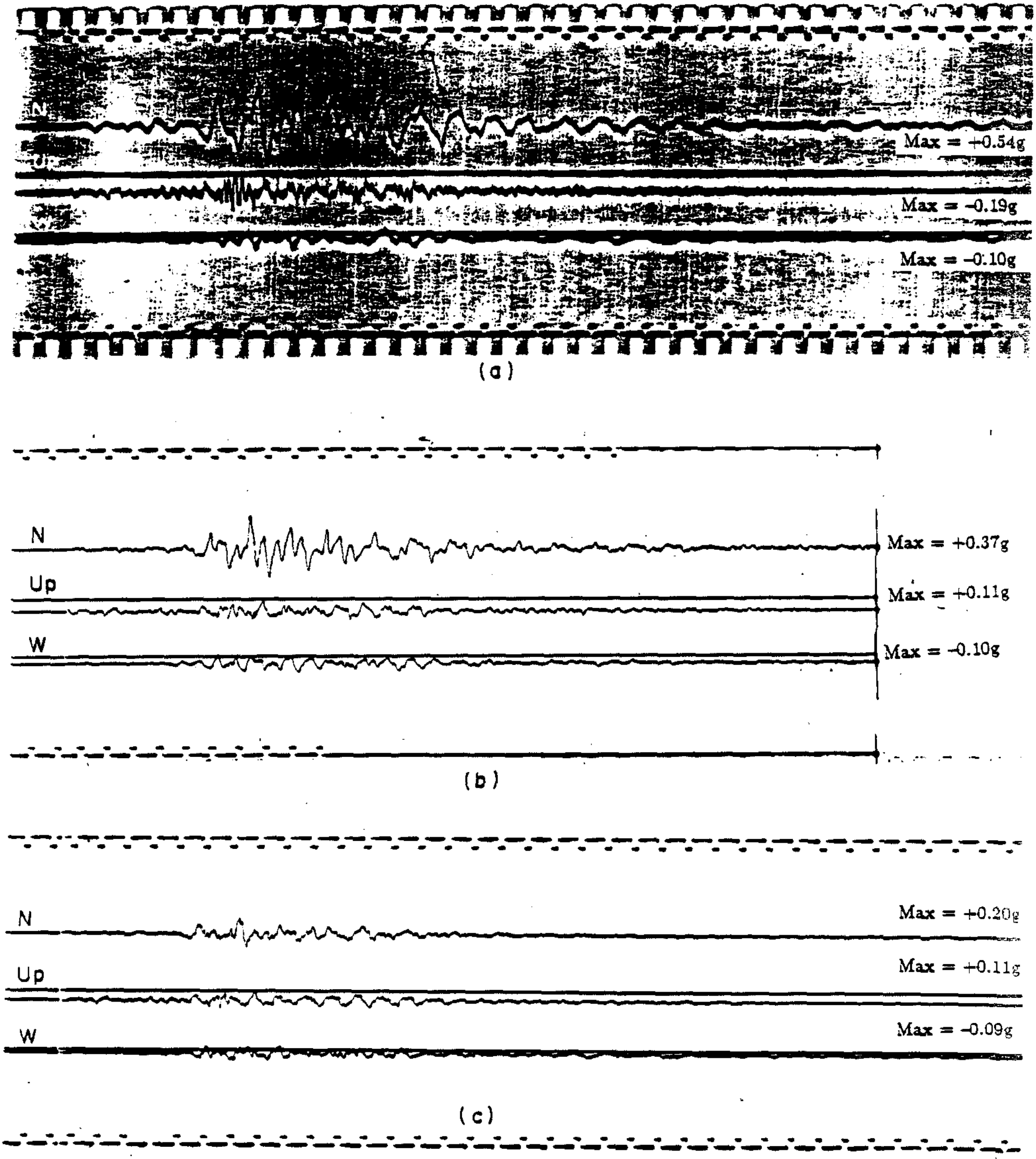


Figure A.13 JPL Building 183, Whittier earthquake, October 1, 1987 ($M_L = 5.9$)
(a) Roof, eighth floor
(b) Fifth floor
(c) First floor

APPENDIX B.
INSTRUMENT LOCATIONS AND SENSITIVITIES

Listed in Table B-1 are the locations and instrument sensitivities for the fifteen stations of the Caltech strong-motion array. The directions are for the configuration illustrated below in Fig. B-1.

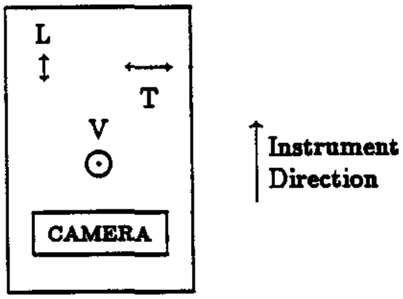


Fig. B-1. Accelerometer directions

		Instrument Location	Instrument Identification Number	Inst. Dir. (L)	Sensitivities (cm/g)			Nat. Freq. (Hz) Damping (%)		
					L	V	T	L	V	T
ON CAMPUS	CENTER	Millikan Library:	CR1 Sn 146	E						
		roof—west			1.83	1.86	1.79	53.4 .62	53.4 .65	54.1 .65
		roof—east			1.82		1.86	51.9 .62		54.4 .66
		6th floor			1.85		1.75	54.5 .65		53.4 .65
		basement			1.77	1.79	1.76	53.8 .64	53.0 .61	54.9 .68
	INNER RING	Mudd Laboratory	Sn 821	E	1.75	1.77	1.80	26.1 .60	25.4 .60	25.8 .60
		Bridge Laboratory	Sn 1255	E	1.83	1.93	1.90	25.2 .60	24.7 .60	25.6 .60
		Keck Laboratory	Sn 126	N	1.71	1.90	1.78	26.8 .60	26.0 .60	25.9 .60
		Thomas Laboratory	Sn 822	N	1.73	1.91	1.85	* .60	* .60	* .60
	OUTER RING	Brown Athletic Ctr.	Sn 1257	W	1.86	1.91	1.92	25.9 .60	24.8 .60	24.8 .60
		1308 California Blvd.	Sn 914	N	1.74	1.78	1.71	26.2 .60	26.4 .60	27.2 .60
		Athenaeum Fac. Club	Sn 124	N	1.90	1.95	1.90	25.8 .60	26.2 .60	26.5 .60
		Industrial Rel. Ctr.	Sn 911	S	1.79	1.90	1.85	* *	* *	* *
		330 S. Wilson (Lura St.)	Sn 1468	S	1.75	1.72	1.80	26.6 .60	27.0 .60	25.8 .60
OFF CAMPUS		Kresge Laboratory	Sn 3000	E	1.75	1.74	1.87	26.1 .60	26.4 .60	25.8 .60
		JPL Building 180:								
		roof	Sn 199	N	1.90	1.90	1.90	* .60	* .60	* .60
		basement	Sn 195	N	1.90	1.90	1.90	* .60	* .60	* .60
		JPL Building 183:								
		roof	Sn 3571	W	1.67	1.73	1.95	* *	* *	* *
		5th floor	Sn 3570	W	1.75	1.84	1.80	* *	* *	* *
		1st floor	Sn 3569	W	1.75	1.75	1.70	* *	* *	* *
		JPL Building 238	CR1 Sn 202	S						
		7th floor			1.81		1.75	53.9 .63		54.0 .62
		3rd floor (center)			1.81	1.79	1.80	51.8 .61	54.7 .61	52.4 .61
		3rd floor (NW)			1.77	1.77	1.81	54.1 .60	54.7 .62	54.0 .64
		1st floor			1.77		1.80	54.4 .63		53.1 .62

Table B-1. Instrument locations, lateral orientations, and calibration data for the 14 stations of the Caltech strong-motion accelerometer array. The instruments' sensitivity, natural frequency and damping are listed for the three recording components: lateral (L), vertical (V), and transverse (T).
(Campus coordinates: 34.14N 118.12W. Kresge Lab coordinates: 34.9N 118.7W)

* unavailable

CALIFORNIA INSTITUTE OF TECHNOLOGY

Reports Published
by
Earthquake Engineering Research Laboratory*
Dynamic Laboratory
Disaster Research Center

Note: Numbers in parenthesis are Accession Numbers assigned by the National Technical Information Service; these reports may be ordered from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia, 22161. Accession Numbers should be quoted on orders for reports (PB — —). Reports without this information either have not been submitted to NTIS or the information was not available at the time of printing. An N/A in parenthesis indicates that the report is no longer available at Caltech.

1. Alford, J.L., G.W. Housner and R.R. Martel, "Spectrum Analysis of Strong-Motion Earthquake," 1951. (Revised August 1964). (N/A)
2. Housner, G.W., "Intensity of Ground Motion During Strong Earthquakes," 1952. (N/A)
3. Hudson, D.E., J.L. Alford and G.W. Housner, "Response of a Structure to an Explosive Generated Ground Shock," 1952. (N/A)
4. Housner, G.W., "Analysis of the Taft Accelerogram of the Earthquake of 21 July 1952." (N/A)
5. Housner, G.W., "A Dislocation Theory of Earthquakes," 1953. (N/A)
6. Caughey, T.K., and D.E. Hudson, "An Electric Analog Type Response Spectrum," 1954. (N/A)
7. Hudson, D.E. and G.W. Housner, "Vibration Tests of a Steel-Frame Building," 1954. (N/A)
8. Housner, G.W., "Earthquake Pressures on Fluid Containers," 1954. (N/A)
9. Hudson, D.E., "The Wilmot Survey Type Strong-Motion Earthquake Recorder," 1958. (N/A)

---

* To order directly by phone, the number is (703) 487-4650.

10. Hudson, D.E., and W.D. Iwan, "The Wilmot Survey Type Strong-Motion Earthquake Recorder, Part II," 1960. (N/A)

11. Caughey, T.K., D.E. Hudson and R.V. Powell, "The CIT Mark II Electric Analog Type Response Spectrum Analyzer for Earthquake Excitation Studies," 1960. (N/A)

12. Keightley, W.O., G.W. Housner and D.E. Hudson, "Vibration Tests of the Encino Dam Intake Tower," 1961. (N/A)

13. Merchant, Howard Carl, "Mode Superposition Methods Applied to Linear Mechanical Systems Under Earthquake Type Excitation," 1961. (N/A)

14. Iwan, Wilfred D., "The Dynamic Response of Bilinear Hysteretic Systems," 1961. (N/A)

15. Hudson, D.E., "A New Vibration Exciter for Dynamic Test of Full-Scale Structures," 1961. (N/A)

16. Hudson, D.E., "Synchronized Vibration Generators for Dynamic Tests of Full-Scale Structures," 1962. (N/A)

17. Jennings, P.C., "Velocity Spectra of the Mexican Earthquakes of 11 May and 19 May 1962," 1962. (N/A)

18. Jennings, P.C., "Response of Simple Yielding Structures to Earthquake Excitation," 1963. (N/A)

19. Keightley, W.O., "Vibration Tests of Structures," 1963. (N/A)

20. Caughey, T.K. and M.E.J. O'Kelly, "General Theory of Vibration of Damped Linear Dynamic Systems," 1963. (N/A)

21. O'Kelly, M.E.J., "Vibration of Viscously Damped Linear Dynamic Systems," 1964. (N/A)

22. Nielsen, N.N., "Dynamic Response of Miltistory Buildings," 1964. (N/A)

23. Tso, W.K., "Dynamics of Thin-Walled Beams of Open Section," 1964. (N/A)

24. Keightley, W.O., "A Dynamic Investigation of Bouquet Canyon Dam," 1964. (N/A)

25. Malhotra, R.K., "Free and Forced Oscillations of a Class of Self-Excited Oscillators," 1964.

26. Hanson, R.D., "Post-Elastic Response of Mild Steel Structures," 1965.

27. Masri, S.F., "Analytical and Experimental Studies of Impact Dampers," 1965.

28. Hanson, R.D., "Static and Dynamic Tests of a Full-Scale Steel-Frame Structures," 1965.

29. Cronin, D.L., "Response of Linear, Viscous Damped Systems to Excitations Having Time-Varying Frequency," 1965.

30. Hu, P.Y.-F., "Analytical and Experimental Studies of Random Vibration," 1965.

31. Crede, C.E., "Research on Failure of Equipment when Subject to Vibration," 1965.

32. Lutes, L.D., "Numerical Response Characteristics of a Uniform Beam Carrying One Discrete Load," 1965. (N/A)

33. Rocke, R.D., "Transmission Matrices and Lumped Parameter Models for Continuous Systems," 1966. (N/A)

34. Brady, A.G., "Studies of Response to Earthquake Ground Motion," 1966. (N/A)

35. Atkinson, J.D., "Spectral Density of First Order Piecewise Linear Systems Excited by White Noise," 1967. (N/A)

36. Dickerson, J.R., "Stability of Parametrically Excited Differential Equations," 1967. (N/A)

37. Giberson, M.F., "The Response of Nonlinear Multi-Story Structures Subjected to Earthquake Excitation," 1967. (N/A)

38. Hallanger, L.W., "The Dynamic Stability of an Unbalanced Mass Exciter," 1967.

39. Husid, R., "Gravity Effects on the Earthquake Response of Yielding Structures," 1967. (N/A)

40. Kuroiwa, J.H., "Vibration Test of a Multistory Building," 1967. (N/A)

41. Lutes, L.D., "Stationary Random Response of Bilinear Hysteretic Systems," 1967.

42. Nigam, N.C., "Inelastic Interactions in the Dynamic Response of Structures," 1967.

43. Nigam, N.C. and P.C. Jennings, "Digital Calculation of Response Spectra from Strong-Motion Earthquake Records," 1968.

44. Spencer, R.A., "The Nonlinear Response of Some Multistory Reinforced and Prestressed Concrete Structures Subjected to Earthquake Excitation," 1968. (N/A)

45. Jennings, P.C., G.W. Housner and N.C. Tsai, "Simulated Earthquake Motions," 1968.

46. "Strong-Motion Instrumental Data on the Borrego Mountain Earthquake of 9 April 1968," (USGS and EERL Joint Report), 1968.

47. Peters, R.B., "Strong Motion Accelerograph Evaluation," 1969.

48. Heitner, K.L., "A Mathematical Model for Calculation of the Run-Up of Tsunamis," 1969.

49. Trifunac, M.D., "Investigation of Strong Earthquake Ground Motion," 1969. (N/A)

50. Tsai, N.C., "Influence of Local Geology on Earthquake Ground Motion," 1969. (N/A)

51. Trifunac, M.D., "Wind and Microtremor Induced Vibrations of a Twenty-Two Steel Frame Building," EERL 70-01, 1970.

52. Yang, I-M., "Stationary Random Response of Multidegree-of-Freedom Systems," DYNL-100, June 1970. (N/A)

53. Patula, E.J., "Equivalent Differential Equations for Non-linear Dynamic Systems," DYNL-101, June 1970.

54. Prelewicz, D.A., "Range of Validity of the Method of Averaging," DYNL-102, 1970.

55. Trifunac, M.D., "On the Statistics and Possible Triggering Mechanism of Earthquakes in Southern California," EERL 70-03, July 1970.

56. Heitner, K.L., "Additional Investigations on a Mathematical Model for Calculation of Run-Up of Tsunamis," July 1970.

57. Trifunac, M.D., "Ambient Vibration Tests of a Thirty-Nine Story Steel Frame Building," EERL 70-02, July 1970.

58. Trifunac, M.D. and D.E. Hudson, "Laboratory Evaluations and Instrument Corrections of Strong-Motion Accelerographs," EERL 70-04, August 1970. (N/A)

59. Trifunac, M.D., "Response Envelope Spectrum and Interpretation of Strong Earthquake Groun Motion," EERL 70-06, August 1970.

60. Keightley, W.O., "A Strong-Motion Accelerograph Array with Telephone Line Interconnections," EERL 70-05, September 1970.

61. Trifunac, M.D., "Low Frequency Digitization Errors and a New Method for Zero Baseline Correction of Strong-Motion Accelerograms," EERL 70-07, September 1970.

62. Vijayaraghavan, A., "Free and Forced Oscillations in a Class of Piecewise-Linear Dynamic Systems," DYNL-103, January 1971.

63. Jennings, P.C., R.B. Mathiesen and J.B. Hoerner, "Forced Vibrations of a 22-Story Steel Frame Building," EERL 71-01, February 1971. (N/A) (PB 205 161)

64. Jennings, P.C., "Engineering Features of the San Fernando Earthquake of February 9, 1971," EERL 71-02, June 1971. (PB 202 550)

65. Bielak, J., "Earthquake Response of Building-Foundation Systems," EERL 71-04, June 1971. (N/A) (PB 205 305)

66. Adu, R.A., "Response and Failure of Structures Under Stationary Random Excitation," EERL 71-03, June 1971. (N/A) (PB 205 304)

67. Skattum, K.S., "Dynamic Analysis of Coupled Shear Walls and Sandwich Beams," EERL 71-06, June 1971. (N/A) (PB 205 267)

68. Hoerner, J.B., "Model Coupling and Earthquake Response of Tall Buildings," EERL 71-07, June 1971. (N/A) (PB 207 635)

69. Stahl, K.J., "Dynamic Response of Circular Plates Subjected to Moving Massive Loads," DYNL-104, June 1971. (N/A)

70. Trifunac, M.D., F.E. Udwadia and A.G. Brady, "High Frequency Errors and Instrument Corrections of Strong-Motion Accelerograms," EERL 71-05, 1971. (PB 205 369)

71. Furuike, D.M., "Dynamic Response of Hysteretic Systems With Application to a System Containing Limited Slip," DYNL-105, September 1971. (N/A)

72. Hudson, D.E. (Editor), "Strong-Motion Instrumental Data on the San Fernando Earthquake of February 9, 1971," (Seismological Field Survey, NOAA, C.I.T. Joint Report), September 1971. (PB 204 198)

73. Jennings, P.C. and J. Bielak, "Dynamics of Building-Soil Interaction," EERL 72-01, April 1972. (PB 209 666)

74. Kim, B.-K., "Piesewise Linear Dynamic Systems with Time Delays," DYNL-106, April 1972.

75. Viano, D.C., "Wave Propagation in a Symmetrically Layered Elastic Plate," DYNL-107, May 1972.

76. Whitney, A.W., "On Insurance Settlements Incident to the 1906 San Francisco Fire," DRC 72-01, August 1972. (PB 213 256)

77. Udwadia, F.E., "Investigation of Earthquake and Microtremor Ground Motions," EERL 72-02, September 1972. (PB 212 853)

78. Wood, J.H., "Analysis of the Earthquake Response of a Nine-Story Steel Frame Building During the San Fernando Earthquake," EERL 72-04, October 1972. (PB 215 823)

79. Jennings, P.C., "Rapid Calculation of Selected Fourier Spectrum Ordinates," EERL 72-05, November 1972.

80. "Research Papers Submitted to Fifth World Conference on Earthquake Engineering, Rome, Italy, 25–29 June 1973," EERL 73-02, March 1973. (PB 220 431)

81. Udwadia, F.E., and M.D. Trifunac, "The Fourier Transform, Response Spectra and Their Relationship Through the Statistics of Oscillator Response," EERL 73-01, April 1973. (PB 220 458)

82. Housner, G.W., "Earthquake-Resistant Design of High-Rise Buildings," DRC 73-01, July 1973. (N/A)

83. "Earthquake and Insurance," Earthquake Research Affiliates Conference, 2-3 April, 1973, DRC 73-02, July 1973. (PB 223 033)

84. Wood, J.H., "Earthquake-Induced Soil Pressures on Structures," EERL 73-05, August 1973. (N/A)

85. Crouse, C.B., "Engineering Studies of the San Fernando Earthquake," EERL 73-04, March 1973. (N/A)

86. Irvine, H.M., "The Veracruz Earthquake of 28 August 1973," EERL 73-06, October 1973.

87. Iemura, H. and P.C. Jennings, "Hysteretic Response of a Nine-Story Reinforced Concrete Building During the San Fernando Earthquake," EERL 73-07, October 1973.

88. Trifunac, M.D. and V. Lee, "Routine Computer Processing of Strong-Motion Accelerograms," EERL 73-03, October 1973. (N/A) (PB 226 047/AS)

89. Moeller, T.L., "The Dynamics of a Spinning Elastic Disk with Massive Load," DYNL 73-01, October 1973.

90. Blevins, R.D., "Flow Induced Vibration of Bluff Structures," DYNL 74-01, February 1974.

91. Irvine, H.M., "Studies in the Statics and Dynamics of Simple Cable Systems," DYNL-108, January 1974.

92. Jephcott, D.K. and D.E. Hudson, "The Performance of Public School Plants During the San Fernando Earthquake," EERL 74-01, September 1974. (PB 240 000/AS)

93. Wong, H.L., "Dynamic Soil-Structure Interaction," EERL 75-01, May 1975. (N/A) (PB 247 233/AS)

94. Foutch, D.A., G.W. Housner and P.C. Jennings, "Dynamic Responses of Six Multistory Buildings During the San Fernando Earthquake," EERL 75-02, October 1975. (PB 248 144/AS)

95. Miller, R.K., "The Steady-State Response of Multidegree-of-Freedom Systems with a Spatially Localized Nonlinearity," EERL 75-03, October 1975. (PB 252 459/AS)

96. Abdel-Ghaffar, A.M., "Dynamic Analyses of Suspension Bridge Structures," EERL 76-01, May 1976. (PB 258 744/AS)

97. Foutch, D.A., "A Study of the Vibrational Characteristics of Two Multistory Buildings," EERL 76-03, September 1976. (PB 260 874/AS)

98. "Strong Motion Earthquake Accelerograms Index Volume," Earthquake Engineering Research Laboratory, EERL 76-02, August 1976. (PB 260 929/AS)

99. Spanos, P-T.D., "Linearization Techniques for Non-Linear Dynamical Systems," EERL 76-04, September 1976. (PB 266 083/AS)

100. Edwards, D.B., "Time Domain Analysis of Switching Regulators," DYNL 77-01, March 1977.

101. Abdel-Ghaffar, A.M., "Studies of the Effect of Differential Motions of Two Foundations upon the Response of the Superstructure of a Bridge," EERL 77-02, January 1977. (PB 271 095/AS)

102. Gates, N.C., "The Earthquake Response of Deteriorating Systems," EERL 77-03, March 1977. (PB 271 090/AS)

103. Daly, W., W. Judd and R. Meade, "Evaluation of Seismicity at U.S. Reservoirs," USCOLD, Committee on Earthquakes, May 1. (PB 270 036/AS)

104. Abdel-Ghaffer, A.M. and G.W. Housner, "An Analysis of the Dynamic Characteristics of a Suspension Bridge by Ambient Vibration Measurements," EERL 77-01, January 1977. (PB 275 063/AS)

105. Housner, G.W. and P.C. Jennings, "Earthquake Design Criteria for Structures," EERL 77-06, November 1977 (PB 276 502/AS)

106. Morrison, P., R. Maley, G. Brady and R. Porcella, "Earthquake Recordings on or Near Dams," USCOLD, Committee on Earthquakes, November 1977. (PB 285 867/AS)

107. Abdel-Ghaffar, A.M., "Engineering Data and Analyses of the Whittier, California Earthquake of January 1, 1976," EERL 77-05, November 1977. (PB 283 750/AS)

108. Beck, J.L., "Determining Models of Structures from Earthquake Records," EERL 78-01, June 1978 (PB 288 806/AS)

109. Psycharis, I., "The Salonica (Thessaloniki) Earthquake of June 20, 1978," EERL 78-03, October 1978. (PB 290 120/AS)

110. Abdel-Ghaffar, A.M. and R.F. Scott, "An Investigation of the Dynamic Characteristics of an Earth Dam," EERL 78-02, August 1978. (PB 288 878/AS)

111. Mason, A.B., Jr., "Some Observations on the Random Response of Linear and Nonlinear Dynamical Systems," EERL 79-01, January 1979. (PB 290 808/AS)

112. Helmberger, D.V. and P.C. Jennings (Organizers), "Strong Ground Motion: N.S.F. Seminar-Workshop," SL-EERL 79-02, February 1978.

113. Lee, D.M., P.C. Jennings and G.W. Housner, "A Selection of Important Strong Motion Earthquake Records," EERL 80-01, January 1980. (PB 80 169196)

114. McVerry, G.H., "Frequency Domain Identification of Structural Models from Earthquake Records," EERL 79-02, October 1979. (PB-80-194301)

115. Abdel-Ghaffar A.M., R.F.Scott and M.J.Craig, "Full-Scall Experimental Investigation of a Modern Earth Dam," EERL 80-02, February 1980. (PB-81-123788)

116. Rutenberg, A., P.C. Jennings and G.W. Housner, "The Response of Veterans Hospital Building 41 in the San Fernando Earthquake," EERL 80-03, May 1980. (PB-82-201377)

117. Haroun, M.A., "Dynamic Analyses of Liquid Storage Tanks," EERL 80-04, February 1980. (PB-81-123275)

118. Liu, W.K., "Development of Finite Element Procedures for Fluid-Structure Interaction," EERL 80-06, August 1980. (PB 184078)

119. Yoder, P.J., "A Strain-Space Plasticity Theory and Numerical Implementation," EERL 80-07, August 1980. (PB-82-201682)

120. Krousgrill, C.M., Jr., "A Linearization Technique for the Dynamic Response of Nonlinear Continua," EERL 80-08, September 1980. (PB-82-201823)

121. Cohen, M., "Silent Boundary Methods for Transient Wave Analysis," EERL 80-09, September 1980. (PB-82-201831)

122. Hall, S.A., "Vortex-Induced Vibrations of Structures," EERL 81-01, January 1981. (PB-82-201849)

123. Psycharis, I.N., "Dynamic Behavior of Rocking Structures Allowed to Uplift," EERL 81-02, August 1981. (PB-82-212945)

124. Shih, C.-F., "Failure of Liquid Storage Tanks Due to Earthquake Excitation," EERL 81-04, May 1981. (PB-82-215013)

125. Lin, A.N., "Experimental Observations of the Effect of Foundation Embedment on Structural Response," EERL 82-01, May 1982. (PB-84-163252)

126. Botelho, D.L.R., "An Empirical Model for Vortex-Induced Vibrations," EERL 82-02, August 1982. (PB-84-161157)

127. Ortiz, L.A., "Dynamic Centrifuge Testing of Cantilever Retaining Walls," SML 82-02, August 1982. (PB-84-162312)

128. Iwan, W.D., Editor, "Proceedings of the U.S. National Workshop on Strong-Motion Earthquake Instrumentation, April 12-14, 1981, Santa Barbara, California," California Institute of Technology, Pasadena, California, 1981.

129. Rashed, A., "Dynamic Analysis of Fluid-Structure Systems," EERL 82-03, July 1982. (PB-84-162916)

130. National Academy Press, "Earthquake Engineering Research—1982."

131. National Academy Press, "Earthquake Engineering Research—1982, Overview and Recommendations."

132. Jain, S.K., "Analytical Models for the Dynamics of Buildings," EERL 83-02, May 1983. (PB-84-161009)

133. Huang, M.-J., "Investigation of Local Geology Effects on Strong Earthquake Ground Motions," EERL 83-03, July 1983. (PB-84-161488)

134. McVerry, G.H. and J.L. Beck, "Structural Identification of JPL Building 180 Using Optimally Synchronized Earthquake Records." EERL 83-01, August 1983. (PB-84-162833)

135. Bardet, J.P., "Application of Plasticity Theory to Soil Behavior: A New Sand Model," SML 83-01, September 1983. (PB-84-162304)

136. Wilson, J.C., "Analysis of the Observed Earthquake Response of a Multiple Span Bridge," EERL 84-01, May 1984. (PB-85-240505/AS)

137. Hushmand, B., "Experimental Studies of Dynamic Response of Foundations," SML 83-02, November 1983. (PB-86-115383/A)

138. Cifuentes, A.O., "System Identification of Hysteretic Structures," EERL 84-04, 1984. (PB-240489/AS14)

139. Smith, K.S., "Stochastic Analysis of the Seismic Response of Secondary Systems," EERL 85-01, November 1984. (PB-85-240497/AS)

140. Maragakis, E., "A Model for the Rigid Body Motions of Skew Bridges," EERL 85-02, December 1984. (PB-85-248433/AS)

141. Jeong, G.D., "Cumulative Damage of Structures Subjected to Response Spectrum Consistent Random Process," EERL 85-03, January 1985. (PB-86-100807)

142. Chelvakumar, K., "A Simple Strain-Space Plasticity Model for Clays," EERL 85-05, 1985. PB-

143. Pak, R.Y.S., "Dynamic Response of a Partially Embedded Bar Under Transverse Excitations," EERL 85-04, May 1985. (PB-87-232856/A06)

144. Tan, T.-S., "Two Phase Soil Study: A. Finite Strain Consolidation, B. Centrifuge Scaling Considerations," SML 85-01, August 1985. PB-

145. Iwan, W.D., M.A. Moser and C.-Y. Peng, "Strong-Motion Earthquake Measurement Using a Digital Accelerograph," EERL 84-02, April 1984.

146. Beck, R.T. and J.L. Beck, "Comparison Between Transfer Function and Modal Minimization Methods for System Identification," EERL 85-06, November 1985. (PB-87-234688/A04)

147. Jones, N.P., "Flow-Induced Vibration of Long Structures," DYNL 86-01, May 1986. (PB-88-106646/A08)

148. Peek, R., "Analysis of Unanchored Liquid Storage Tanks Under Seismic Loads," EERL 86-01, April 1986. (PB-87-232872/A12)

149. Paparizos, L.G., "Some Observations on the Random Response of Hysteretic Systems," EERL 86-02. 1986. PB-

150. Moser, M.A., "The Response of Stick-Slip Systems to Random Seismic Excitation," EERL 86-03, September 1986. PB-

151. Burrige, P.B., "Failure of Slopes," SML 87-01, March 1987. PB-

152. Jayakumar, P., "Modeling and Identification in Structural Dynamics," EERL 87-01, May 1987. PB-

153. Dowling, M.J., "Nonlinear Seismic Analysis of Arc Dams," EERL 87-03, September 1987. PB-

154. Duron, Z.H., "Experimental and Finite Element Studies of a Large Arch Dam," EERL 87-02, September 1987. PB-

155. Whirley, R.G., "Random Response of Nonlinear Continuous Systems," EERL 87-04, September 1987. PB-

156. Peng, C.-Y., "General Model Identification of Linear and Nonlinear Dynamic Systems," EERL 87-05, September 1987. PB-