

**QUANTIFYING THE EFFECT OF SOIL-STRUCTURE  
INTERACTION FOR USE IN BUILDING DESIGN**

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Data Utilization Report CSMIP/00-02 (OSMS 00-04)

California Strong Motion Instrumentation Program

February 2000

**CALIFORNIA DEPARTMENT OF CONSERVATION  
DIVISION OF MINES AND GEOLOGY  
OFFICE OF STRONG MOTION STUDIES**

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### **DISCLAIMER**

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Division of Mines and Geology  
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## PREFACE

The California Strong Motion Instrumentation Program (CSMIP) in the Division of Mines and Geology of the California Department of Conservation promotes and facilitates the improvement of seismic codes through the Data Interpretation Project. The objective of this project is to increase the understanding of earthquake strong ground shaking and its effects on structures through interpretation and analysis studies of CSMIP and other applicable strong motion data. The ultimate goal is to accelerate the process by which lessons learned from earthquake data are incorporated into seismic code provisions and seismic design practices.

The specific objectives of the CSMIP Data Interpretation Project are to:

1. Understand the spatial variation and magnitude dependence of earthquake strong ground motion.
2. Understand the effects of earthquake motions on the response of geologic formations, buildings and lifeline structures.
3. Expedite the incorporation of knowledge of earthquake shaking into revision of seismic codes and practices.
4. Increase awareness within the seismological and earthquake engineering community about the effective usage of strong motion data.
5. Improve instrumentation methods and data processing techniques to maximize the usefulness of SMIP data. Develop data representations to increase the usefulness and the applicability to design engineers.

This report is part of CSMIP data utilization reports designed to transfer recent research findings on strong-motion data to practicing seismic design professionals and earth scientists. CSMIP extends its appreciation to the members of the Strong Motion Instrumentation Advisory Committee and its subcommittees for their recommendations regarding the Data Interpretation Research Project.

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## ABSTRACT

This research investigates the effects of soil-structure interaction (SSI) for regular buildings, validate current analysis techniques, and investigate the degree to which SSI contributes to the code based R factor for a variety of building and soil conditions. The research includes the analysis of strong-motion records for 11 CSMIP building/free field pairs to investigate the reduction in building response due to soil-structure interaction. The research also includes SSI analyses using the FLUSH computer program for four CSMIP buildings sites, comparison of recorded with model response, and comparison of the predicted base shear reduction using FLUSH and ATC 3-06 to the actual reduction recorded.



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# Chapter 1

## INTRODUCTION

The successful performance of buildings subjected to strong earthquake ground motions depends on their strength, the selected structural system and configuration, as well as the detailing and interconnection of the structural elements. Strong-motion recordings taken during large earthquakes continue to show that properly designed buildings are capable of sustaining large recorded ground accelerations with little or no damage even though these motions far exceed their calculated strength. Recent experience in the Loma Prieta earthquake demonstrated that structures subjected to 30 to 40%g peak ground acceleration did not experience the kind of damage that would have been predicted using purely elastic analysis techniques.

Current seismic design provisions used in the United States include large reduction coefficients called R factors to account for this phenomena. These factors range in value from 1.5 to 12 and are used to define a suitable design base shear from an elastic response spectrum. From a structural design point of view, the key components making up the R factor include over-strength, redundancy, damping, multi-mode effects, system ductility, radiation damping, and soil-structure interaction (SSI). It is commonly recognized that extensive research is needed to justify and refine the arbitrarily established R values and tailor their use for new design.

It is often assumed that the motion experienced at the base of a building is the same as the free field ground motion. This is only true if the structure is supported on soil which is rigid. For most soil conditions, the motions at the base of the building are significantly different than in the free field, and may even include a rocking component in addition to horizontal translational and vertical components (ATC, 1978). In addition, the motion at the building's base may be larger or smaller than the motion of the free-field. This phenomena has been commonly termed soil-structure interaction.

Currently, building designers often rely on geotechnical engineers to develop response spectra for the design of a new building. The information used to develop these spectra is derived from a number of sources including boring logs, site trenches, and strong-motion records from previous earthquakes that have occurred on nearby faults or similar sites. All available information is averaged and smoothed to develop the final "site-specific" design spectra. This spectra serves to represent the ground-motion characteristics of the free field at one specific site. It does not always represent the motion at the base of the building however, as the effect between the soil and the future building have not been investigated.

When using this spectra for the design of a new building, the engineer scales it according to the building code to account for inelastic effects using the previously mentioned R factors. If the engineer would like to specifically account the SSI effects between the soil and building, the options are currently limited to using the hand procedure in ATC 3-06 or doing a more complex soil-structure interaction analysis. These more complex SSI analyses, however, have generally concentrated on rigid structures such as nuclear power plants rather than more flexible building structures. For all but the most important buildings, the engineer will probably ignore the effects of SSI or simply rely on the code based R factor to take into account the SSI effect.

With each strong earthquake however, we have the opportunity to improve our current analysis techniques and design assumptions, including the design of structures to take into account the effects of SSI. Soil-structure interaction is just one of the many areas of research that benefits tremendously from instrumented buildings which record strong ground motions. These records provide researchers a large amount of information about a building's behavior during an earthquake and provide us clues to learn how to refine and improve our current design techniques to resist seismic forces.

### **1.1 Objectives of the Study**

This research seeks to investigate the beneficial effects of soil-structure interaction for regular buildings, validate current analysis techniques used to quantify SSI effects, and investigate the degree to which SSI contributes to the code based R factor for a variety of buildings an soil conditions.

The idea for this project came out of a soil-structure interaction analysis performed by the authors as part of the design of a new hospital for the Department of Veterans Affairs, Palo Alto Medical Center. Using commercially available SSI analysis procedures, a significant reduction in the design response spectrum in the range near the fundamental period of the building was justified and used in the dynamic analysis of the building. As a check on the results, a hand analysis was performed using the NEHRP SSI procedure first outlined in ATC-03. It appears that because of the conservative assumptions built in to the NEHRP procedure, that procedure greatly underestimated the base shear reduction attributable to SSI by a factor of almost three.

These results prompted the conclusion that additional analyses may identify trends which would help to better quantify the level of SSI reductions that could be obtained for various types of buildings and soil conditions. In addition, the opportunity to verify and correlate reductions calculated with analytical and hand-based SSI procedures with those calculated from actual strong-motion records became obvious.

### **1.2 Overview of Soil-Structure Interaction Effects**

Many researchers have noted that earthquake ground motions measured at the base of massive containment buildings in nuclear power plants were usually smaller than those

recorded in the free field at the same facility. This phenomenon, which results from the interaction of seismic ground motions between the rigid, embedded building and the surrounding foundation material is commonly called soil-structure interaction (SSI). Early research in the SSI field was mostly conducted by the nuclear industry. The study of SSI effects on regular buildings began around the early to mid-70's, most notably by A.S. Valestos. Dr. H. Bolton Seed made probably the most comprehensive summary of SSI effects on buildings in his Nabor Cabrillo Lecture presented at the annual convention of Mexican Society of Civil Engineers in 1986 (Seed, 1986).

Seed summarized the general effect of soil-structure interaction as follows. As earthquake-generated waves arrive at the ground surface, they generate motions in building structures. The motions that the building experiences depend on the vibration characteristics of the building and the layout of the building in plan. In order for the building to respond to the earthquake motion, it must first overcome its own inertia and thus a complex effect of interaction between the soil and the structure is initiated. This soil-structure interaction can be generally categorized into 3 separate effects: kinematic interaction, inertial interaction, and foundation sliding.

#### *1.2.1 Kinematic Interaction*

Kinematic interaction is characterized as motion of the structure due to rigid-body displacement of the ground surface (Wolf, 1985). Kinematic interaction theory assumes that the building structure has no mass, the building foundation has no mass, and the building foundation is completely rigid. During an earthquake, the seismic waves travel up through the soil and reflect and scatter off the rigid foundation. As the waves reach the surface, their motion will be amplified. This is similar to whipping a rope with a free end where it is observed that the largest motion occurs at the free end. The motion of the rigid foundation will be essentially the same as that of the ground around it, because the massless structure does not change the response at the base. The foundation moves as a rigid body and the scattering of waves modifies the motion compared to the motion that would occur if the foundation was not present (Fenves & Serino, 1992).

The amount of kinematic interaction in a soil-structure system depends largely on the type, frequency content, and angle of incidence of the seismic waves, and the embedment and flexibility of the foundation (Fenves & Serino, 1992).

#### *1.2.2 Inertial Interaction*

Inertial interaction is characterized by the motion and deformation of the foundation and structure apart from the motion of the surrounding soil. Assume that the building's foundation is completely flexible and that the building has been completely separated from the ground. During an earthquake, the ground motion is transmitted from the soil into the building. The building mass develops an inertia force to resist this change in motion since a body at rest has a tendency to remain at rest. The inertial resistance will be followed by the modal vibration of the structure throughout the period of ground

shaking and then after the shaking stops, the building will vibrate freely until the motions are damped out and the building returns to rest. Oftentimes, the surrounding ground will deform as a result of the structure trying to lift off its foundation.

The amount of inertial interaction depends on the mass of the building, the mass and type of foundation, and the properties of the soil.

### 1.2.3 *Foundation Sliding*

Foundation sliding is characterized as the relative lateral motion between the structure's foundation and the surrounding soil. Sometimes during an earthquake, the structure will break free of the surrounding soil and move laterally. The tremendous friction generated by this behavior dissipates the earthquake's energy until the structure returns to rest. Foundation sliding essentially short-circuits the normal effects of both kinematic and inertial interaction. Because the foundation breaks free, the surrounding soil cannot displace the foundation as a rigid body so kinematic interaction is reduced. Also, since large inertia forces are never transmitted into the building but are instead dissipated by friction through sliding, inertial interaction is also reduced.

Foundation sliding depends primarily on the type of foundation system and the properties of the surrounding soil. Generally, only mat foundations and spread footings with tie beam foundations exhibit foundation sliding effects.

The study described here will focus primarily on the first two effects of SSI described above, kinematic interaction and inertial interaction. Foundation sliding is assumed to be negligible for the majority of the buildings investigated in this report.

## 1.3 **Organization of Report**

The project work plan includes a number of different tasks culminating with the preparation of this report.

The initial tasks were to conduct a literature search and to collect strong-motion records. The literature search focused on studies of the effects of soil-structure interaction on conventional buildings. Chapter 2 summarizes the literature studied for this report. Numerous strong-motion records were collected from the California Strong Motion Instrumentation Program (CSMIP) for three principal earthquakes: 1987 Whittier, 1989 Loma Prieta, and 1990 Upland. The more recent events in Petrolia, Landers, Big Bear and Northridge were unable to be studied as no building records had yet been digitized for those events.

The first phase of the study focused on the reduction in peak ground acceleration with depth of structure embedment. Twenty-eight buildings with records from ten recent mainshocks and aftershocks are included in this study. Included in the tables is data

from the recent January 17, 1994 Northridge earthquake in Southern California. The description and conclusions of this phase of the study are presented in Chapter 3.

For the second phase of the study, 11 different free field/building instrument pairs were chosen and strong-motion records obtained from CSMIP. Included for each building site in Appendix A are: a building description, a picture and site plan of the building and free field instrument location (if available), orientation of the strong-motion accelegraphs, time histories of the free field, basement, and roof records (each direction), response spectrums of the free field and building basement (each direction), fourier spectrums including spectrums utilizing 5 second time windows (each direction). The analysis conducted on each pair of records sought to determine the reduction in response due to soil-structure interaction by measuring the difference in response between the free field and base response spectrums at the building's fundamental period. The description and conclusions of this phase of the study are included in Chapter 4.

For the third phase of the study, 4 of the 11 buildings were chosen for further study and structural drawings obtained from CSMIP. A soil-structure interaction analysis was performed for each of the four buildings using the FLUSH computer program. The results of the FLUSH analysis were compared with the results of time history analyses using a simple stick model of the building, the results of the previous two parts of the study as well as with the hand procedure developed in ATC-03, and the results of other researchers. The description and conclusions of this phase of the study are included in Chapter 5.

Chapter 6 contains overall conclusions encompassing all phases of the study and recommendations for further research. Chapter 7 contains a comprehensive list of references used for this report.



# Chapter 2

## SUMMARY OF PREVIOUS WORK

Although the concept of soil-structure interaction has been in the literature for a number of years, most of the research has been centered on important and monumental structures such as nuclear reactors. These type of reactor structures are most often configured as rigid and heavy concrete boxes and are often embedded deeply into the ground. As a result, the amount of kinematic interaction between the structure and the soil is quite large. The large earthquake forces used for the design of these structures prompted the nuclear industry and designers to investigate the beneficial effects of reductions due to soil-structure interaction and take advantage of them in the structure's design.

The behavior of conventional buildings is quite different than these reactor structures. Conventional buildings are usually much lighter and much more flexible. Even though inertial and kinematic soil-structure interaction are less in conventional buildings than in nuclear structures, SSI can have a significant effect on a buildings behavior. In the past few years, the number of researchers investigating the response of conventional buildings has been steadily increasing.

### 2.1 Seed - Nabor Cabrillo Lecture

Dr. H. Bolton Seed has published a number of papers on the effects of soil-structure interaction on conventional buildings. Probably the most comprehensive summary of SSI effects on buildings is Seed's Nabor Cabrillo Lecture presented at the annual convention of Mexican Society of Civil Engineers in 1986 (Seed, 1986).

In his lecture, Dr. Seed states that as the ground motions arrive at the ground surface, they generate motions in any overlying structures. The motions at the ground surface depend on the vibration characteristics and the layout of the super-structure. In order to excite the super-structure, the motions must overcome the inertial resistance of the structure and thus a complex effect of interaction between the soil and the structure is initiated. Dr. Seed categorizes the effects of SSI into three groups: base slab averaging, inertial interaction, and kinematic interaction.

Seed explains that base-slab averaging is due to the interaction of a stiff base mat in contact with the soil. If the structure has a stiff concrete base-slab, the slab will essentially be forced to move with one common motion regardless of the travel paths and

arrival time of the various incident waves reaching the underside of the slab. Since the peak amplitudes of motions at points on the bottom of the slab will not occur at the same instant, it follows that the "average" motion experienced by the rigid mat would be somewhat less than the maximum amplitudes that would have developed in the soil should there not be a rigid mat; that is, in the free field. The magnitude of this effect depends on geometry and characteristics of the base-slab as well as the uniformity (or spatial variation) of the free field motions. Dr. Seed believed that this effect is not likely to be very significant, except for low-period structures with large lateral dimensions. This observation has been reported by a number of investigators including Yamahara (1970), Ambraseys (1975), Scanlon (1976) and Newmark et al. (1970). For the purposes of our work, we have grouped what Seed has termed base-slab averaging with the effects of kinematic interaction. Because conventional buildings rarely have rigid foundation mats and are generally not configured like nuclear power plants, the effect of base-slab averaging is expected to be small.

The most complex and most widely studied aspect of soil-structure interaction is inertial interaction. According to Seed, as the ground motion is transmitted from the ground surface to the overlying structure, there will be an inertial resistance in response to the incoming motions. The inertial resistance will be followed by the modal vibration of the structure throughout the period of ground shaking and sometimes even after the incoming excitation has ceased. As a result, the contact pressures between the base structure and the ground will change during the strong ground shaking. Oftentimes there will also be associated deformations of the ground as well as the structure.

Seed postulated that "inertial interaction tends to cause a slight reduction in the intensity of motions developed at the base of the structure compared with the intensity of motions developed in the free field... for most structures the effect will be small, of the order of about +10 to -20%" (Seed, 1986). He based this observation not on building base shear reductions, but rather on his observation of the reduction in peak ground acceleration.

The effect of depth of embedment on structures is characterized by Seed as kinematic interaction. Seed observed that the peak acceleration due to earthquake ground motion decreases significantly as the depth of the soil deposit increases. This was verified by records from a number of strong-motion sites with downhole, vertical arrays of instruments such as the USGS instrument at Menlo Park, California, an array at Narimasu, Japan, and the EERC array at Richmond Field Station, California. As further evidence, a number of nearby pairs of buildings, each pair with one building constructed on the ground surface and one with a full basement, were compared by Seed and Lysmer (1980) for peak ground accelerations during the San Fernando, California earthquake of 1971. In 7 of the 8 cases studied, the peak acceleration recorded at the base of the building with a basement was on the average about 27% less than the building founded on the ground surface. Seed neglected to add the case to the table of data presented where an increase was observed. He concluded that this reduction in pga with depth was "not a chance phenomena, but a pattern attributable to deterministic effects" (Seed, 1986).

It is not clear why Seed chose to leave out the specific pair which showed an increase in response. It is clear from our research that increases do occur, and these increases are repeated at some sites for different earthquakes. In addition, we discuss in Chapter 3 the disadvantages of looking solely at pga values to base predictions on reductions due to soil-structure interaction.

Although kinematic interaction is very significant for deeply embedded structures, it can also be important for more shallow structures. Dr. Seed reported that changes in structural response on the order of 50% due to deep embedment (around 75 ft) are clearly possible and that even shallow depths of embedment (around 15 ft) may reduce the response of a structure on the order of 20% compared with the corresponding response for a structure resting on ground surface (Seed and Lysmer, 1980). Seed also suggested that the difference in pga between the ground and the basement can be used as a rough indication of the amount of reduction in motion due to SSI.

## 2.2 Valestos - ATC 3-06

The effect of inertial soil-structure interaction on building response is well documented in ATC 3-06 (ATC, 1978), published by the Applied Technology Council in 1978 and based on the work of Andrew Valestos and others. Procedures are given to calculate a reduction due to soil-structure interaction using either a static lateral force procedure or a modal analysis procedure. Using the static lateral force method outlined in Chapter 6 of ATC 3-06 is similar to the procedure for normal buildings without taking into account SSI effects. The procedure can be calculated by hand and is based primarily on the period of the building, shear wave velocity and shear modulus of the soil, and amount of foundation damping.

The static lateral force procedure is based on the behavior of a single-degree-of-freedom oscillator, with both foundation spring and foundation dashpot. Inherent in the procedure is the assumption that SSI affects only the fundamental mode of vibration of the building. References are made to papers by Valestos, Jennings, and Bielak that corroborate this assumption for normal buildings. Also, the effective height and effective weight of the structure used for the calculations are set at 70% of the actual height and weight of buildings greater than 1-story in height.

In general, the ATC procedure shows that the effects due to SSI are small, on the order of +0 to -15% for most buildings. In addition, the total amount of reduction due to SSI is limited to -30%. ATC 3-06 states in the *Commentary* to Chapter 6 that "it is expected, however, that this limit will control only infrequently, and that the calculated reduction will in most cases be less." (ATC, 1978)

The procedure is based primarily on analytical solutions and classical mechanics of single-degree-of-freedom systems. The authors are not aware of any attempts to compare the results of the hand analysis with actual reductions calculated from strong-motion

records. It is clear from this research that greater reductions can occur and may make a significant contribution to a structure's response to a particular earthquake motion.

### 2.3 Fenves and Serino - Hollywood Storage Building

Fenves and Serino studied soil-structure interaction effects for the Hollywood Storage Building in Los Angeles using the 1987 Whittier earthquake for their analysis (1992). They chose the Hollywood Storage Building because the building has a regular configuration and both strong-motion and soil data were available for the site.

A 3-D linear-elastic finite-element model was created for the structure using SUPER-ETABS. The building was idealized as "a system of independent frames and shear walls interconnected by floor diaphragms which are assumed rigid in their own plane." Element sizes were based on gross section properties using the 1925 building design drawings. A modulus of elasticity for concrete was chosen to match the vibration period of the model with the periods obtained from the response of the building to the Whittier earthquake. A final modulus of elasticity of  $E_c=2800 \text{ kip/in}^2$  was used and thought to reasonably account for limited cracking and creep over the life of the structure.

The soil model was developed using the substructure approach with the building as one substructure and the soil as a second substructure. The foundation was idealized as a "rigid, circular disk attached to the surface of the soil." The soil supporting the foundation was represented with springs and dashpots to represent a homogeneous viscoelastic halfspace. Nonlinear behavior of the soil was included in the analysis by "selecting elastic and damping properties consistent with the level of strain."

Fenves and Serino concluded that soil-structure interaction had little effect on the maximum base shear force in the flexible, transverse direction, but substantial effect in the stiffer, longitudinal direction. Their analysis showed a reduction of 3% in base shear in the transverse direction and 17% in the longitudinal direction due to the effects of soil-structure interaction. They also found that the results using the hand procedure of ATC-03 "conservatively estimate the reduction in base shear force in the longitudinal direction of the Warehouse building" but "fairly accurately" predict those in the transverse direction.

In the conclusions, Fenves proposes, "a more detailed modelling of the foundation and soil, accounting for embedment, piles, and soil layers, may improve the correlation between the model and the recorded transfer functions, particularly in the short period range" (Fenves and Serino, 1992). This research includes a more detailed study of the Hollywood Storage Building as recommended by Fenves and Serino.

# Chapter 3

## INVESTIGATION OF REDUCED PGA WITH DEPTH

One of the basic parameters traditionally used to judge the strength of earthquake ground motion is the peak ground acceleration (pga) at a free field or base of a building site. The peak ground acceleration is the largest recorded acceleration (positive or negative) during a strong-motion event at any time point in the strong-motion record. Engineers have included this parameter in the static lateral force provisions of the building code as a measure of a particular site's probable earthquake intensity. Although other parameters such as Richter magnitude represent the energy content of an earthquake more accurately, pga values are easy to obtain and give a general measure with which to compare the intensity of different earthquakes.

The intensity of the groundmotion and in turn the peak ground acceleration is dependent on a number of different factors including distance to the ruptured fault, epicenter and hypocenter, duration of strong shaking, local site conditions, depth to bedrock and so on. It has also been theorized by Seed and others that peak ground acceleration decreases with increasing depth. To verify this hypothesis, numerous records have been collected from downhole arrays of strong-motion instruments in Japan and California. Although the trend of decreasing pga with depth has been proven, it is a non-linear phenomena that is highly dependent on local soil conditions.

This "depth factor" has been used by the nuclear industry for many years to justify large reductions in ground motion (and thus also reduction in pga) due to soil-structure interaction caused by embedment of the structure. For this reason, most nuclear power plants are purposely embedded into the ground.

Seed and Lysmer studied the effect of embedment on conventional structures by comparing nearby pairs of buildings with strong-motion records from the 1971 San Fernando earthquake, each pair with one building constructed on the ground surface and one building with a full basement (1980). Seed observed a reduction in pga between the building on the ground surface and the building with the basement in 7 out of 8 cases (see Section 2.1 for additional discussion). With the large number of records available today, we can continue this investigation and improve upon the results.

In Tables 3.1 and 3.2, the peak ground accelerations in two orthogonal directions at a number of CSMIP building sites are listed along with each corresponding free field station. The records in the table are separated by those having been taken in basements

(Table 3.1) and those taken in buildings constructed on the ground surface (Table 3.2). In addition, Figure 3.1 and 3.3 show this information in graphical form for all buildings in Table 3.1 and 3.2 respectively. Figure 3.2 and 3.4 restrict the points on the graph to buildings that experienced a free field pga of 0.05g or greater.

### 3.1 Comparison with Seed & Lysmer Study

The data presented in Table 3.1 and 3.2 is an extension of the work presented by Seed and Lysmer for the 1971 San Fernando earthquake but it has a number of significant differences.

Table 3.1 reports peak ground acceleration values at the base of each building as compared to a nearby free field instrument whereas Seed and Lysmer used nearby pairs of buildings, one on the ground surface and one embedded. Since Seed and Lysmer performed their study, a number of recent earthquakes have produced an abundance of free field data for ground sites in California. The use of these free field stations is probably more appropriate since it more accurately represents the true response at the ground surface without any direct building influence.

The Seed and Lysmer study did not differentiate between the orthogonal directions of each building, nor whether the two instruments being compared (ground vs. basement) were in the same plan configuration. Table 3.1 shows the two directions independently from each other and the closest corresponding free field direction. Many buildings have both the building and free field instruments oriented in the same direction. However, some do not have free field and building instruments oriented together (see Appendix A, Figures 3 and 5 for each building's strong-motion instrumentation scheme). The correlations reported here are stronger when similar components are paired.

Seed and Lysmer used records with particularly strong shaking. On average, the ground motions they used had a pga of about 0.20g. Looking at Table 3.1 and Figure 3.1, it is apparent that as the shaking becomes stronger, the reduction of pga for buildings with basements becomes more pronounced and more repeatable.

### 3.2 Influential Factors

One problem with comparing pga values from two different sources is that the pga for each record can occur at different times. Because CDMG does not give a time of occurrence when reporting pga values, the only way to determine when the peak occurred is to look at the actual strong-motion trace. Included in Appendix A are copies of all the strong-motion traces used to establish pga values in Table 3.1 and 3.2, except those for the recent Northridge earthquake. If the acceleration peaks for a building free field pair occur within 2 seconds of each other, we deemed the pair a valid comparison. All pairs with peak time differences of greater than 2 seconds are notated in Table 3.1 and 3.2 with an asterisk (\*) after the base direction, and as an open diamond in Figures 3.1 through 3.4. Note that since this value of 2 sec. is arbitrary, no records were

discarded from the study and all records were used for both Table 3.1 and 3.2 and Figures 3.1 through 3.4.

Another problem with free field and building pairs of pga are that many times the instruments are not lined up but are offset by some angle. For the calculation of the response spectrums, this angle is taken into account by rotation of one of the pair of records. Pga values, however, are usually not corrected for rotation. In Tables 3.1 and 3.2, the pairs of base and free field values with the closest angle orientation are shown with no additional correction.

In addition to free field and building instruments not being aligned with each other, many times the instruments do not trigger at the same time and therefore do not have common timing. It is rare that a free field station is directly connected to a building so that a common trigger can be used. Often the free field instrument is located at some distance from the building without a clear, unobstructed path. Newer instruments use radio receivers to intercept the time radio broadcast from NOAA in Denver and stamp the trigger time onto the strong-motion trace. However, until recently, only free field instruments used this type of receiver. In order to correct for these obvious timing differences, a time shift has been applied to many of the time histories as needed to align the records. In all cases, the building record has been moved relative to the free field station.

In many areas, especially in downtown Los Angeles and San Francisco, it is very difficult to get a true "free field" record. Because of the many tightly spaced, large buildings in these locations, it is not clear that a ground instrument can accurately record the motion of waves in the free field without being influenced from surrounding buildings. To test for this phenomena, the fourier spectrum for both the free field and base of building records were plotted together and included in Appendix A as Figure 10. If the first mode frequencies for the ground and building are different, it can be reasoned that the ground record has not been "colored" by the adjacent building. This information is provided for each of the 11 buildings used for the more detailed study described in Chapter 4, but not for every pair in Table 3.1 and 3.2 because not all of the records have been digitized at the time of this report. Although interpretation is difficult, most of the 11 building studied have distinct free field records. Only the San Jose - 3 Story Office Building and the Imperial County Services Building have similar fourier spectrums for both building and free field time histories.

### **3.3 Buildings with Basements**

Looking at Table 3.1 and Figure 3.1 and 3.2, it can be seen that as the intensity of the ground motion increases, the reduction in motion between base and free field also increases, even for different earthquakes at the same building site. For small earthquakes, those with peak ground accelerations less than 0.06g, there is wide variability in the reduction in motion from +25% to -33%. For stronger earthquakes, Table 3.1 shows that buildings with basements consistently show a reduction in motion,

in some cases a reduction of as much as 50% for the Hollywood Storage Building during the Whittier earthquake, and the Seal Beach office building during the Northridge earthquake.

Part of the reason that weak earthquakes result in small reductions is likely due to the fact that small motions result in small pga values which have only one significant digit of accuracy. As in the case of Pomona - First Federal Savings Building during the Big Bear earthquake, a change in pga of .01g results in a percent change of 33%, certainly an anomalous situation.

Evidence that stronger earthquakes increase the reduction in pga between free field and building base for buildings with basements can be seen by looking at the Pomona - First Federal Savings Building records for Whittier, Landers, and Upland earthquakes. During the Whittier and Landers events, the Pomona building experienced about 0.07g and a reduction on average (both earthquakes and both directions) of about 15%. For the Upland earthquake, the ground motion increased to about 0.20g and the average reduction increased to about 40%.

At stronger levels of ground shaking, the reductions in pga between free field and base of building become repeatable. A good example of this is the Palm Springs - 4-Story Hospital. This hospital building experienced both the Big Bear and Landers earthquakes at similar levels of ground excitation, around 0.09g. In both cases, similar reductions occurred of -11.1% and -22.2% in the north direction and -33.3% and -33.3% in the west direction for both earthquakes respectively.

The highest reduction in pga recorded in Table 3.1 is for the Hollywood Storage Building in Los Angeles during the Whittier earthquake, which had 42.9% and 50% reductions respectively for the north and east directions. Although other researchers have shown that the Hollywood Storage Building has significant soil-structure interaction, most recently Fenves and Serino (1992), these observed reductions in pga values are quite high (see Chapter 5 and Table 5.1 for more discussion).

The strong-motion records themselves give a good qualitative measure of how much reduction occurs between free field records and base of building records. Figures 3.5 and 3.6 each show a 10 second interval of the strong ground shaking portion of the corrected acceleration time histories for both Richmond City Hall and Hollywood Storage Building respectively. It is quite clear that the reduction between the free field and basement records is not just at the peak ground acceleration for both records but at a number of intermediate peaks as well. The basement record, shown as a dark solid line, seems to have well defined peaks that are clearly reduced from the free field record, shown as a light solid line.

### 3.4 Buildings without Basements

For buildings without basements, shown in Table 3.2, substantial increases and decreases are observed at all levels of shaking. Earthquakes with  $p_{ga}$  values between 0.07g and 0.20g do not show any apparent trend in increasing or reducing motion at the base. However, earthquakes in the intermediate range (with  $p_{ga}$  values between 0.07g and 0.20g) all show significant reductions in base motion (except for the 3-Story Office Building in San Bernardino and the 1-story Supermarket building in Fortuna). Even with stronger shaking ( $p_{ga}$  values greater than 0.20g), some buildings, including the Imperial County Services Building and the 1-Story Warehouse in Hollister, continue to show large increases in motion. In fact, the Imperial County Services Building had the largest increase in response, one of the largest ground  $p_{ga}$  values, and was the only building on the list that was severely damaged.

These results are disturbing though inconclusive. It would be conservative to conclude that for all buildings without basements, no substantial decrease in motion can be justified using only ratios of  $p_{ga}$  values at the free field and building base. Of greatest concern however are those buildings showing substantial increases in motion. The many buildings with increases warrant further study in this area.

For example, although the Hollister Warehouse and the Fortuna Safeway are similar building types, both are concrete tilt-up buildings with a plywood roof, the Fortuna Safeway did not show repeatable increases in motion. Increases were recorded during the Petrolia mainshock but decreases recorded during the first aftershock. Also, both the San Bernardino office building and the Hollister Warehouse experienced large increases in one principal direction only. For the warehouse, the increase was in the flexible transverse direction. The office building, on the other hand, is a fairly symmetric box with perimeter moment-resisting frames in both directions. The available evidence does not lend itself to an obvious conclusion.

The strong-motion records for buildings built on the ground surface are an interesting contrast to those for buildings with basements. Figures 3.7 and 3.8 each show a 10 second interval of the strong ground shaking portion of the corrected acceleration time histories for both Hollister Warehouse and Imperial County Services Building respectively. It is quite clear that at the peak ground acceleration and at a number of intermediate peaks, there is an increase in response from the free field to the base level. This effect can be seen most clearly on the Hollister record at about 12 sec. and on the Imperial record at about 7.75 sec. and 10.5 sec. The basement record, shown as a dark solid line, is clearly larger in response than the free field record, shown as a light solid line. For the Hollister record, both base channels, Channel 7 in the middle of the building, and Channel 9 at the base of one of the end walls are plotted. The results for both channels are similar over most of the record as shown in Figure 3.7.

### 3.5 Trends and Conclusions

From our investigation, a number of important conclusions and recommendations can be made.

It is clear that on average, buildings with basements can experience a substantial reduction of motion at the base of the building when compared with the free field motion. In addition, the reduction tends to be larger during stronger shaking. For buildings built on the ground surface, no consistently significant reduction in response occurs and the increases cannot be overlooked.

If the information in this chapter is correlated with observed building damage for these buildings during each of these earthquakes, another important conclusion can be drawn. We have observed repeated events with ground shaking of up to 0.2g below which no significant structural damage has occurred. This "significant damage" can be described as damage which causes the building to be labelled a life-safety hazard (yellow or red tagged). Note that the data set of buildings studied did not include any unreinforced masonry structures nor any non-engineered structures both of which might be damaged at lower levels of shaking.

To aid future studies of this nature, the following recommendations are offered to improve the Strong Motion Instrumentation Program of CDMG:

1. Free field stations should be installed at all instrumented buildings sites. These free field stations should be unobstructed by other buildings, as much as possible.
2. To achieve common timing, radio transmitters should be installed in both building and free field accelerometer pairs so that accurate time shifts can be applied to the records when the instruments do not trigger at the same time.

Building	Basement Depth	Earthquake	Maximum Acceleration				Percent change
			Base		FF		
Los Angeles - Hollywood Storage Building	20'	Big Bear	N	0.03	360°	0.03	0.0%
			E	0.03	90°	0.03	0.0%
Pomona - First Federal Savings Building	10.5'	Big Bear	N	0.02	360°	0.03	-33.3%
			W	0.02	90°	0.03	-33.3%
Los Angeles - 54-Story Office Building	45'	Landers	N	0.04	179°	0.04	0.0%
			E	0.03	89°	0.04	-25.0%
Los Angeles - 12-Story Commercial/Office	20'	Landers	N	0.04	179°	0.04	0.0%
			E	0.04	89°	0.04	0.0%
Los Angeles - 52-Story Office Building	57'	Landers	N*	0.05	179°	0.04	+25.0%
			E	0.04	89°	0.04	0.0%
Los Angeles - 9-Story Office Building	13'	Landers	N	0.03	179°	0.04	-25.0%
			W*	0.05	89°	0.04	+25.0%
Pomona - First Federal Savings Building	10.5'	Whittier	N	0.05	12°	0.07	-28.6%
			W	0.05	102°	0.06	-16.7%
Pomona - First Federal Savings Building	10.5'	Landers	N	0.06	360°	0.07	-14.3%
			W	0.05	90°	0.05	0.0%
Hayward - 13-Story CSUH Admin. Building	20'	Loma Prieta	N	0.07	360°	0.08	-12.5%
			E	0.09	90°	0.08	+12.5%
Palm Springs - 4-Story Hospital	14'	Big Bear	N	0.08	360°	0.09	-11.1%
			W	0.06	90°	0.09	-33.3%
Palm Springs - 4-Story Hospital	14'	Landers	N	0.07	360°	0.09	-22.2%
			W	0.06	90°	0.09	-33.3%
Seal Beach - 8-Story Office Building	16'	Northridge	N	0.05	360°	0.08	-50.0%
			E	0.08	90°	0.11	-33.0%
Richmond - Richmond City Hall	10'	Loma Prieta	S*	0.12	190°	0.13	-7.7%
			W	0.09	280°	0.11	-18.2%
Los Angeles - 54-Story Office Building	45'	Northridge	N	0.14	179°	0.19	-22.0%
			E	0.09	89°	0.10	-10.0%

\* indicates difference in free field peak occurrence and building peak occurrence greater than 2 seconds apart.

**TABLE 3.1a - Change in Maximum Acceleration Between Basement of Building and Free field for Buildings with Basements**

Building	Basement Depth	Earthquake	Maximum Acceleration				Percent change
			Base		FF		
Los Angeles - 52-Story Office Building	57'	Northridge	N	0.15	179°	0.19	-21.0%
			E	0.11	89°	0.10	+10.0%
Los Angeles - Hollywood Storage Building	20'	Whittier	N	0.12	360°	0.21	-42.9%
			E	0.06	90°	0.12	-50.0%
Pomona - First Federal Savings Building	10.5'	Upland	N	0.13	360°	0.20	-35.0%
			W	0.13	90°	0.23	-43.5%
Los Angeles - Hollywood Storage Building	20'	Northridge	N	0.29	360°	0.41	-30.0%
			E	0.21	90°	0.24	-12.0%

\* indicates difference in free field peak occurrence and building peak occurrence greater than 2 seconds apart.

**TABLE 3.1b - Change in Maximum Acceleration Between Basement of Building and Free field for Buildings with Basements**

Building	Earthquake	Maximum Acceleration				Percent change
		Base		FF		
Los Angeles - 15-Story Government Office Bldg.	Big Bear	N	0.03	180°	0.04	-25.0%
		W	0.02	90°	0.02	0.0%
Los Angeles - 15-Story Government Office Bldg.	Landers	N	0.03	180°	0.04	-25.0%
		W	0.03	90°	0.04	-25.0%
Los Angeles - 17-Story Residential Building	Big Bear	S	0.03	180°	0.04	-25.0%
		E	0.04	90°	0.02	+100.0%
Los Angeles - 17-Story Residential Building	Landers	S	0.04	180°	0.04	0.0%
		E*	0.05	90°	0.04	+25.0%

\* indicates difference in free field peak occurrence and building peak occurrence greater than 2 seconds apart.

**TABLE 3.2a - Change in Maximum Acceleration Between Base of Building and Free field for Buildings Constructed on Top of Ground Surface**

Building	Earthquake	Maximum Acceleration				Percent change
		Base		FF		
Long Beach - 15-Story Government Office Building	Northridge	N	0.03	360°	0.06	-50.0%
		E	0.04	90°	0.06	-33.0%
Lancaster - Medical Office Building	Whittier	N	0.05	10°	0.06	-16.7%
		E	0.06	110°	0.06	0.0%
Sylmar - Olive View Medical Center	Landers	N*	0.06	356°	0.04	+50.0%
		E	0.03	86°	0.06	-50.0%
Sylmar - Olive View Medical Center	Whittier	N	0.06	356°	0.06	0.0%
		E	0.06	86°	0.05	+20.0%
Long Beach - Harbor Administration Building	Whittier	N	0.04	360°	0.05	-20.0%
		E	0.07	90°	0.07	0.0%
Eureka - 5-Story Residential Building	Petrolia Aftershock #1	S	0.08	360°	0.07	+14.3%
		E	0.04	90°	0.06	-33.3%
Lancaster - Medical Office Building	Landers	N	0.04	25°	0.05	-20.0%
		E	0.07	115°	0.08	-12.5%
Piedmont - 3-Story School Building	Loma Prieta	N*	0.08	315°	0.07	+14.5%
		E	0.07	45°	0.08	-12.5%
Palm Desert - 4-Story Medical Office Building	Landers	N	0.06	360°	0.09	-33.3%
		E	0.06	90°	0.09	-33.3%
Palm Desert - 4-Story Medical Office Building	Big Bear	N	0.07	360°	0.09	-22.2%
		E	0.06	90°	0.09	-33.3%
San Bernardino - 3-Story Office Building	Landers	N	0.08	360°	0.09	-11.1%
		W	0.11	270°	0.08	+37.5%
San Bernardino - 3-Story Office Building	Big Bear	N	0.09	360°	0.10	-10.0%
		W	0.13	270°	0.09	+44.4%
Newport Beach - 11-Story Hospital	Northridge	N	0.05	360°	0.08	-50.0%
		E	0.08	90°	0.11	-35.0%

\* indicates difference in free field peak occurrence and building peak occurrence greater than 2 seconds apart.

TABLE 3.2b - Change in Maximum Acceleration Between Base of Building and Free field for Buildings Constructed on Top of Ground Surface

Building	Earthquake	Maximum Acceleration				Percent change
		Base		FF		
San Bernardino - 1-Story Commercial Building	Big Bear	S	0.07	360°	0.11	-36.4%
		E	0.07	270°	0.12	-41.7%
San Bernardino - 9-Story Commercial Building	Big Bear	S*	0.08	360°	0.11	-27.3%
		E	0.08	270°	0.12	-33.3%
San Bernardino - 1-Story Commercial Building	Landers	S	0.09	360°	0.12	-25.0%
		E	0.08	270°	0.12	-33.3%
San Bernardino - 9-Story Commercial Building	Landers	S*	0.10	360°	0.12	-16.7%
		E	0.08	270°	0.12	-33.3%
Fortuna - 1-Story Supermarket Building	Petrolia	S	0.14	360°	0.12	+16.7%
		W	0.13	90°	0.12	+8.3%
Eureka - 5-Story Residential Building	Petrolia	S	0.15	360°	0.16	-6.3%
		E	0.16	90°	0.17	-5.9%
Fortuna - 1-Story Supermarket Building	Petrolia Aftershock #1	S	0.17	360°	0.19	-10.5%
		W*	0.18	90°	0.19	-5.3%
El Centro - Imperial County Services Building	Imperial Valley	N	0.35	2°	0.24	+45.8%
		E	0.32	92°	0.24	+33.3%
San Jose - 3-Story Office Building	Loma Prieta	N*	0.20	135°	0.28	-28.6%
		W	0.20	225°	0.27	-25.9%
Los Angeles - UCLA Math-Science Building	Northridge	N	0.29	360°	0.66	-56.0%
		E	0.25	90°	0.32	-22.0%
Hollister - 1-Story Warehouse	Loma Prieta	N	0.36	360°	0.38	-5.3%
		E	0.25	90°	0.18	+38.9%
Sylmar - 6-Story County Hospital	Northridge	N	0.82	360°	0.91	-10.0%
		W	0.42	90°	0.61	-30.0%

\* indicates difference in free field peak occurrence and building peak occurrence greater than 2 seconds apart.

**TABLE 3.2c - Change in Maximum Acceleration Between Base of Building and Free field for Buildings Constructed on Top of Ground Surface**

# Chapter 4

## GROUND MOTION ANALYSIS

A ground motion analysis technique was developed to quantify in design terms the amount of reduction in response due to soil-structure interaction using simple tools, recorded strong-motion records, and existing design-based techniques. The analysis technique is predicated on pairs of records and their response spectra (building and free field) for each earthquake and building to be investigated. Although CSMIP has many building instruments, only a small percentage have free field instruments in close enough proximity to enable this type of analysis. Because the response of many regular buildings is dominated by their fundamental mode of vibration, response spectra for the free field and for the building base were compared in the fundamental period range.

### 4.1 Determination of Building Period

The first step was to determine the building period. For each building record in each principal direction, the fourier transform was calculated from the time history at the roof and the time history at the base (see Figure 4.1). The roof spectrum was divided by the base spectrum to form a transfer function which was plotted against frequency. An example is shown in Figure 4.2 for the EW direction of Hollywood Storage Building. The first peak is characteristically the building's fundamental frequency. This method was used by Cole et.al. in a recent CSMIP study on building periods (Cole, 1992).

It is often has been suggested that the building period will lengthen as an earthquake progresses and the building begins to yield. However, when an entire time history is used in the calculations of the transforms, this effect is lost. To accurately follow the change in building period over time, the time history was divided into a number of five second time steps or "windows". The transfer function of roof/base fourier transform was computed for each time window and these were plotted together on one graph (see Figure 4.2). In this way, the period of the building during the time of strong ground shaking can be observed separately from the building period after the shaking has stopped and the building is vibrating harmonically. The period used was typically the harmonic period of the structure after shaking had stopped. This was usually always close to the average period over the entire time history range.

For most records studied, the period remained fairly constant during the entire duration of ground shaking. This is not surprising since most of the buildings show little damage. However, in some cases, most notably the Imperial County Services Building, the period lengthens significantly (see Figure 4.5). This building experienced significant structural

damage at about 11 seconds into the strong motion record. The period starts at about 0.63 sec. (between 5 and 10 seconds during the time history), lengthens to 0.83 sec. (between 10 to 15 seconds during the time history), and finally stops at 0.95 (between 15 and 30 seconds during the time history). The overall building period using the entire record is 0.67 which is close to the period of the original undamaged structure.

#### 4.2 Response Spectrum Analysis Technique

After the building period has been determined, the response spectrum at the base of the building was plotted along with the response spectrum for the properly rotated direction of the free field record on the same graph. If a line is drawn at the building's fundamental period, a change in the response spectra values taken from the base and free field records can be observed at or slightly above the building period (see Figure 4.3).

This reduction in response can better be seen by dividing the base by the free field response spectrums and plotting the spectral ratio. Frequencies with a spectral ratio below 1.0 show a reduction in spectral acceleration. At the building's measured fundamental period, this shows as a valley in the graph (see Figure 4.4). Frequencies with a spectral ratio above 1.0 show an increase in spectral acceleration response. This would show as a peak on a graph similar to the one shown in Figure 4.4.

Bldg. Name	Bldg. Type	Foundatio n Type	Site Geology	# Stories	EQ	FF PG A	Dir	Bldg. Perio d	% change
Richmond City Hall	Conc. MF	Spread footings	Alluvium	3/1	Loma Prieta	0.13	S	0.29	+40%
							W	0.25	-20%
LA - Hollywood Storage Bldg.	Conc. shearwalls (EW)	Bearing Piles	Alluvium over shale & sandstone	14/ partial	Whittie r	0.21	N	2.27	+6%
							E	0.60	-32%
Pomona - First Federal Savings	Conc. MF	Spread footings	Alluvium	2/1	Whittie r	0.07	W	0.27	-9%
							N	0.26	+1%

<sup>1</sup>Above ground/below ground

**TABLE 4.1 - Comparison of Percent Reductions in Response for 3 Buildings with Basements**

This increase or reduction in response between the base of the building and the free field at the building's fundamental frequency is recorded in the last column of Table 4.1 and 4.2. Table 4.1 is for the three buildings with basements and Table 4.2 is for the eight buildings constructed on top of the ground surface. In addition, figures similar to 4.1 through 4.4 are included in Appendix A, for each of the 11 buildings listed in Tables 4.1 and 4.2.

Bldg. Name	Bldg. Type	Foundation Type	Site Geology	# Stories	EQ	FF PGA	Dir	Bldg. Period	% change
Hayward - CSUH	Conc. MF	Bearing Piles	Franciscan rock	13	Loma Prieta	0.08	E	1.39	-20%
							N	1.39	-40%
Hollister Warehouse	Tilt-up w/ plywood roof	Spread footings	Alluvium	1	Loma Prieta	0.38	E	0.73	0%
							N	0.15	-15%
Piedmont Jr. High School	Conc. shearwalls	Spread footings w/ tie beams	Weathered serpentine	3	Loma Prieta	0.08	N	0.16	+20%
							E	0.16	-8%
San Jose - Office Bldg.	Steel MF	Spread footings	Rock	3	Loma Prieta	0.28	W	0.71	-10%
							N	0.74	-13%
Lancaster - MOB	Masonry bearing walls	Conc. piers w/ grade beam	Alluvium	3	Whittier	0.06	E	0.12	-12%
							N	0.09	-42%
Long Beach - Harbor Admin. Bldg.	Steel MF	Bearing Piles	Deep alluvium	7	Whittier	0.07	N	1.20	+7%
							E	1.41	+5%
Sylmar - Olive View Medical Center	Conc. and steel shearwalls	Spread footings	Alluvium	6	Whittier	0.06	N	0.30	+34%
							E	0.27	+24%
Imperial County Services Bldg.	Conc. shearwalls (NS)	Spread footings	Alluvium	6	Imperial Valley	0.27	N	0.50	+14%
							E	1.00	0%

TABLE 4.2 - Comparison of Percent Reductions in Response for 8 Buildings Constructed on Top of Ground Surface

### 4.3 Trends and Conclusions

Table 4.1 suggests that the change in motion between the free field and building base varies considerably from +40% to -42%. In light of this, a number of important observations can be made for the 11 pairs of data investigated.

The number of records that showed a reduction in response is equal to the number that showed an increase in response: 9 showed a reduction, 9 showed an increase, and 2 showed no change. The variation in reduction in response and increase in response is also about the same: the reductions vary from -9% to -42% and the increases vary from +1% to +40%. The average reduction in response is about 50% greater than the average increase in response: the average reduction is -24.6% and the average increase is +16.8%. This is obviously not the kind of trend that can be added to the building code.

In Chapter 3, it was apparent that buildings with basements showed a repeatable reduction in response when compared to a nearby free field record. When comparing Table 4.1 and 4.2, the average reduction for buildings with basements is just -2.3% and for buildings constructed on top of the ground surface, the average reduction is actually greater, -3.5%. This does not seem to correlate well with the results listed in Table 3.1 and 3.2.

When individual records are investigated, for example the Hollywood Storage Building for the Whittier earthquake, the results between the pga analysis and the response spectra analysis still do not correlate. Using pga values, the north direction of Hollywood Storage shows a reduction of 42.9% and in the east direction a reduction of 50%. Using response spectral values, the north direction shows an increase of 6% and in the east direction a reduction a reduction of 32%. The other records are similar - a few results are close (within 10%), but many show no correlation.

Overall, this same behavior seems to occur regardless of the size of the earthquake, the type of soil, the type of construction, or the foundation condition. No significant trend of reduction is currently apparent as we had hoped. Also, there seems to be no correlation with the results shown in Tables 3.1 and 3.2 and those in Table 4.1 and 4.2, even for the exact same building and earthquake.

It is apparent that response spectra are unable to reflect the reduction in response between the free field and building base due to soil-structure interaction. Part of this might be due to the uncertainty inherent in determining the building period. Another part might be due to the response spectra itself. While useful as a tool for engineers to investigate overall building response and calculate the building's base shear, response spectra by their nature cannot reflect changes in response over time, but only the maximum response. In this instance, perhaps the more accurate way to measure these reductions are through the direct calculation of base shear demand using actual strong-motion time-histories.

It should also be pointed out that for new design projects, the free-field spectrum that a geotechnical engineer provides the building designer has been "smoothed" for design. Even spectrums generated from SSI analyses are smoothed for design using a number of real and generated time-histories to provide a bounded solution. When these two smoothed spectra are compared, a more direct calculation of the change in response due to SSI can be calculated using the ordinates at the fundamental period of the building. This does not work well for our study however since these smoothing techniques tend to mask the results using actual time-histories.

# Chapter 5

## SOIL-STRUCTURE INTERACTION ANALYSIS

In order to investigate the validity of current SSI analysis techniques for conventional buildings, four sites were chosen for more detailed analysis. The sites chosen, Richmond City Hall, Imperial County Services Building, Hollywood Storage Building, and Hayward - CSUH Administration Building, were selected to represent a variety of different building and soil types (see Table 4.1 and 4.2). The analysis procedure used was based on the FLUSH soil-structure analysis program, using commercially available techniques and procedures. The original intent was to develop a standard technique that would be usable in the design office and available for developing code provisions.

### 5.1 Modelling Procedure

The stick model was developed using a two-dimensional or three-dimensional full-frame SAP90 model of the building. The model was loaded with static unit loads and the displacements computed. If the building is a stiff, shearwall building, like Richmond City Hall, it was assumed to behave like a shear beam. The shear areas were backcalculated from the story shears and the displacements, and the moments-of-inertia assumed to be very large. If the building is a more flexible moment frame building, like the Hayward - CSUH Admin. Building, it was assumed to act like a cantilevered frame. The shear areas and moments-of-inertia were backcalculated from the displacements and rotations at each story. The stick model was then checked against the full-frame model for proper modal behavior, matching displacements, and a consistent fundamental period of vibration. Note that for the most part, only the first mode behavior of the stick matched that of the full building model. Each building model was unique and care was taken to accurately model each building as a multi-degree-of-freedom stick model.

The soil profile was developed from the available geotechnical reports utilizing logs of borings at the building site and shear wave velocity profiles when available. When possible, the data was based on borings that went down to bedrock. In many places however, such as within the Los Angeles basin, borings stop well short of this depth. For these sites, an educated estimate of the shear wave velocity profile past the depth at which the borings stop was made.

The soil-structure interaction analysis developed for this study used FLUSH, an SSI program developed by Lysmer et. al. (1979). The program uses a two-dimensional finite element mesh capable of representing differing soil characteristics with depth and lateral

extent. It can approximate the behavior in three-dimensions by the use of energy-dissipating dashpots in the out-of-plane direction. For buildings with basements, the basement condition was modelled as rigid. The soil finite element used in FLUSH incorporates non-linear material behavior. Each mesh was generated such that the model would be valid for frequencies up to 15 Hz. Since the motions in the soil are assumed to be vertically propagating S-waves, the influence of frictional piles, such as used for Hollywood Storage Building, were neglected since they are not expected to significantly influence the horizontal response motions.

## 5.2 Building Descriptions

### 5.2.1 Richmond City Hall

The 3-story Richmond City Hall is located between 25th and 27th street, near MacDonald Avenue in Richmond. The free field ground station is located at the northwest corner of the city library parking lot, west of MacDonald Avenue and south of Civic Center Plaza as shown in Figure 5.1 and 5.2.

The site is located on Pleistocene alluvium described as consisting principally of weakly consolidate, slightly weathered, poorly sorted, irregularly interbedded clay, silty, sand and gravel. The bedrock depth contour for the Richmond area, see Figure 5.4, shows that the site is located probably at the deepest portion of the buried channel between the East bay hills on the west and the San Pablo hills on the east.

The subsurface soil conditions at the Richmond City Hall parking lot were studied by Woodward-Clyde in 1992. They consist of 2 feet of fill (silty sand) overlying 6 feet of firm, bluish gray, highly plastic Bay mud. Underlying the Bay mud is a layer of grayish brown to orange-brown silty clay with moderate plasticity and firm to stiff consistency that extends to 160 feet. Between 160 feet 190 feet, the soil consists of olive-brown clayey gravels and orange-brown sandy clay with occasional gravel lenses. Highly weathered sandstone is uncounted at 190 feet, becomes less weathered at 204 feet, and extends to the bottom of the borehole at 243 feet (see Figure 5.3). Shear wave velocity measurements of the borehole were conducted by Agbabian Associates in 1993 under the sponsorship of Electric Power Research Institute (see Figure 5.5). The finite element mesh extends vertically to the top of sandstone at about 100-ft where a sharp velocity was encountered.

Richmond City Hall is a 3-story, 37.5' tall building with a 10' deep basement (see Figure 5.6). It is rectangular in plan, measuring 68' X 260' and is built on spread footings. A reinforced concrete moment resisting frame carries both gravity and lateral loads. Additional lateral resistance is provided in both directions by reinforced concrete shear walls at the basement level in both the interior and on the perimeter of the building. Shear walls at the corners of the structure extend to the roof. All other shear walls terminate at the ground floor.

### 5.2.2 *Imperial County Services Building*

The Imperial County Services Building (ICSB) was located in El Centro, California. El Centro lies in the central portion of the arid Salton Trough, a low-lying, alluvium-floored depression extending from the San Bernardino Mountains of the Transverse Range southeastward to the Gulf of California (see Figure 5.7).

The near surface alluvial deposits of Quaternary age in the El Centro area consist of Lake Coahuilla deposits (Ql) and Quaternary alluvium (Qal). In 1982, USGS conducted geotechnical investigations at a number of strong-motion stations in the Imperial Valley of California. The exploration at the ICSB site extended to a depth of 100 feet. The boring log, included as Figure 5.9, shows that the 100 feet of soil encountered at the site consists of interbedded layers of medium dense to dense silty sand stiff clayey silt to silty clay. Below 100 feet, Shanon and Wilson (1976) conducted a detailed subsurface soil investigation and took velocity measurements at a nearby station (El Centro Array No. 9), about 2500 feet northeast of the ICSB site (see Figure 5.8). This information was used to construct the finite element mesh below 100 feet to a depth of 400 feet (see Figure 5.12).

The Imperial County Services Building itself was a 6-story, 81'-8" tall structure measuring 85'-4" X 136'-10" in plan (see Figure 5.13). It was founded on spread concrete footings and had Raymond step-taper piles located under each column. Gravity loads were carried by 5" reinforced concrete slabs supported by reinforced concrete pan joists spanning in the transverse direction into a reinforced concrete moment frame. Lateral loads were resisted by the concrete moment frame in the longitudinal direction and by concrete shear walls in the transverse direction. There are four shear walls at the ground level, positioned at interior column bays. Shear walls in the upper stories are located at the east and west ends of the building and terminate at the second floor. The building was structurally damaged in the 1979 Imperial County Earthquake and subsequently demolished.

### 5.2.3 *Hollywood Storage Building*

The Hollywood Storage Building is located at the southwest corner of Santa Monica Boulevard and Highland Avenue in Hollywood, California (see Figure 5.14). The site is underlain by Lakewood formation of upper Pleistocene age to a depth of about 340 ft, followed by Puente formation of upper Miocene age, and in turn by Monterey formation and Topanga formation of middle Miocene age to a depth of about 8000 feet (see Figure 5.15). The subsurface soil conditions at the site were obtained using a compilation of oil exploration logs, water logs, and soil boring logs (Duke et al., 1970 and USGS, 1982). Shear wave velocities of the near surface materials, where SSI effects are most pronounced, were measured either by downhole geological logging (USGS, 1982) or by surface refraction tests (Duke et al., 1970). The deeper portions of the velocity profile were inferred from geological sources and oil well velocity logs (Duke et al., 1970 and Duke and Leeds, 1962). A sharp velocity contrast is noted at a depth of about 340 feet

where the Miocene age formation is encountered. This sharp velocity contrast provides an excellent boundary for the vertical dimension of the finite element mesh. Thus the SSI mesh and was extended from ground surface to a depth of about 340 feet. The velocity profile used for the analysis is shown in Figure 5.16.

The Hollywood Storage Building is a 149' tall, 14-story structure with a 9' deep basement under the west half of the building (see Figure 5.17). The building measures 51' X 217' in plan and has a foundation consisting of reinforced concrete footings on concrete piles. Gravity loads are carried by reinforced concrete slabs supported by joists tied into a concrete moment frame. In the transverse direction, the concrete moment frame also provides lateral resistance in conjunction with concrete shear walls at the ground floor. In the longitudinal direction, two exterior coupled shearwalls provide lateral resistance.

#### 5.2.4 Hayward - CSUH Administration Building

The CSUH Administration Building is located at the Hayward campus of California State University as shown on Figure 5.18. The Hayward campus is located on the San Francisco East bay hills, not far from the Hayward fault (see Figure 5.22).

The free field ground station sits between the stadium and the baseball field, approximately 1600 feet northwest from the administration building (see Figure 5.19). In March 1975, a 100-ft deep soil boring was made 20 feet north of this free field station by USGS to study the subsurface ground condition at the CSUH site and to perform a downhole velocity survey. The boring log and velocity profile were published by USGS in an Open-File Report 76-731 (USGS, 1976). The subsurface profile at the site consists of approximately 3 feet of surficial sandy clay (CL) over rhyolite which has been deeply weathered to sandy clay loam (CL to SC) and gravelly sandy clay loam (CL to GC). At a depth of about 11 feet, the material grades to moderately weathered rhyolite with close to very close fractures (see Figure 5.20). We have assumed that the partial basement of the Administration building is seated on this moderately weathered rhyolite.

A finite element mesh was used to model the rhyolite foundation material to a depth of 200 feet with shear wave velocity of 2700 ft/sec and a Poisson ratio of 0.23. These values were selected based on the previously mentioned USGS Open-File Report. Pertinent subsurface information including boring log, velocity profile and geologic map are included as Figures 5.20, 5.21 and 5.22 respectively for reference.

The CSUH Administration building in Hayward is a 13-story, 201' tall structure which is 122' X 125' in plan (see Figure 5.23). The foundation is an 18" reinforced concrete slab on grade supported by bearing piles. Gravity loads are carried by metal deck floor slabs filled with 2.5" of lightweight reinforced concrete and supported by a steel moment frame. Lateral loads are resisted in both directions by a steel moment frame on the interior, a reinforced concrete moment frame around the perimeter, and shear walls around the elevators from the basement to the second floor.

### 5.3 Analysis Comparisons and Conclusions

The four buildings were analyzed in both principal directions except CSUH which is symmetric in both directions. The results of the seven soil-structure interaction analyses completed for this study are summarized in Table 5.1, in the column labelled "FLUSH". In addition, the response spectrums and a portion of the time-history trace are plotted in Figures 5.24 through 5.37. On each graph is plotted the results recorded at the base of the building versus the corresponding results for the FLUSH analysis.

Five other columns are included in Table 5.1 for comparison. The "TH Stick" column shows the amount of base shear reduction calculated using a stick model analysis with the time history recorded at the base of the building as the input motion compared to using the free field time history as the input motion. The "pga" column repeats the results from Chapter 3 for each of the seven cases - the difference in peak ground acceleration between the building base and the free field. The "RS Ratio" column repeats the results of Chapter 4 for each case - the difference in each response spectrum at the base of the building vs. the free field. The "ATC 3-06" column shows the amount of reduction in base shear calculated using the hand procedure in ATC 3-06. The "Other Studies" column shows the results of other SSI analyses which looked at the same buildings (Fenves and Serino, 1992).

Building Name	Direction	Percent Reduction in Base Shear					
		TH Stick	FLUSH	pga	RS Ratio	ATC 3-06	Other Studies*
Richmond City Hall	NS	+8%	-20%	-8%	+40%	-25%	-
	EW	-18%	-16%	-18%	-20%	-30%	-
Imperial County Services	NS	-3%	+30%	+46%	+14%	-15%	-
	EW	+15%	-2%	+33%	0%	0%	-
Hollywood Storage	NS	-6%	-2%	-43%	+6%	-8%	-3%
	EW	-34%	-15%	-50%	-32%	-15%	-17%
Hayward CSUH	NS/EW	-28%	+3%	-12%	-40%	-4%	-

\* Fenves and Serino, (1992)

TABLE 5.1 - Comparison of Reduction in Response Due to Soil-Structure Interaction

It is assumed that the control case for comparison in Table 5.1 is the time history analysis using the building stick model, column "TH Stick". This is the only analysis which correctly takes into account the actual mass, stiffness, and time-dependent motion of the actual building using the strong motion records collected from the earthquake directly.

In some cases, the FLUSH procedure accurately captures the spectral shape at the base of the structure. The response spectrum comparisons for Richmond City Hall in both directions, and Hollywood Storage Building in the EW direction show very good agreement over the entire period range (see Figures 5.24, 5.25, and 5.33). Two of these three results also agree well with the time history building stick models shown in the "TH Stick" column. The spectrums for Hollywood Storage Building in the NS direction and Hayward - CSUH Admin. Building show good agreement in the period of interest (see Figures 5.32 and 5.36) but not as good agreement in the low period range. These two results do not agree as well with the time history building stick models.

For the Imperial County Services Building, the results of the FLUSH analysis are not in good agreement with the recorded motions (see Figures 5.28 and 5.29). First, the motion at the base of the building is significantly higher than the free field motion over the entire period range. This is contrary to all other records looked at for this study. It is unusual that the base of the building amplified the free field motion even at very low periods (high frequency motion) which is usually not amplified by typical structures. Second, the computed building response using FLUSH is strongly influenced by the free field control motion. The model is not able to reproduce the high spectral amplification in the period range between 0.2 to 0.5 seconds seen in the building base record. In addition, the results from the FLUSH analyses do not seem to correlate well with what is seen by the building strong-motion time histories (see Figures 5.30 and 5.31).

The comparison of the results using FLUSH for Hollywood Storage Building look very comparable to the results obtained by Fenves and Serino (1992) (see Table 5.1) and to those using the time-history building stick model. This suggests that for stiff, uniform sites, low to moderate levels of seismic excitation, and a regular shaped building, current analysis techniques are adequate to determine the amount of reduction in base shear due to soil-structure interaction.

It is clear that the results of using "pga" values as described in Chapter 3 are good for determining overall trends in amounts of base shear reductions for groups of buildings but not final values for individual buildings. Peak ground acceleration values are time independent, vary with building orientation, and seem to only be valid for stronger earthquakes, those with motion greater than about .08g.

The results using "RS Ratio" as described in Chapter 4 seem to give erratic results in most cases. In general, since response spectra do not take time into consideration and are difficult to use for design until smoothed, they are not recommended to calculate reductions in base shear due to soil-structure interaction.

The results of the "ATC 3-06" hand analyses are in generally good correlation with those of the time-history analysis using the building stick model. However, in some cases it overpredicts the amount of reduction, (Richmond City Hall in the EW direction and Imperial County Services in the NS direction) and in some cases it underpredicts the results (Hollywood Storage Building in the EW direction and CSUH Hayward). It also is unable to predict increases in response that are calculated using the actual strong motion time histories. In theory, this would seem to be difficult to calculate using a hand procedure until the reason for the increase in response is known. It is also interesting to note that the ATC 3-06 results are generally close to those of the FLUSH analysis results except for the Imperial County Services Building in the NS direction. For a preliminary assessment, it appears that the ATC 3-06 hand procedures provide the right order of magnitude estimate of base shear reductions due to SSI for many sites. However, it should be pointed out that increases in response can occur and that the hand procedure cannot predict these occurrences.



# Chapter 6

## CONCLUSIONS AND RECOMMENDATIONS

An analysis of strong-motion records for 11 CSMIP building/free field pairs to investigate the reduction in building response due to soil-structure interaction has been completed. Soil-structure interaction analyses using the FLUSH computer program for four CSMIP buildings sites, comparison of recorded with model response, and comparison of the base shear reduction using FLUSH with the results of stick models, an ATC 3-06 hand analysis, and previous analyses have also been completed. Based upon the data collected, the following observations were made:

- (1) Buildings are not simple, static structures, but are complex and respond non-linearly during dynamic excitation. Soil-structure interaction is a complex phenomena and is difficult to predict.
- (2) There is an observed reduction of base motion for regular buildings with basements when the pga during a strong-motion event is greater than 0.08g. This seems to occur even for small basements, say those that cover only half of the building plan area, and shallow basements, those which are only as deep as ten feet.
- (3) CSMIP should be encouraged to place free field instruments near instrumented buildings so we may obtain more data pairs and continue to investigate the effects of SSI on building response.
- (4) CSMIP should also be encouraged to install radio transmitters in both building and free-field instruments so that time shifts can be applied to the records when the instruments do not trigger at the same time.
- (5) Using the difference in actual recorded spectral accelerations to predict the amount of base shear reduction due to soil-structure interaction is not supported by the records studied. It seems clear that when available, time histories should be used for post-earthquake dynamic analysis and response spectra reserved for design.
- (6) Buildings have a significant effect on the response they experience during an earthquake. As a consequence, free field spectra should be replaced with base of building spectra if possible for design of buildings. Also, base of building records

instead of free field records should be used to make attenuation relationships applicable to buildings.

- (7) Increases in building response at the base of a building when compared with the free field should be investigated further. As engineers, we need to understand why these increases occur so that they can be predicted and taken into account where applicable.
- (8) Our current hand techniques for investigating the effects of SSI on conventional building structures such as those outlined in ATC 3-06 provide the right order of magnitude estimate of base shear reductions due to SSI for many sites. However, it should be pointed out that increases in response can occur and that the hand procedure cannot predict these occurrences.
- (9) Our current analytical techniques for investigating the effects of SSI on conventional building structures such as FLUSH are adequate in most, but not all cases. More analyses need to be performed to refine and improve the techniques presented here to achieve better results for different types of buildings and soil conditions.
- (10) It is too early to propose any method to incorporate into building codes to account for soil-structure interaction. More research needs to be done utilizing the most recent CSMIP strong-motion records for instrumented buildings.

# Chapter 7

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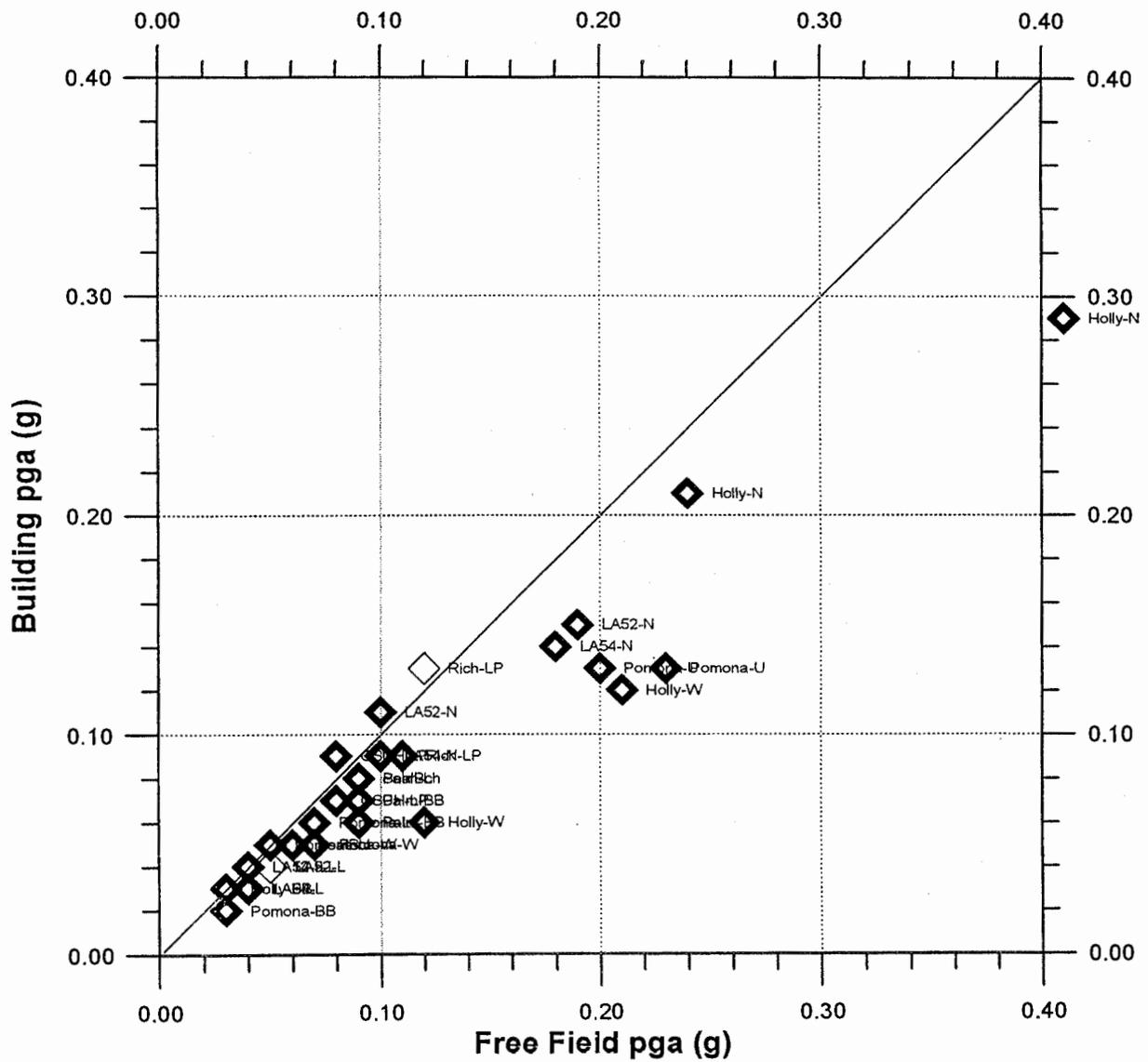
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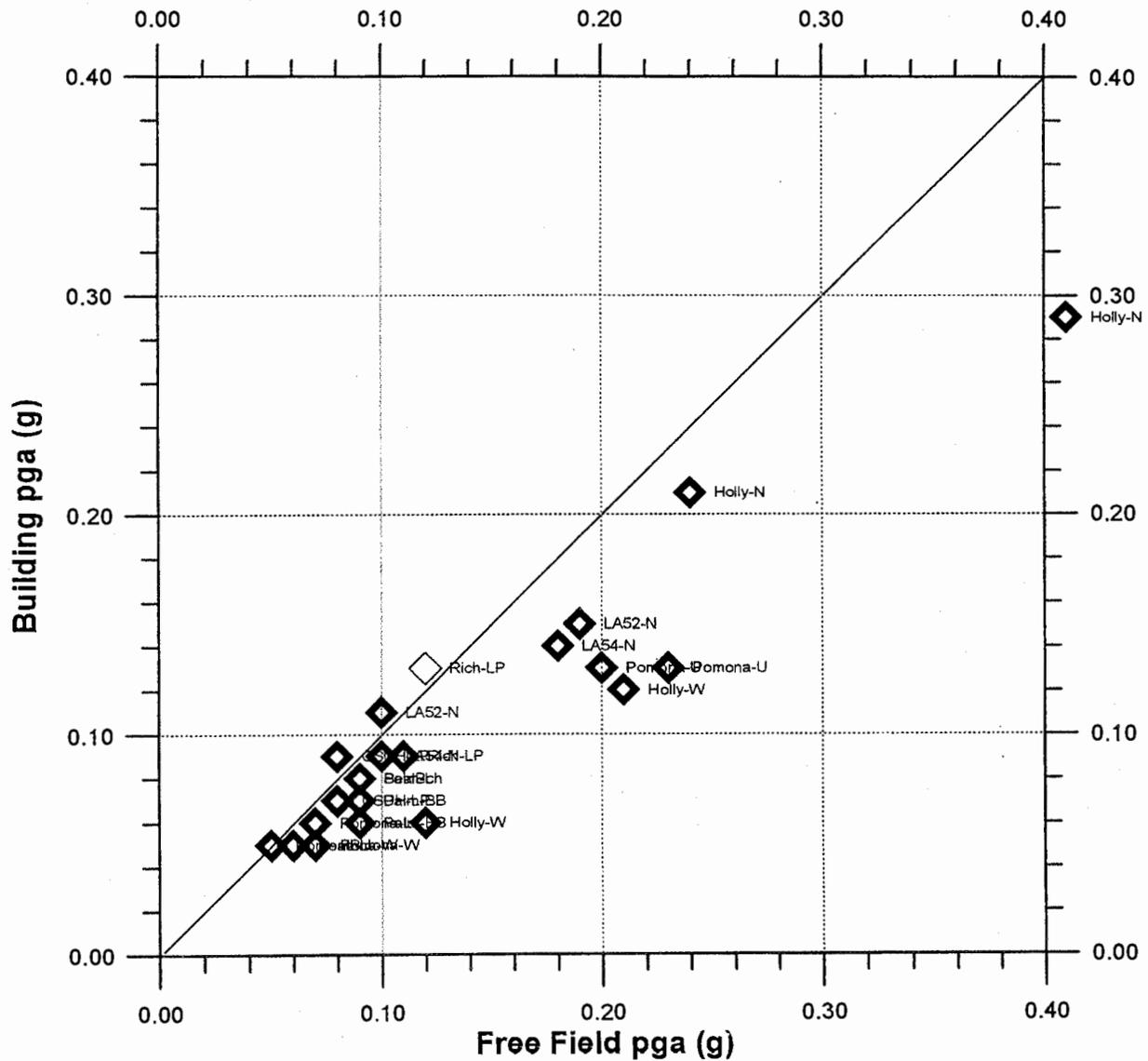
# FIGURES



SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

Free Field pga vs. Basement pga  
 for Buildings with Basements  
 (all records)

Figure  
 3.1



SMIP Directed Research Project

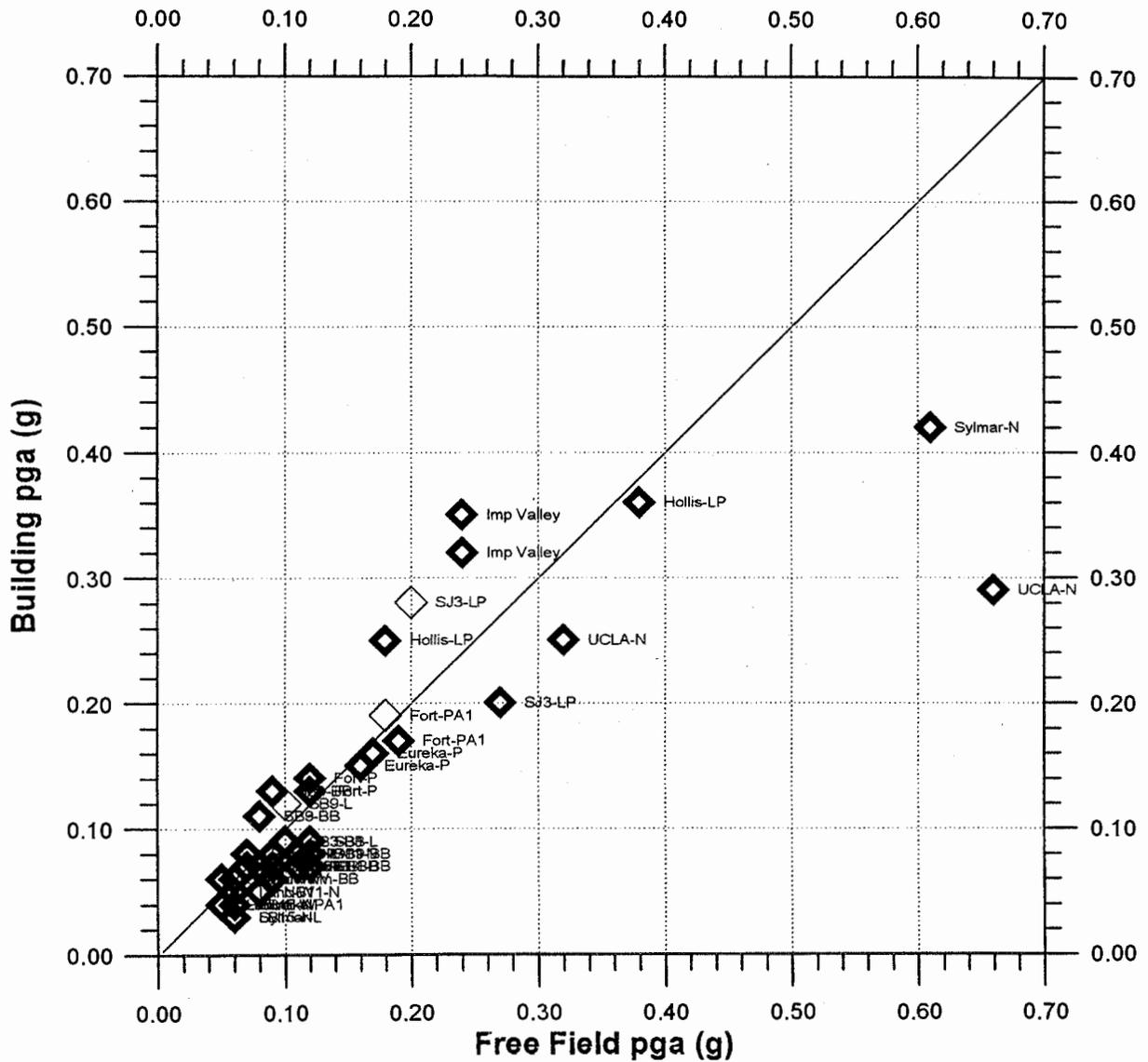
Poland, Mejia, Soulages, Sun

Free Field pga vs. Basement pga  
for Buildings with Basements  
(records with pga > 0.05)

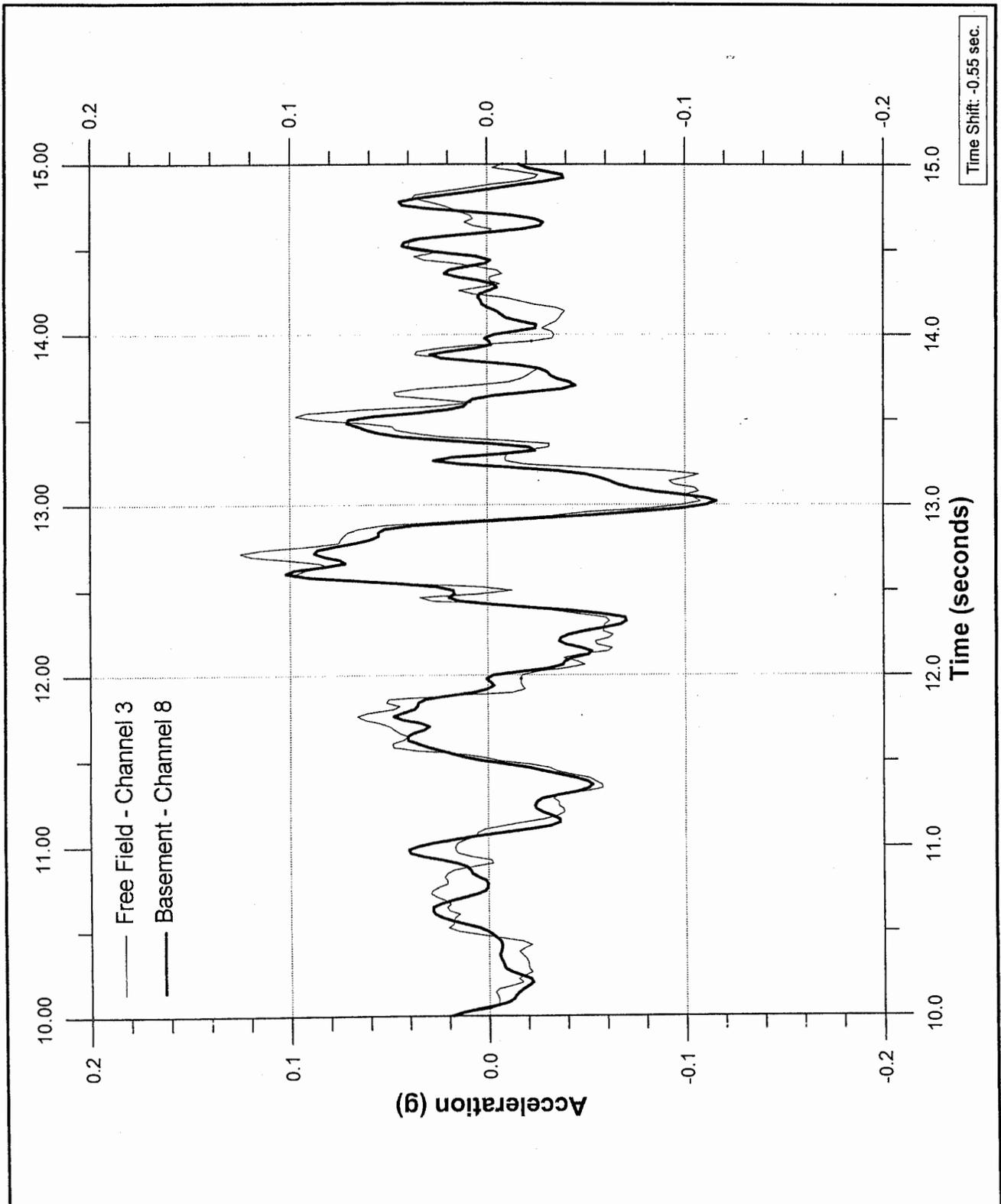
Figure

3.2

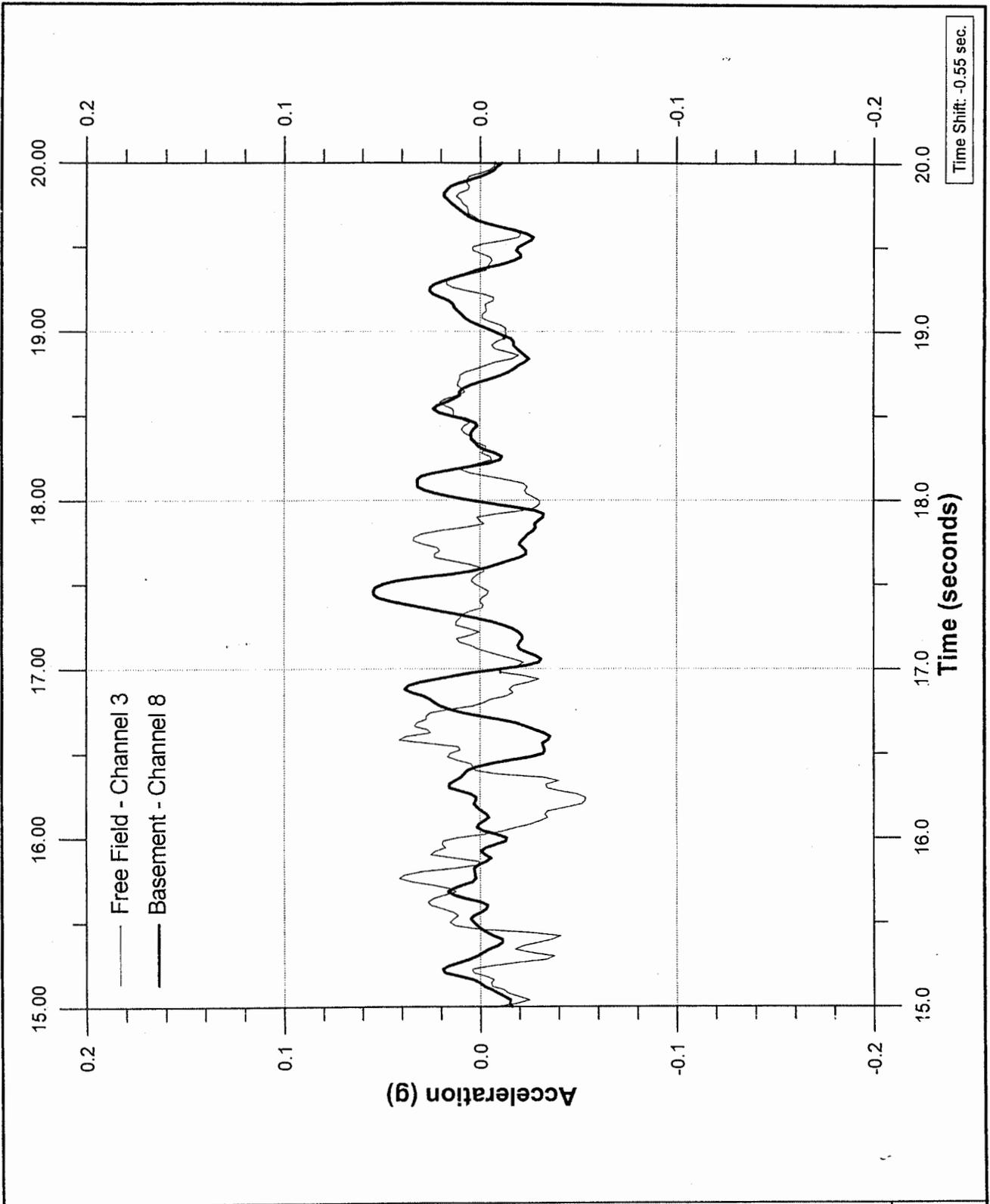




SMIP Directed Research Project	Free Field pga vs. Base pga	Figure
Poland, Mejia, Soulages, Sun	for Buildings without Basements	3.4
	(records w/ pga > 0.05)	



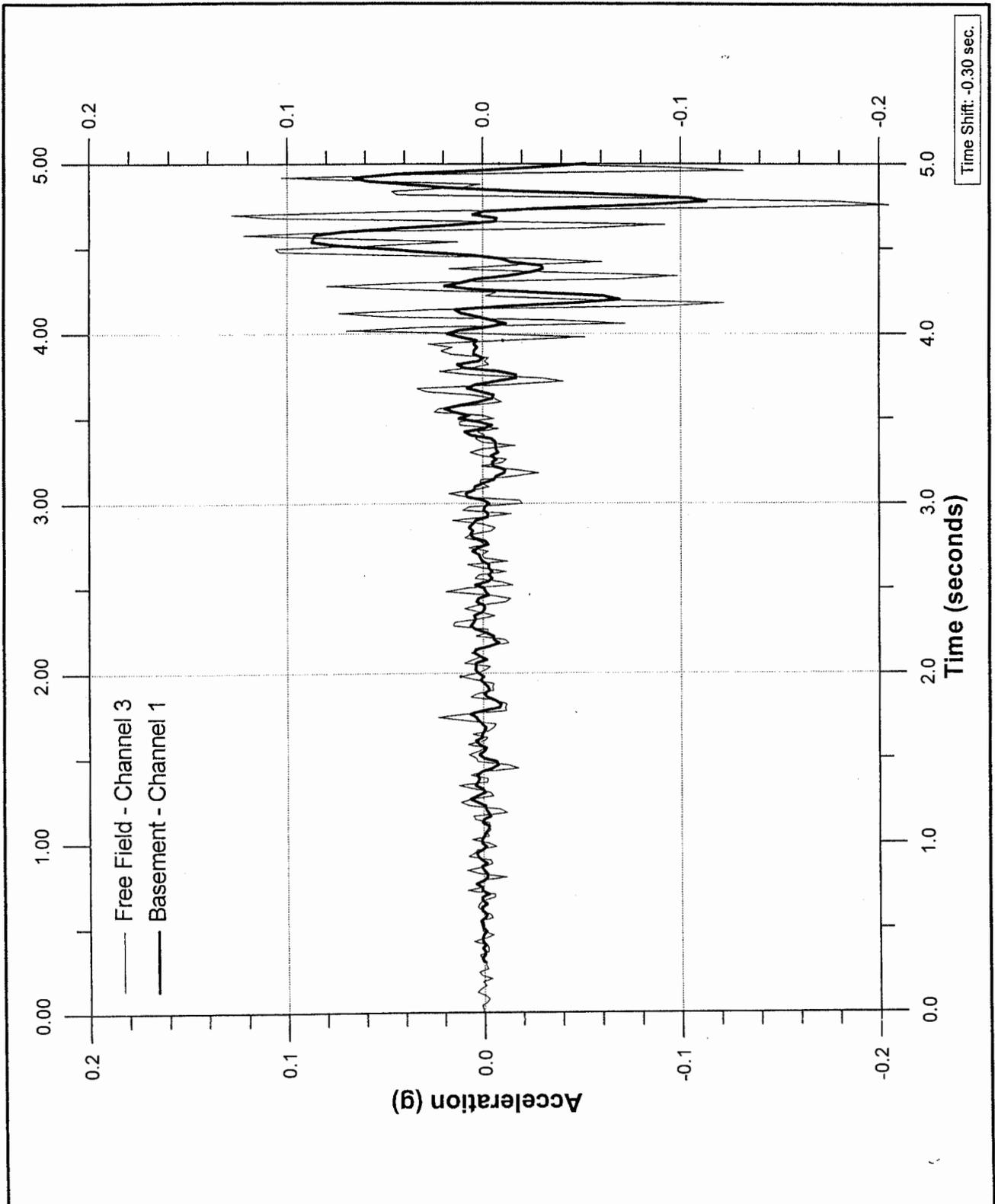
SMIP Directed Research Project	Corrected Time Histories at Basement and FF	Figure
Poland, Mejia, Soulages, Sun	Richmond City Hall Loma Prieta - 10/17/89 - South	3.5a



SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

Corrected Time Histories at Basement and FF  
 Richmond City Hall  
 Loma Prieta - 10/17/89 - South

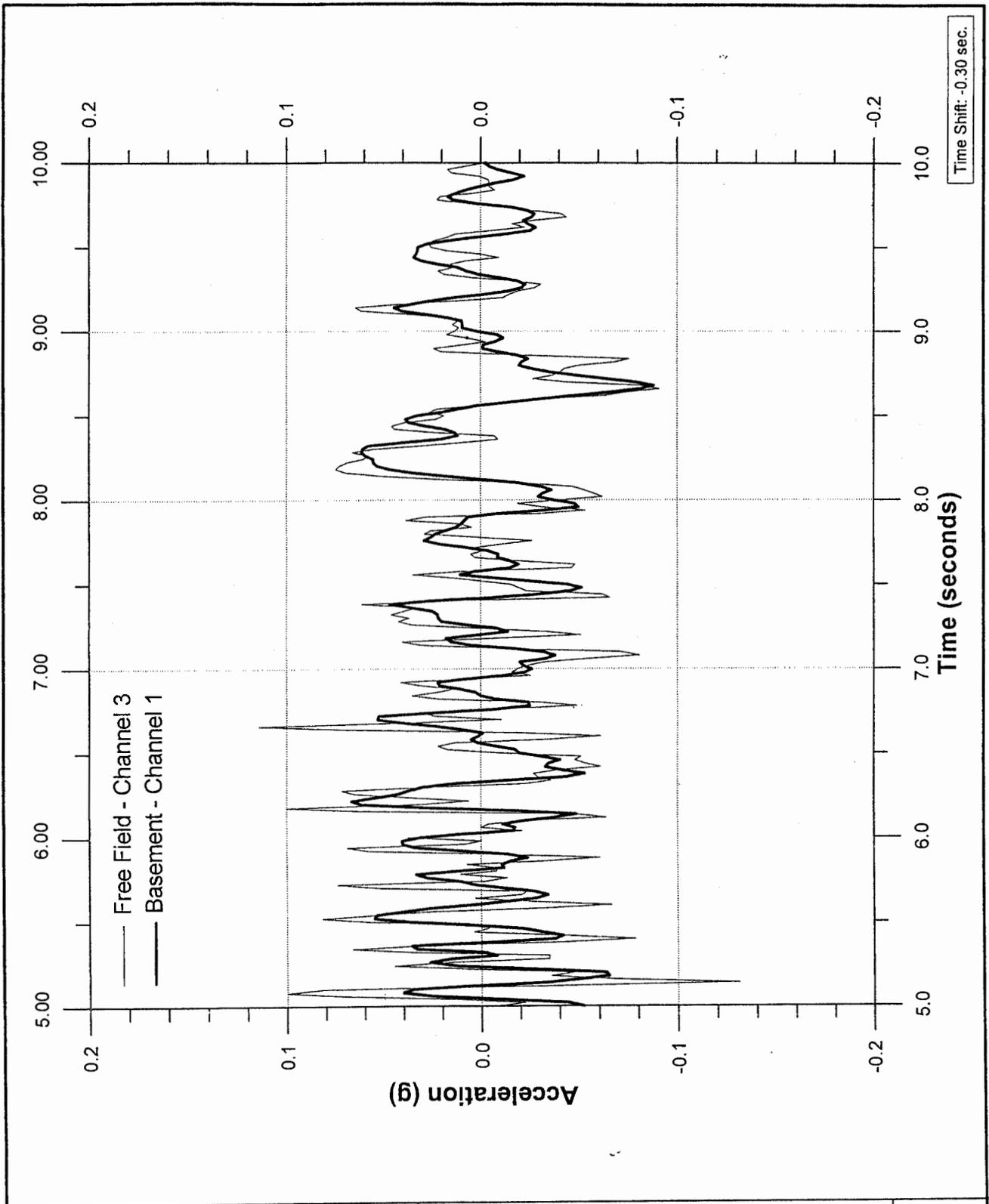
Figure  
**3.5b**



SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

Corrected Time Histories at Basement and FF  
 Hollywood Storage Building  
 Whittier - 10/1/87 - North

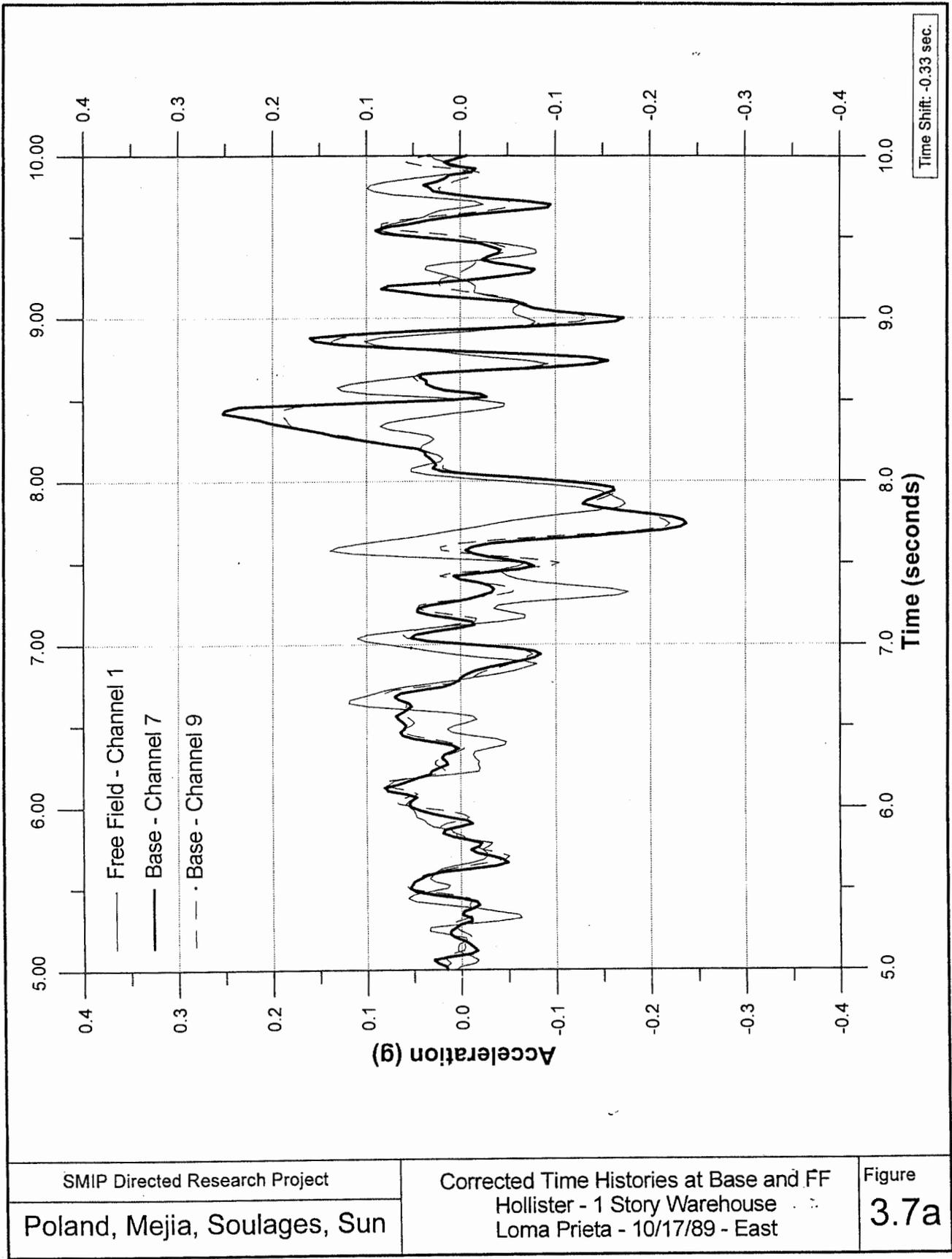
Figure  
**3.6a**



SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

Corrected Time Histories at Basement and FF  
 Hollywood Storage Building  
 Whittier - 10/1/87 - North

Figure  
**3.6b**



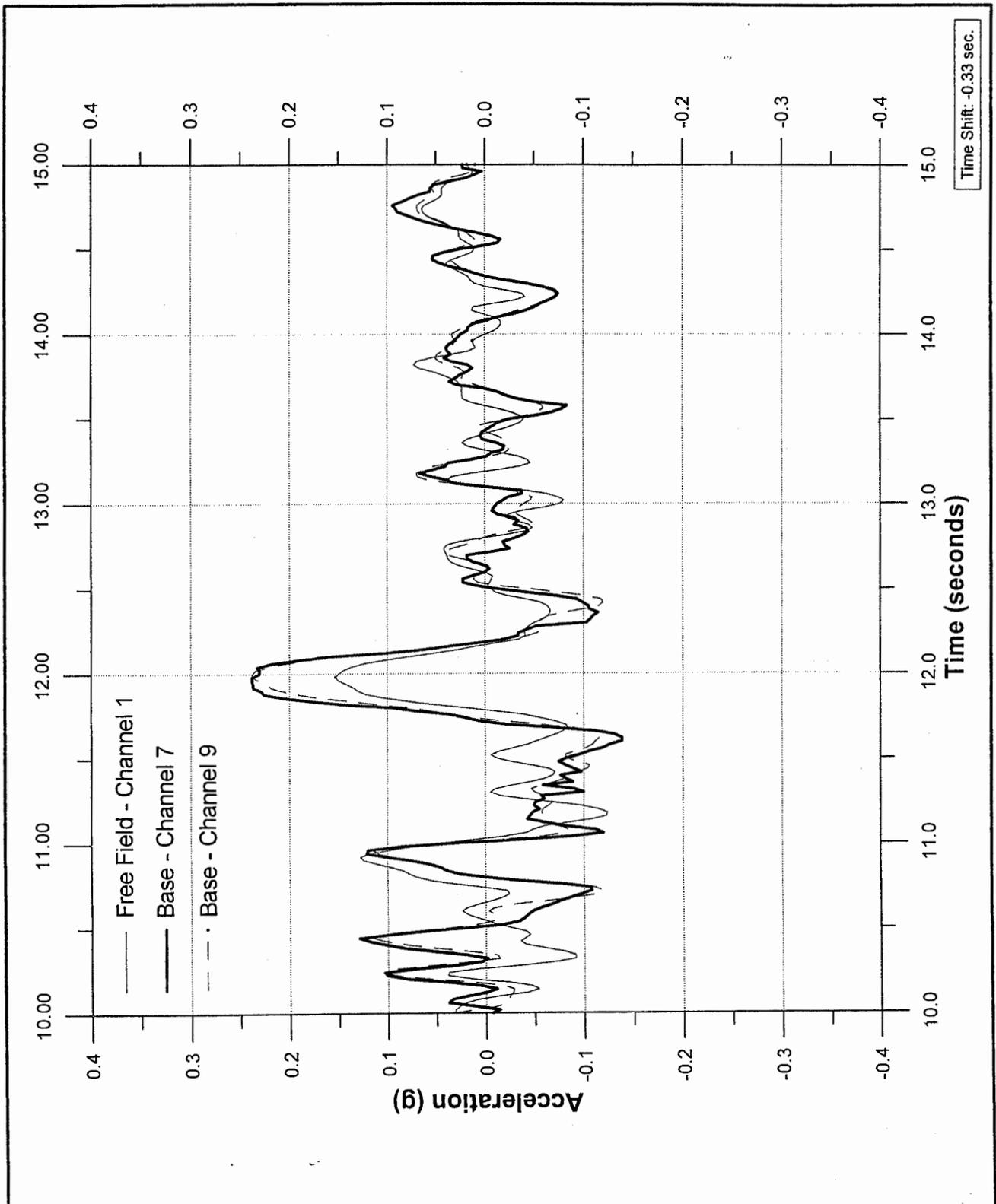
SMIP Directed Research Project

Poland, Mejia, Soulages, Sun

Corrected Time Histories at Base and FF  
 Hollister - 1 Story Warehouse  
 Loma Prieta - 10/17/89 - East

Figure

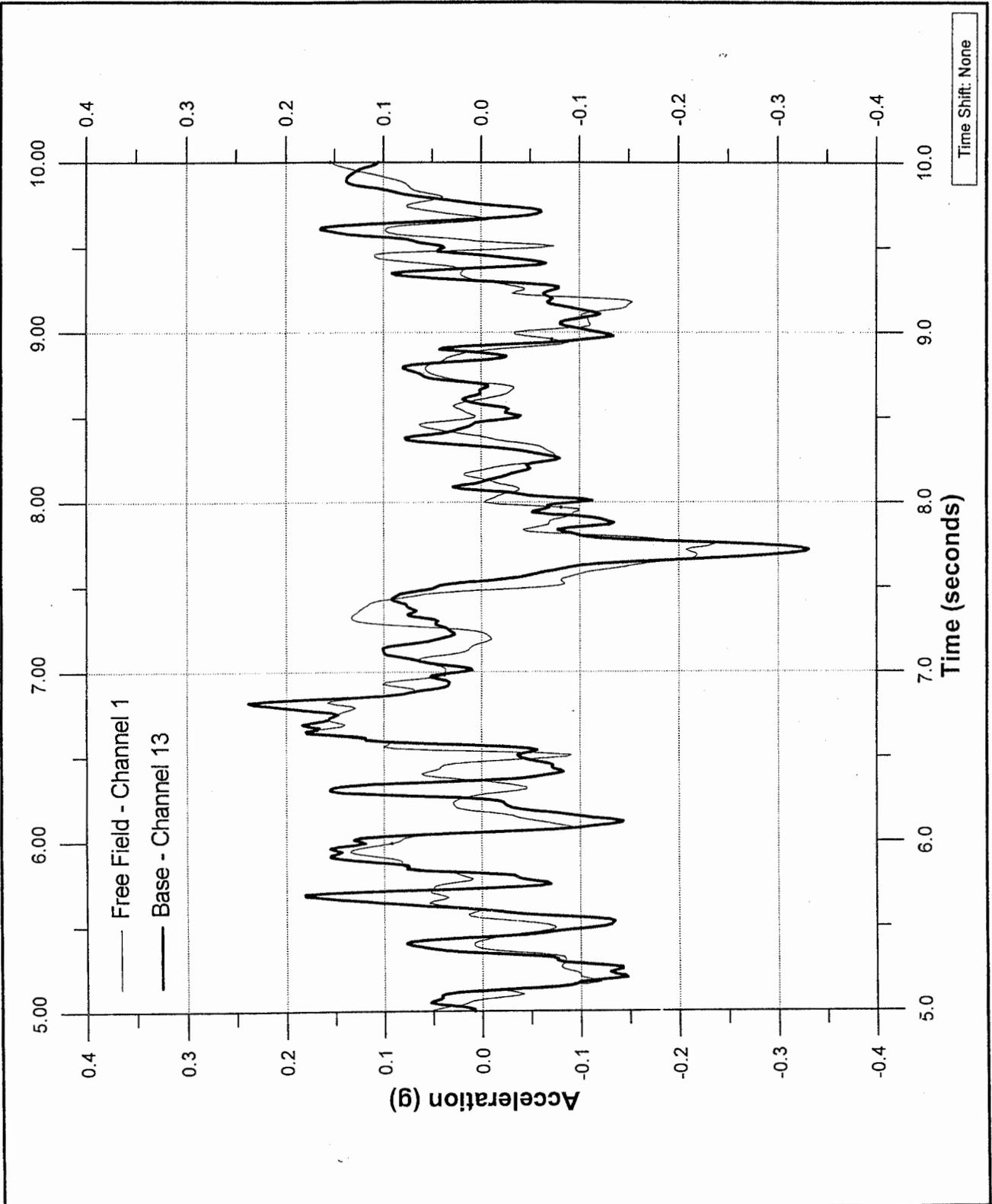
3.7a



SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

Corrected Time Histories at Base and FF  
 Hollister - 1 Story Warehouse  
 Loma Prieta - 10/17/89 - East

Figure  
**3.7b**

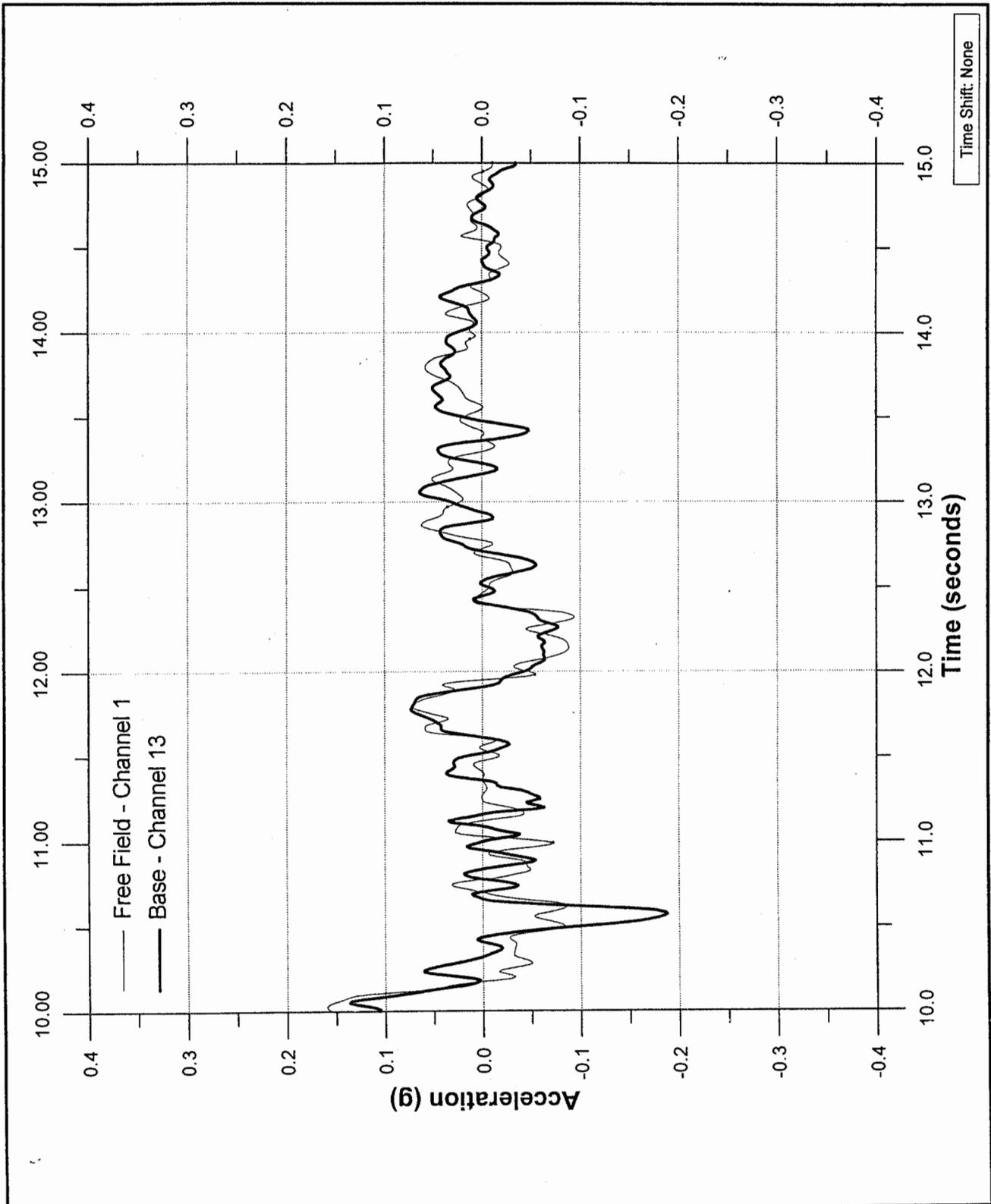


Time Shift: None

SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

Corrected Time Histories at Base and FF  
 Imperial County Services Building  
 Imperial Valley - 10/15/79 - East

Figure  
**3.8a**

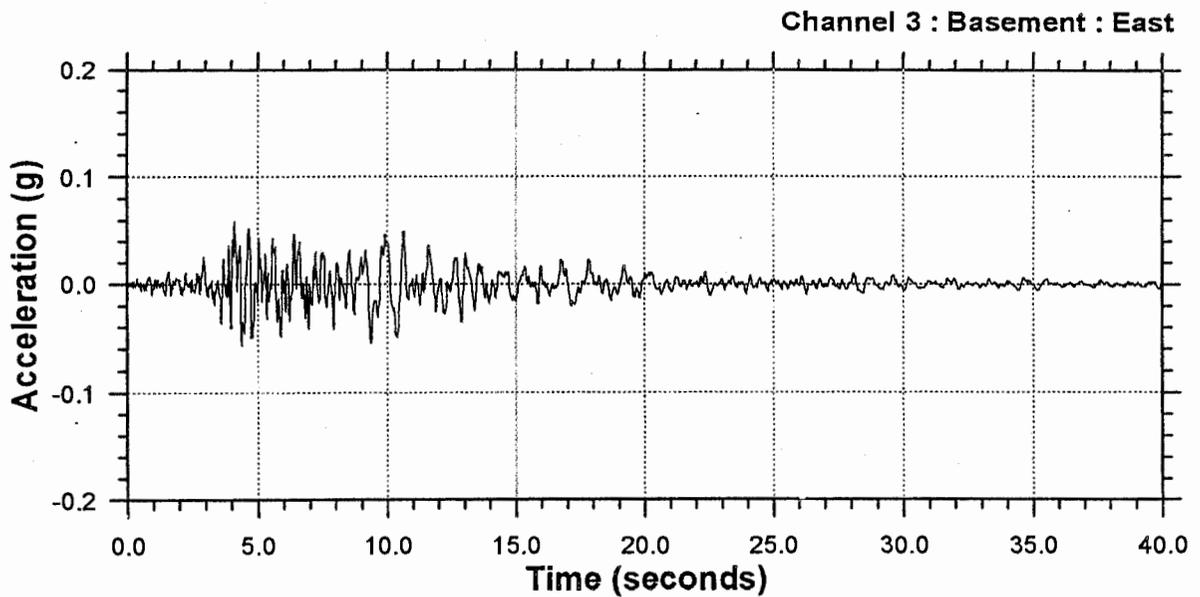
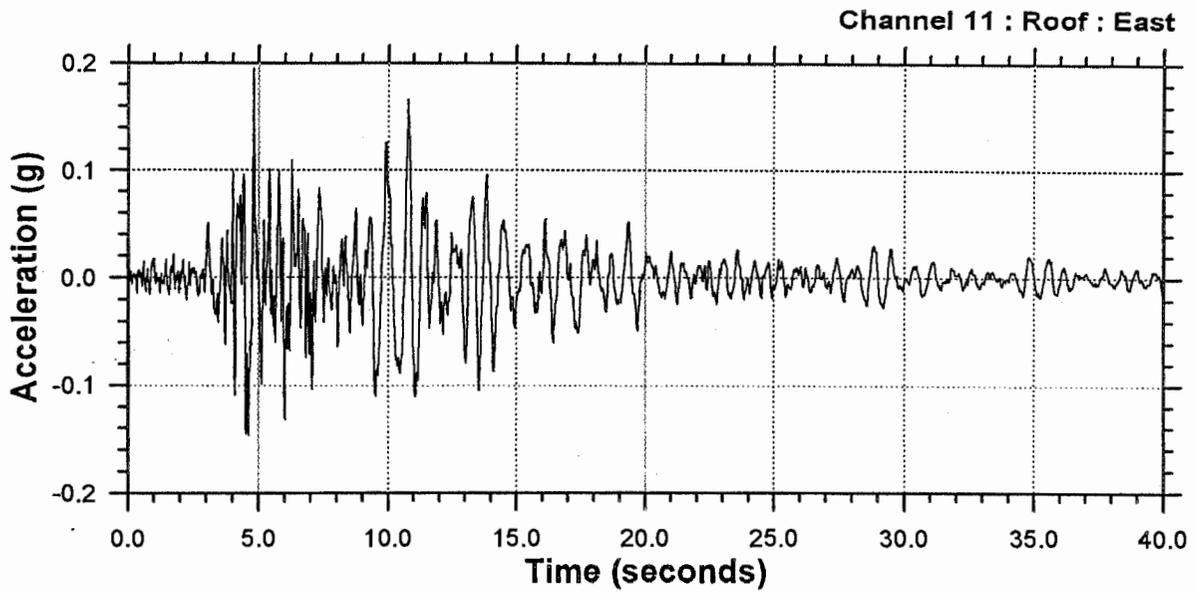


Time Shift: None

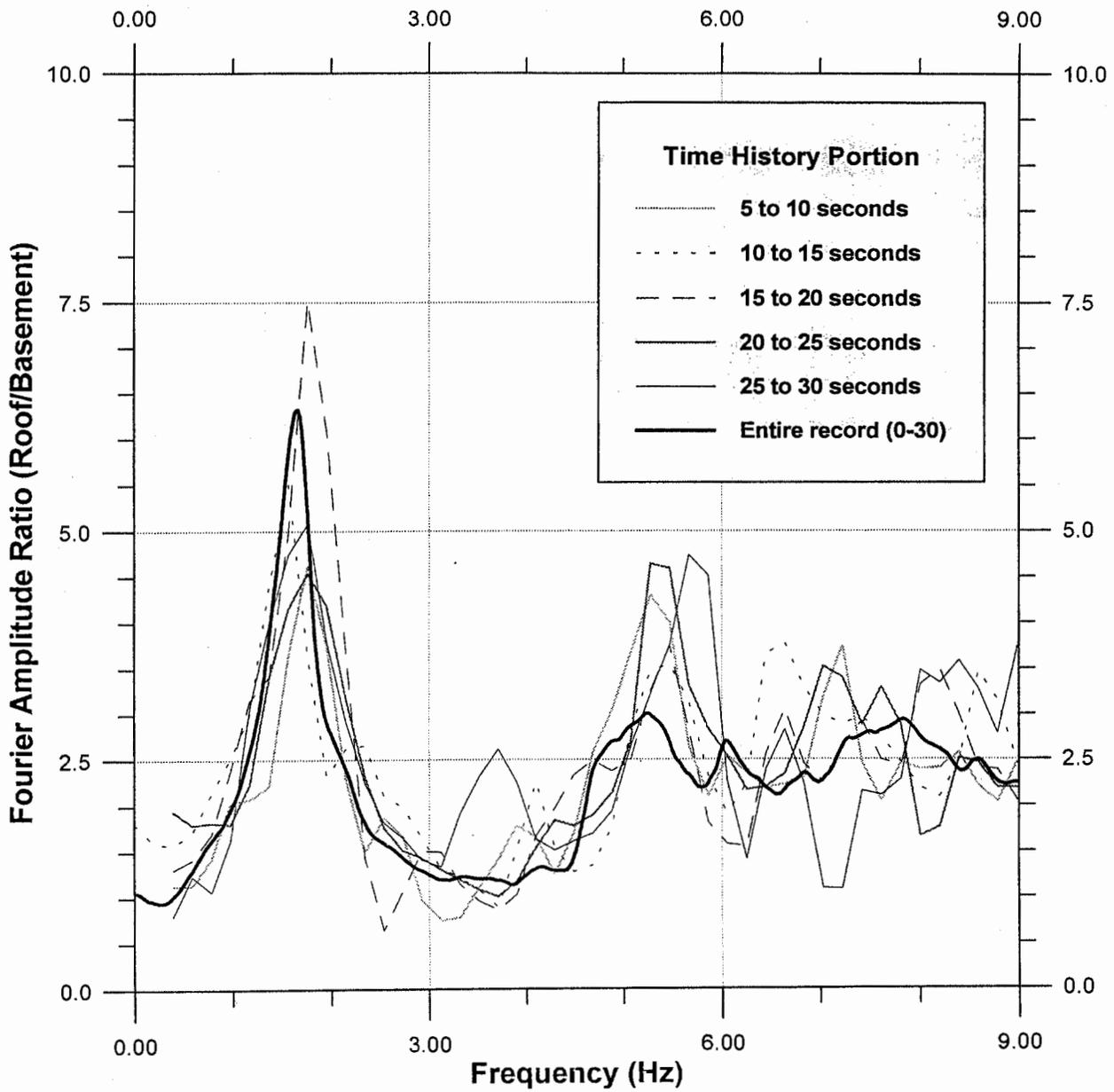
SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

Corrected Time Histories at Base and FF  
 Imperial County Services Building  
 Imperial Valley - 10/15/79 - East

Figure  
**3.8b**



SMIP Directed Research Project	Corrected Time Histories - East-West Direction Hollywood Storage Building Whittier - 10/1/87	Figure
Poland, Mejia, Soulages, Sun		<b>4.1</b>

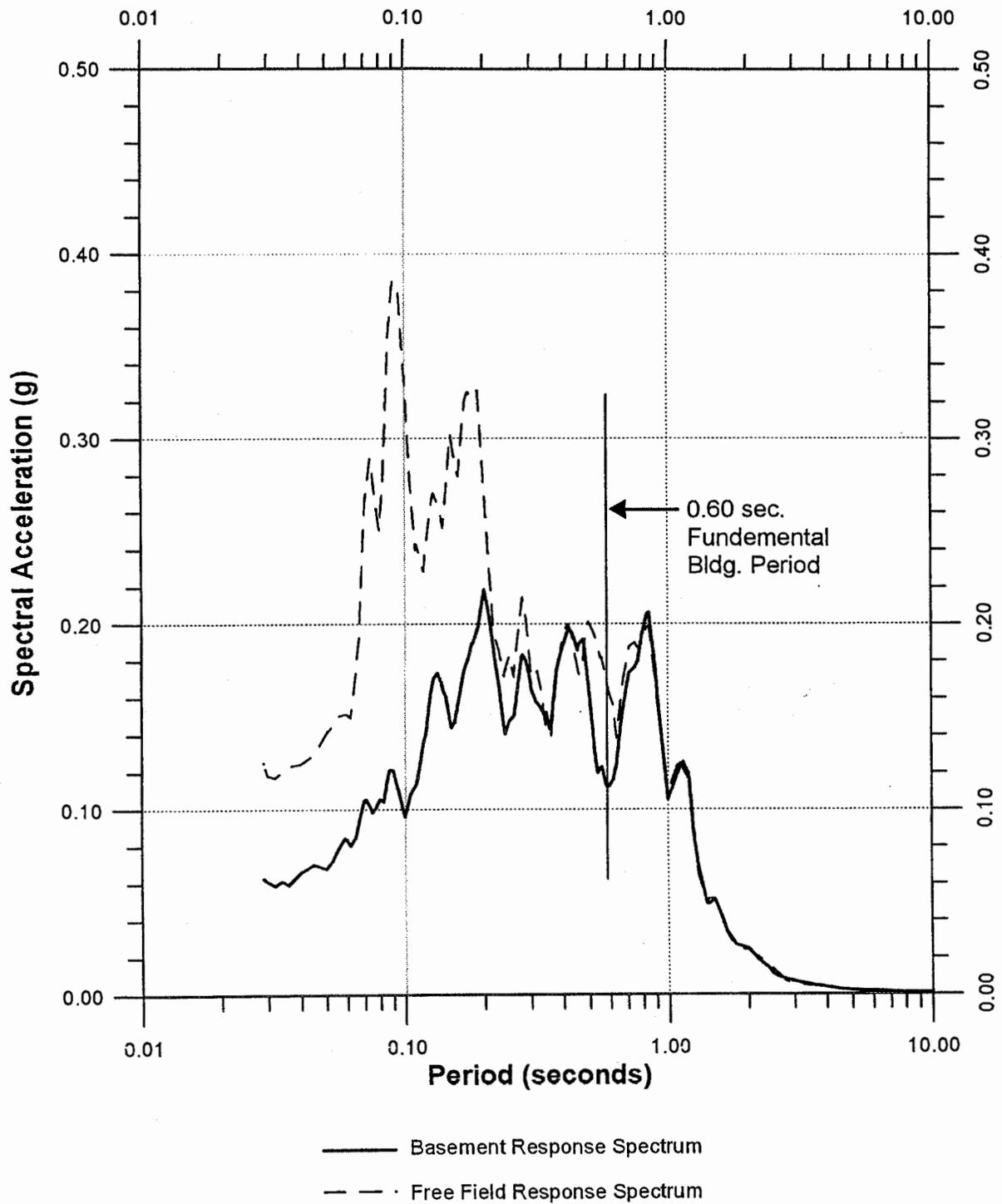


SMIP Directed Research Project

Poland, Mejia, Soulages, Sun

Fourier Amplitude Ratio  
 Hollywood Storage Building  
 Whittier - 10/1/87 - East

Figure  
**4.2**

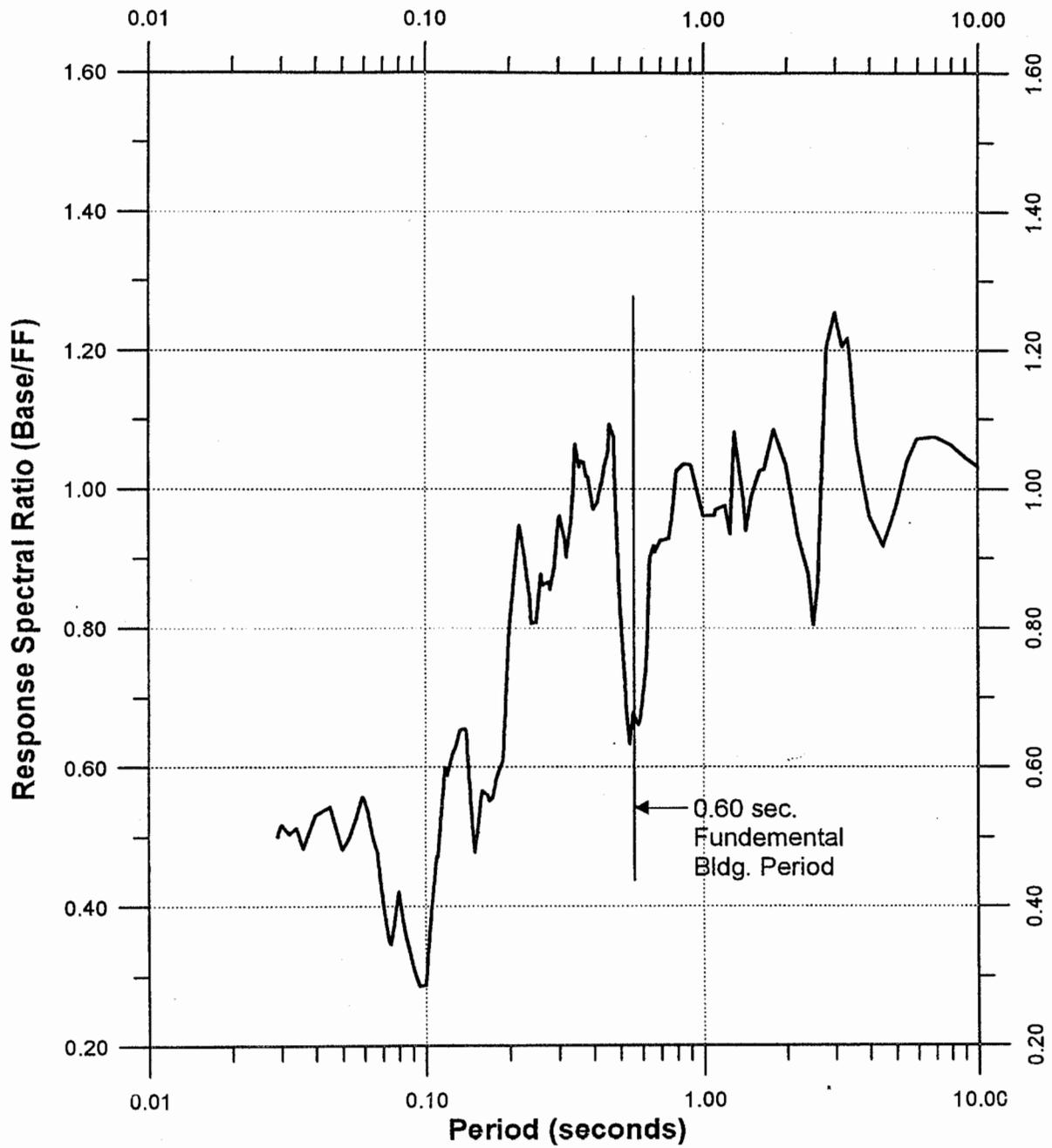


SMIP Directed Research Project

Poland, Mejia, Soulages, Sun

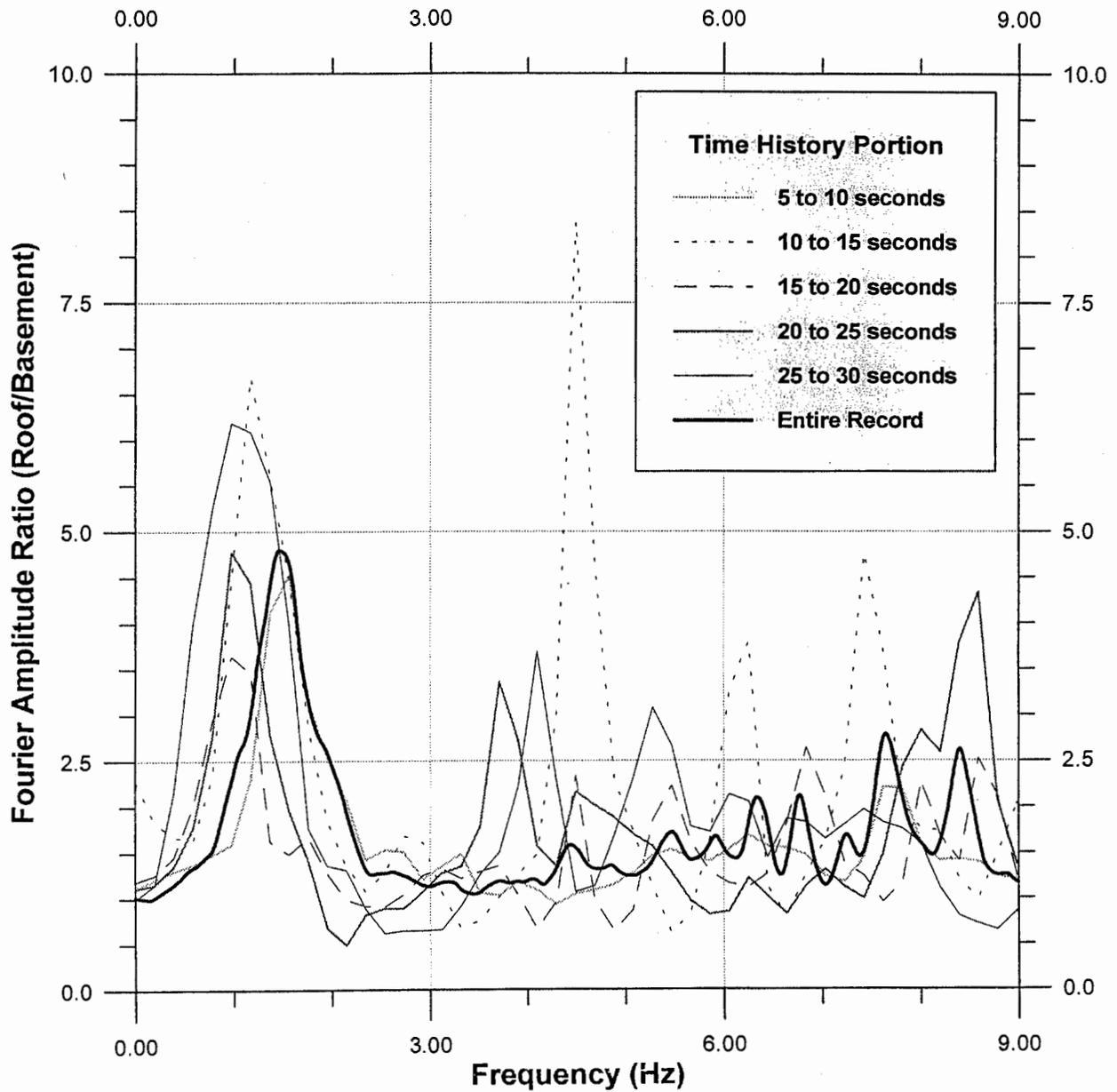
Response Spectrums  
 Hollywood Storage Building  
 Whittier - 10/1/87 - East

Figure  
**4.3**



— Hollywood Storage - Whittier - East

SMIP Directed Research Project	SSI Response Reduction Hollywood Storage Building Whittier - 10/1/87 - East	Figure <b>4.4</b>
Poland, Mejia, Soulages, Sun		



SMIP Directed Research Project

Poland, Mejia, Soulages, Sun

Fourier Amplitude Ratio  
 Imperial County Services Building  
 Imperial Valley - 10/15/79 - North

Figure

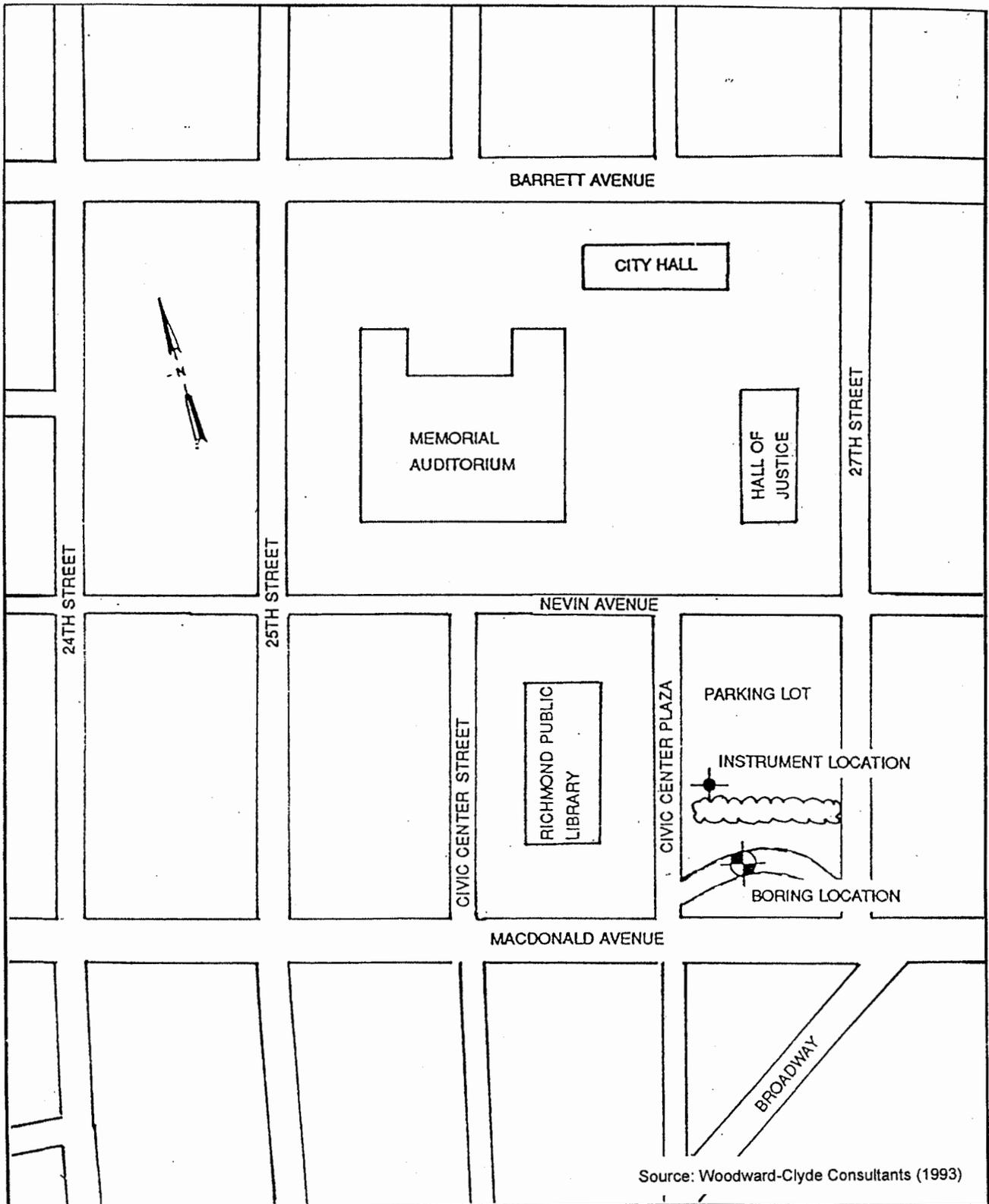
4.5



SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

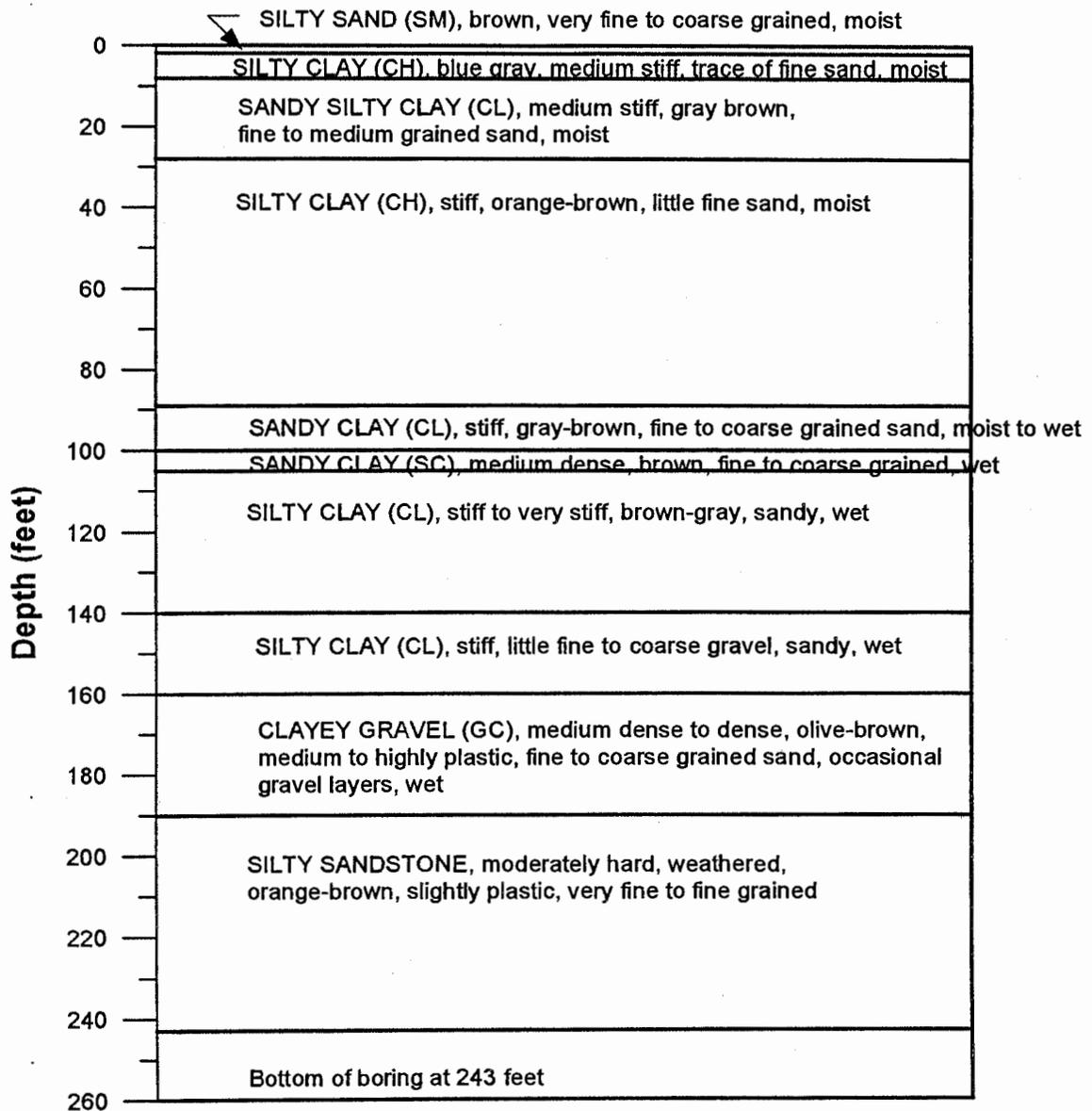
Site Plan  
 Richmond City Hall Parking Lot

Figure  
 5.1



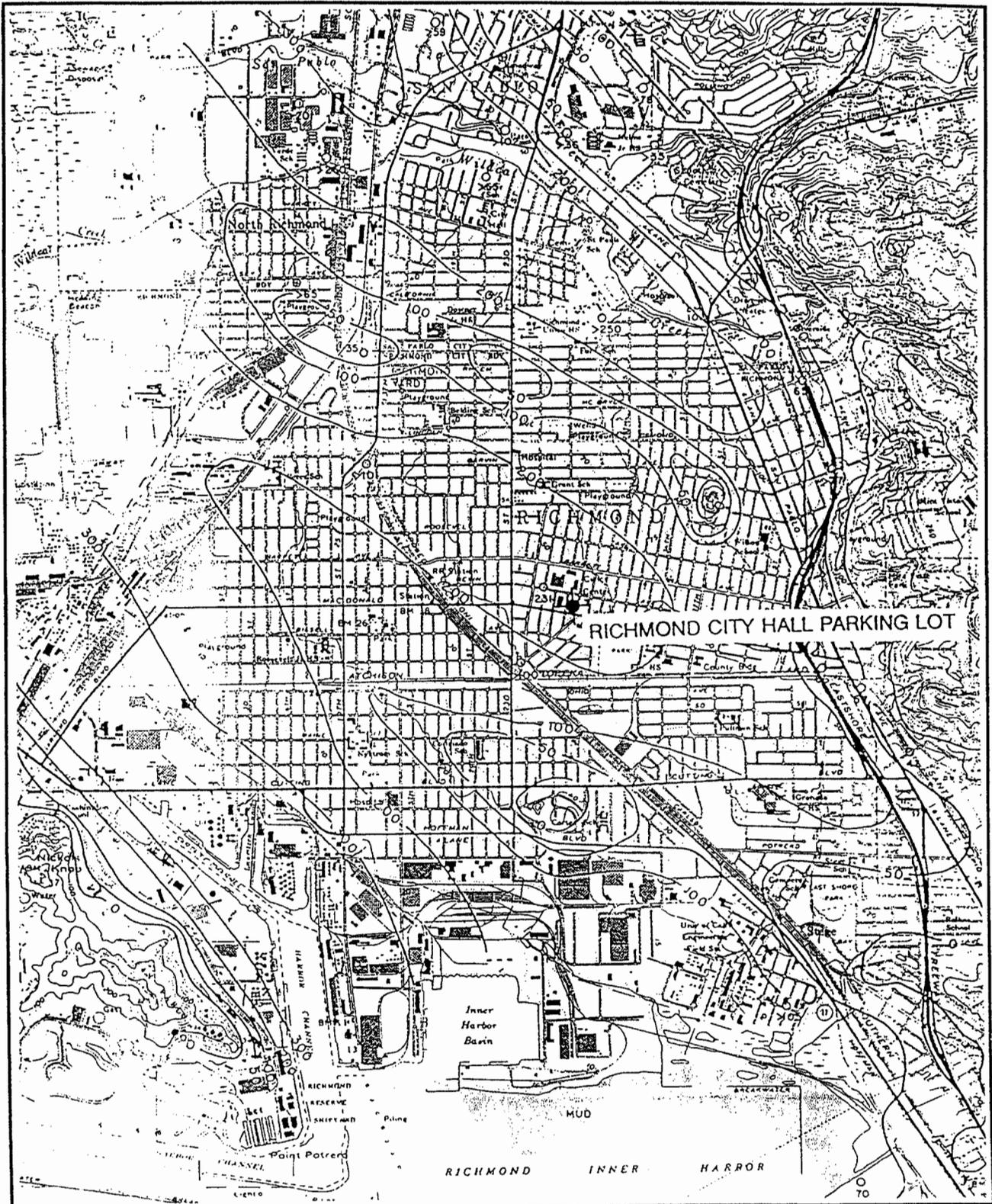
Source: Woodward-Clyde Consultants (1993)

SMIP Directed Research Project	<b>Boring Location Plan</b> Richmond City Hall Parking Lot	Figure
Poland, Mejia, Soulages, Sun		<b>5.2</b>



Source: Woodward-Clyde Consultants (1993)

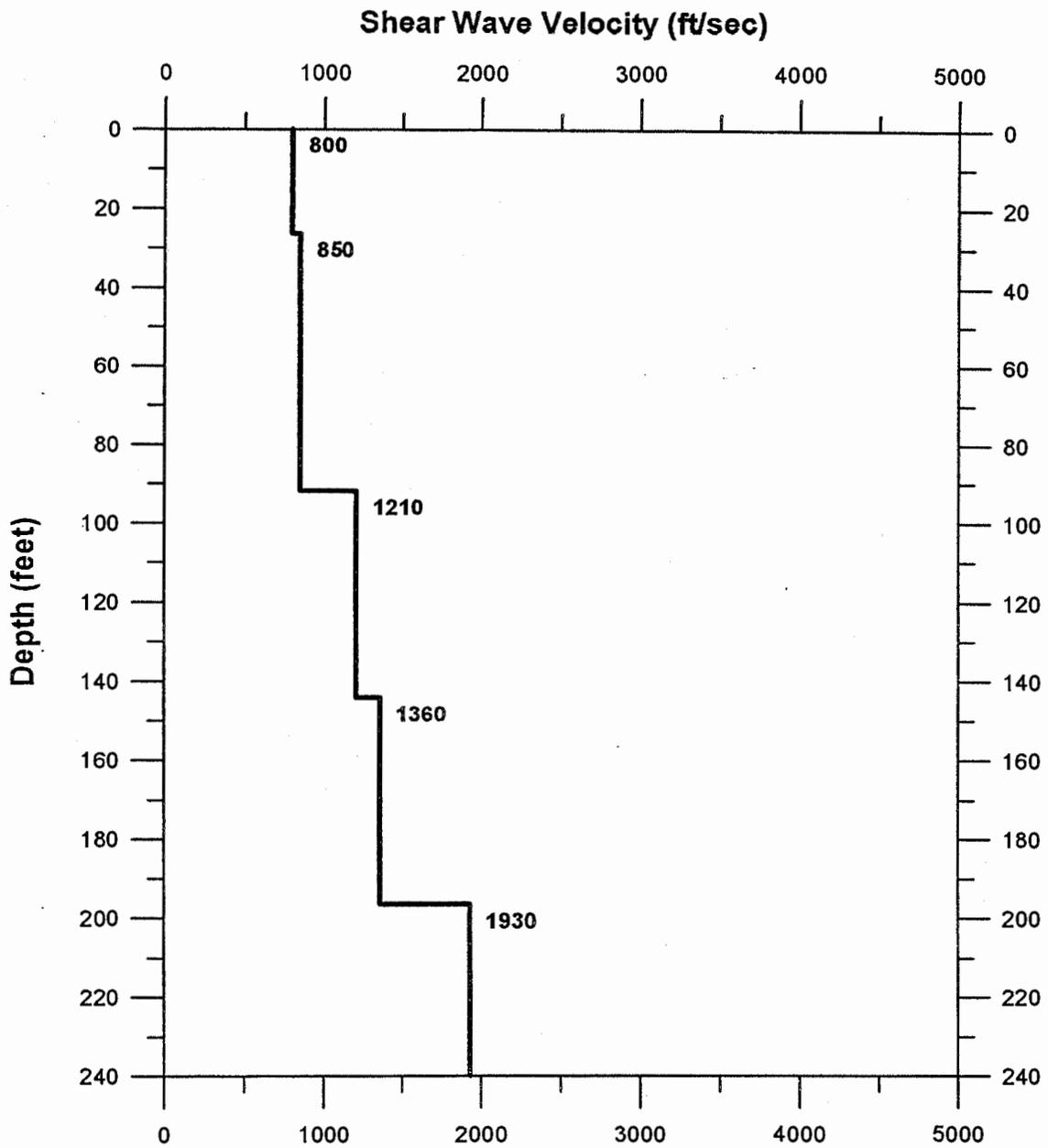
SMIP Directed Research Project	Boring Log Richmond City Hall Parking Lot	Figure
Poland, Mejia, Soulages, Sun		5.3



SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

Depth to Bedrock Contour Map  
 Richmond, San Pablo, El Cerrito Area

Figure  
**5.4**



Source: Agbabian Associates (1993)

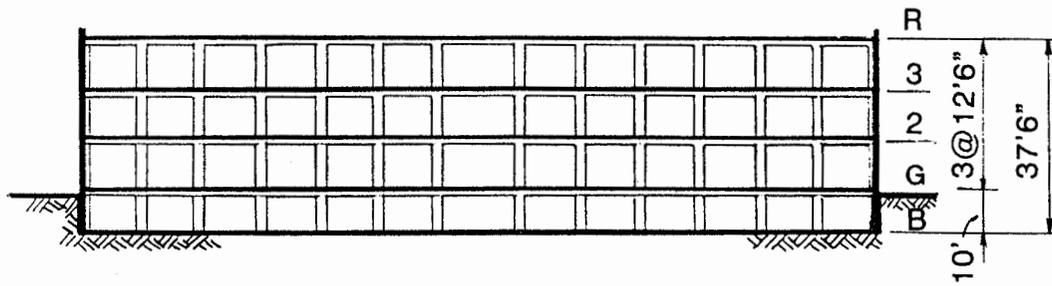
SMIP Directed Research Project

Poland, Mejia, Soulages, Sun

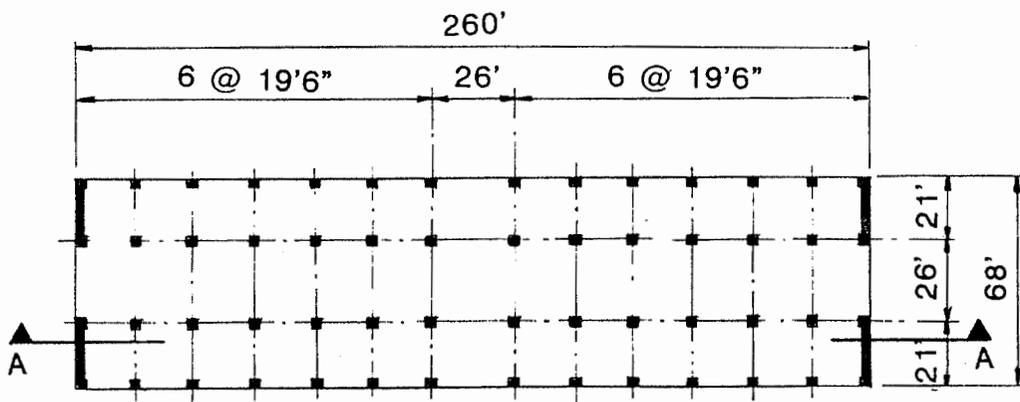
Shear Wave Velocity Profile  
Richmond City Hall Parking Lot

Figure

5.5



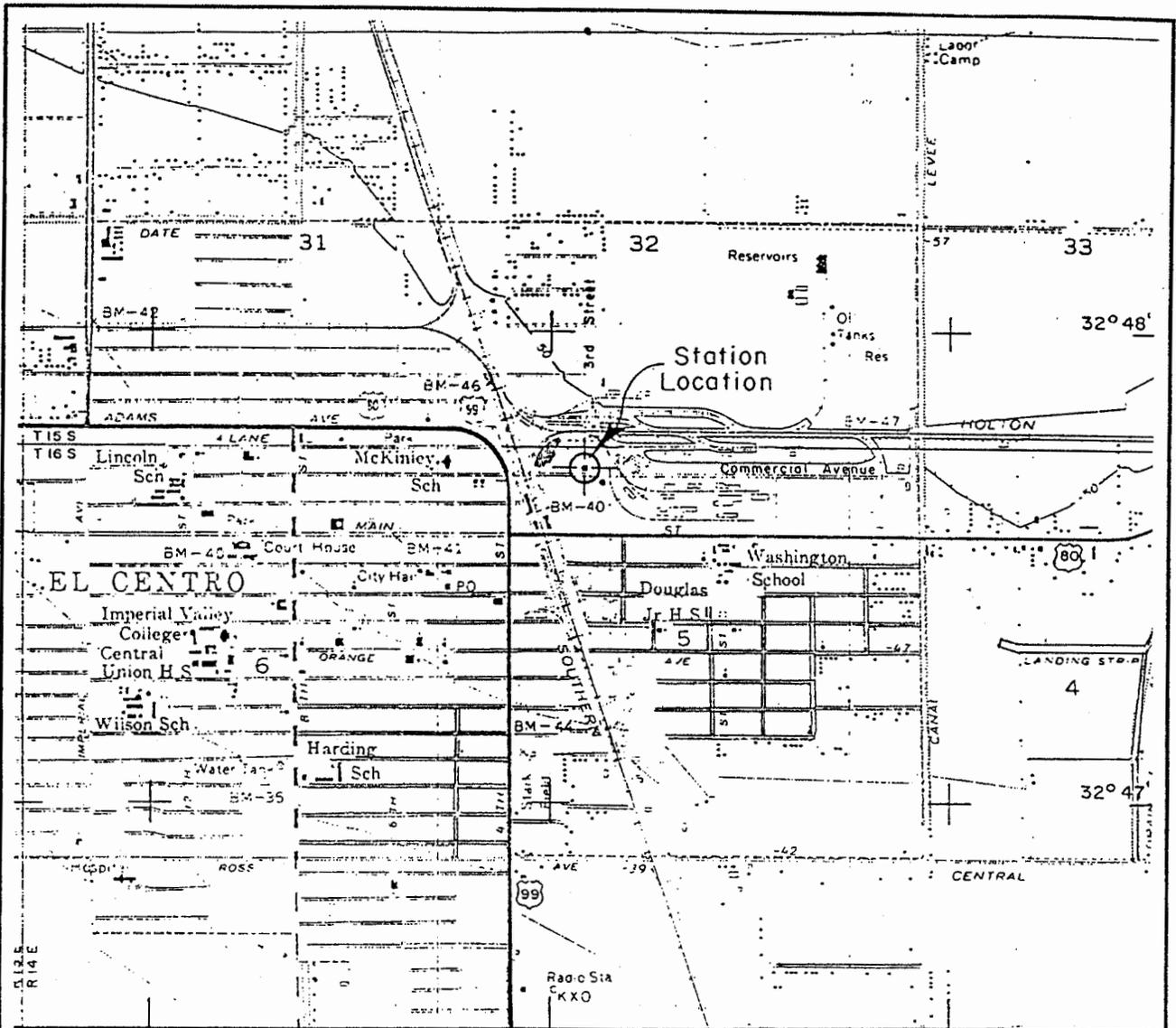
SECTION A-A



TYPICAL FLOOR PLAN



SMIP Directed Research Project	Structural Configuration Richmond City Hall	Figure
Poland, Mejia, Soulages, Sun		5.6



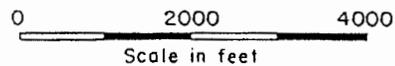
El Centro - Terminal Substation

USGS Topographic  
 Quadrangle: El Centro, California.

Coordinates: 32° 47' 42" N  
 115° 32' 55" W

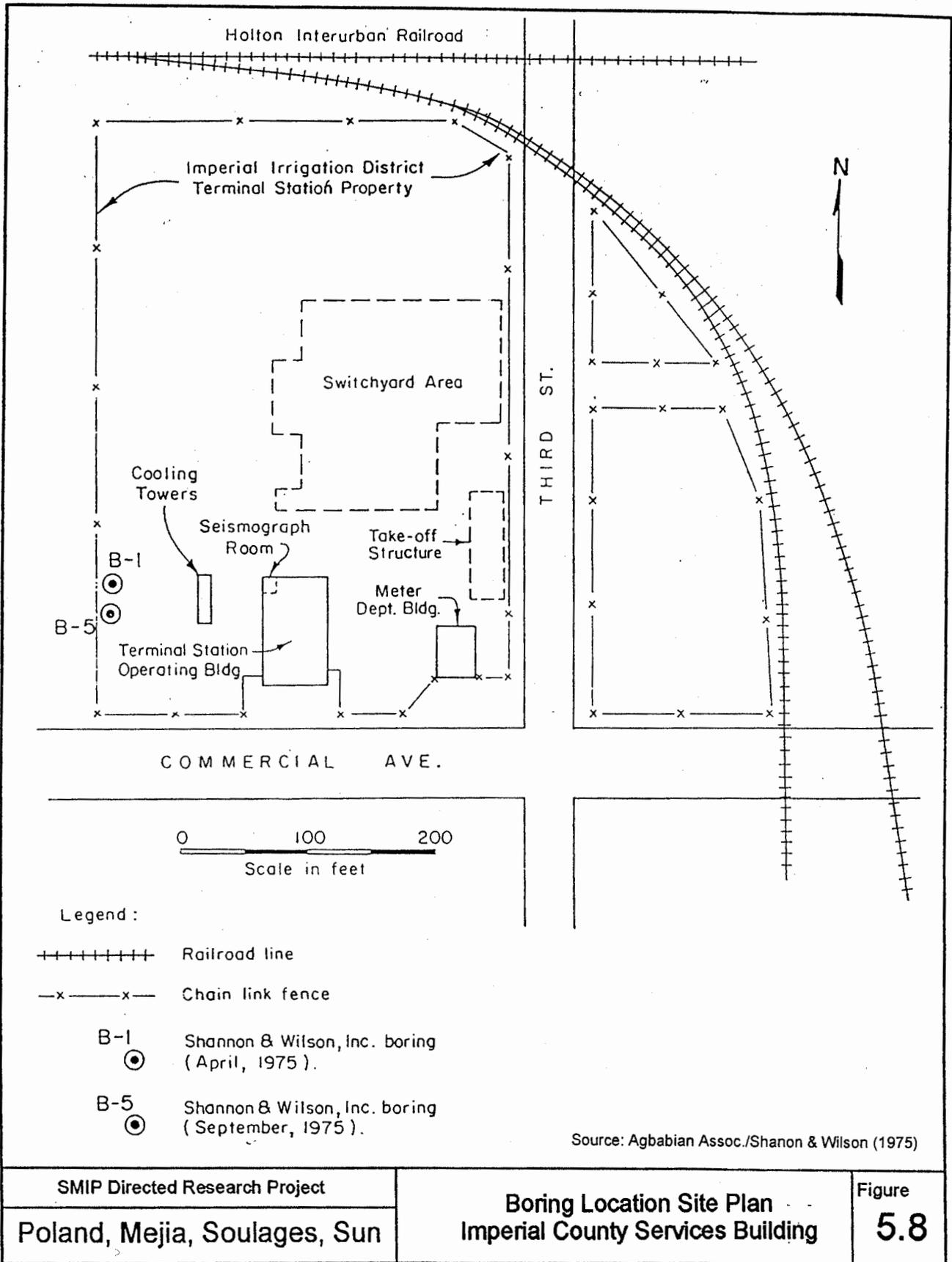
Location: N.W. corner of intersection of  
 Third St. and Commercial Ave.

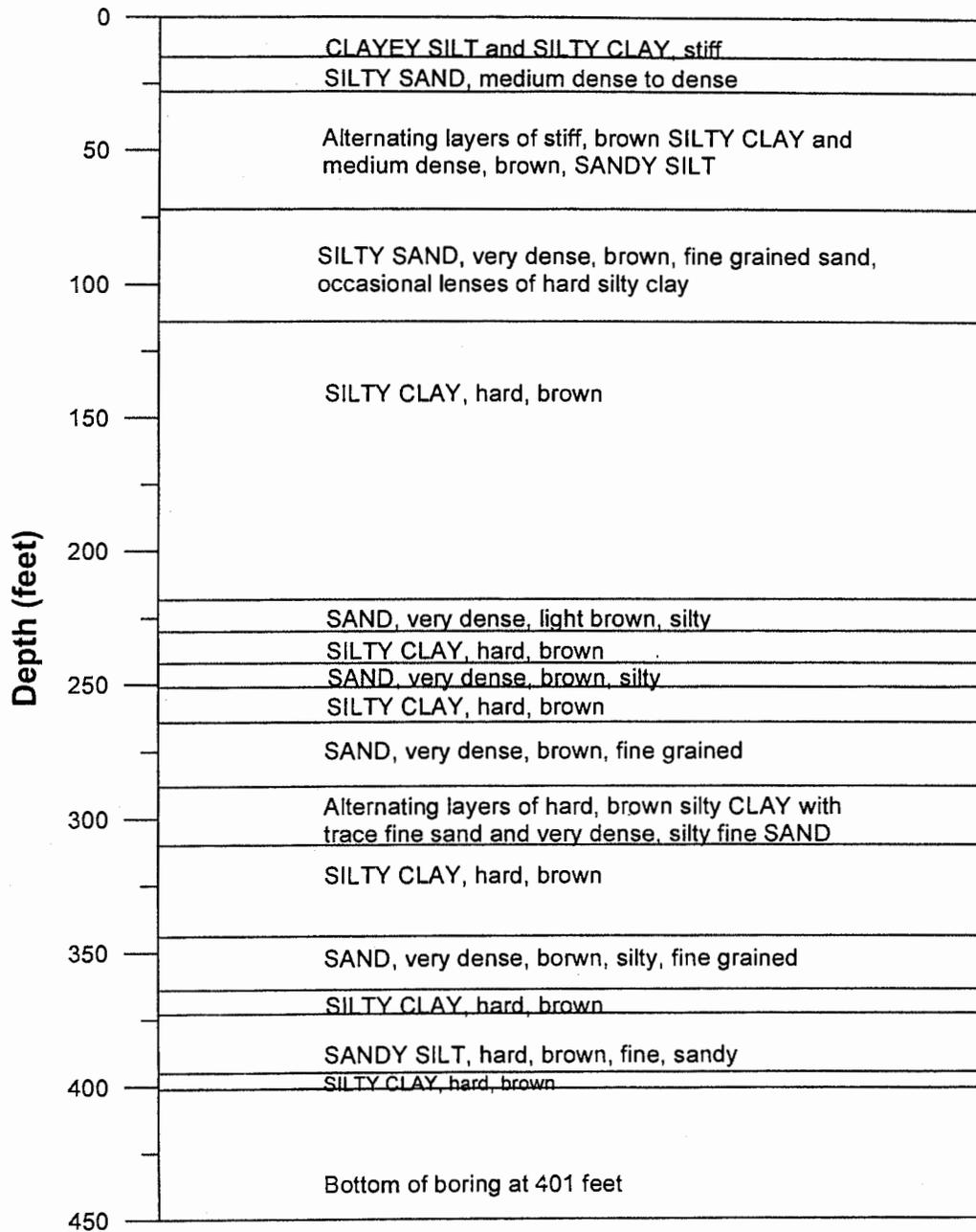
Structure: Two story, heavy, reinforced concrete  
 building.



Source: Agbajian Assoc./Shanon & Wilson (1975)

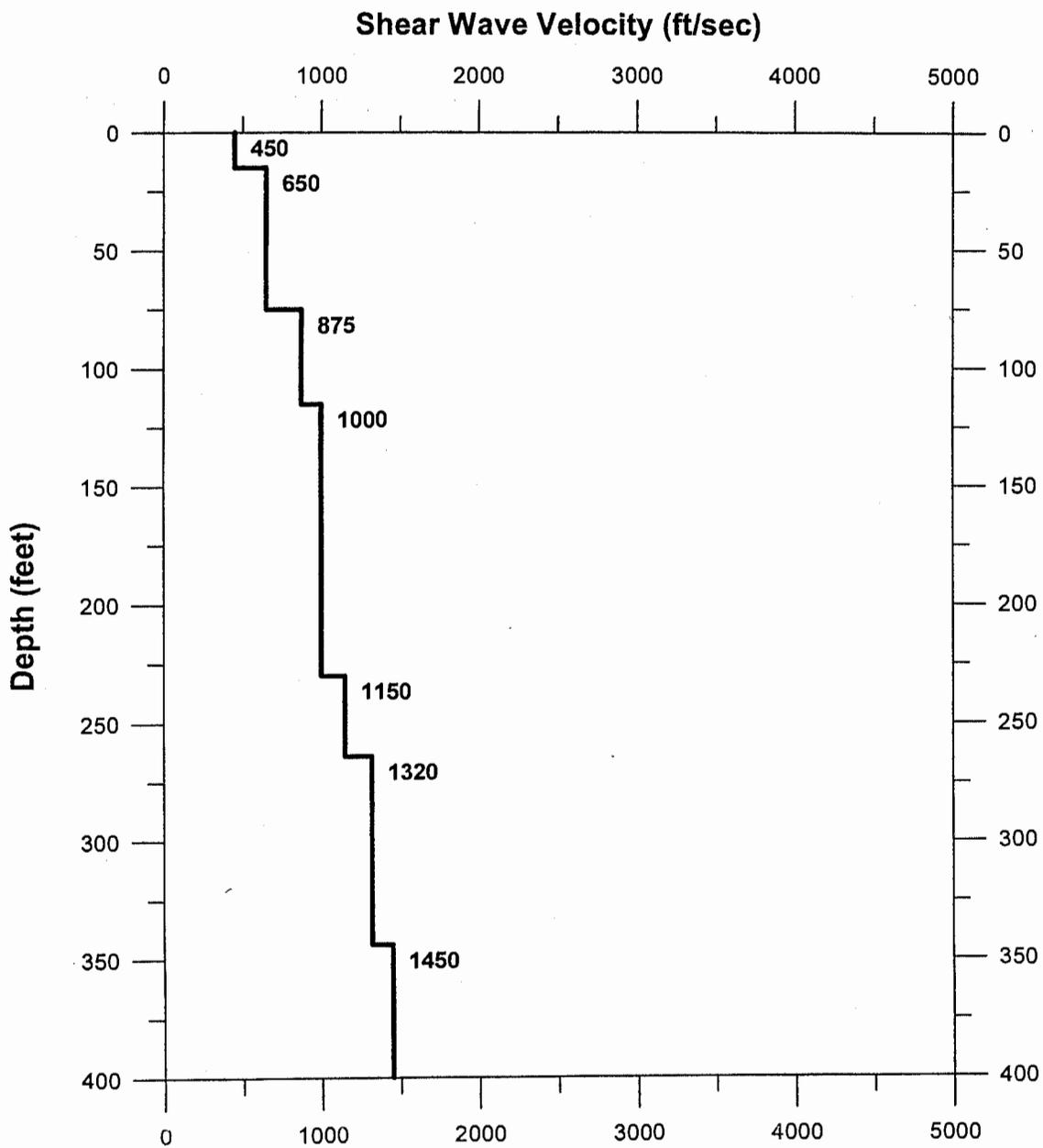
SMIP Directed Research Project	<b>Station Location Plan</b> <b>Imperial County Services Building</b>	Figure
Poland, Mejia, Soulages, Sun		<b>5.7</b>





Source: USGS Open-File Report 84-562  
 Agbajian Assoc./Shanon & Wilson (1975)

SMIP Directed Research Project	Boring Log Imperial County Services Building	Figure
Poland, Mejia, Soulages, Sun		5.9



Source: USGS Open File Report 84-562  
 Agbabian Assoc./Shanon & Wilson (1975)

SMIP Directed Research Project

Poland, Mejia, Soulages, Sun

Shear Wave Velocity Profile  
 Imperial County Services Building

Figure  
**5.10**

# B-1 BORING LOG<sup>1</sup>

Depth, (ft.)	USC	Unconfined Shear Strength (tsf)	TEST <sup>3</sup> Dry Density, (pcf)
0			
Approx. surface Elev. -47 ft. MSL <sup>2</sup>			
16	CH	.39	96
Stiff, brown, silty CLAY with occ. lenses of silty, fine sand.			
72	CH	1.81	94 98 96
Alternating layers of very stiff, brown, silty CLAY and Medium, brown, silty fine SAND and sandy SILT (micaceous).			
114	CL-ML		94
	CH	.51	94
	CL-ML	.60	93
			94
148	SM		85
Very dense, brown, silty fine SAND with occ. lenses of hard, silty clay.			
221	SM		94
	SM		
	CH	2.10	98
Hard, brown, silty CLAY with occ. lenses sandy, clayey silt. (Very dense, brown, silty, fine SAND encountered 138- 148 ft. and below 215 ft.)			
	SM		97
	CL	.71	95
	CH	1.44	101
	CH		92
	CH		

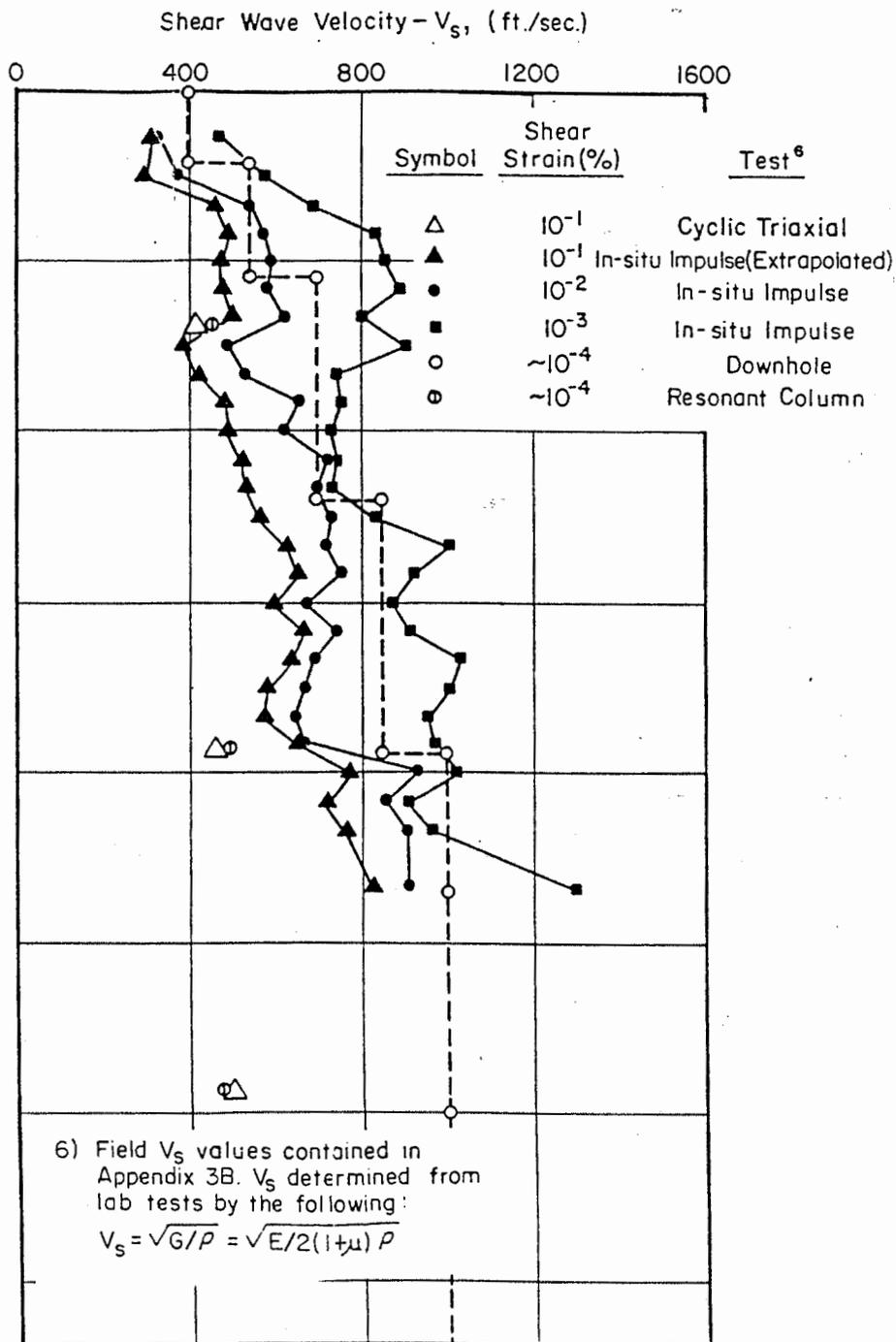
▽  
Approx.  
groundwater  
level

- 1) The stratification lines in the boring log represent the approximate boundaries between soil types, and the transition may be gradual.
- 2) Elevation obtained by hand leveling from USGS B.M. (elev. 953.72) at Railway Station, (See Fig. 3-1)
- 3) Laboratory tests performed on samples from indicated depth, (Appendix 3C).

Bottom of boring at 221 feet

Source: Agbabian Assoc./Shanon & Wilson (1975)

SMIP Directed Research Project	Boring Log B-1 El Centro Array	Figure <b>5.11</b>
Poland, Mejia, Soulages, Sun		



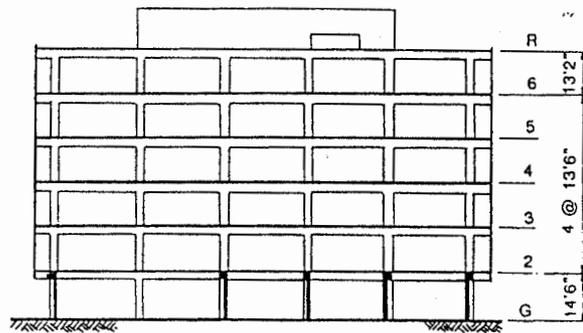
Source: Agbabian Assoc./Shanon & Wilson (1975)

SMIP Directed Research Project

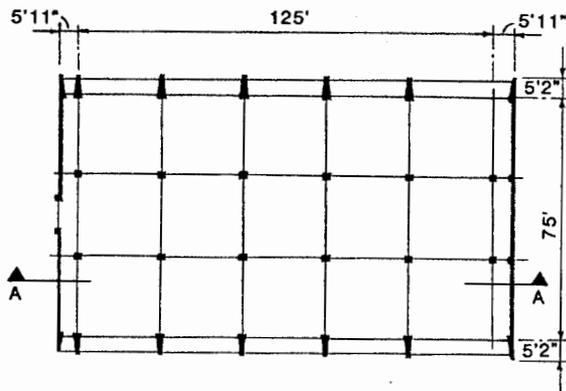
Poland, Mejia, Soulages, Sun

Shear Wave Velocity Profile - B-1  
 El Centro Array

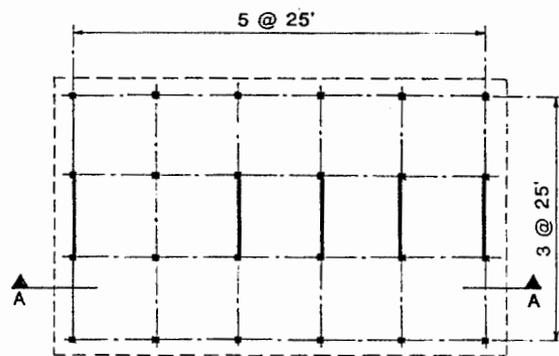
Figure  
**5.12**



SECTION A-A



UPPER FLOOR PLAN



GROUND FLOOR PLAN

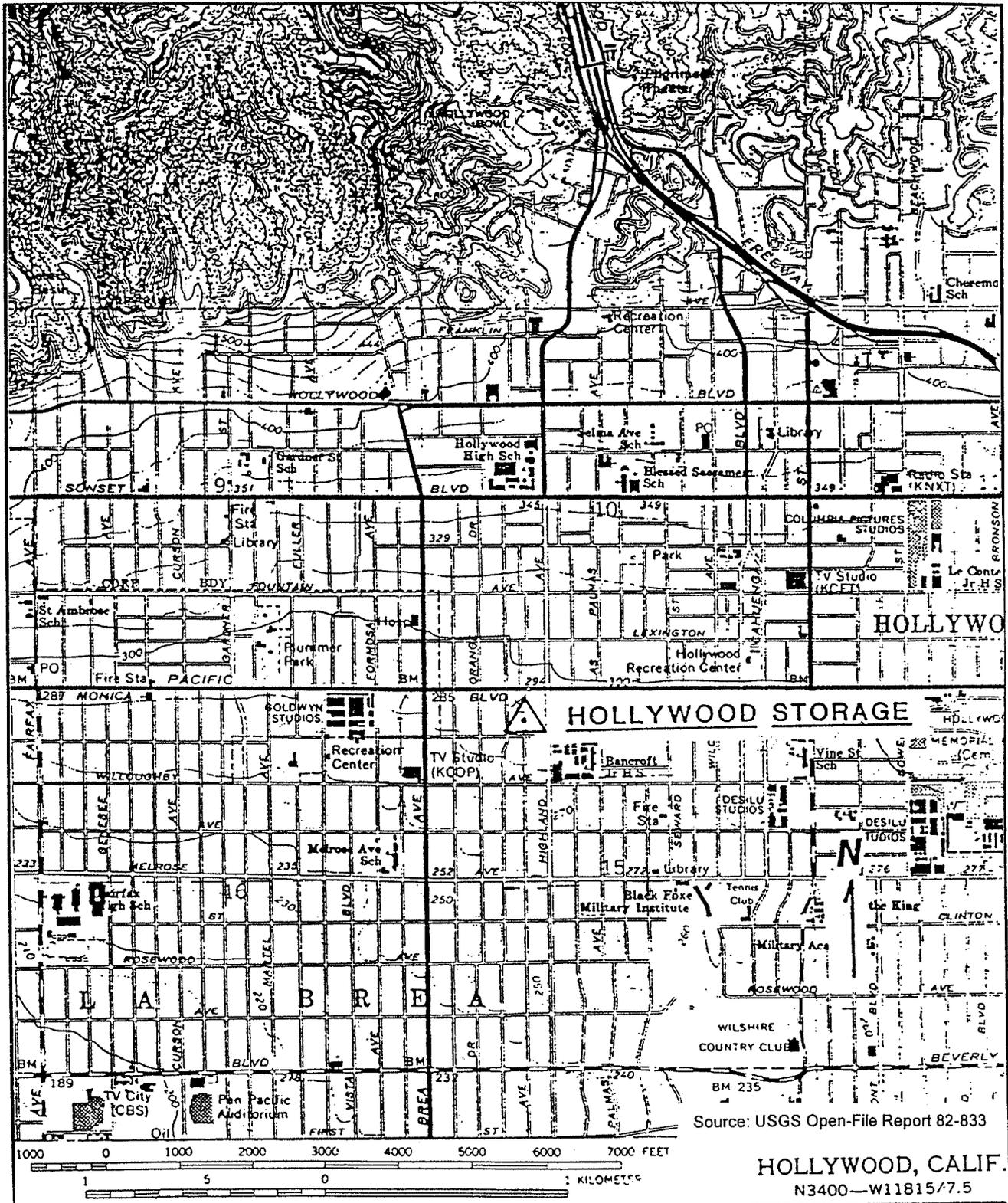


SMIP Directed Research Project

Poland, Mejia, Soulagés, Sun

Structural Configuration  
Imperial County Services Building

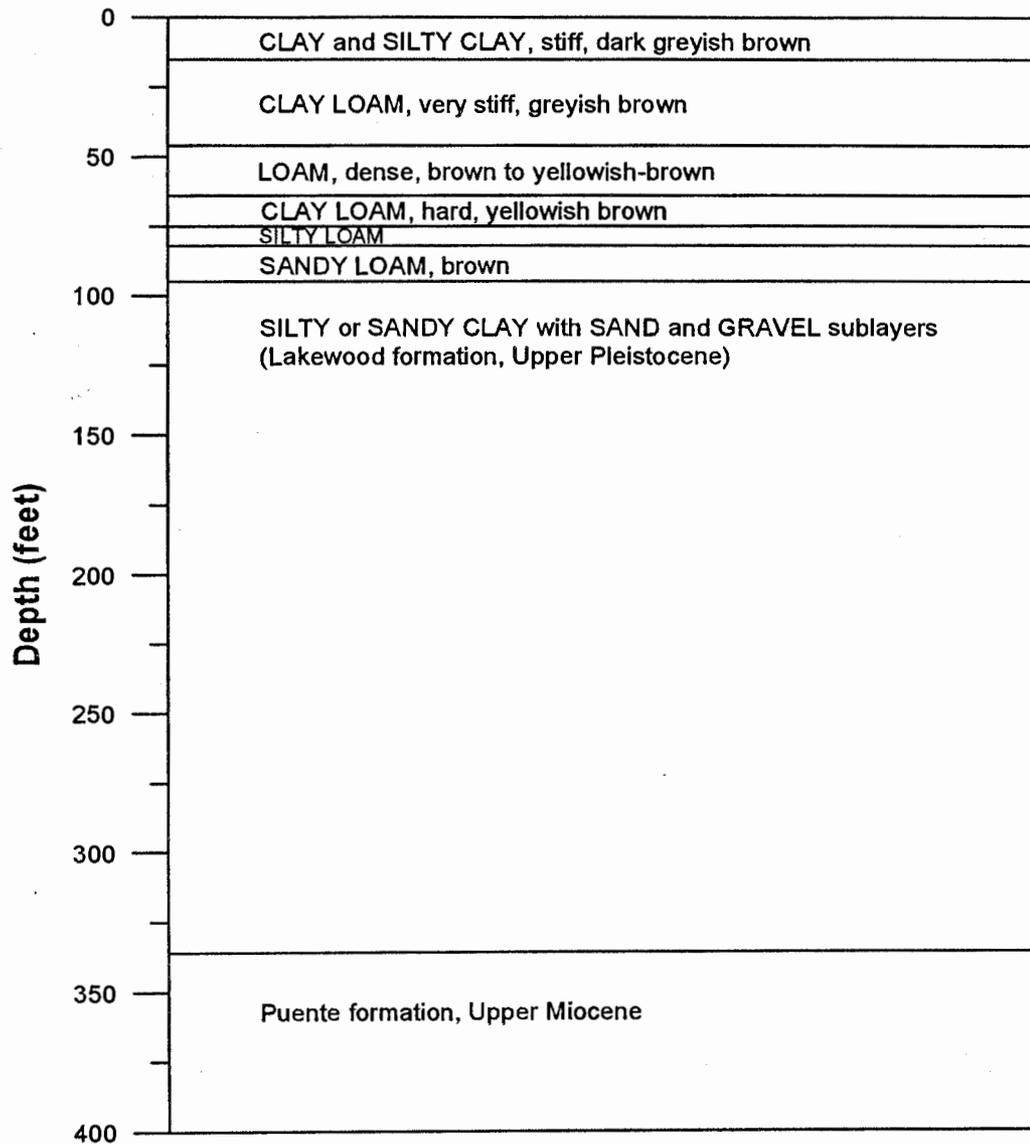
Figure  
5.13



SMIP Directed Research Project  
Poland, Mejia, Soulages, Sun

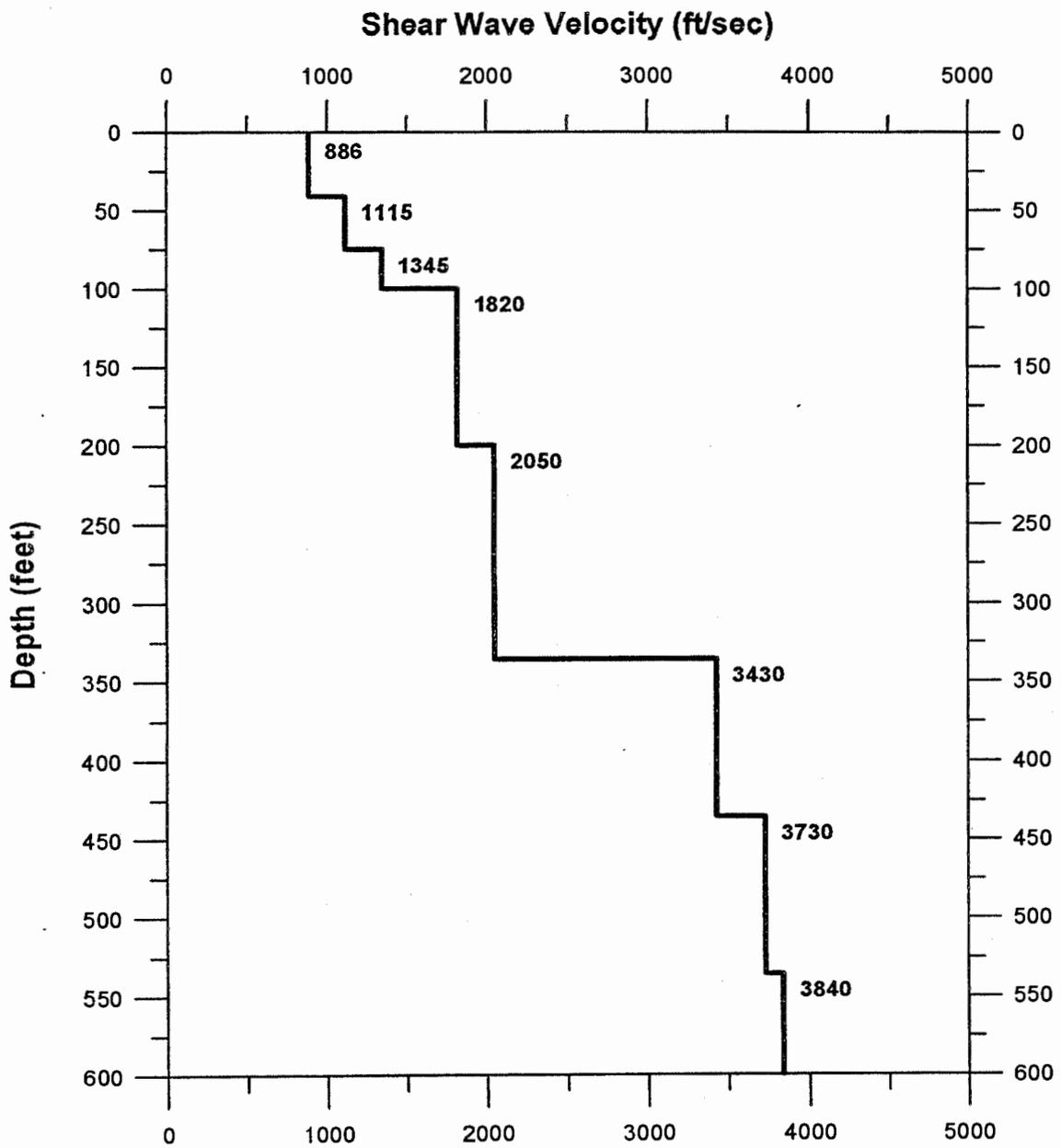
Station Location Plan  
Hollywood Storage Building

Figure  
**5.14**



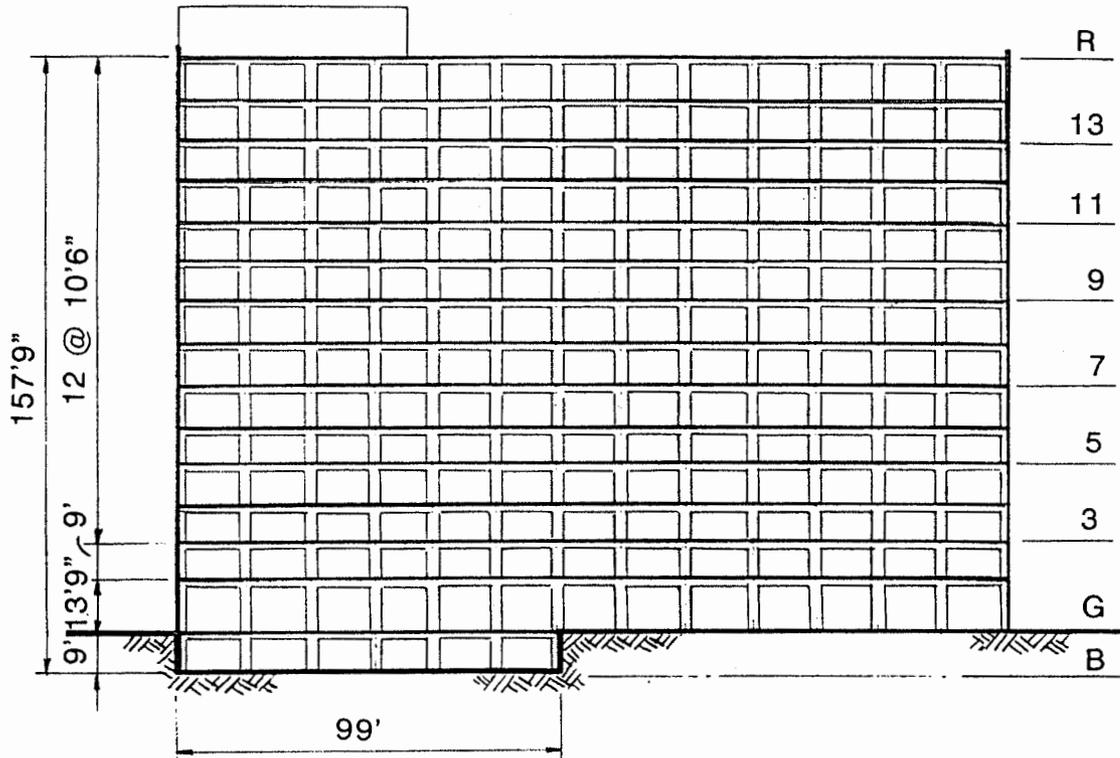
Source: USGS Open-File Report 82-833  
 Duke, et al. (1970)  
 USGS (1971)  
 Duke and Leeds (1962)

SMIP Directed Research Project	Boring Log Hollywood Storage Building	Figure
Poland, Mejia, Soulages, Sun		5.15

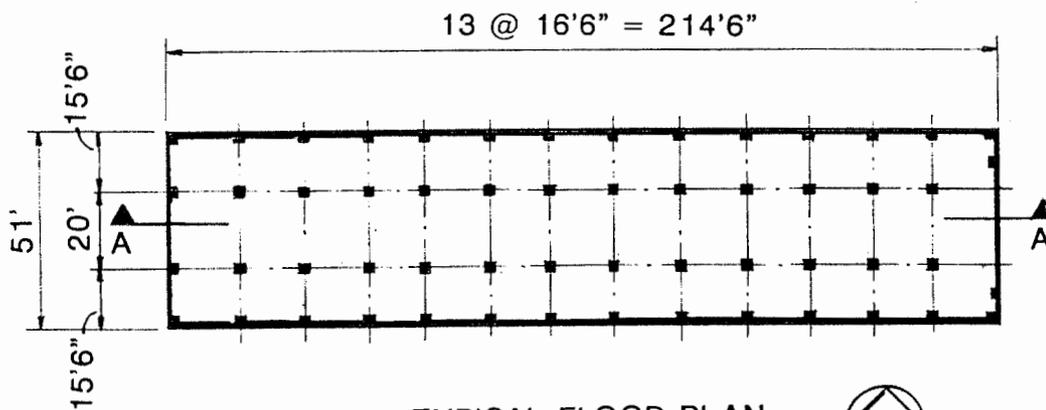


Source: USGS Open File Report 82-833  
 Duke, et. al. (1970)  
 USGS (1971)  
 Duke and Leeds (1962)

SMIP Directed Research Project	Shear Wave Velocity Profile Hollywood Storage Building	Figure
Poland, Mejia, Soulages, Sun		5.16



SECTION A-A



TYPICAL FLOOR PLAN

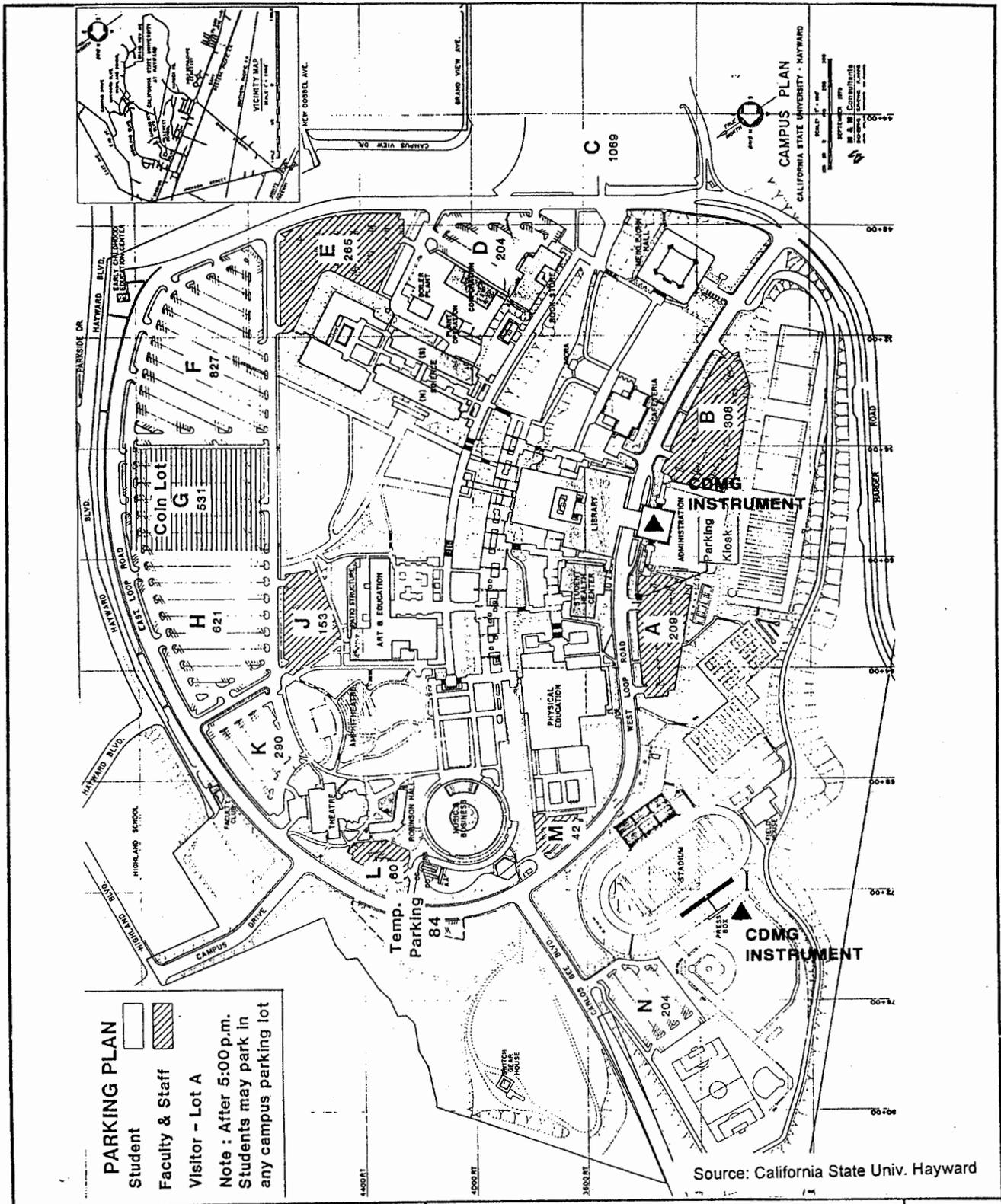


SMIP Directed Research Project

Poland, Mejia, Soulages, Sun

Structural Configuration  
Hollywood Storage Building

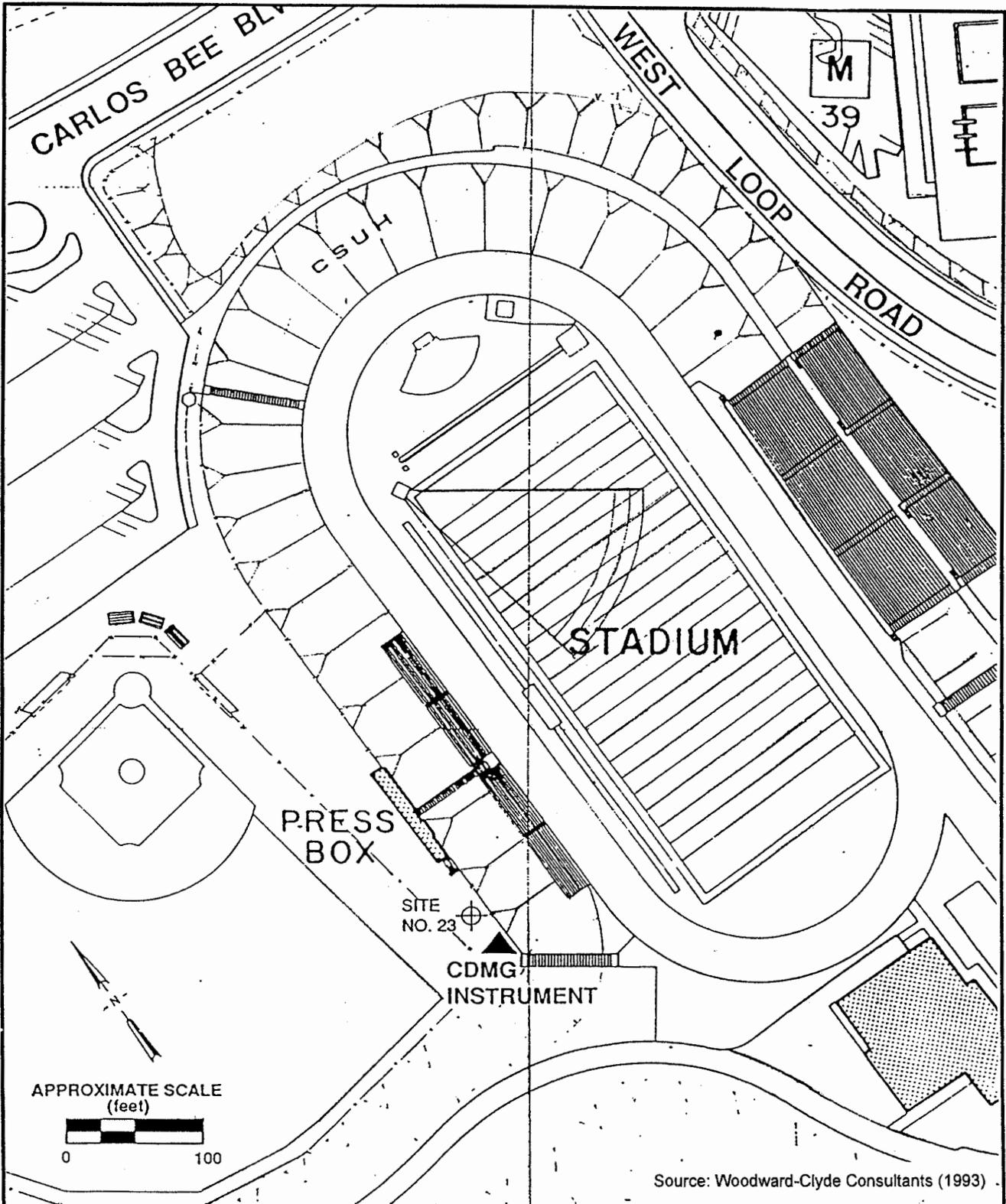
Figure  
**5.17**



SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

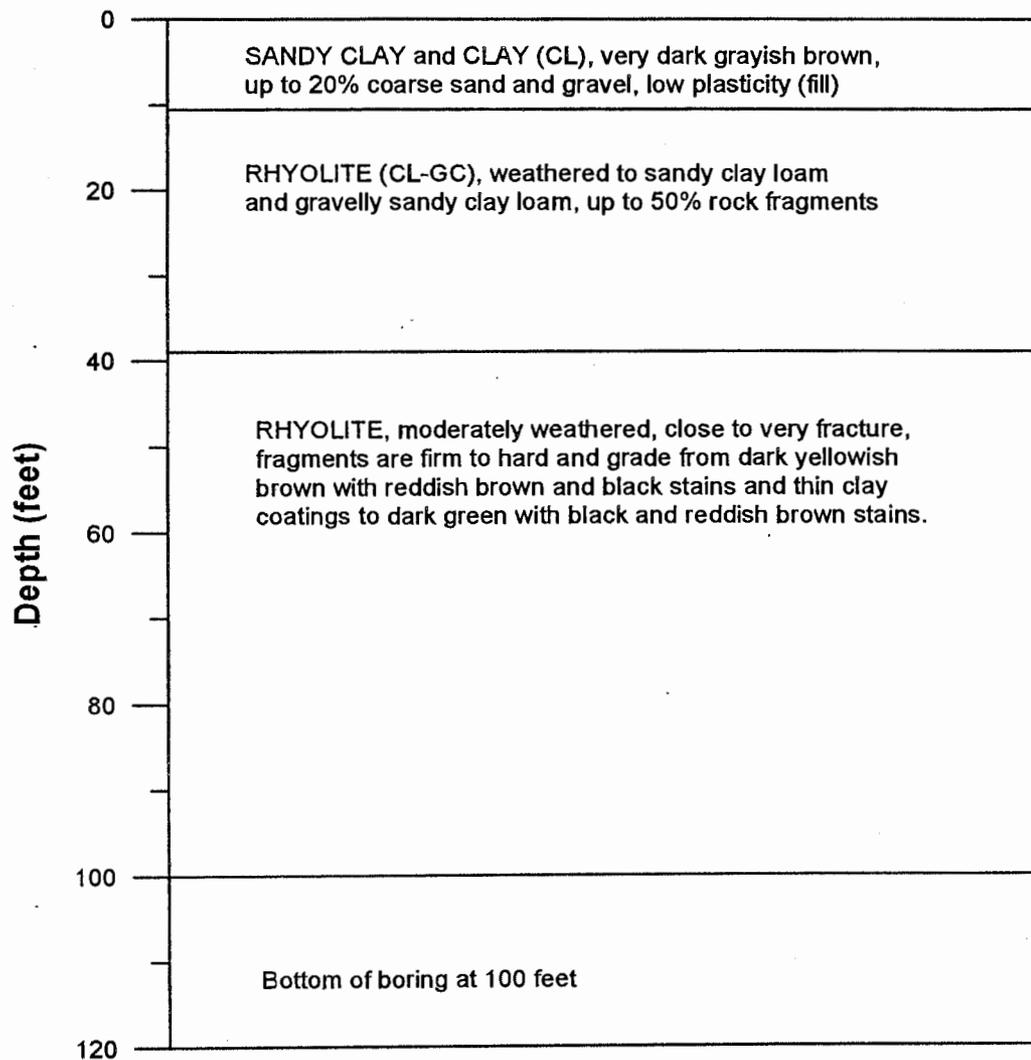
Site Plan  
 California State University  
 Hayward

Figure  
**5.18**



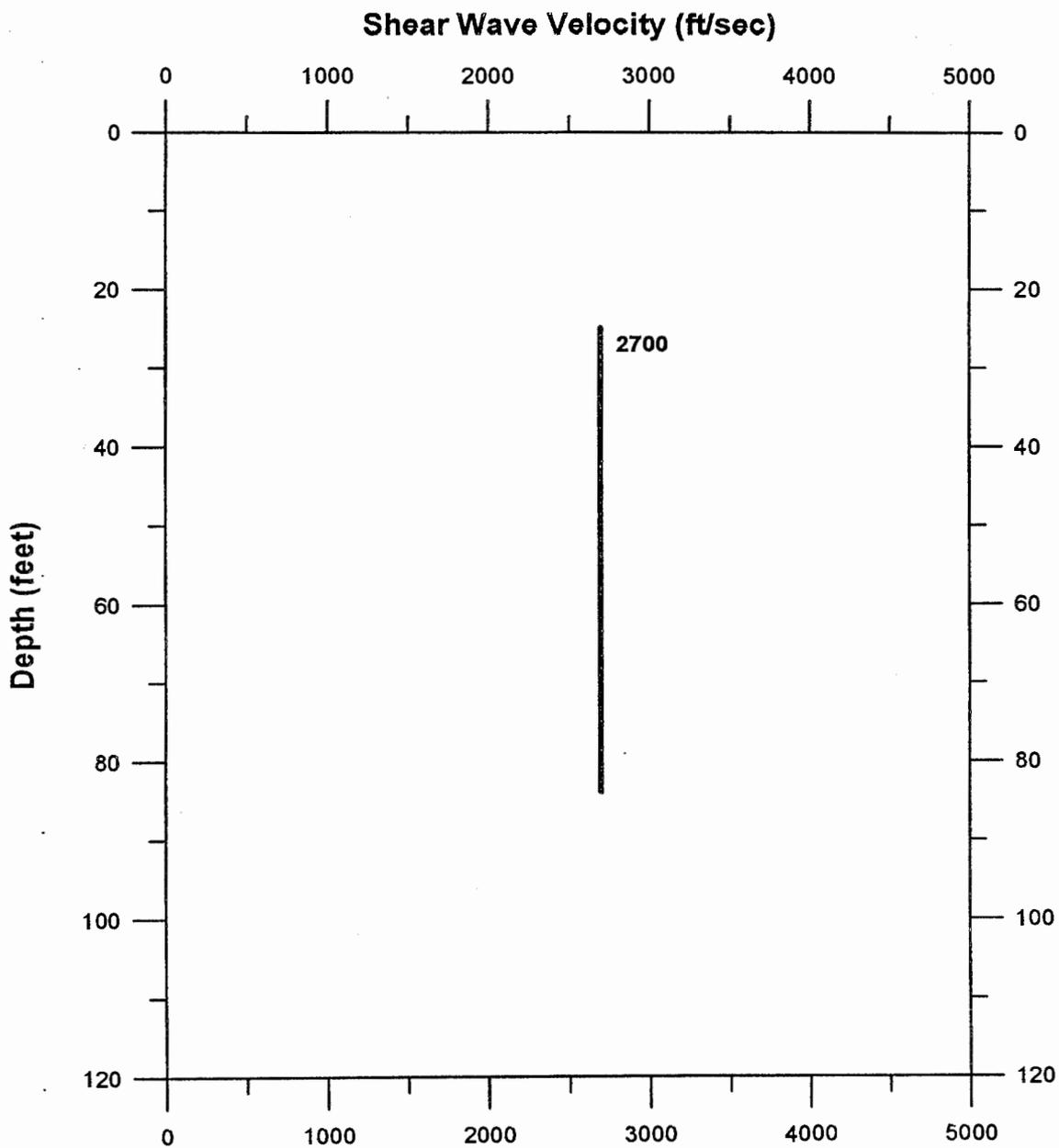
Source: Woodward-Clyde Consultants (1993)

<p>SMIP Directed Research Project</p>	<p>Site and Boring Location Plan</p>	<p>Figure</p>
<p>Poland, Mejia, Soulages, Sun</p>	<p>California State University, Hayward Stadium Grounds</p>	<p>5.19</p>



Source: USGS Open-File Report 76-731

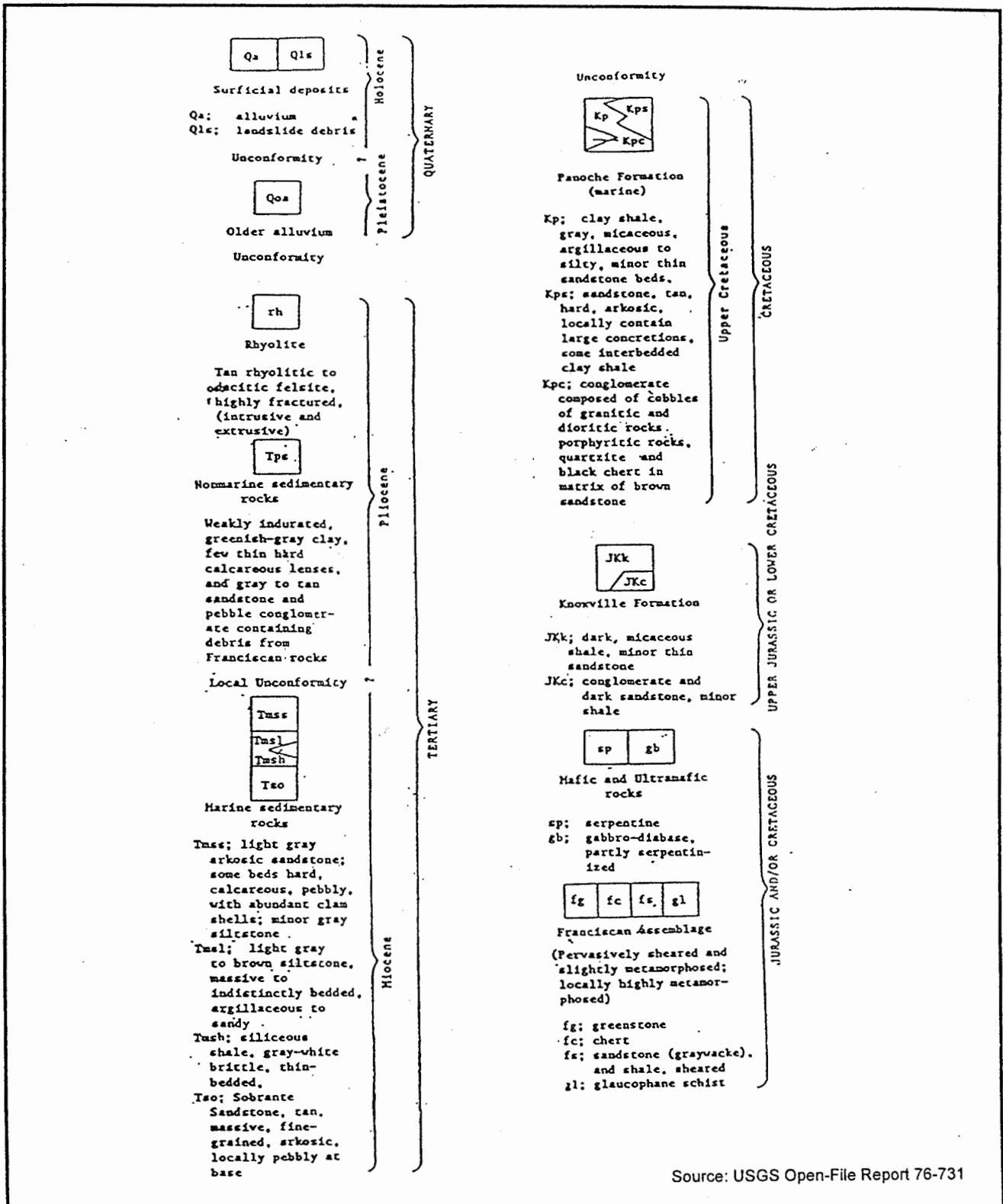
SMIP Directed Research Project	Boring Log California State University Hayward Stadium Grounds	Figure
Poland, Mejia, Soulages, Sun		5.20



Source: USGS Open File Report 76-731

SMIP Directed Research Project	Shear Wave Velocity Profile California State University Hayward Stadium Grounds	Figure <b>5.21</b>
Poland, Mejia, Soulages, Sun		



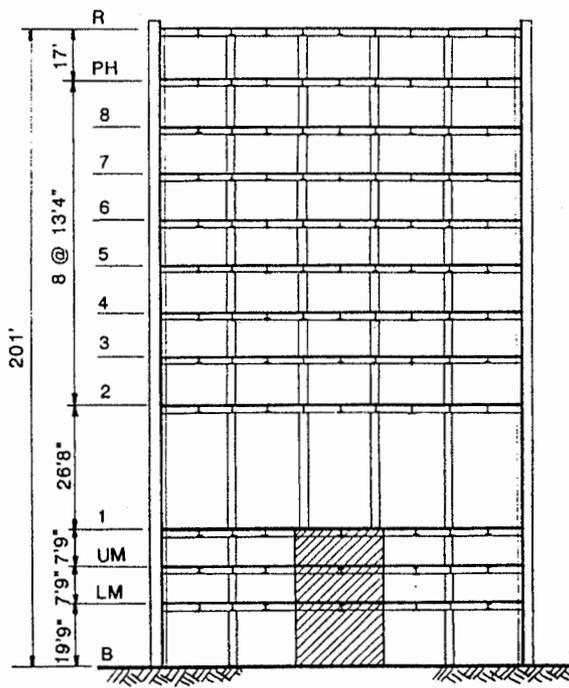


Source: USGS Open-File Report 76-731

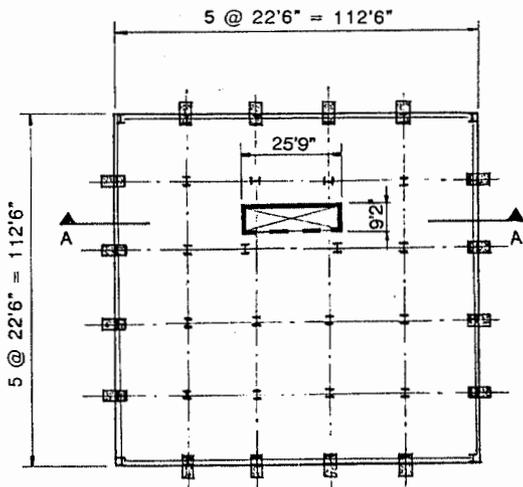
SMIP Directed Research Project  
Poland, Mejia, Soulages, Sun

Legend for the Preliminary Geologic Map of the Hayward Quadrangle

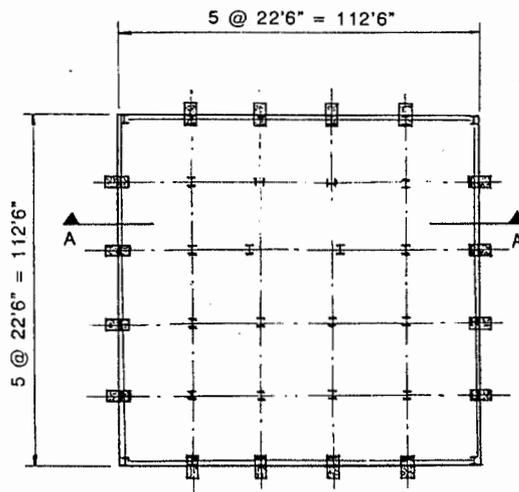
Figure 5.22b



SECTION A-A



TYPICAL FLOOR PLAN  
(1ST FLOOR, UPPER & LOWER MALL)

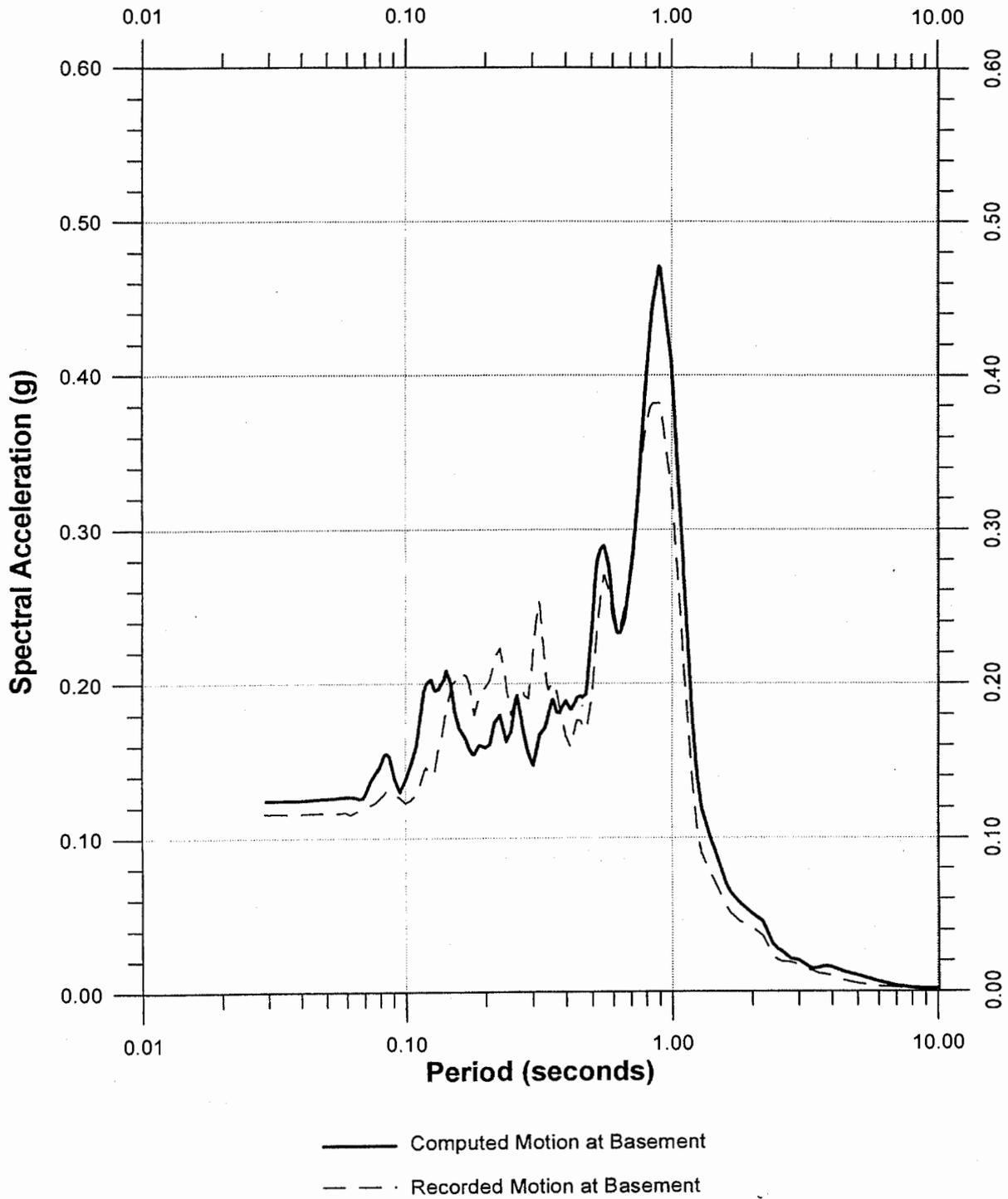


TYPICAL FLOOR PLAN  
(2ND FLOOR TO ROOF)

SMIP Directed Research Project  
Poland, Mejia, Soulages, Sun

Structural Configuration  
Administration Building at California  
State University, Hayward

Figure  
**5.23**

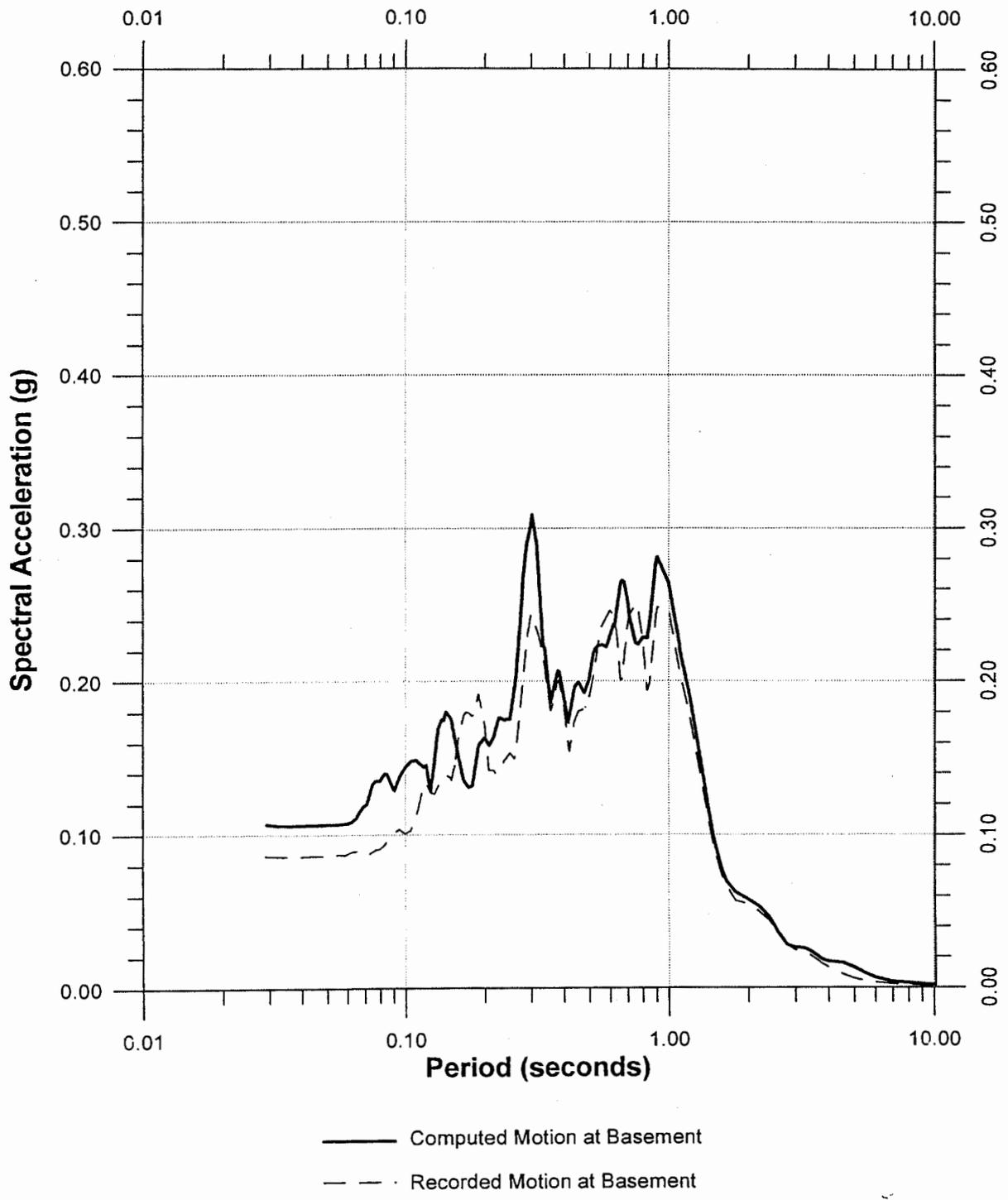


SMIP Directed Research Project

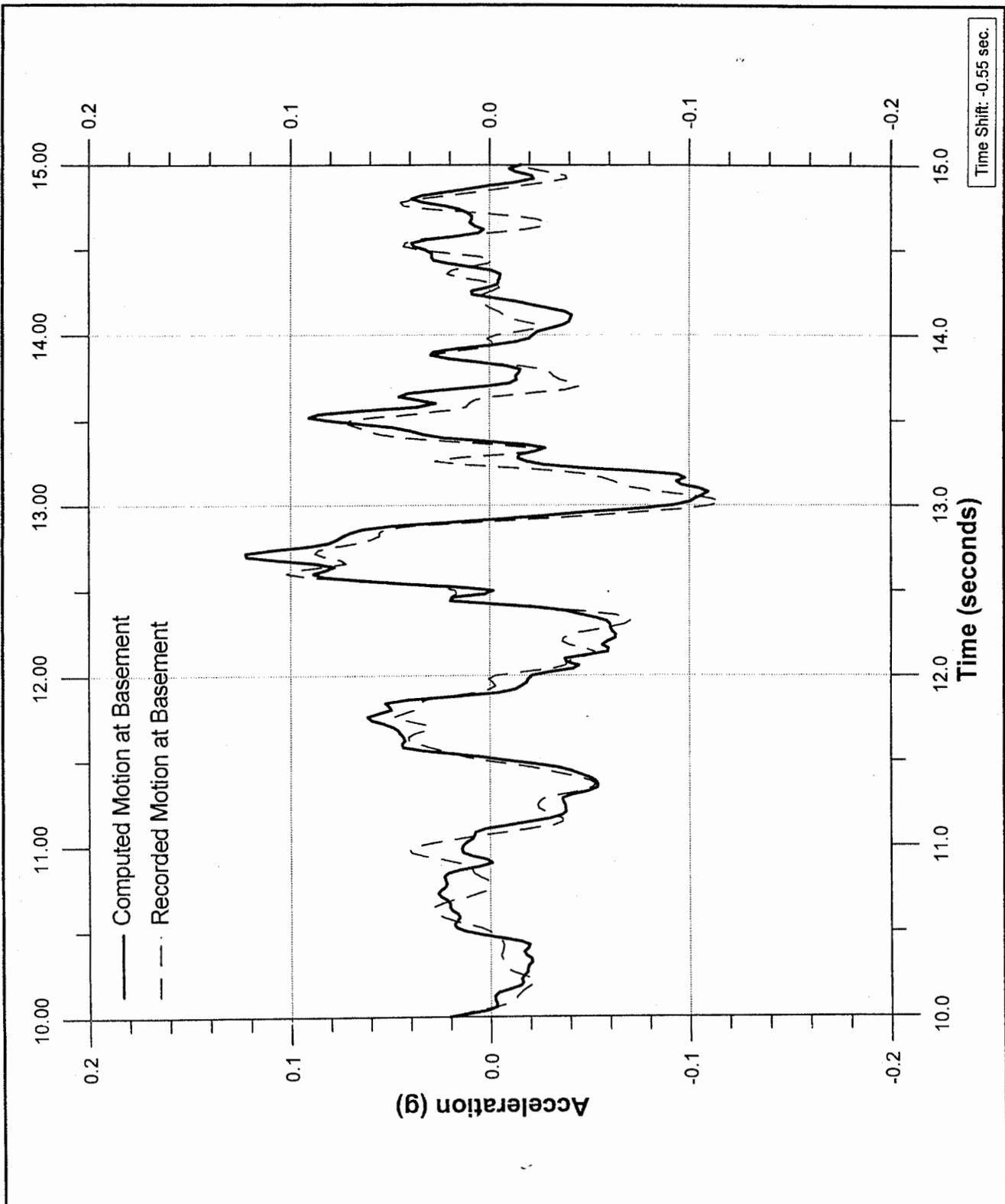
Poland, Mejia, Soulages, Sun

Response Spectrums  
 Computed vs. Recorded Motion at Basement  
 Richmond City Hall  
 Loma Prieta - 10/17/89 - South

Figure  
**5.24**



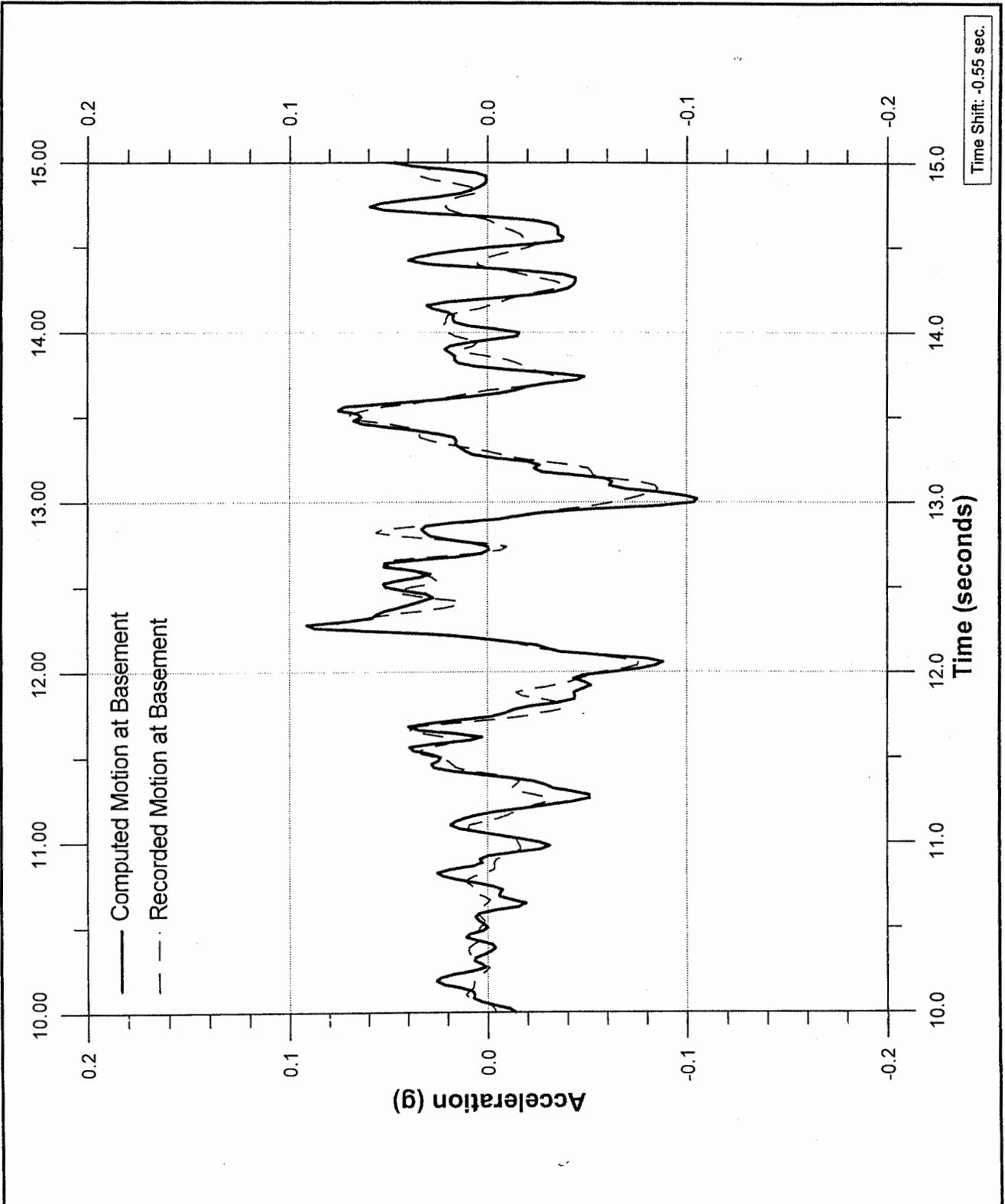
SMIP Directed Research Project	Response Spectrums Computed vs. Recorded Response at Basement	Figure
Poland, Mejia, Soulages, Sun	Richmond City Hall Loma Prieta - 10/17/89 - West	5.25



SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

Time Histories  
 Computed vs. Recorded Motion at Basement  
 Richmond City Hall  
 Loma Prieta - 10/17/89 - South

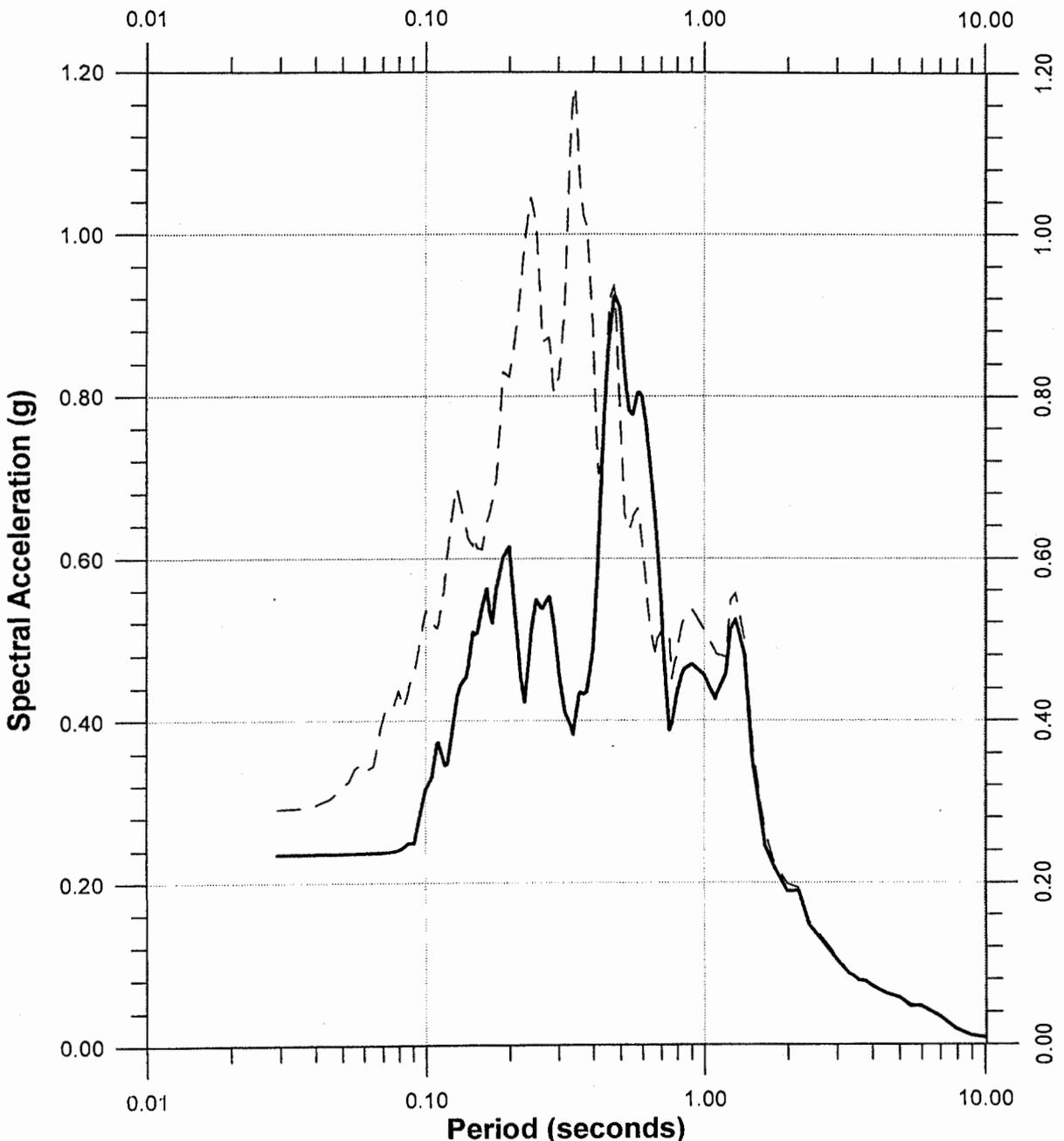
Figure  
**5.26**



SMIP Directed Research Project  
Poland, Mejia, Soulages, Sun

Time Histories  
Computed vs. Recorded Motion at Basement  
Richmond City Hall  
Loma Prieta - 10/17/89 - West

Figure  
**5.27**

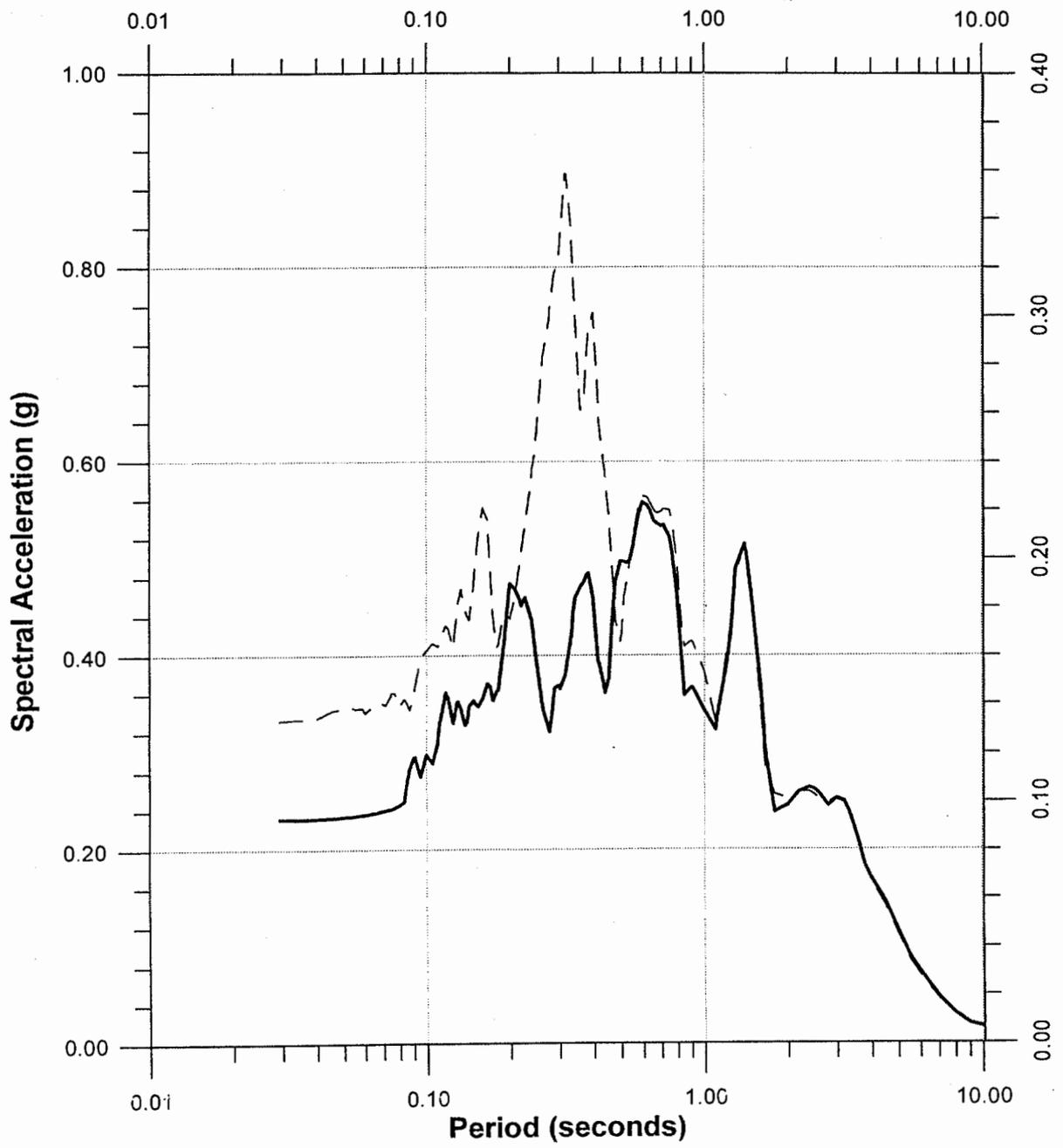


——— Computed Motion at Base  
 - - - Recorded Motion at Base

SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

Response Spectrums  
 Computed vs. Recorded Motion at Base  
 Imperial County Sevcies Bldg.  
 Imperial Valley - 10/15/79 - North

Figure  
**5.28**

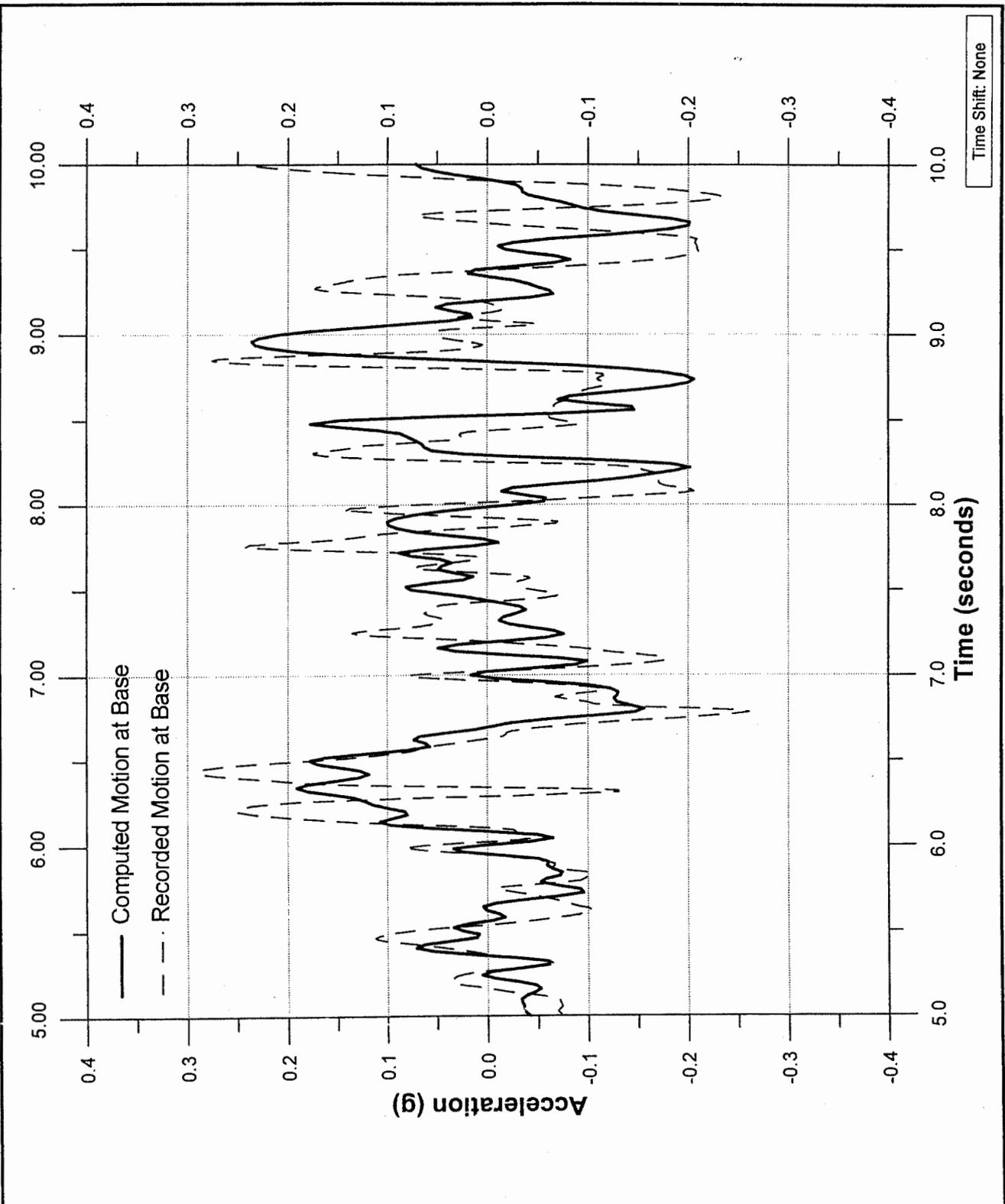


——— Computed Motion at Base  
 - - - Recorded Motion at Base

SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

Response Spectrums  
 Computed vs. Recorded Motion at Base  
 Imperial County Sevcies Bldg.  
 Imperial Valley - 10/15/79 - East

Figure  
**5.29**

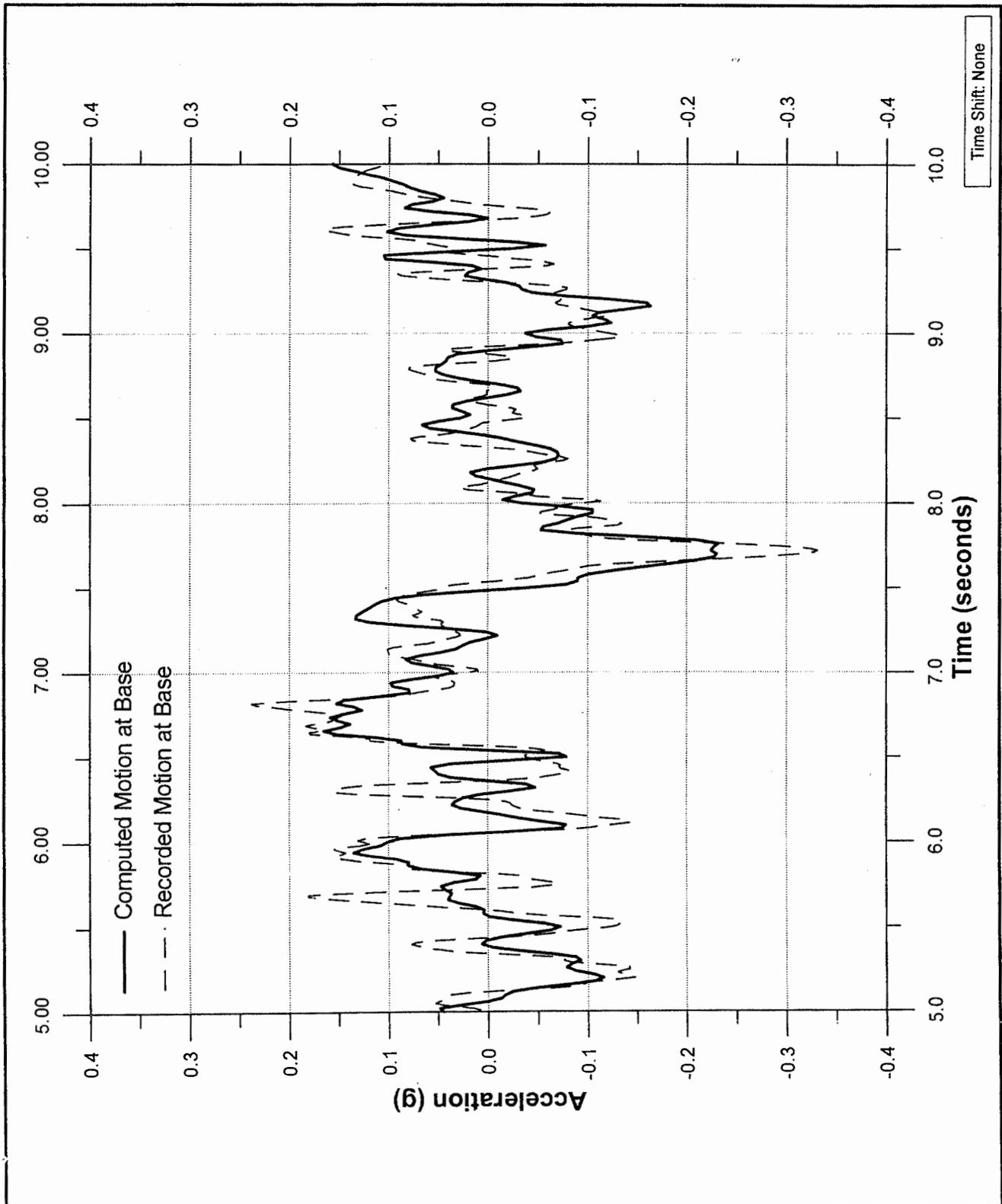


Time Shift: None

SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

Time Histories  
 Computed vs. Recorded Motion at Base  
 Imperial County Services Building  
 Imperial Valley - 10/15/79 - North

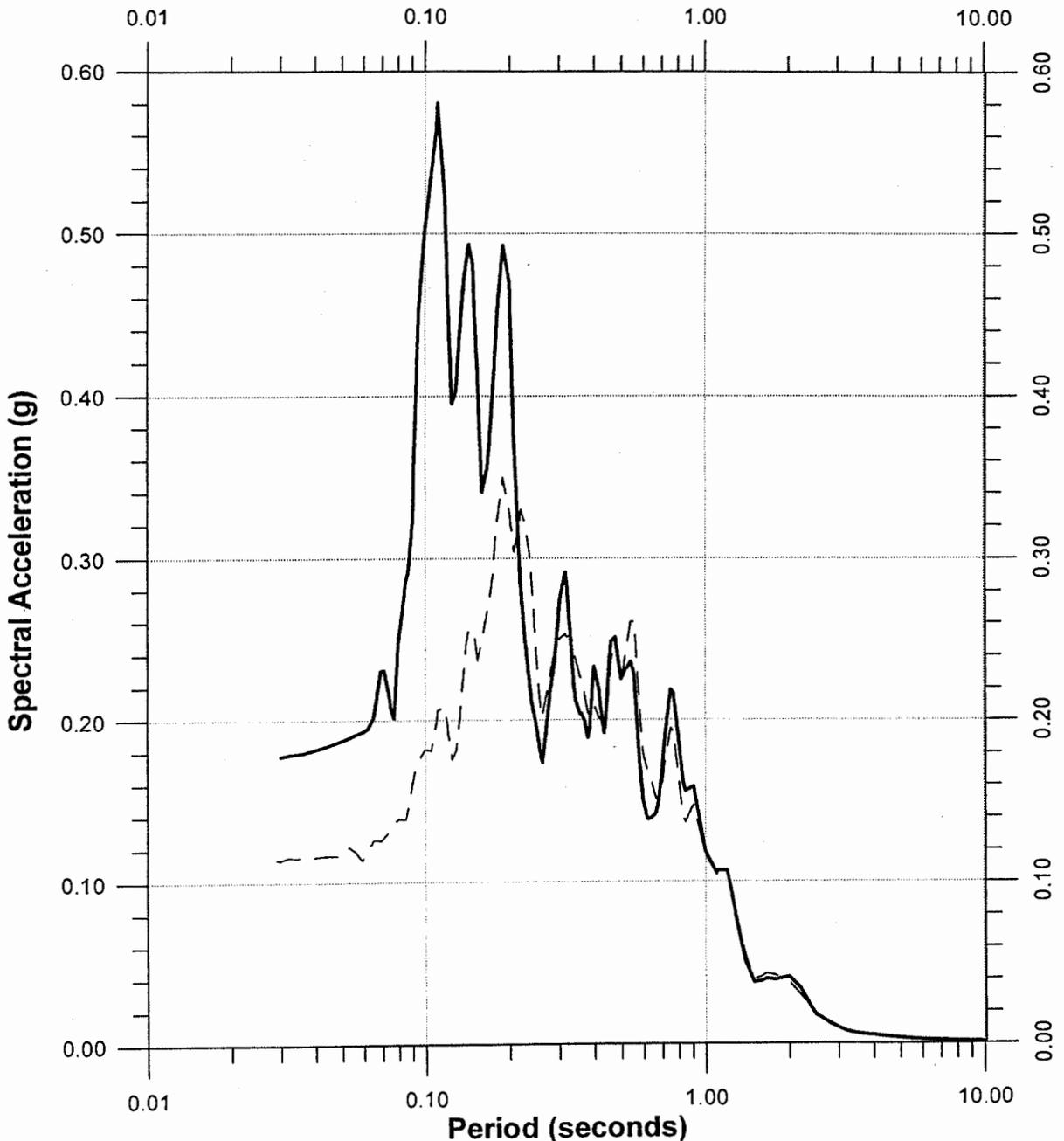
Figure  
**5.30**



SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

Time Histories  
 Computed vs. Recorded Motion at Base  
 Imperial County Services Building  
 Imperial Valley - 10/15/79 - East

Figure  
**5.31**

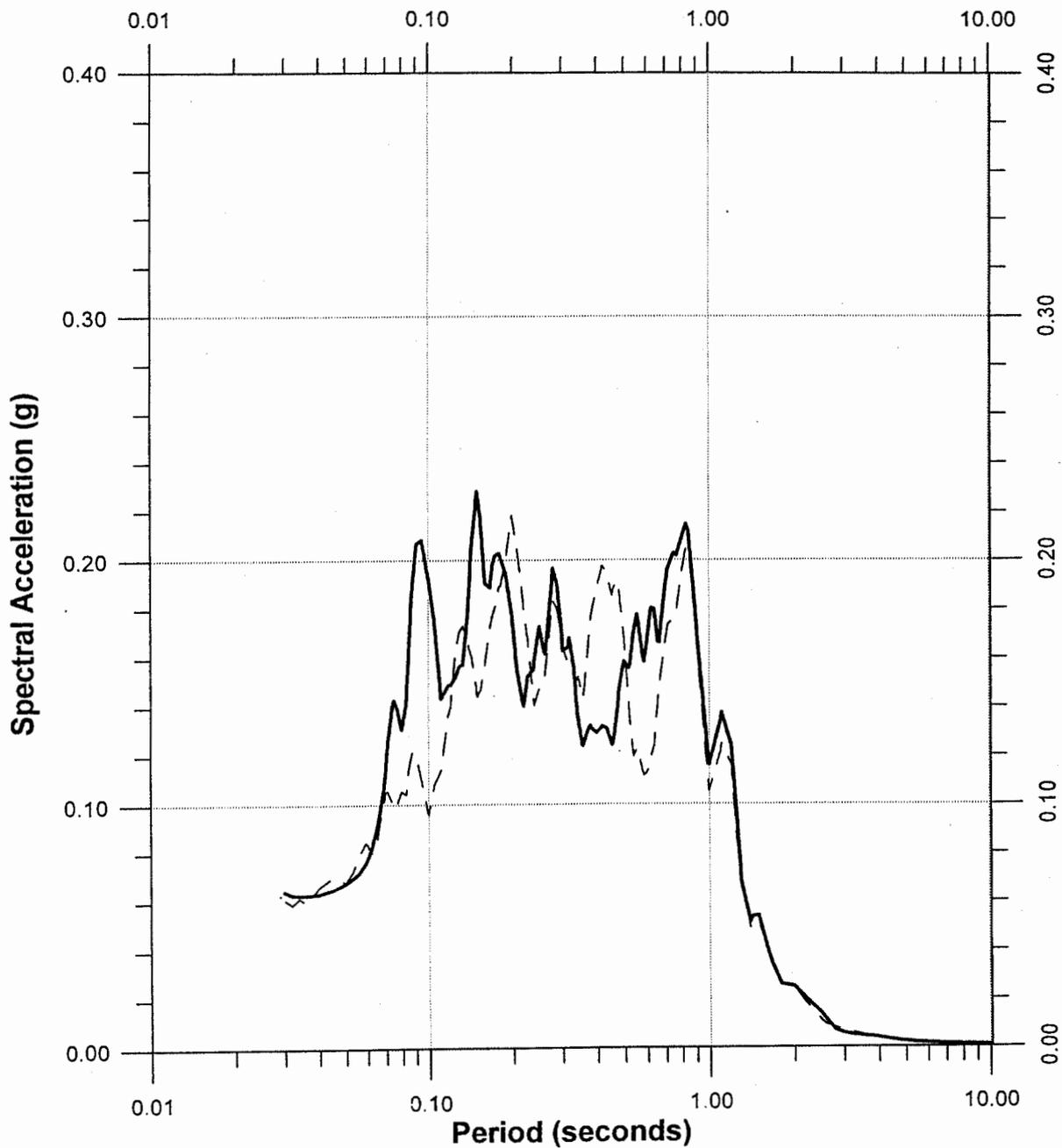


——— Computed Motion at Basement  
 - - - Recorded Motion at Basement

SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

Response Spectrums  
 Computed vs. Recorded Motion at Basement  
 Hollywood Storage Building  
 Whittier - 10/1/87 - North

Figure  
**5.32**

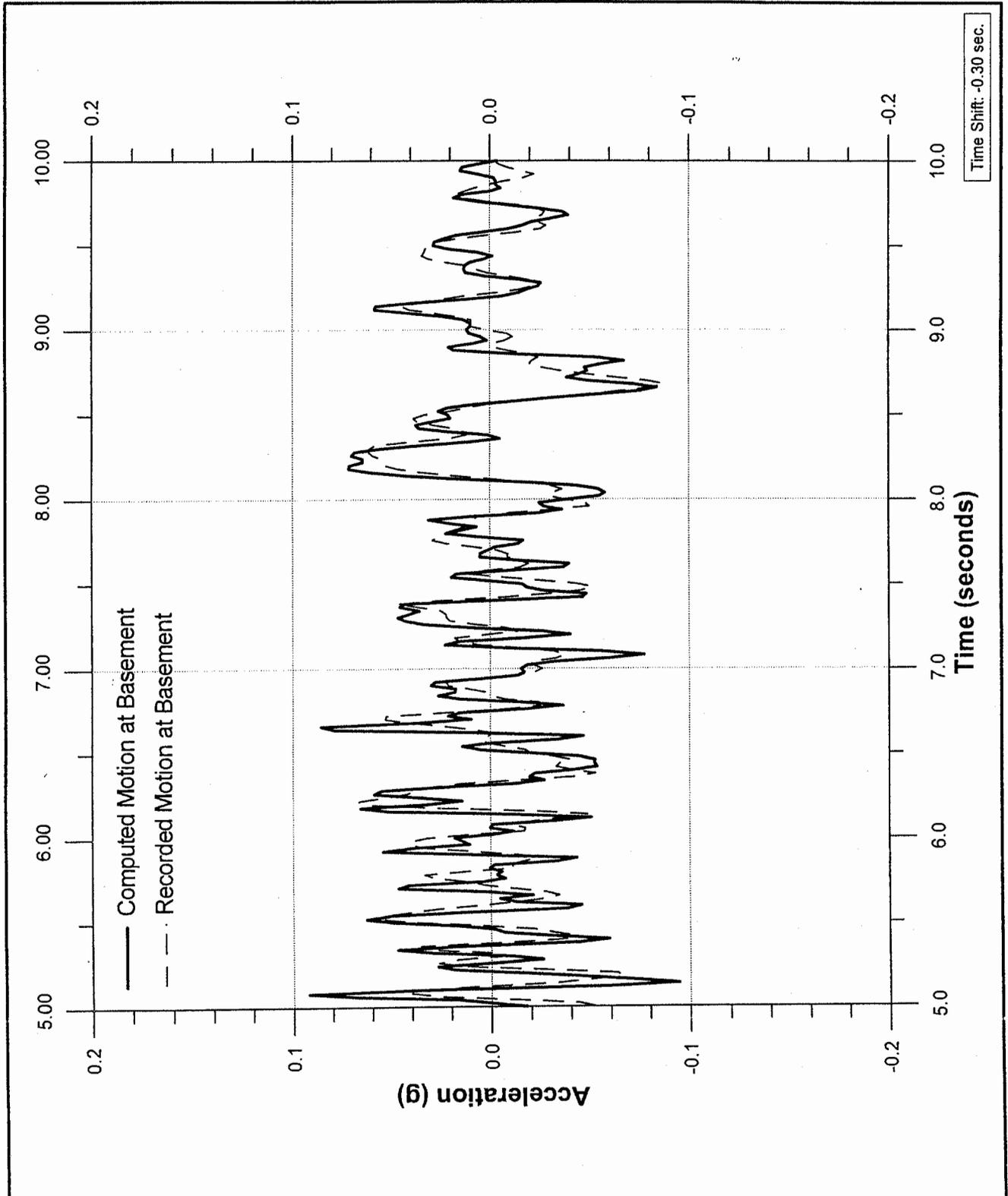


——— Computed Motion at Basement  
 - - - Recorded Motion at Basement

SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

Response Spectrums  
 Computed vs. Recorded Motion at Basement  
 Hollywood Storage Building  
 Whittier - 10/1/87 - East

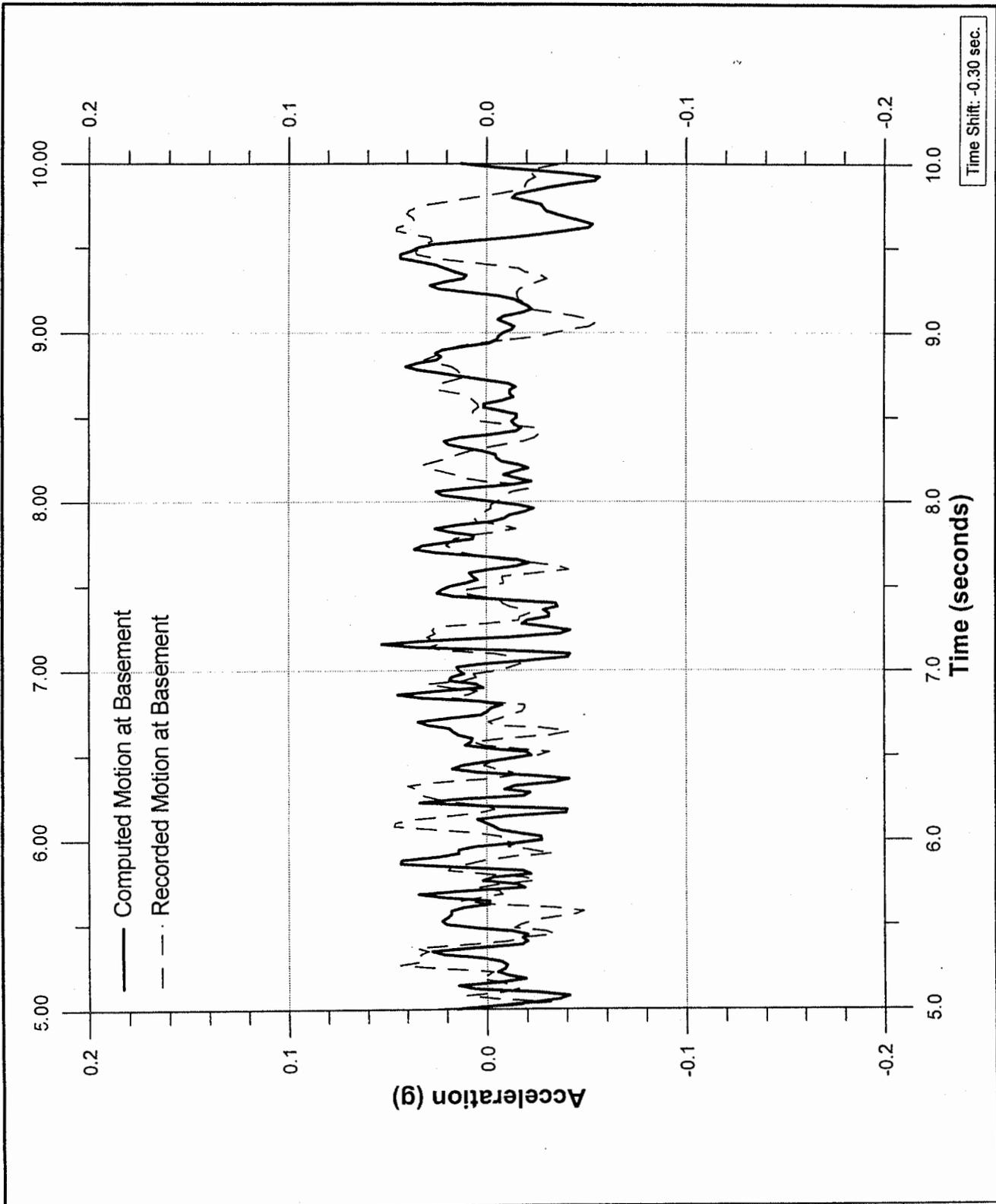
Figure  
**5.33**



SMIP Directed Research Project  
 Poland, Mejia, Soulages, Sun

Time Histories  
 Computed vs. Recorded Motion at Basement  
 Hollywood Storage Building  
 Whittier - 10/1/87 - North

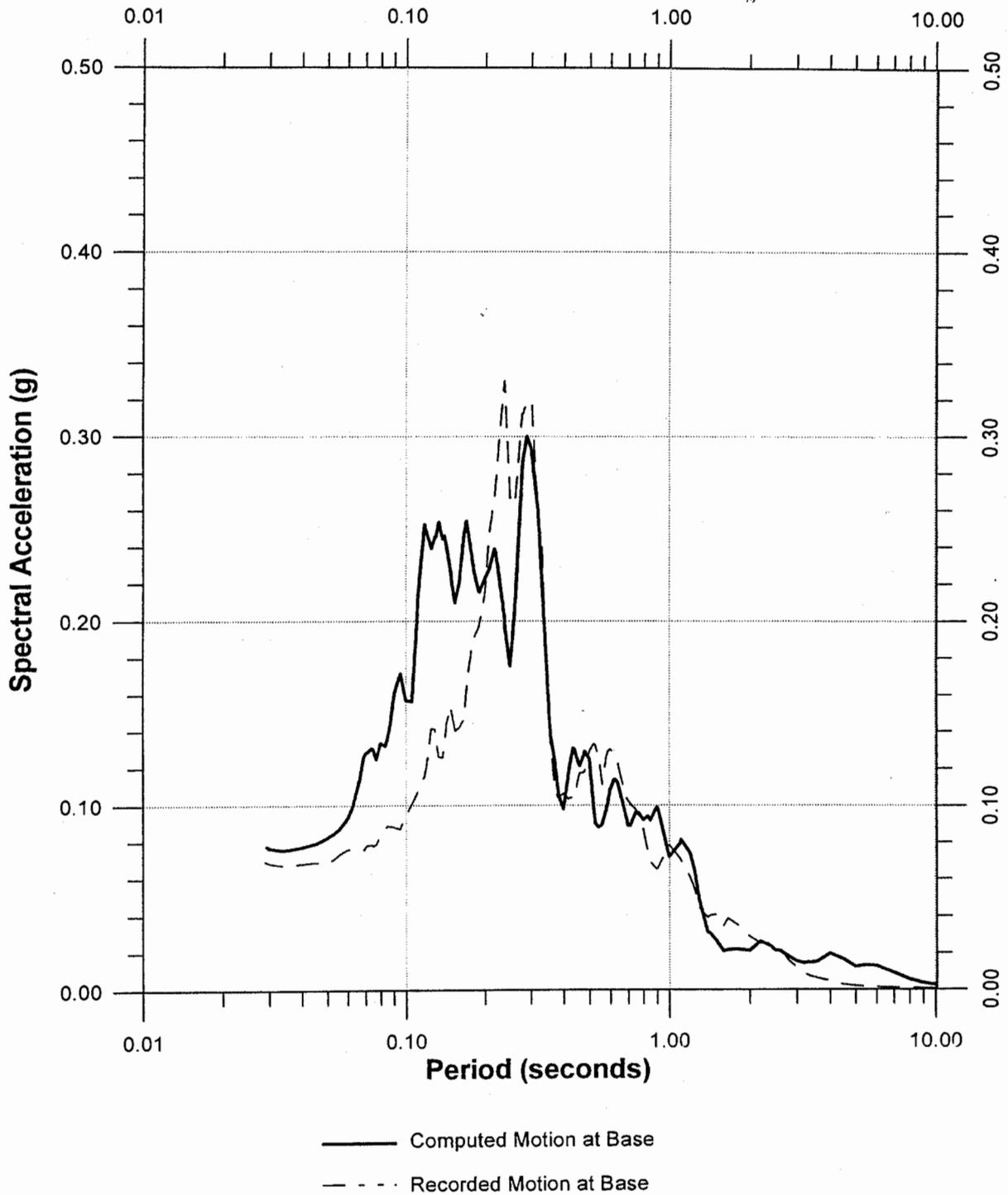
Figure  
**5.34**



SMIP Directed Research Project  
Poland, Mejia, Soulages, Sun

Time Histories  
Computed vs. Recorded Motion at Basement  
Hollywood Storage Building  
Whittier - 10/1/87 - East

Figure  
5.35

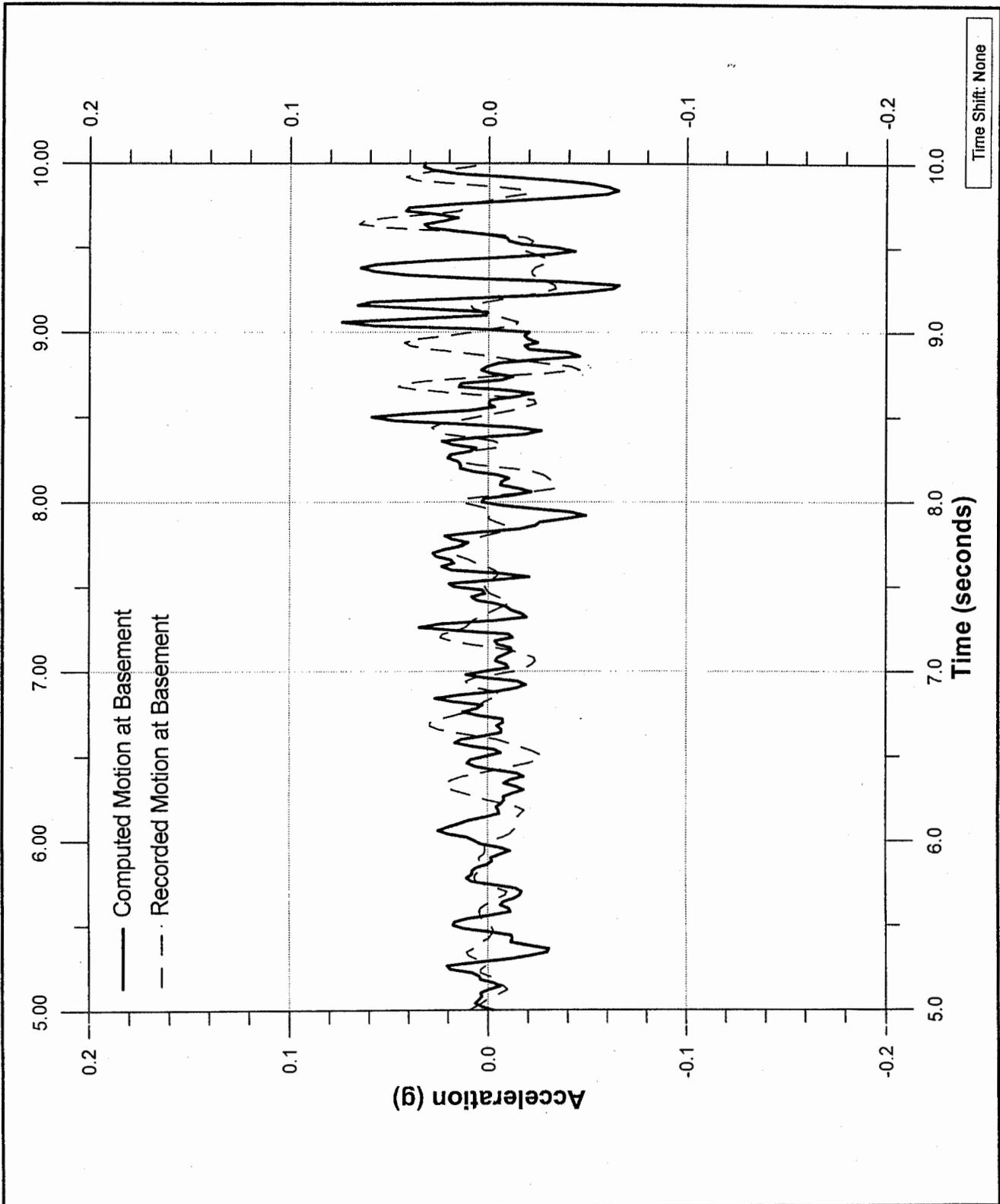


SMIP Directed Research Project

Poland, Mejia, Soulages, Sun

Response Spectrums  
 Computed vs. Recorded Motion at Base  
 13 Story CSUH Admin. Bldg.  
 Loma Prieta - 10/17/89 - North

Figure  
**5.36**



SMIP Directed Research Project  
Poland, Mejia, Soulages, Sun

Time Histories  
Computed vs. Recorded Motion at Base  
13 Story CSUH Admin. Bldg.  
Loma Prieta - 10/17/89 - North

Figure  
**5.37**

## LIST OF CSMIP DATA UTILIZATION REPORTS

California Department of Conservation  
Division of Mines and Geology  
Office of Strong Motion Studies  
California Strong Motion Instrumentation Program (CSMIP)

The California Strong Motion Instrumentation Program (CSMIP) publishes data utilization reports as part of the Data Interpretation Project. These reports were prepared by investigators funded by CSMIP. Results obtained by the investigators were summarized in the papers included in the proceedings of the annual seminar. These reports and seminar proceedings are available from CSMIP at nominal cost. Requests for the reports, seminar proceedings and/or for additional information should be addressed to: Data Interpretation Project Manager, Office of Strong Motion Studies, Division of Mines and Geology, California Department of Conservation, 801 K Street, MS 13-35, Sacramento, California 95814-3531. Phone: (916)322-3105

- CSMIP/92-01 "Evaluation of Soil-Structure Interaction in Buildings during Earthquakes," by G. Fenves and G. Serino, June 1992, 57 pp.
- CSMIP/92-02 "Seismic Performance Investigation of the Hayward BART Elevated Section," by W. Tseng, M. Yang and J. Penzien, September 1992, 61 pp.
- CSMIP/93-01 "Influence of Critical Moho Reflections on Strong Motion Attenuation in California," by P. Somerville, N. Smith and D. Dreger, December 1993, 84 pp.
- CSMIP/93-02 "Investigation of the Response of Puddingstone Dam in the Whittier Narrows Earthquake of October 1, 1987," by J. Bray, R. Seed and R. Boulanger, December 1993, 60 pp.
- CSMIP/93-03 "Investigation of the Response of Cogswell Dam in the Whittier Narrows Earthquake of October 1, 1987," by R. Boulanger, R. Seed and J. Bray, December 1993, 53 pp.
- CSMIP/94-01 "Torsional Response Characteristics of Regular Buildings under Different Seismic Excitation Levels," by H. Sedarat, S. Gupta, and S. Werner, January 1994, 43 pp.
- CSMIP/94-02 "Degradation of Plywood Roof Diaphragms under Multiple Earthquake Loading," by J. Bouwkamp, R. Hamburger and J. Gillengerten, February 1994, 32 pp.
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