R-7211-13-2859

DOE/NBM--6006717

DE86 006717

PROCEDURES AND CRITERIA FOR INCREASING THE EARTHQUAKE RESISTANCE LEVEL OF ELECTRICAL SUBSTATIONS AND SPECIAL INSTALLATIONS

September 30, 1973

Contract No. 14-03-2262N

Prepared for

BONNEVILLE POWER ADMINISTRATION
Portland, Oregon

AGBABIAN ASSOCIATES
El Segundo, California

DISCLAIMER

THIS REPORT WAS PREPARED PURSUANT TO A MEMORANDUM FROM THE DEPUTY ASSISTANT SECRETARY FOR MANAGEMENT AND BUDGET TO THE UNITED STATES DEPARTMENT OF THE INTERIOR, DATED NOVEMBER 9, 1971. PUBLICATION OF THE FINDINGS AND RECOMMENDATIONS HEREIN SHOULD NOT BE CONSTRUED AS REPRESENTING EITHER THE APPROVAL OR DISAPPROVAL OF THE SECRETARY OF THE INTERIOR. THE PURPOSE OF THIS REPORT IS TO PROVIDE INFORMATION AND ALTERNATIVES FOR FURTHER CONSIDERATION BY THE BONNEVILLE POWER ADMINISTRATION, THE SECRETARY OF THE INTERIOR, AND OTHER FEDERAL AGENCIES.

FOREWORD

This report was prepared for the Bonneville Power Administration, Division of Engineering and Construction, by Agbabian Associates, Engineering Consultants, El Segundo, California, under Contract 14-03-2262N. Project Manager for the study was Dr. George A. Young. Major contributions to the study were made by Dr. M. S. Agbabian, Technical Director; Philip J. Richter, Project Engineer for the portion of the study reported in Chapters 3, 4, and 5; Kelvin L. Merz, Principal Investigator for the work reported in Chapters 3 and 4; and Donald G. Cross, Principal Investigator for the work reported in Chapter 5. Contributions on analyses and design concepts reported in Chapters 4 and 5 were provided by Art K. Patel and Fred Sheybani. Response spectra for the Celilo Converter Station studies were developed by Dr. Samy A. Adham and probability studies of earthquake ground accelerations reported in Chapter 2 were provided by Dr. H. S. Ts'ao.

Two formally constituted seismic committees composed of independent consultants and Government officials participated in formulating the seismic criteria recommendations in this report. The first seismic committee was formed at the outset of the investigation to review the seismic criteria used in the studies reported in Chapters 3 and 4 relative to the Celilo Converter Station. The second seismic committee was formed near the conclusion of the study to evaluate the seismic criteria reported in Chapter 2 and the Seismic Regionalization Study for the Bonneville Power Administration Service Area reported in Appendix E-1. The study reported in Appendix E-1 was performed under this contract by Shannon and Wilson, Inc., Portland, Oregon. Authors of this report were Dr. Richard W. Couch and Robert J. Deacon.

The need for the installation of strong motion recorders by the Bonneville Power Administration was evaluated under this contract by Dr. Stewart W. Smith. A report by Dr. Smith is included as Appendix D-1.

As an integral part of the execution of this contract, Agbabian Associates conducted an Earthquake Engineering Seminar hosted by the Bonneville Power Administration in Portland, November 13-17, 1972. This Seminar was designed for technical staffs of interested organizations to improve their expertise in seismic designs of electrical facilities and equipment. Among the organizations which joined to sponsor this Seminar were:

U. S. Army Corps of Engineers
Pacific Power & Light Company
City of Seattle, Department of Lighting
Puget Sound Power & Light Company
Portland General Electric Company
City of Tacoma, Light Division
General Electric Company

Technical monitor of the study for the Bonneville Power Administration was Mr. Norman R. Drulard. Important contributions of information on the design of electrical equipment and facilities, and in reviewing design concepts were provided by Messrs. C. E. Bragunier, D. C. Johnson, Dwight Raikoglo and P. L. White, of the Bonneville Power Administration. Mr. Donald C. Birch, Head Geologist (now retired), Bonneville Power Administration, provided valuable assistance in the collection of information relative to the preparation of Chapter 2.

Acknowledgement is made of the interest and contributions to the study made by Mr. George S. Bingham, Assistant Administrator for Engineering and Construction and Chief Engineer, and Mr. Clifford C. Diemond, Assistant Chief Engineer (now retired) Bonneville Power Administration.

CONTENTS

CONTENTS (CONTINUED)

CONTENTS (CONTINUED)

CONTENTS (CONTINUED)

CHAPTER 1

INTRODUCTION AND SUMMARY

INTRODUCTION

This report defines a procedure and provides basic information needed to determine the modifications required to make electrical substations and special installations of the Bonneville Power Administration (BPA) more resistant to strong earthquake ground motion. It also provides a procedure for developing an effective plan for establishing the sequence, or priority, of providing the required modifications. Background information essential to the study and a summary of the five major study areas of the report are provided in this chapter.

BACKGROUND INFORMATION

The BPA service area includes the states of Washington, Oregon, Idaho and the portion of Montana west of the continental divide, see Figure 1-1. This represents an area of more than 250,000 square miles. The transmission network within this area includes 7500 miles of high voltage transmission lines, more than 300 electrical substations, and several special installations, such as the Celilo AC-DC Converter Station and the Dittmer Control Center. The total capital investment in facilities and equipment exceeds one billion dollars.

One of the important elements in this system is the Pacific Intertie which provides two 500-kv AC transmission lines and one 800-kv DC transmission line between central Oregon and northern and southern California, respectively. The Intertie permits the exchange of power between these general regions. In February 1971, the San Fernando earthquake caused severe damage to the AC-DC converter station located at the southern terminus of the 800-kv DC transmission line in Sylmar, California. This disaster caused more than twenty million dollars in damage to this one facility and the DC portion of the Intertie was rendered inoperable for nearly eighteen months.

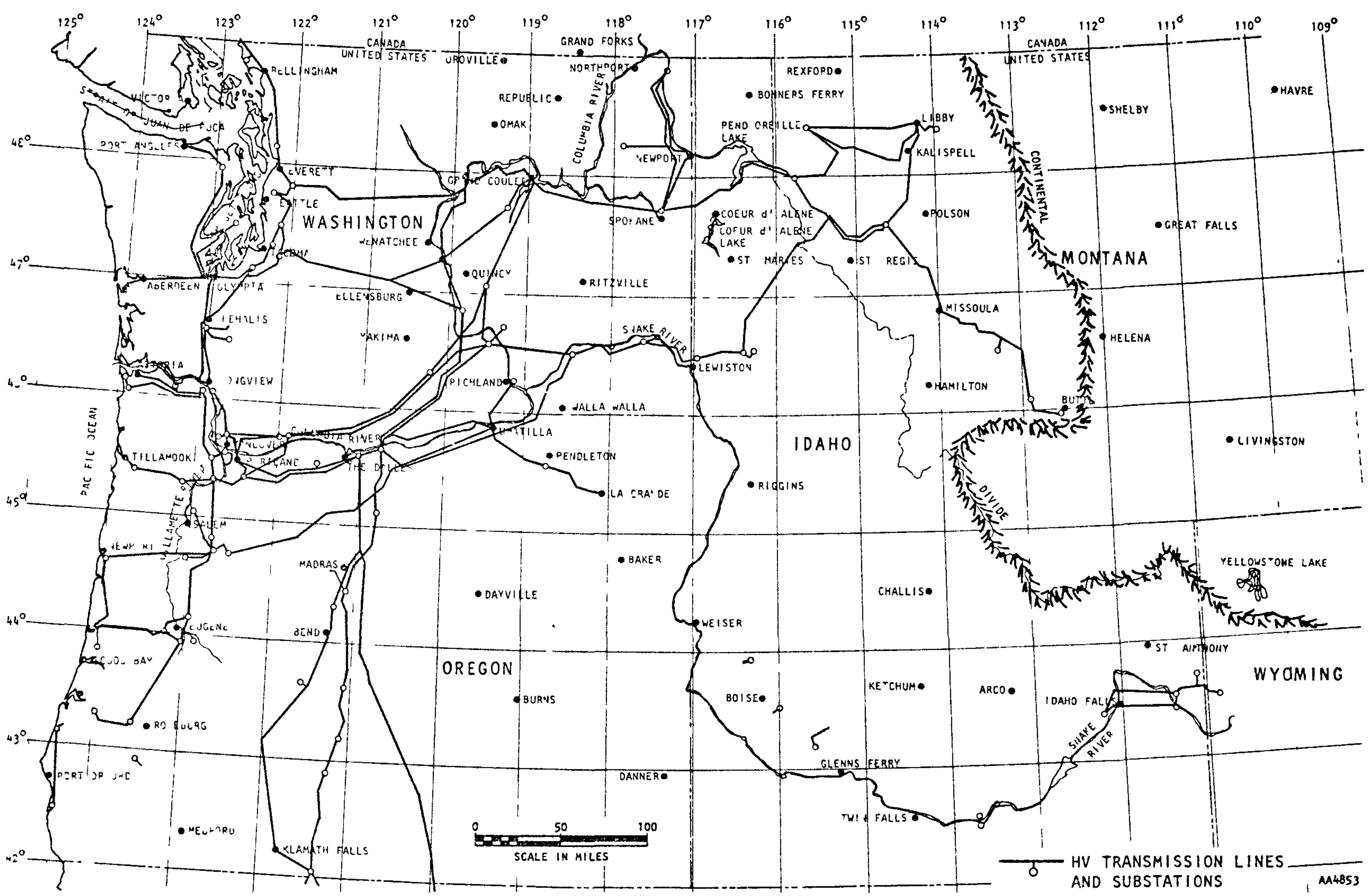


FIGURE 1-1. MAP OF BONNEVILLE POWER ADMINISTRATION SERVICE AREA

R-7211-13-2859

Immediately following the earthquake, studies were initiated to provide an assessment of the earthquake resistant design of the Celilo Converter Station and the Big Eddy and John Day AC Substations which are located in Oregon at the northern terminus of the Intertie (Reference 1-1). This was followed by studies covered in this report which provides design modification concepts required to increase the earthquake resistance level of equipment and structures at these stations. These studies required the definition of design earthquake ground motion for the Celilo-Big Eddy-John Day area, and the development of criteria for equipment classification and design modifications.

From the experience obtained during the assessment study, it became evident that other important BPA facilities should also be investigated and increased resistance to earthquake ground motion provided. This required the development of criteria earthquake ground motion applicable to the entire service area and the development of a generalized procedure for classifying equipment, developing design modifications and establishing priorities for implementing the modifications. To provide additional basic information on the susceptibility of substation equipment to seismic damage, two additional substations, Allston and Custer, were investigated. Also, because of its importance as a special installation, a separate preliminary investigation was made of the Dittmer Control Center.

Earthquake ground motion for substation design for the BPA service area is provided in Chapter 2. The procedure used to provide a seismic classification of equipment and support structures for the Celilo Converter Station and for the Big Eddy, John Day, Allston and Custer substations is summarized in generalized form in Chapter 3. Concepts for design modifications required at these stations are compared and sketches of recommended concepts are provided in Chapter 4. The results of the preliminary investigation of the Dittmer Control Center is provided in Chapter 5. Finally, a generalized procedure for developing an implementation plan for modifications required to increase the earthquake resistance level of substations in the BPA system is presented in Chapter 6. Modification priorities for the Celilo Converter Station and Dittmer Control Center, which are special installations, are also provided in Chapter 6. A brief summary of the studies reported in each of these chapters follows.

DEFINITION OF EARTHQUAKE GROUND MOTION

Chapter 2 defines earthquake ground motion for the BPA service area. It has been developed for use in procurement specifications for new equipment, and for use in developing design modifications for existing electrical equipment and support structures in the electrical substations and special installations distributed throughout the service area. Because there is a wide variation in the severity and frequency of occurrence of strong earthquake ground motion within this broad geographical region, local variations in seismic risk have been considered so that widely dispersed facilities of equal importance can be designed with comparable factors of safety.

The criteria is considered to be generally applicable to small substations. It is assumed that before applying the criteria, the services of competent geological and soils engineers will be utilized to ascertain that local instabilities, such as slope failures, soil liquefaction, and foundation faulting, will not occur within or in the immediate proximity of the sites being considered. On this assumption the criteria has been restricted to vibratory ground motion.

The approach followed in developing the earthquake ground motion criteria can be summarized as follows:

a. Significant potential epicentral areas have been defined based upon the seismic history of the region and a general consideration of known tectonic structures. The maximum earthquake magnitude that can reasonably be expected to occur in each significant potential epicentral area has been predicted by a statistical consideration of historical earthquakes of different magnitudes and a consideration of known fault conditions.

b. Ground motion intensity attenuation relationships have been developed which relate attenuated Modified Mercalli (MM) intensities with distance from each significant potential epicentral area. These curves have been developed from information obtained from past earthquakes in the BPA service area and have been used to construct isoseisms that would be radiated by the maximum earthquakes predicted for each potential epicentral area. A seismic regionalization map has then been provided which primarily uses the (MM) Intensity VIII and IX isoseisms as boundaries between zones of different levels of seismic risk.

c. Normalized (0.1 g) response spectra have been developed for three different site profile conditions. Normalized curves have then been scaled linearly to provide criteria spectra for each seismic zone. Acceleration records applicable to each site profile condition have also been identified.

d. Basic probability relationships on the occurrence of strong earthquake ground motion for the Puget Sound and Portland areas have been developed for equipment modification justification studies using the seismic history for these areas and procedures defined by Housner.

SEISMIC CLASSIFICATION PROCEDURE FOR EQUIPMENT AND SUPPORT STRUCTURES

Chapter 3 outlines the procedures used to provide a seismic classification of the electrical equipment and support structures located at the Celilo Converter Station and at the Big Eddy, John Day, Allston, and Custer substations. The objective of the classification was to identify the electrical equipment susceptible to damage from earthquake ground motion. The general procedure is illustrated by flow chart in Figure 1-2.

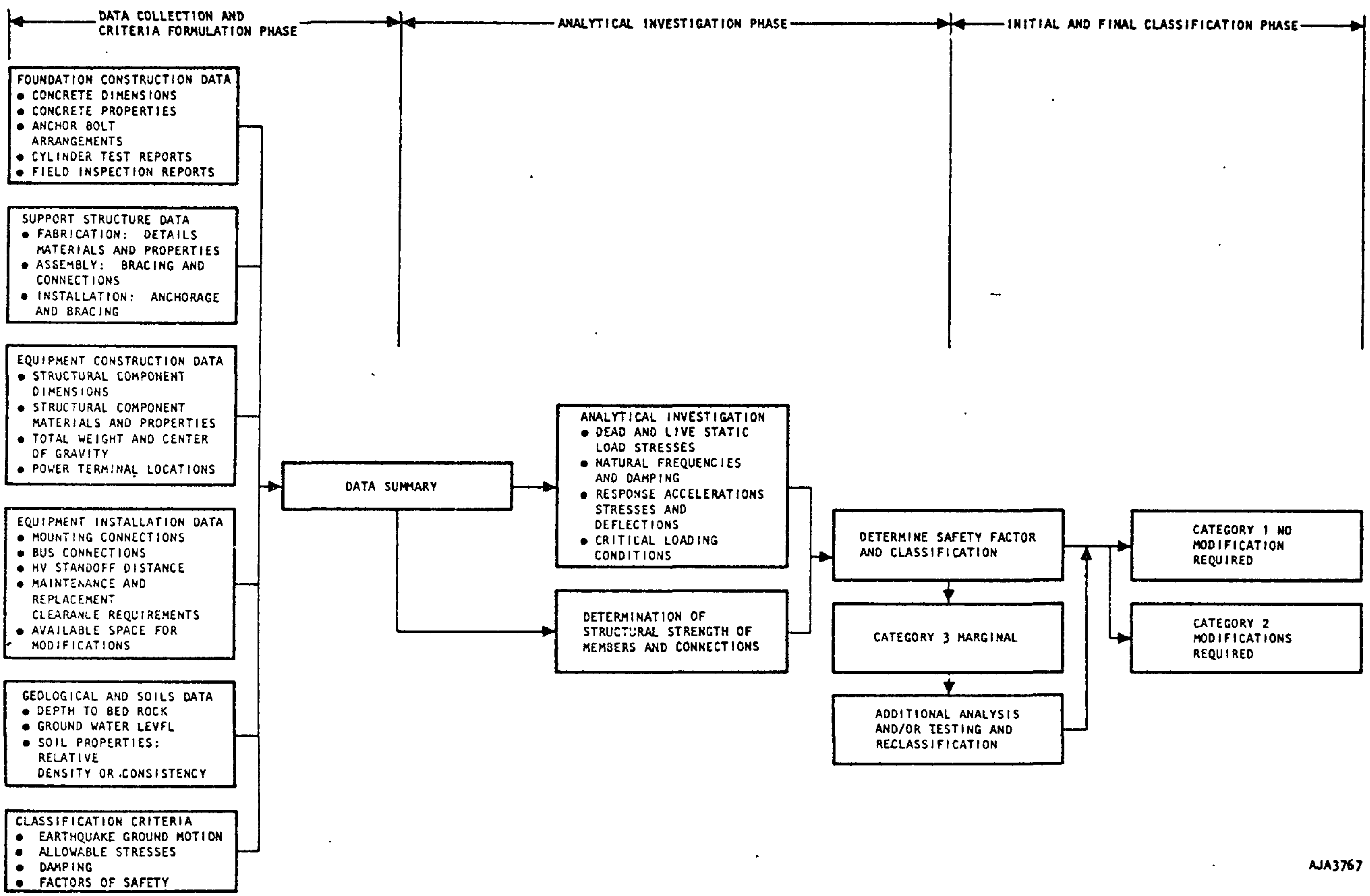


FIGURE 1-2, GENERAL CLASSIFICATION PROCEDURE

The classification procedure illustrated in Figure 1-2 is divided into three phases which are the following:

Data collection and criteria formulation

Analytical investigation

Initial and final classification

Under data collection, information concerning foundation, support structure and equipment construction and installation was first collected and reviewed. The soils data and available test information were then combined with the equipment and support information in a data summary for each item of equipment. Classification criteria was established for earthquake ground motion, material properties, allowable stresses and factors of safety during this initial phase. A dynamic structural model of the equipment was then formulated consistent with the general overall configuration to obtain an estimate of the equipment frequencies. A preliminary analysis followed using the applicable response spectrum, damping and model to obtain an estimate of the stresses in the major components. The resulting stresses or internal forces were then compared with the classification criteria and the factor of safety computed to provide an initial classification into one of the following three categories:

Category 1 - No modification required

Category 2 - Modification required

Category 3 - Marginal, additional investigation required

Items placed in Category 3 were subjected to a more detailed analysis before finally being placed in Categories 1 or 2.

Documentation of the classification criteria and of the analysis and classification procedures is provided in Chapter 3. A summary of the final equipment classification for the Celilo Converter Station and the Big Eddy and John Day Substations is provided in Table 3-1 and for the Allston and Custer Substations in Table 3-2 of Chapter 3.

DESIGN MODIFICATIONS FOR EQUIPMENT AND SUPPORT STRUCTURES

Design concepts are compared and sketches of preferred concepts are provided in Chapter 4 for the modifications needed to improve the seismic resistance of selected equipment and support structures at the Celilo Converter Station, and at the Big Eddy, John Day, Allston and Custer substations. The modifications, in general, require one of the following four approaches:

a. The system response to seismic excitation is to be minimized by changing the natural frequency, increasing the damping, or by providing shock isolation.

b. The existing support structure is to be reinforced.

c. New structural supports are to be provided to brace existing structures.

d. The existing support structures are to be replaced with redesigned structures.

In cases where the existing equipment has low damping and moderate overstress, reducing the natural frequency and increasing the damping was often found to be the simplest and most economical modification. An example, is the use of Belleville spring washers at the base of the lightning arresters which is illustrated in Figures 4-12 through 4-15 of Chapter 4. In contrast, the 500 KV Air Power Circuit Breakers (APCB) were found to be fragile and functionally more complex. As a result, shock isolation of the complete unit was recommended. Figure 4-5 illustrates an acceptable concept. An example of increasing the seismic resistance by reinforcing the support structure is illustrated in Figure 4-18 for the DC Filter Capacitor and in Figure 4-23 for the Filter Reactors.

Providing new structural supports to brace the existing support structures requires a careful consideration of the horizontal and vertical clearances with high voltage circuits and for maintenance operations. The additional bracing arrangement normally requires the use of insulator stacks used as diagonal guys or braces, or overhead insulator columns, braces or diagonal suspension strings supported by new steel framing which spans the entire equipment group. The modifications proposed for the AC Harmonic Filter Capacitor Racks in Figures 4-26 through 4-28 illustrate the latter.

Examples of complete replacement of existing structures or components with redesigned structures are provided by the new capacitor rack concepts considered but ultimately rejected for the overhead bracing concept referred to above. For new installations, a redesigned rack will prove more cost effective. The replacement of the precast concrete canopies of the Celilo Terminal Building with lighter weight fiberglass canopies is an additional example of this type of modification.

Each of the equipment items and support structures classified in Category 2 at these stations was reviewed to determine the most direct and cost-effective method of providing the required seismic resistance. Design sketches of recommended concepts were reduced and are provided in Chapter 4. Final construction drawings and specifications are to be prepared by BPA as the modifications are implemented.

SEISMIC CLASSIFICATION PROCEDURES AND DESIGN MODIFICATIONS FOR DITTMER CONTROL CENTER

A preliminary classification and some modification concepts for high priority items for the Dittmer Control Center are provided in Chapter 5. Additional study is required before final classification of all essential items can be made at this facility and modification concepts completed. This installation is a vital command communication facility requiring a different approach for earthquake assessment than that used for substation review. Because of the interdependence of subsystem elements, and a limitation on contract scope, an item-by-item analysis was not considered a realistic or practical approach. An alternate approach of assessing equipment and structures by generic class (e.g., concrete structures, heavy equipment, metal racks, etc.) was adopted in order to assess the greatest number of items as possible within the time available. Generic classes of potentially earthquake-sensitive items were then analyzed by order of their priority of importance to the Dittmer control operation.

Items initially considered for investigation and seismic classification were selected by BPA. This list identified the building power equipment (primary power, emergency power, and uninterruptible computer power) and computer equipment (cabinets and consoles) as critical items. In addition to these items, other equipment and structures were also included after a site visit and plan review. Priorities for classification were then established based on three considerations concerning the effect of earthquake damage on Control Center operation. These considerations were operational criticality, system redundancy and shock sensitivity. Based on these considerations, equipment and structures were grouped into three priorities for classification to establish the order and depth of critical review that could be applied within the scope of the contract. A summary of the results of the classification study is provided in Table 5-1 of Chapter 5.

IMPLEMENTATION PLAN FOR INCREASING EARTHQUAKE RESISTANCE LEVEL OF ELECTRICAL SUBSTATIONS AND SPECIAL INSTALLATIONS

Chapter 6 has two objectives. First it presents a procedure for developing an effective implementation plan to establish the sequence, or priority, of providing modifications to items of electrical equipment and support structures located in BPA's electrical substations. Second, it provides a priority plan for implementing the modifications described in Chapters 4 and 5 to the special installations, the Celilo Converter Station and the Big Eddy and John Day substations, and the Dittmer Control Center, respectively. These installations require a high degree of operational reliability and represent a large capital investment.

The essential steps to be followed in developing the implementation plan for substation modifications are illustrated in the flow chart in Figure 1-3. The procedure requires the collection of three separate categories of information which are operational priorities, earthquake ground motion criteria and design modification identification and cost information. The objective is to provide an economical justification of the modifications

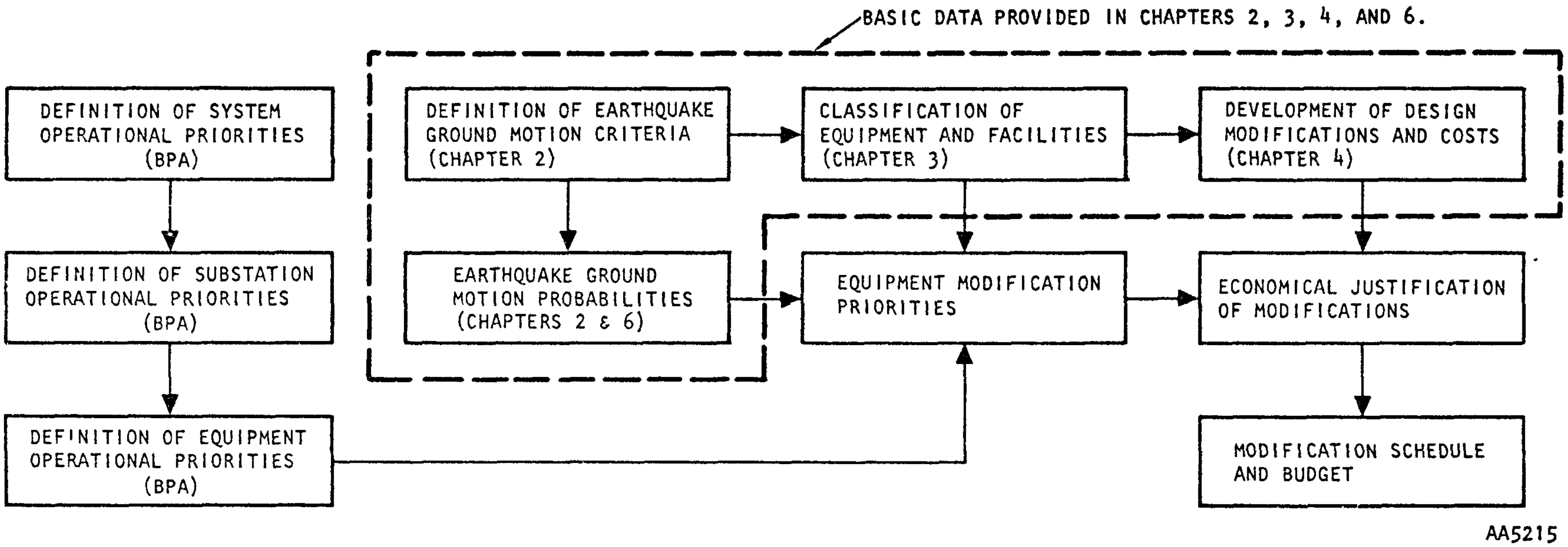


FIGURE 1-3. FLOW CHART OF IMPLEMENTATION PLAN FOR SCHEDULING MODIFICATIONS TO INCREASE EARTHQUAKE RESISTANCE LEVEL OF SUBSTATION EQUIPMENT AND SUPPORT STRUCTURES

R-7211-13-2859

prior to establishing a schedule and budget. The items given the highest priority for modification should be those which have the highest operational priority, as well as a favorable benefit to cost ratio. A discussion of the general procedure follows.

A careful systematic study of the overall transmission system is required to establish the relative operational importance of the individual items of equipment in the various substations. The first step is to establish the relative operational importance of major elements, or segments, of the transmission system. The relative operational importance of the individual substations associated with each of the major segments of the system can then be identified and finally the relative operational importance of the various items of equipment in each substation can be determined. These three steps are indicated by the left column of boxes in Figure 1-3. This information is to be provided by BPA. However, to demonstrate the procedure, operational priorities and costs were assumed in a hypothetical example presented in Chapter 6. Earthquake ground motion criteria for use in classification of equipment and in the development of design modifications and costs are provided for the entire BPA service area in Chapter 2 as indicated in Figure 1-3 in the second column of boxes. Ground motion probability data for the Puget Sound and Portland areas are also provided in Chapter 2. The scope of the contract did not permit extending the probability analysis to the entire service area. Application of the basic probability data to the hypothetical example problem is provided in Chapter 6.

Seismic classification procedures and design modifications for example substations are provided in Chapter 3 and 4, respectively. Costs of the modifications are to be provided by BPA. However, assumed costs have been used in the example problem in Chapter 6. With this basic information, equipment modification priorities and an economical justification is provided as an example for 500 KV Air Power Circuit Breakers (APCB). Based on this example, earthquake damage to the APCB's is eight times more likely to occur in Puget Sound seismic Zone C than in the John Day Zone A area, and roughly four times more likely to occur in the Portland Zone B area than in the John

Day Zone A area. The modifications, based on assumed costs were found to be economically justified in the Puget Sound Zone C area, marginally justified in Zone B, and not economically justified in Zone A. However, the costs are not factual and no final conclusions should be made from the example problem other than for the relative probability of damage in the different seismic zones.

An important step in progessively increasing the earthquake resistance level of BPA substations, is to make certain that new equipment procurement specifications include provisions for seismic resistance that are consistent with the seismic risk that can be experienced in the different seismic zones of the BPA service area. A seismic specification for new equipment procurement is discussed in Chapter 6 and provided in Appendix C-1 which can be keyed to the seismic environment defined for the different seismic zones in Chapter 2.

The Celio Converter station and the Dittmer Control Center represent special installations which require a high operational reliability and each represent a large capital investment. Equipment and structure modification priorities for these installations are provided in Chapter 6. The priorities for the items requiring modification in these special installations have been based upon a subjective rating using such factors as damage potential, functional importance, safety hazard, proximity damage potential (damage caused to adjacent equipment), cost of replacement and cost of modification. An economical justification of the modifications cannot be provided until factual data on losses from power outages resulting from loss of stations becomes available. Priorities for the Celilo Converter Station and the John Day and Big Eddy substations are provided in Table 6-6 and for the Dittmer Control Center in Table 6-7 of Chapter 6. Additional classification studies are also required for the latter.

REFERENCES

1-1. *Assessment of Earthquake Resistance Design of AC-DC Converter Stations, HV-DC Pacific Intertie"*, Agbabian-Jacobsen Associates, R-7119-1984, August 1971

CHAPTER 2

DEFINITION OF EARTHQUAKE GROUND MOTION

INTRODUCTION

This chapter defines earthquake ground motion for the Bonneville Power Administration (BPA) service area. It has been developed for use in procurement specifications for new equipment, and for use in developing design modifications for existing electrical equipment and support structures in the more than 300 electrical substations and special installations distributed throughout the service area. Because there is a wide variation in the severity and frequency of occurrence of strong earthquake ground motion within this broad geographical region, local variations in seismic risk have been considered so that widely dispersed facilities of equal importance can be designed with comparable factors of safety.

The criteria is considered to be generally applicable to small substations. It is assumed that before applying the criteria, the services of competent geological and soils engineers will be utilized to ascertain that local instabilities, such as slope failures, soil liquefaction, and foundation faulting, will not occur within or in the immediate proximity of the sites being considered. On this assumption the criteria has been restricted to vibratory ground motion which has been defined by response spectra. Acceleration records are also provided which can be scaled to comparable spectra levels.

The criteria provided in this chapter can also be used for the preliminary design of major electrical facilities in which a large investment is to be concentrated at one specific site, or for special facilities which require a high operational reliability. However, it is recommended that the criteria for final design for such facilities be supported by a detailed geological study and a careful review of the seismic history in the region of each site. The Celilo Converter Station and the Dittmer Control Center are examples of major and special facilities falling in this category.

It should be specifically noted that the criteria presented is *not applicable nor should it be considered comparable* to seismic criteria required for the design of nuclear power plants or large dams. These types of facilities represent major installations with special safety requirements that dictate an entirely different approach to criteria formulation.

The approach followed in developing the criteria can be summarized as follows:

a. Significant potential epicentral areas were defined based upon the seismic history of the region and a general consideration of known tectonic structures. The maximum earthquake magnitude that can reasonably be expected to occur in each significant potential epicentral area was predicted by a statistical consideration of historical earthquakes of different magnitudes and a consideration of known fault conditions.

b. Ground motion intensity attenuation relationships were developed which relate attenuated Modified Mercalli (MM) Intensities with distance from each significant potential epicentral area. These curves were developed from information obtained from past earthquakes in the BPA service area and were used to construct isoseisms that would be radiated by the maximum earthquakes predicted for each potential epicentral area. A seismic regionalization map was then provided primarily by using the predicted (MM) Intensity VIII and IX isoseisms as boundaries between zones of different levels of seismic risk.

c. Normalized (0.1 g) response spectra were developed for three different site profile conditions. Normalized curves were then scaled linearly to provide criteria spectra for each seismic zone. Acceleration records applicable to each site profile condition were also identified.

d. Basic probability relationships for the occurrence of strong earthquake ground motion in the Puget Sound and Portland areas were developed for equipment modification justification studies using the seismic history for these areas and procedures defined by Housner.

Discussion of each of these steps follows.

SIGNIFICANT POTENTIAL EPICENTRAL AREAS

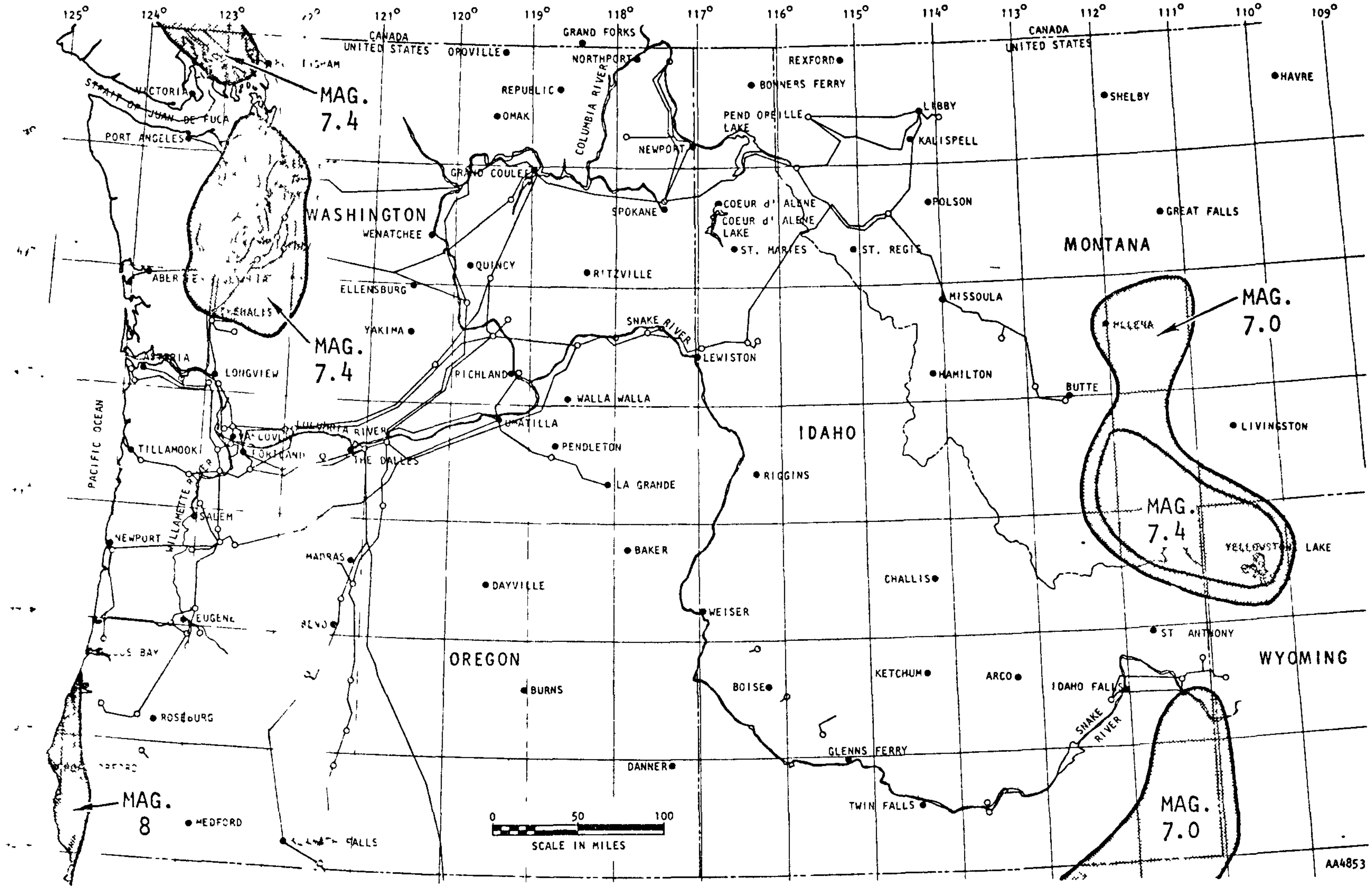
The first step in the development of the seismic regionalization map was to identify those areas which historically have experienced the largest and most frequent earthquakes. By associating this record with the geology and tectonic structures of the region, the significant potential epicentral areas were defined and the maximum earthquake magnitudes that could reasonably be expected to occur in each epicentral area were predicted. This task was performed as a separate study by Shannon and Wilson, Inc. (Reference 2-1).

The maximum earthquakes predicted for each potential epicentral area were based primarily upon a statistical consideration of historical earthquakes of different magnitudes. The maximum earthquake magnitudes predicted for each area were based on a 130 year recurrence interval. As explained in Reference 2-1, this interval was selected on the assumption that 130 years is long enough to permit strain energy sufficient to produce the largest earthquakes to accumulate and to be released. Consideration was also given to the length and characteristics of known and postulated fault structures when establishing these values. The boundary of each potential epicentral area was then defined by assuming that the maximum predicted earthquake for the area could occur along a fault or structure lineation as far out from the center of seismicity of the area as the point where the observed seismicity decreased by a magnitude interval of 2.6. The selection of this numerical value was primarily based upon judgement but gave results which were reasonable and consistent. Careful study of the work of Couch and Deacon in Reference 2-1 is suggested for more detailed information on the selection of the recurrence interval, and the delineation of the potential epicentral areas.

Potential Epicenters of Magnitude 7^{+} Earthquakes

Potential epicentral areas of Richter magnitude 7 earthquakes, or larger, are shown in Figure 2-1. The most significant epicentral areas to the BPA service area are those in the Puget Sound area where a maximum earthquake magnitude of 7.4 has been predicted. Historically, a magnitude 7.1 earthquake has been experienced in this area. Although the structural features are deeply covered with alluvium, geologic information indicates faulting in this area and fault lengths of roughly 50 miles have been postulated. Thus, the 7.4 magnitude prediction is well supported.

Off the coast of Port Orford, an epicentral area associated with the Cape Blanco fault system has been identified in Figure 2-1 for which a maximum earthquake magnitude of 8 has been predicted. This area has experienced a magnitude 7.6 earthquake about 40 miles offshore. Two additional potential epicentral areas have also been identified in Figure 2-1 which are located near the southeastern edge of the BPA service area. One extends south from near Helena, Montana to Yellowstone Lake in Wyoming. Earthquake magnitudes of 7 to 7.4 have been predicted for this area. This area has experienced a magnitude 7.1 earthquake near Hebgen Lake in 1959. The second area is located farther to the south in eastern Idaho for which a maximum earthquake magnitude of 7 has been predicted.



POTENTIAL EPICENTRAL AREAS OF MAGNITUDE 7 AND LARGER

HV TRANSMISSION LINES AND SUBSTATIONS

FIGURE 2-1. MAP SHOWING POTENTIAL EPICENTRAL AREAS OF MAGNITUDE 7 EARTHQUAKES AND LARGER (BASIC DATA FROM REFERENCE 2-1)

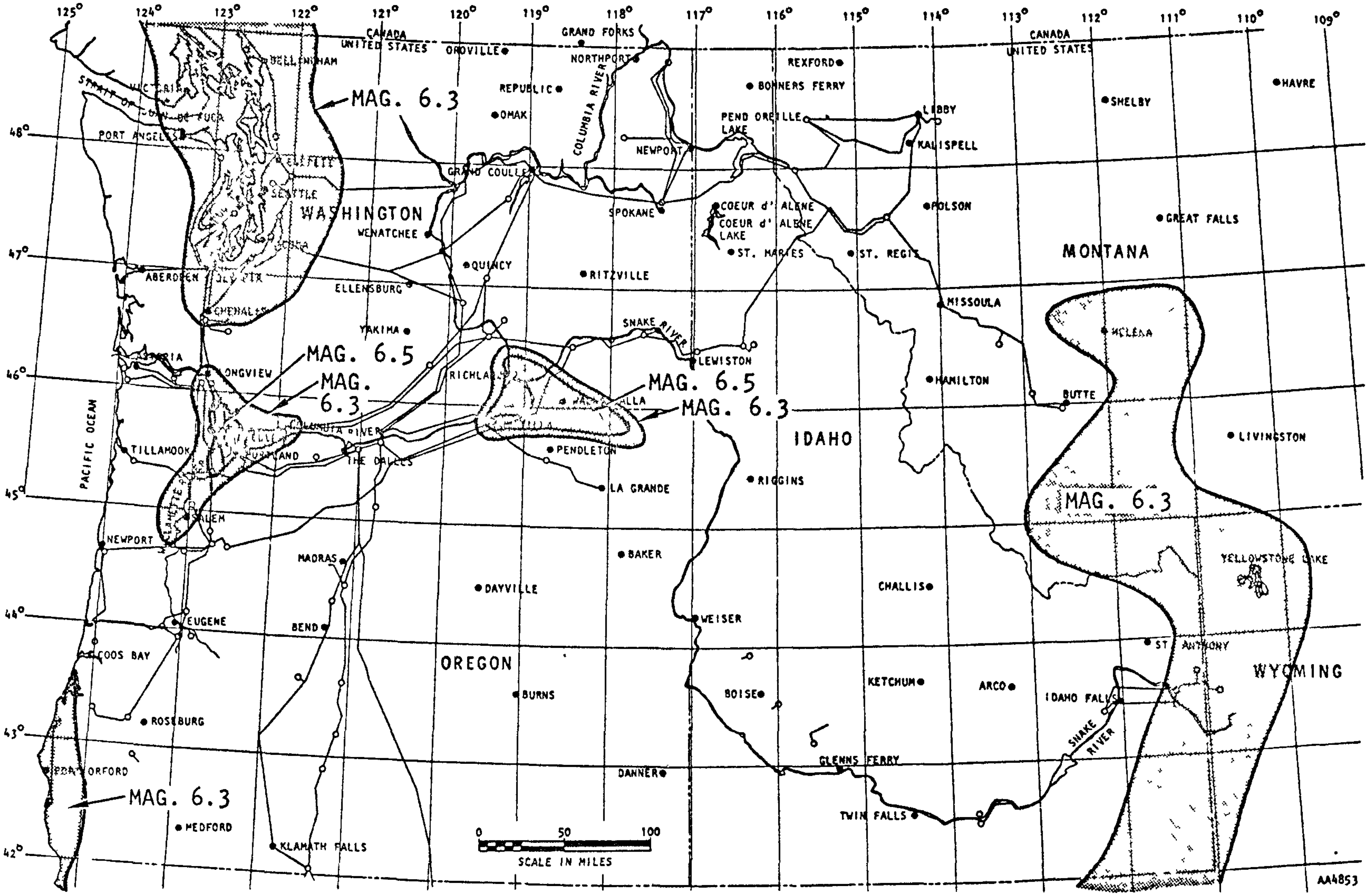
R-7221-13-2859

A review of the potential magnitude 7 epicentral areas by a Seismic Committee brought forth the opinion that there is little significant difference in the fault structures extending from just south of Kalispell, Montana to the Helena, Montana area from those identified with magnitude 7 earthquakes in the Helena-Yellowstone Lake area. This area was not included in the magnitude 7 potential epicentral area as it was not supported by the seismic history. A more conservative analysis would justify including this area.

Potential Epicenters of Magnitude 6^+ Earthquakes

Figure 2-2 indicates the potential epicentral areas in which maximum earthquake magnitudes of 6.3 to 6.5 have been predicted. The areas indicated include the Puget Sound, Cape Blanco, Idaho and western Montana areas discussed above, and two additional areas. One of the new areas is the Portland-Willamette River area and the other is the Umatilla-Walla Walla area near the confluence of the Snake and Columbia Rivers. The prediction of maximum earthquake magnitudes of 6.5 for these two additional areas is not as well defined as the predictions for the other areas and are therefore slightly more conservative. The maximum earthquake magnitudes of records for the Portland-Willamette River area is 5.2, and for the Umatilla-Walla Walla area it is 5.8. A fault length of roughly 20 miles has been postulated for the Portland-Willamette River area.

A Seismic Committee review also brought forth the opinion that the fault structures north from the Richland area along the Columbia River are not significantly different from those associated with the magnitude 6.5 earthquakes in the Umatilla-Walla Walla area. This area was not included in the potential epicentral area of Magnitude 6.5 earthquakes as it is not supported by the seismic history of the area. Again, a more conservative analysis would justify the inclusion of this area.



POTENTIAL EPICENTRAL AREAS OF MAGNITUDE 6.3 TO 6.5

HV TRANSMISSION LINES AND SUBSTATIONS

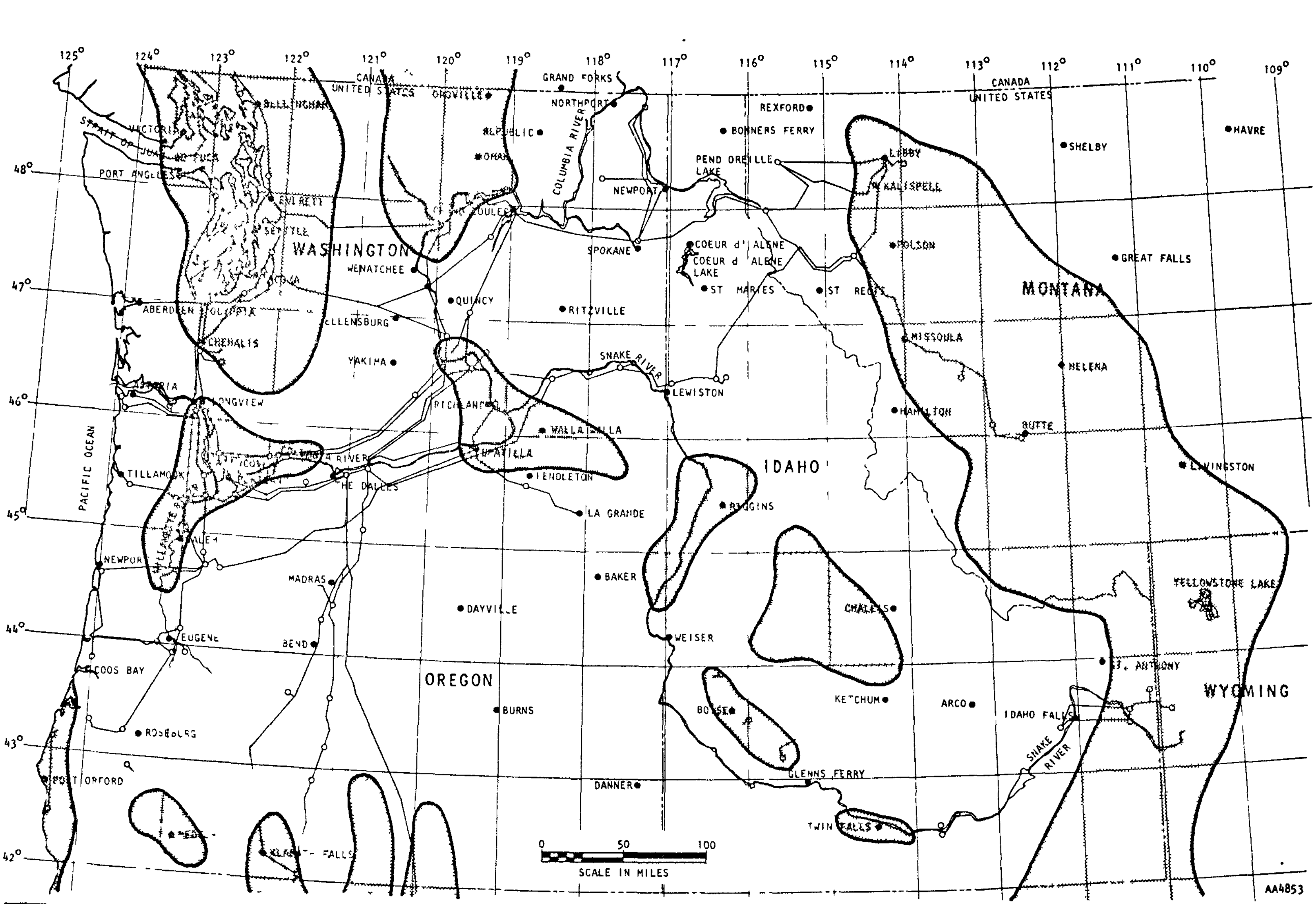
FIGURE 2-2. MAP SHOWING POTNETIAL EPICENTRAL AREAS OF MAGNITUDE 6.3 TO 6.5 (BASIC DATA FROM REFERENCE 2-1)

Potential Epicenters of Magnitude 5 Earthquakes

The potential epicentral areas indicated in Figures 2-1 and 2-2 define the six significant areas used to define the proposed seismic regionalization map for the BPA service area. There are other epicentral areas of lower magnitude earthquake potential defined in Reference 2-1. These include the potential epicentral areas of earthquake magnitude 5 shown in Figure 2-3. These areas do not need individual study, as the epicentral intensity of ground motion associated with these areas is not predicted to exceed the intensity of motion defined for the lowest seismic zone for the seismic regionalization map. This will be discussed in more detail later. It can be noted in Figure 2-3 that the area northwest of Helena to the Kalispell area and an area north of Richland have been included as potential epicentral areas of magnitude 5 earthquakes. These are areas which the Seismic Committee has indicated have geologic structures capable of producing larger magnitude earthquakes.

SEISMIC INTENSITY ATTENUATION RELATIONSHIPS

In order to establish zones of different seismic risk, the intensity of earthquake ground motion in the potential epicentral areas noted in Figures 2-2 and 2-3 were estimated, and attenuation relationships were established with distance from the potential epicentral areas. Unfortunately, prior to 1962, few instruments that could record the strength of motion quantitatively (i.e., peak ground acceleration) were deployed in the Pacific Northwest and there is little local data upon which to base quantitative attenuation curves. In the absence of better information, MM Intensity data were used to develop intensity attenuation relationships. The procedure followed is explained in Reference 2-1 as follows:



POTENTIAL EPICENTRAL AREAS OF MAGNITUDE 5 EARTHQUAKES

HV TRANSMISSION LINES AND SUBSTATIONS

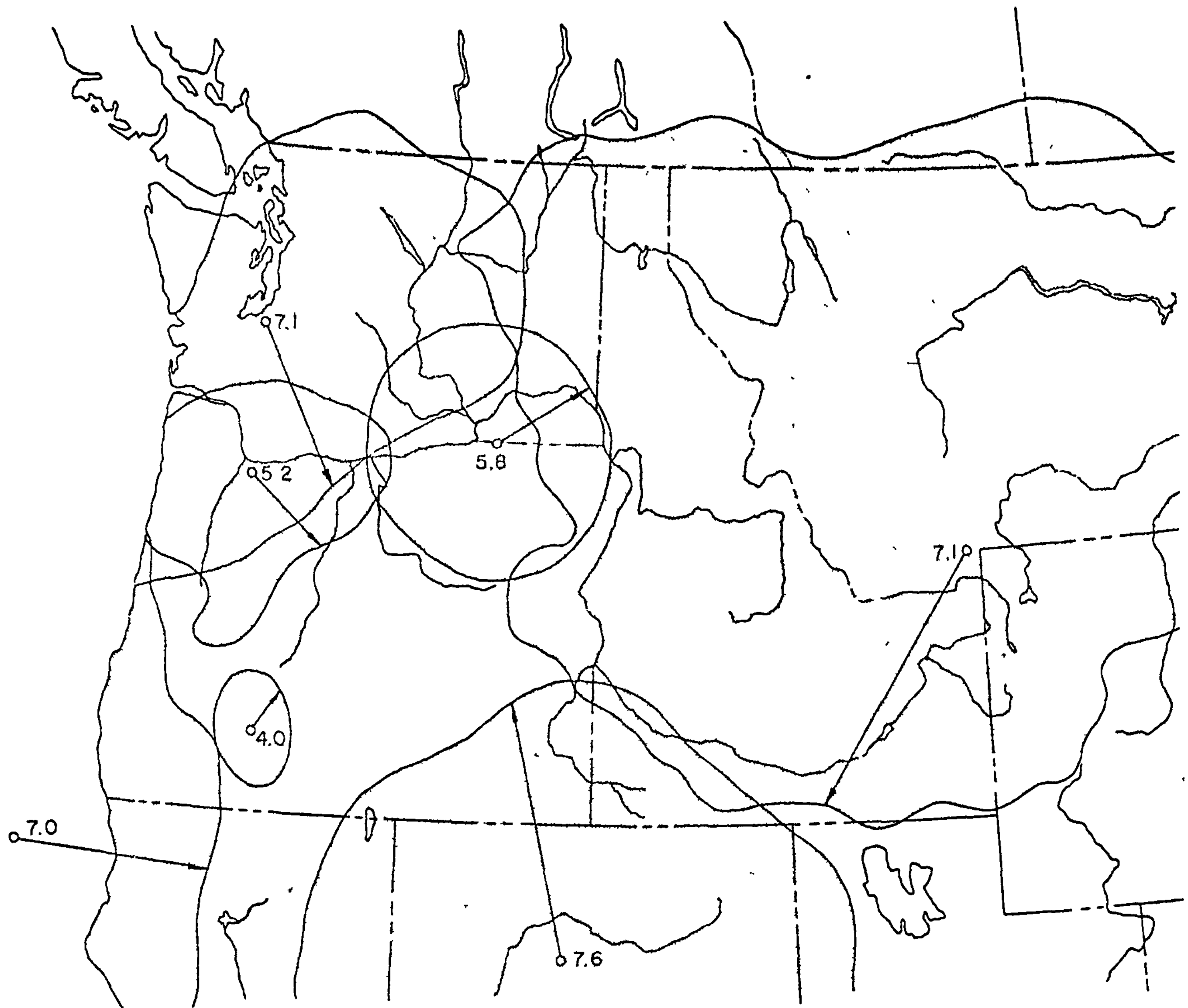
FIGURE 2-3. MAP SHOWING POTENTIAL EPICENTRAL AREAS OF MAGNITUDE 5 EARTHQUAKES (BASIC DATA FROM REFERENCE 2-1)

> Seismic energy is not uniformly radiated from the source; in addition, variations in attenuation characteristics along the ray paths of the propagating waves cause non-uniform attenuation, hence, the surface excitation pattern is not symmetric and in fact is very complex and difficult to predict. Figure [2-4] shows the asymmetry in the surface excitation pattern of several earthquakes in the BPA Service Area. Figure [2-5] illustrates the changes in earthquake intensity as a function of distance....,for the following earthquakes; Strait of Georgia, British Columbia, 1946; Olympia, Washington, 1949; Tacoma, Washington, 1965; and Portland, Oregon, 1962. The ends of the vertical bars, marked by appropriate symbols, indicate the maximum and minimum distance at which a given intensity was observed. A similar symbol between the ends of the bars indicates a length of the radius of a circle which best fits the area of observed intensity. A smooth curve is drawn through the symbols which indicate the length of the radii to obtain the shape of the attenuation curves. The curves indicate that although attenuation varies greatly with azimuth the general shape of the curves is similar for these earthquakes all of which occurred west of the Cascade Range. ..... Figure [2-6] shows the attenuation curves applicable to the region west of the Cascade Range. ..... Further analysis shows a significant difference in attenuation characteristics between sources within Puget Sound and sources outside Puget Sound, but west of the Cascade Range. Consequently a second set of attenuation curves was derived solely for the Puget Sound region. ....., Similarly, attenuation curves were derived for southwest Oregon.....for seismic waves originating off the coast primarily in the vicinity of the margin of the continental slope. Attenuation curves were also derived for the Basin and Range area.....and for the area east of the Cascade Range.

Reference 2-1 should be consulted for more detailed information on the intensity attenuation relationships.

SEISMIC INTENSITY AND REGIONALIZATION MAPS

Seismic intensity maps were constructed (Reference 2-1) by postulating that the epicenters of the predicted maximum earthquake could occur anywhere along a fault or structural lineation within each potential epicentral area. The empirical Gutenberg-Richter magnitude-intensity relationship, $I = 3/2\ (M-1)$, was used to estimate the epicentral intensities. Here, I is MM Intensity and M is the Richter Magnitude. The appropriate intensity attenuation curves discussed above were then used to construct attenuated



(FROM REFERENCE 2-1)

FIGURE 2-4. INTENSITY IV ISOSEISMAL FOR MAJOR EARTHQUAKES IN PACIFIC NORTHWEST

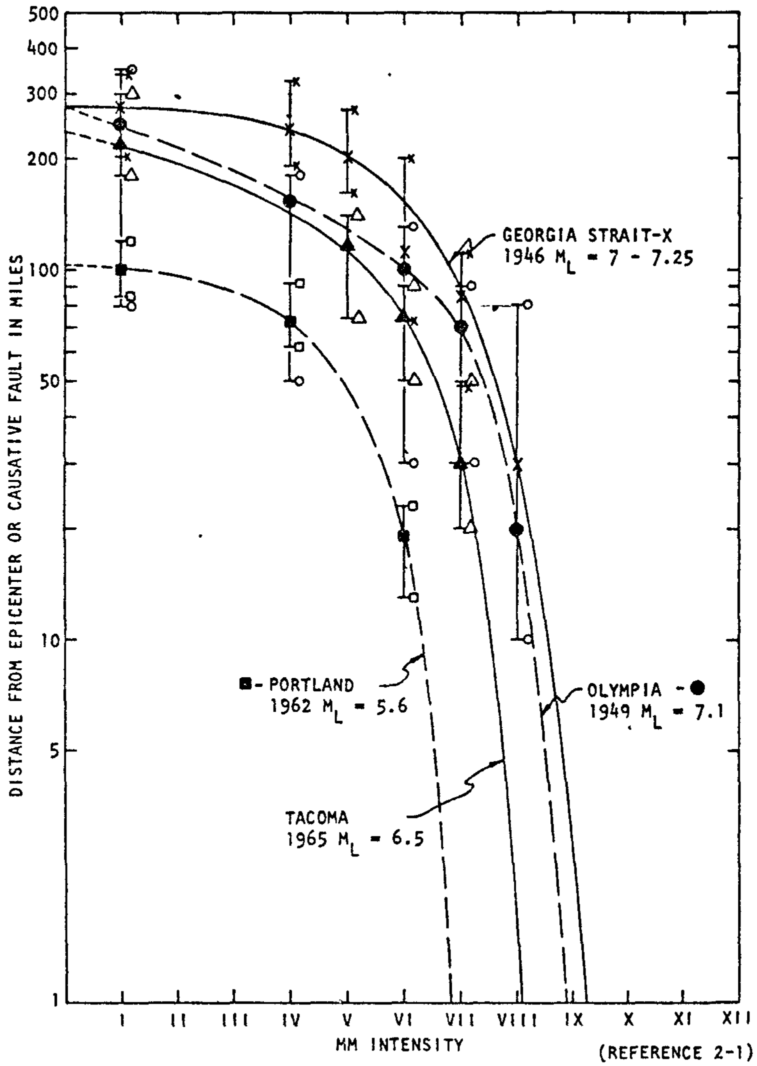


FIGURE 2-5. OBSERVED AVERAGE INTENSITY ATTENUATION CURVES FOR EARTHQUAKES WEST OF CASCADES

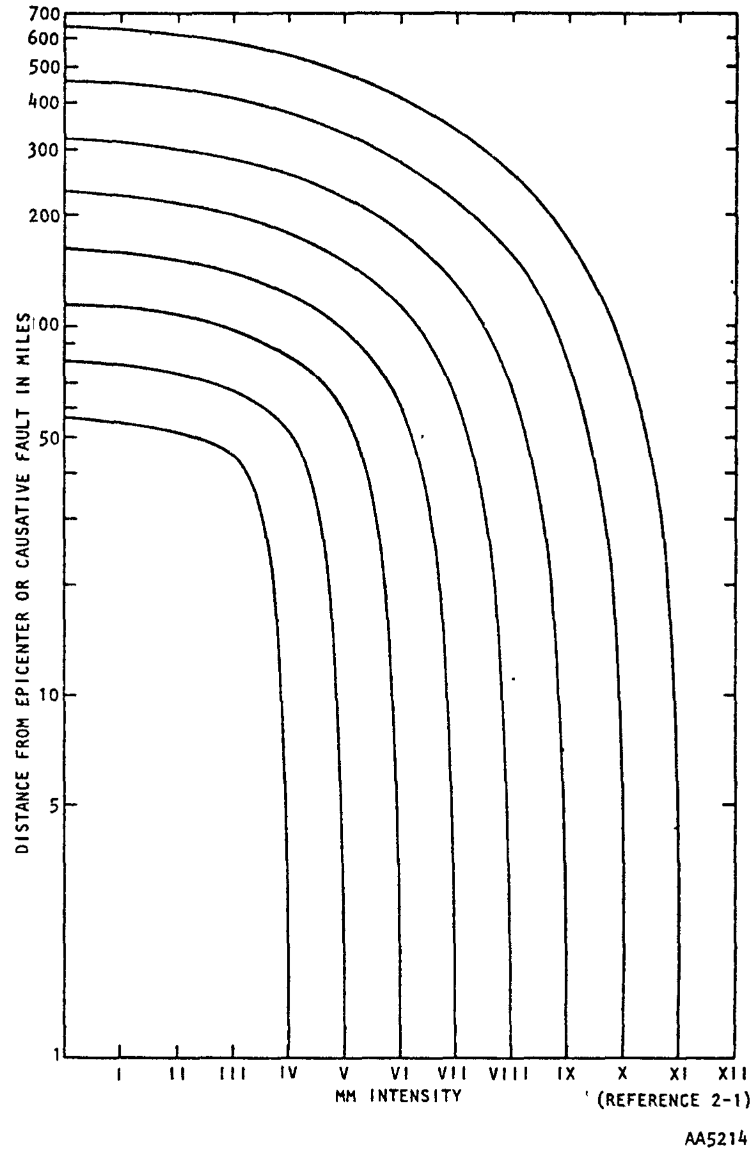


FIGURE 2-6. EARTHQUAKE INTENSITY ATTENUATION CURVES FOR REGION WEST OF CASCADE RANGE (EXCLUSIVE OF PUGET SOUND)

isoseismal curves and provide seismic intensity maps based on the potential epicentral areas shown in Figures 2-1 and 2-2. The seismic regionalization map shown in Figure 2-7 was constructed from a composite seismic intensity map by using the Intensity VIII and IX isoseismal lines as boundaries between three zones of different seismic risk. These zones were defined as Zones A, B and C. Zone D was defined in a slightly different manner and will be discussed later.

CONVERSION OF MM INTENSITIES TO QUANTITATIVE DATA

The isoseisms used as zone boundaries for the seismic regionalization map have assigned MM Intensities which are qualitative ratings based on damage experienced from past earthquakes. Brief descriptions of the damage levels identified with these intensities are as follows:

> Intensity VIII: Panel walls thrown out of frames; fall of walls, monuments, chimneys; sand and mud ejected.
>
> Intensity IX: Buildings shifted off foundations; cracked, thrown out of plumb; ground cracked; underground pipes broken.

Reference 2-2 indicates that the most severe damage to electrical equipment occurring in California during the 1952 Kern County and 1971 San Fernando earthquakes occurred in areas experiencing Intensity VIII to XI ground motion. Moderate damage occurred in areas experiencing Intensity VII to VIII ground motion. Based on these experiences, electrical equipment as presently designed could experience severe damage from strong earthquake ground motion in Zones B, C, and D in the BPA service area, see Figure 2-7, and moderate damage in much of Zone A. This information is useful and qualitative, but it is necessary to convert the intensity levels to a quantitative basis in order to define ground motion criteria for design. While there is no single ideal quantitative indicator of the strength of motion, peak horizontal ground surface acceleration is the most accepted, or conventional, indicator and has been used in this study. There is some

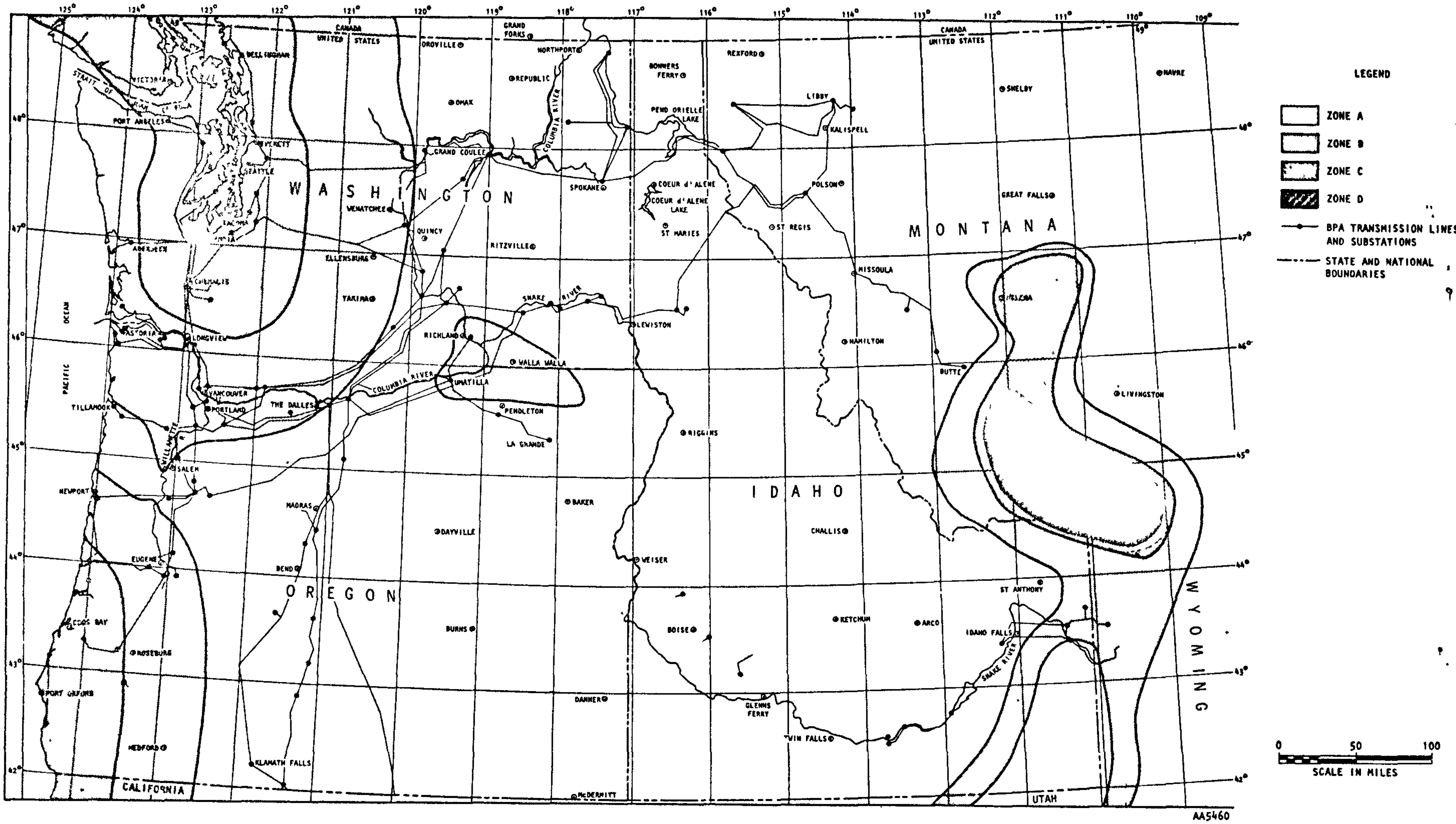
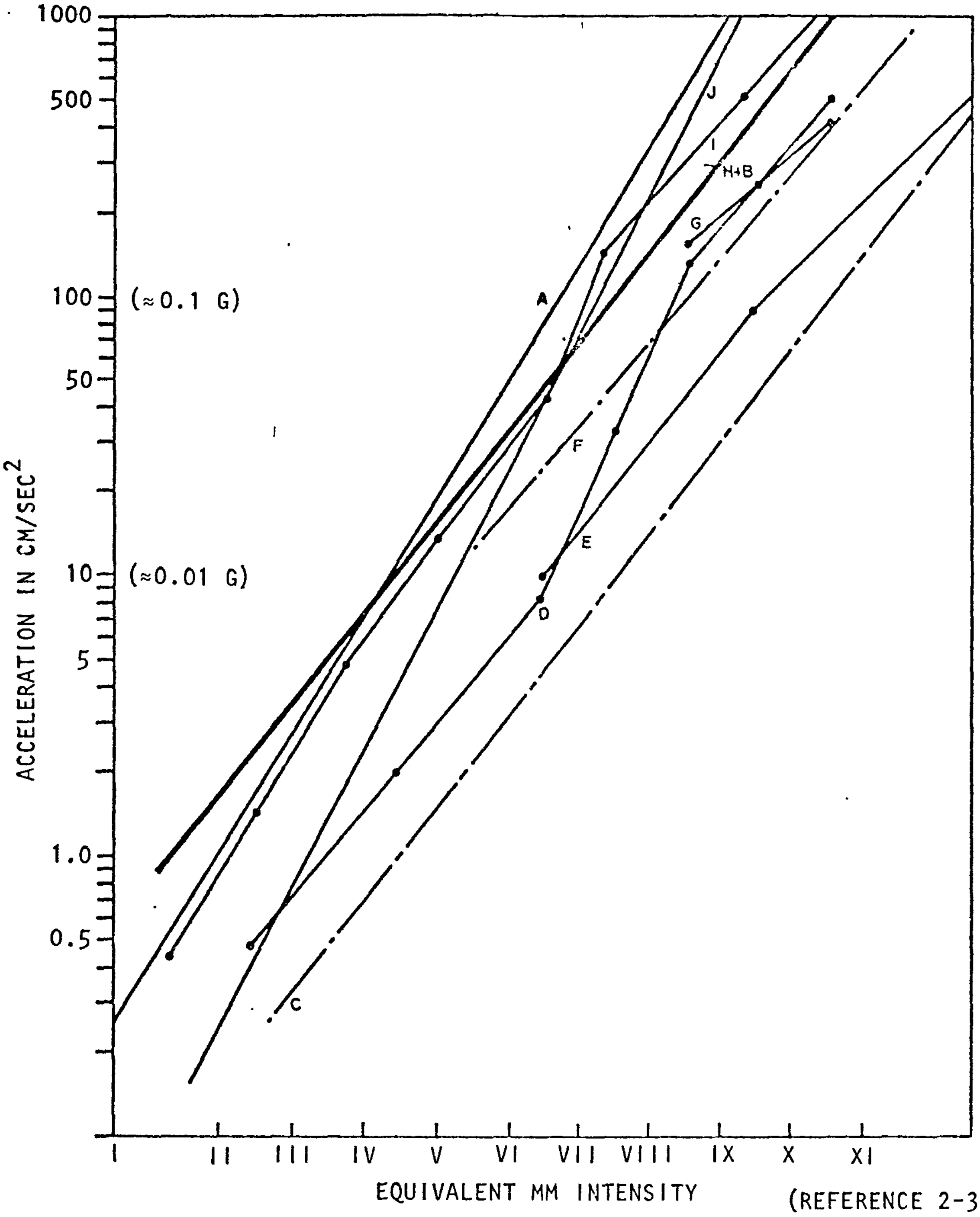


FIGURE 2-7. SEISMIC RISK MAP FOR BONNEVILLE POWER ADMINISTRATION SERVICE AREA (BASIC DATA FROM REFERENCE 2-1)

uncertainty associated with making this conversion and considerable judgement was exercised. A discussion of the problems considered follows.

There are many published correlations which relate MM Intensities to peak horizontal ground acceleration from which a wide variation in results can be obtained. Figure 2-8 illustrates some of the published correlations. The wide variations result from the fact that the MM Intensity rating is affected by many factors other than peak horizontal ground surface acceleration. The quality, age and type of building construction, the soil conditions and the difference in the reaction of individual observers to the earthquake and resulting damage all exert an influence on the MM Intensity rating. The influence of soil conditions is clearly shown in Figure 2-4 by the shape of the MM Intensity IV isoseism for the 1962 Portland earthquake. In this Figure, the MM Intensity isoseism extends an abnormal distance south of Portland along the Willamette River valley. Obviously, the deeper river bottom sediments experienced motions that were more damaging than were experienced by the more firm soil profiles lying east and west of the valley, but the same distance from the causative fault. Even though the bedrock in both zones may have experienced the same peak horizontal acceleration, the ground surface in the river valley probably experienced a lower peak acceleration than points on firm ground. However, greater peak ground surface velocities and displacements probably resulted at the lower frequencies in the river valley than on firm ground and these rather than peak ground acceleration were probably the cause of damage in the river valley area. Thus, ground motion characteristics other than peak horizontal ground surface acceleration helped establish the zone of damage in this region. This localized effect of the soil conditions, however, was moderated by using smoothed intensity radiation curves based on equivalent circular radiation patterns as discussed in the above section.



(REFERENCE 2-3)

A - HERSHBERGER (1956)
B - GUTENBERG AND RICHTER (1942)
C - CANCANI (1904)
D - ISHIMOTO (1932)
E - SAVARENSKY AND KIRNOS (1955)
F - MEDVEDEV ET AL (1963)
G - N.Z. DRAFT BY-LAW
H - TID - 7024 (1963)
I - KAWASUMI (1951)
J - PETERSCHMITT (1951)

FIGURE 2-8. EARTHQUAKE INTENSITY-ACCELERATION RELATIONSHIPS

At the range of the MM Intensity VIII and IX isoseisms from the potential epicentral areas, it was concluded that the Gutenberg-Richter correlation would be used to convert intensity to peak horizontal ground surface acceleration since it is based on a broad correlation of data. This relationship is indicated by Curve B in Figure 2-8 and has the equation

$$I = 3 \log a + 1.5$$

Here, I is the MM intensity and (a) is the peak horizontal ground surface acceleration in centimeters per second squared. On this basis, a peak horizontal ground surface acceleration of 0.32 g was predicted for the range indicated by the boundary between Zones B and C (MM Intensity IX), and 0.15 g was predicted for the range indicated by the MM Intensity VIII isoseism. In order to simplify later scaling relationships, the boundary between Zones A and B was shifted slightly towards the epicentral areas to indicate a range where a peak horizontal ground surface acceleration of 0.16 g would be predicted. On this basis, Zone C was defined to be the zone where a peak horizontal ground surface acceleration of 0.32 g, or greater, can be expected and Zone A was defined to be the zone in which 0.16 g should not be exceeded.

Consideration was next given to the probable variations in peak horizontal ground surface accelerations to be expected in each zone. For Zone A, consideration was given to the fact that potential epicenters of magnitude 5 earthquakes were not considered in defining the boundaries of the seismic regionalization map. Figure 2-3 indicates significant areas of Zone A could experience magnitude 5 earthquakes. A detailed study of the Spokane area (Reference 2-4) which is in Zone A revealed that MM Intensity VI ground motion has been experienced and that MM Intensity VII is potentially possible. These are consistent with magnitude 5 earthquake epicentral intensities. A value of 0,08 to 0.10 g would be consistent for peak horizontal ground surface accelerations occurring in the epicentral areas of magnitude 5 earthquakes, although values as high as 0.15 g have been indicated by Housner (Reference 2-4). On this basis, peak horizontal ground surface accelerations in Zone A were concluded to vary from about 0.08 g to 0.16 g with an average for Zone A being about 0,12 g.

The probable variations in peak horizontal ground surface acceleration in Zones B and C were defined by considering the possible epicentral accelerations of large magnitude earthquakes. This was difficult to quantify as peak ground surface accelerations in these regions are significantly affected by local factors such as the depth and type of faulting and the depth and type of soil profile present. There is also very little quantitative information available in the region close in to the epicenter and scaling relationships such as those by Gutenberg-Richter are less reliable in this region. More consideration, therefore, was given to local geology and soils conditions than present scaling relationships.

In the Oregon and Washington Zone B and C areas, the earth's crust is apparently experiencing a dilational effect so that normal and strike-slip faulting predominates rather than reverse thrust faulting (Reference 2-1). Also, in the Puget Sound area (Zone C) the epicenters have been deeper (45 miles) than for most California earthquakes (7 to 15 miles) and faulting has not penetrated to the ground surface. In both the Puget Sound and Portland (Zone B) epicentral areas, the bedrock is at depth and in general is overlain with deep sediments. In the Port Orford area (Zone C), the major fault structures are offshore and in the Walla Walla-Umatilla area (Zone B) the major energy release has been beneath the deep basaltic flows which are several thousand feet thick. Based upon these considerations, as well as the judgement and opinions of a Seismic Committee, a peak horizontal ground surface acceleration of 0.48 g was assigned to the Zone C regions in the Puget Sound area and to the on shore Port Orford area. Peak horizontal ground surface accelerations in these two Zone C areas were, therefore, assumed to vary from 0.48 g to the zone boundary value of 0.32 g. Since the areas experiencing the highest values in a given earthquake are relatively small, the average value for these two Zone C areas was assumed to be 0.36 g. Based on these same considerations, it was concluded that the peak horizontal ground surface accelerations in the Portland and Walla Walla-Umatilla Zone B epicentral areas should not exceed 0.32 g, and the average Zone B values should be about 0.24 g.

In the active seismic zone extending to the northwest from the Yellowstone Lake area, reverse thrust faulting is prevalent which indicates that this area is subjected to compressional rather dilational tectonic effects. Faulting in this region also penetrates to the ground surface. On this basis a Zone D was defined in which the maximum peak horizontal ground surface acceleration was considered to be between 0.6 to 0.7 g over a limited area with an average peak acceleration of 0.48 g. The maximum and average peak horizontal ground acceleration in Zones B and C in this region were taken to be the same as discussed above for the Oregon and Washington Zones. It should be noted that a more conservative approach to this region would justify extending these three zones farther to the northwest towards Kalispell based on the fault structures present.

The maximum and the average peak horizontal ground surface accelerations in the four zones defined have been summarized in Table 2-1 for easy reference. Scaling values for response spectra will be selected from this table in later discussions.

TABLE 2-1. SUMMARY OF PEAK HORIZONTAL GROUND SURFACE ACCELERATION

Seismic Zone	Peak Horizontal Ground Surface Acceleration (g)	
	Maximum	Average
A	0.16	0.12
B	0.32	0.24
C	0.48	0.36
D	0.64	0.48

NORMALIZED RESPONSE SPECTRA

The next step in the study was to define earthquake ground motion for each seismic risk zone indicated in Figure 2-7 by the use of response spectra for single degree of freedom oscillators. There are three general procedures for developing response spectra that do not involve detailed mathematical modeling and analysis of the soil profile. Two do not give direct consideration to effects of different site soil conditions on the response spectra. These are procedures developed by Housner (Reference 2-5) and by Newmark and Hall (Reference 2-6) and both have had wide acceptance. A third procedure is to utilize a smoothed (average) curve derived from two or more spectra from earthquake records of sites having site soil conditions similar to the site being considered. Since it was considered desirable to investigate the effect of variations in soil profile conditions on the response spectra before selecting criteria spectra, the third procedure was adopted. Three general site profile conditions were selected for study which have been characterized as Hard, Firm and Deep Soil sites.

Since response spectra for single degree of freedom isolators theoretically converge to the peak input (ground) acceleration at the higher frequencies (i.e., say above 20 to 30 cps), it was convenient to first work with response spectra normalized to a 0.1 g peak ground surface acceleration and then to scale the normalized spectra linearly by values of peak ground surface acceleration selected for each seismic zone to obtain criteria spectra. A discussion of the development of normalized spectra for each generalized site condition will be presented first, followed by the selection of zone scaling factors.

Hard Sites

Hard sites were defined as being rock sites, or rock overlain by not more than 15 to 25 ft of soil. The S80E component of the Golden Gate record of March 1957 and the N21E component of the Lake Hughes Array 12 record of February 9, 1971 were two of the records selected as characteristic of hard sites. The Golden Gate record is for a magnitude 5.3 earthquake and resulted from a strike-slip fault movement at a distance of about 8 miles from the recorder. The Lake Hughes record resulted from a magnitude 6.3 earthquake that was associated with reverse thrust faulting. The fault distance in this case was about 25 miles. The site of the Golden Gate recorder is recognized as a rock site while the Lake Hughes Array 12 recorder is located on shallow alluvium (less than 10 ft) overlying sandstone. Response spectra for 5 percent critical damping normalized to 0.1 g peak horizontal ground acceleration are shown in Figure 2-9. The similarity of the two records is striking. It is noted that there is no significant difference in the two spectra that can be associated with the type of faulting. A smooth (average) curve for these two records is also shown in Figure 2-9. Since these two records were associated with recorders that were considerable distance from the fault, and did not exhibit large amplitude vibratory displacements, it was considered desirable to have a third curve for hard sites which could be associated with such motions. The S16E component of the Pacoima record for the February 9, 1971 earthquake, normalized to 0.1 g peak horizontal ground acceleration, was selected and plotted in Figure 2-9. This is a rock site. The fault surface for the February 9, 1971 earthquake passed beneath the Pacoima recording station at a distance of 2 to 3 miles and large amplitude vibratory displacements were experienced at the recorder. A smooth curve has been drawn relative to the Pacoima curve in the low frequency range of the spectrum as representative of the response that might be anticipated on hard sites which experience large amplitude displacements. Its position was also determined by considering the relative position of smooth spectra developed for other site profile conditions which follow.

Firm Sites

Firm sites were characterized as sites having up to 200 ft of dense cohesionless materials, or stiff clays overlying bedrock. Response spectra selected for firm sites were the N-S component of the El Centro May 18, 1940 record, and the S69E component of the Taft July 21, 1952 record. Response spectra normalized to 0.1 g peak horizontal ground acceleration are shown in Figure 2-10 for 5 percent of critical damping. The El Centro recorder was approximately six miles from a zone where strike-slip faulting occurred, while the Taft recorder was approximately 35 miles from the zone of reverse

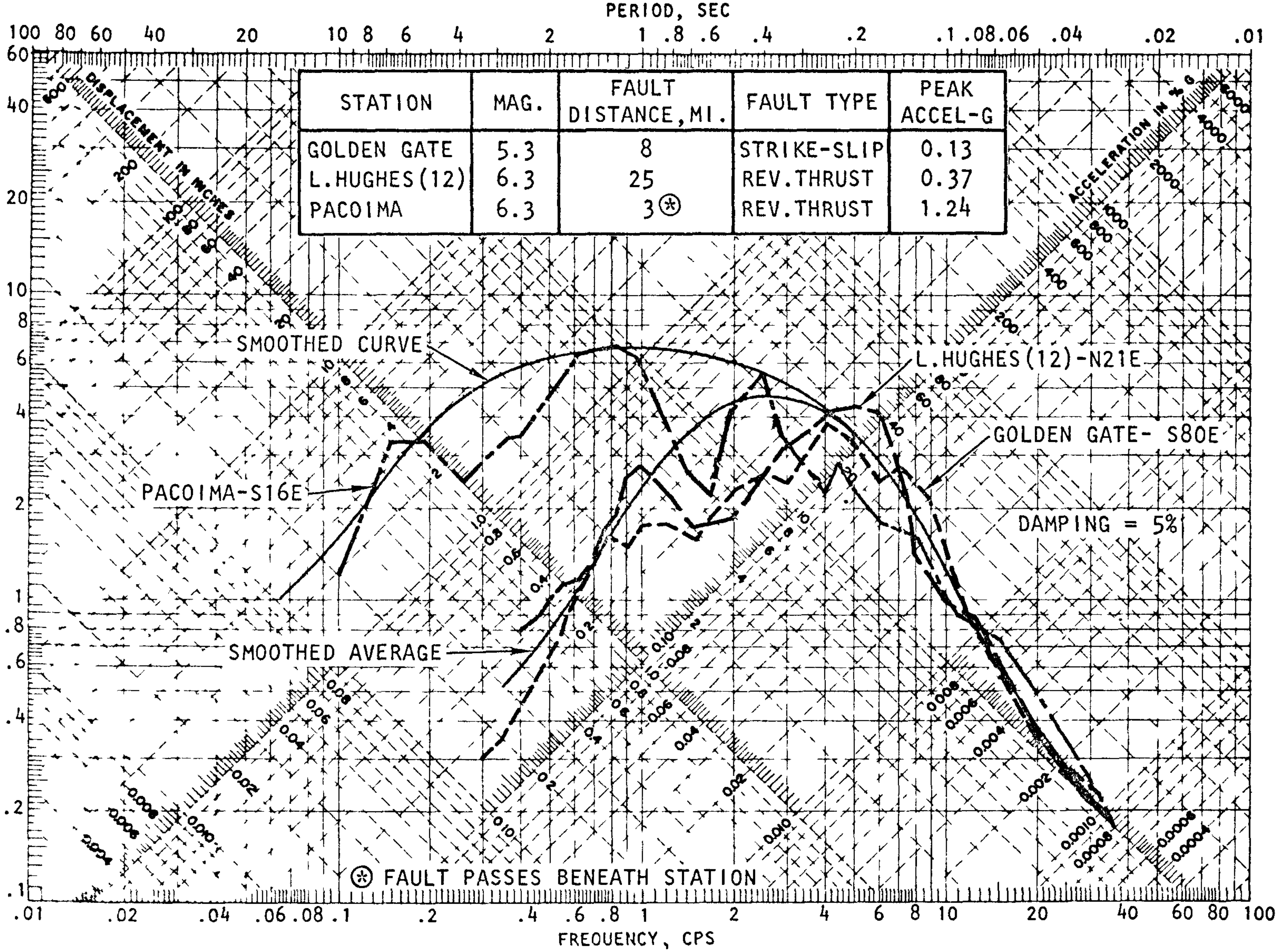
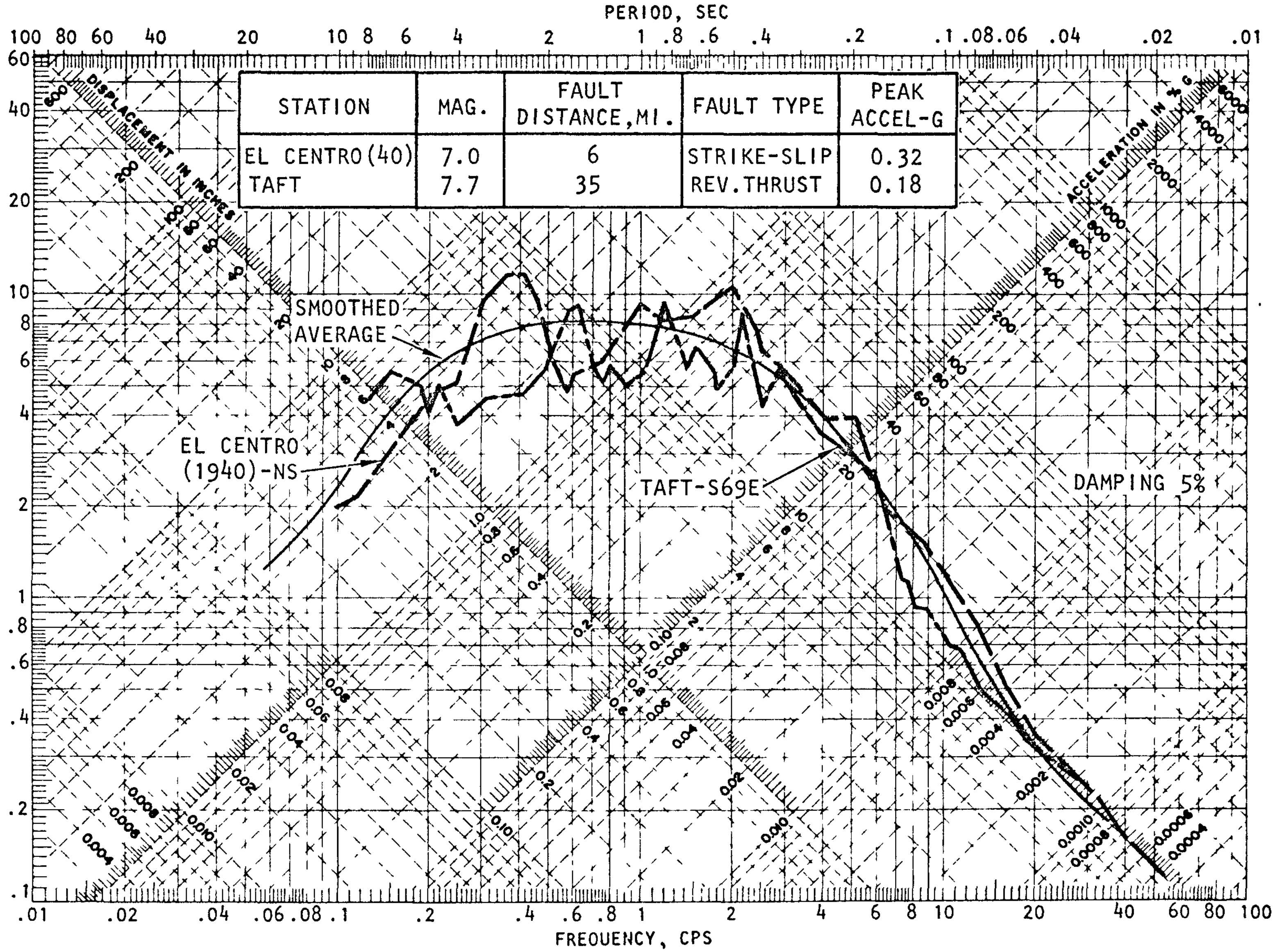


FIGURE 2-9. COMPARISON OF A HORIZONTAL RESPONSE SPECTRA FROM HARD SITES NORMALIZED TO 0.1G PEAK GROUND SURFACE ACCELERATION



STATION	MAG.	FAULT DISTANCE, MI.	FAULT TYPE	PEAK ACCEL-G
EL CENTRO (40)	7.0	6	STRIKE-SLIP	0.32
TAFT	7.7	35	REV. THRUST	0.18

FIGURE 2-10. COMPARISON OF RESPONSE SPECTRA FROM FIRM SOIL SITES NORMALIZED TO 0.1G PEAK HORIZONTAL GROUND SURFACE ACCELERATION

thrust faulting. The El Centro site profile is described in Reference 2-7 as having approximately 60 ft of clay near surface, underlain by about 40 ft of silty clay loam, below which there is more than 1000 ft of Pleistocene gravel, clay and sand. The seismic velocity for the top 60 ft is indicated to be about 1200 ft/sec. Below 60 ft the seismic velocity is indicated to be 5900 ft/sec, or higher. The soil profile of the Taft site is also indicated in Reference 2-7. The first 40 ft is indicated to be silt and sandy loam with a seismic velocity of about 1200 ft/sec. Below 40 ft the seismic velocity is postulated to be about 5000 ft/sec and the material is thought to be the Tulare formation of the upper Pliocene but factual data below 40 ft is not available. Again it will be noted that there is good agreement between the two normalized response spectra and there is no apparent significant difference resulting from strike-slip or reverse thrust faulting. The smooth (average) curve is also shown in Figure 2-10. It should also be noted that the earthquake magnitudes considered for the firm sites are significantly larger than considered for the hard sites. Seismologically there should be a difference in wave length in the resulting ground motion. From an engineering point of view this may have caused the maximum response accelerations to occur at a slightly lower frequency for the firm sites. However, this shift in frequency could also have resulted from the deeper soil profile.

Deep Soil Sites

Deep soil sites were defined as sites having more than 200 ft of cohesionless materials or soft clays overlying bedrock. The north-south components of the Hollywood Storage Building record of the July 21, 1952 Kern County earthquake, and the Orion (Holiday Inn) record from the San Fernando earthquake of February 9, 1971 were used to provide response spectra for deep soil sites. At both sites, saturated sand several hundred feet deep exists. Soil conditions at the Hollywood Storage Building site are described in Reference 2-7. Sandy clay, gravel and sand are indicated for a depth of about 300 ft. Reference 2-8 indicates that the Orion site has more than 600 ft of saturated sands and silts. Response spectra for 5 percent of critical damping normalized to 0.1 g peak horizontal ground acceleration are shown in Figure 2-11. It should be noted that the fault distance for one record is only 8 miles but for the other it is 75 miles. Also, the peak horizontal acceleration in one case was 0.045 g but for the other it was 0.27 g. However, when both records were normalized to 0.1 g, the resulting spectra compare closely. This probably would not have resulted for two such distinctly different fault distances had the comparison been made for hard or firm sites. It is also probable that the Hollywood Storage Building site is a more firm site than the Orion site and that the filtering effect of distance on the bedrock motion resulted in a spectra comparable to a deeper soil site. The smooth (average) curve is also shown in Figure 2-11.

Normalized Response Spectra

The smooth (average) curves for the three different site profile conditions for 5 percent critical damping are plotted in Figure 2-12 for comparative purposes. The procedures discussed above were then repeated for the same acceleration records but for critical damping values of 0, 2 and 10 percent. The smooth (average) curves from these determinations are shown in Figures 2-13, 2-14, and 2-15.

Comparison of Spectra

A comparison was made between the site spectra used to derive the smooth spectra for each site condition and comparable Newmark-Hall spectra since a relatively few records were used in the BPA study. Figure 2-16 through 2-18 provides a comparison of spectra for hard sites, firm sites and deep soil sites, respectively, for 5 percent of critical damping. It is significant that the Newmark-Hall spectrum provides a good average response for the hard sites for frequencies above 4 cps, but lies above the hard site spectra for all lower frequencies. For the firm sites, the Newmark-Hall spectrum provides a good average for the response between 2 and 6 cps, but lies generally above the firm site spectra at all other frequencies. For the deep soil sites, the agreement is for all frequencies below 2 cps. If

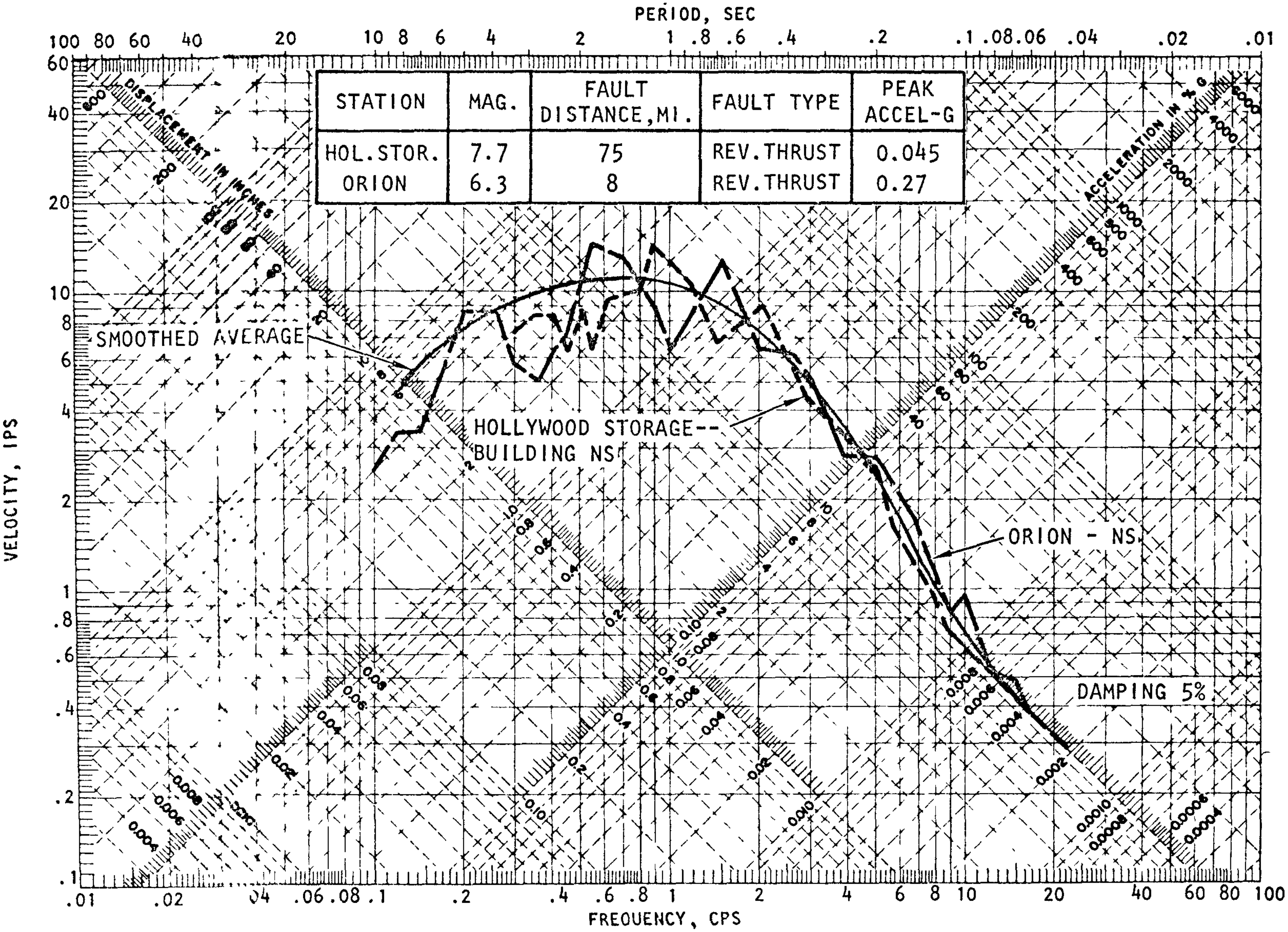


FIGURE 2-11. COMPARISON OF HORIZONTAL RESPONSE SPECTRA FROM DEEP SOIL SITES NORMALIZED TO 0.1G PEAK GROUND SURFACE ACCELERATION

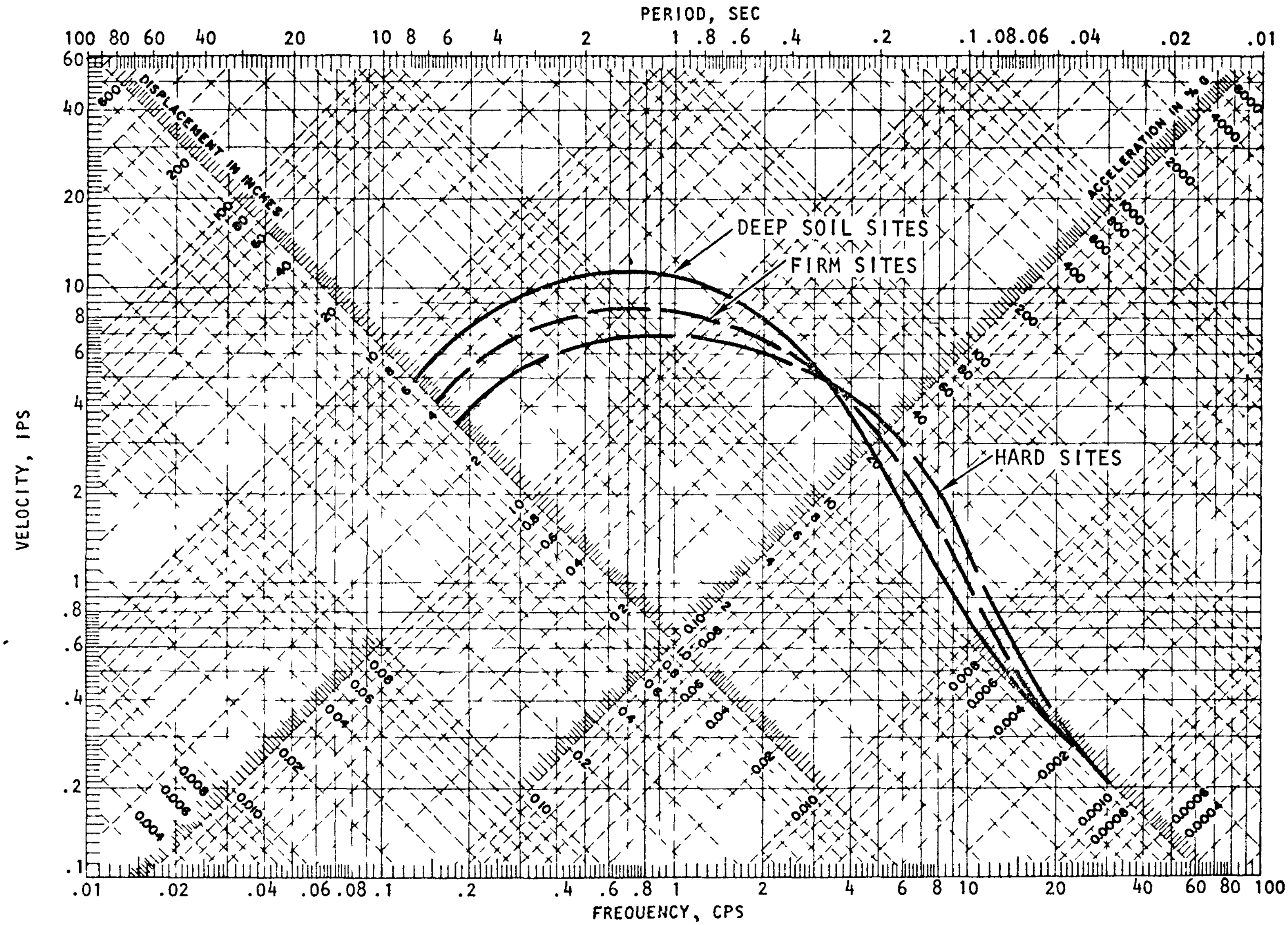


FIGURE 2-12. AVERAGE HORIZONTAL RESPONSE SPECTRA FOR DIFFERENT SITE CONDITIONS NORMALIZED TO 0.1G PEAK GROUND SURFACE ACCELERATION--5 PERCENT CRITICAL DAMPING

R-7211-13-2859

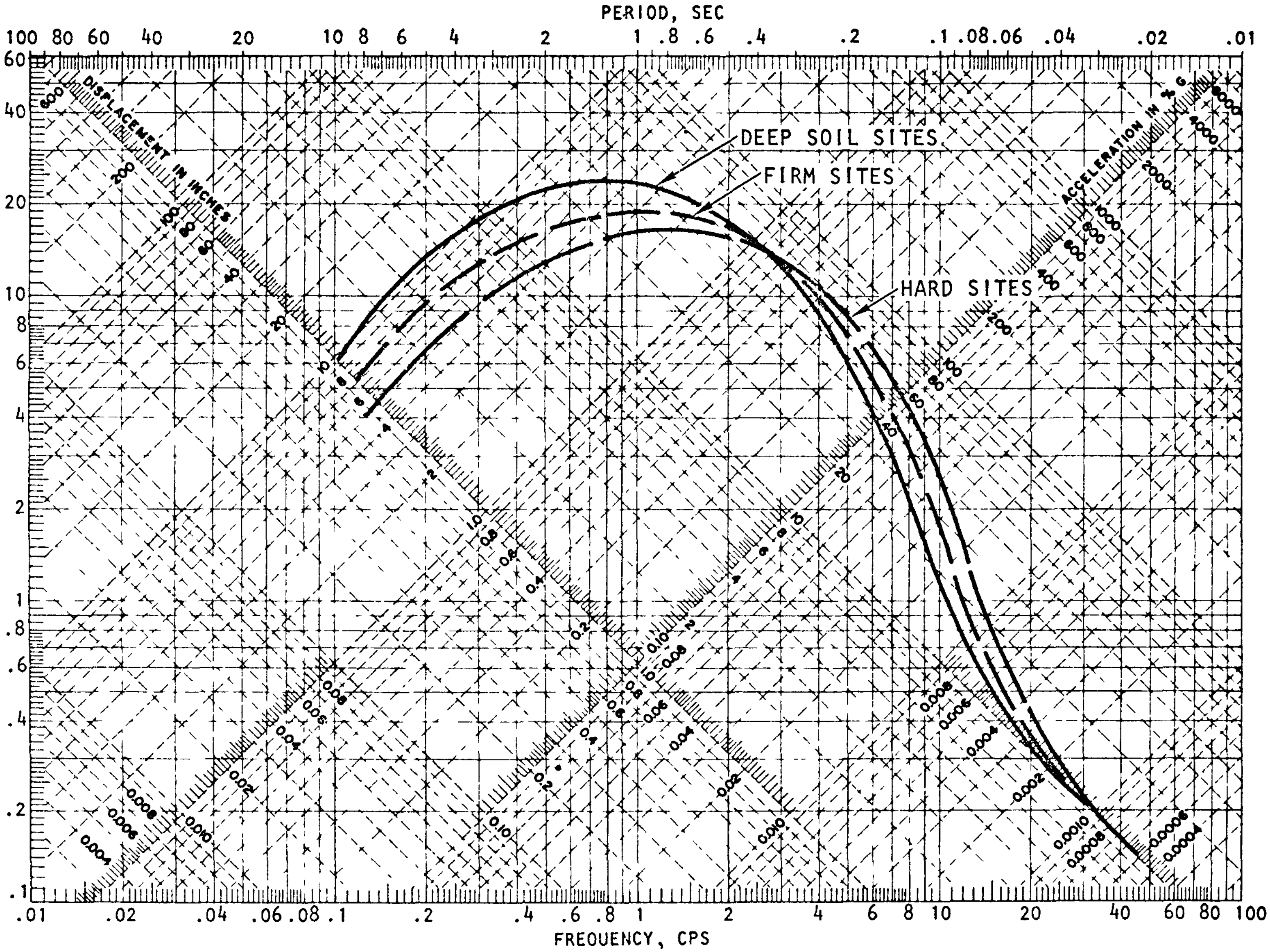


FIGURE 2-13. AVERAGE HORIZONTAL RESPONSE SPECTRA FOR DIFFERENT SITE CONDITIONS NORMALIZED TO 0.1G PEAK GROUND SURFACE ACCELERATION--0 PERCENT CRITICAL DAMPING

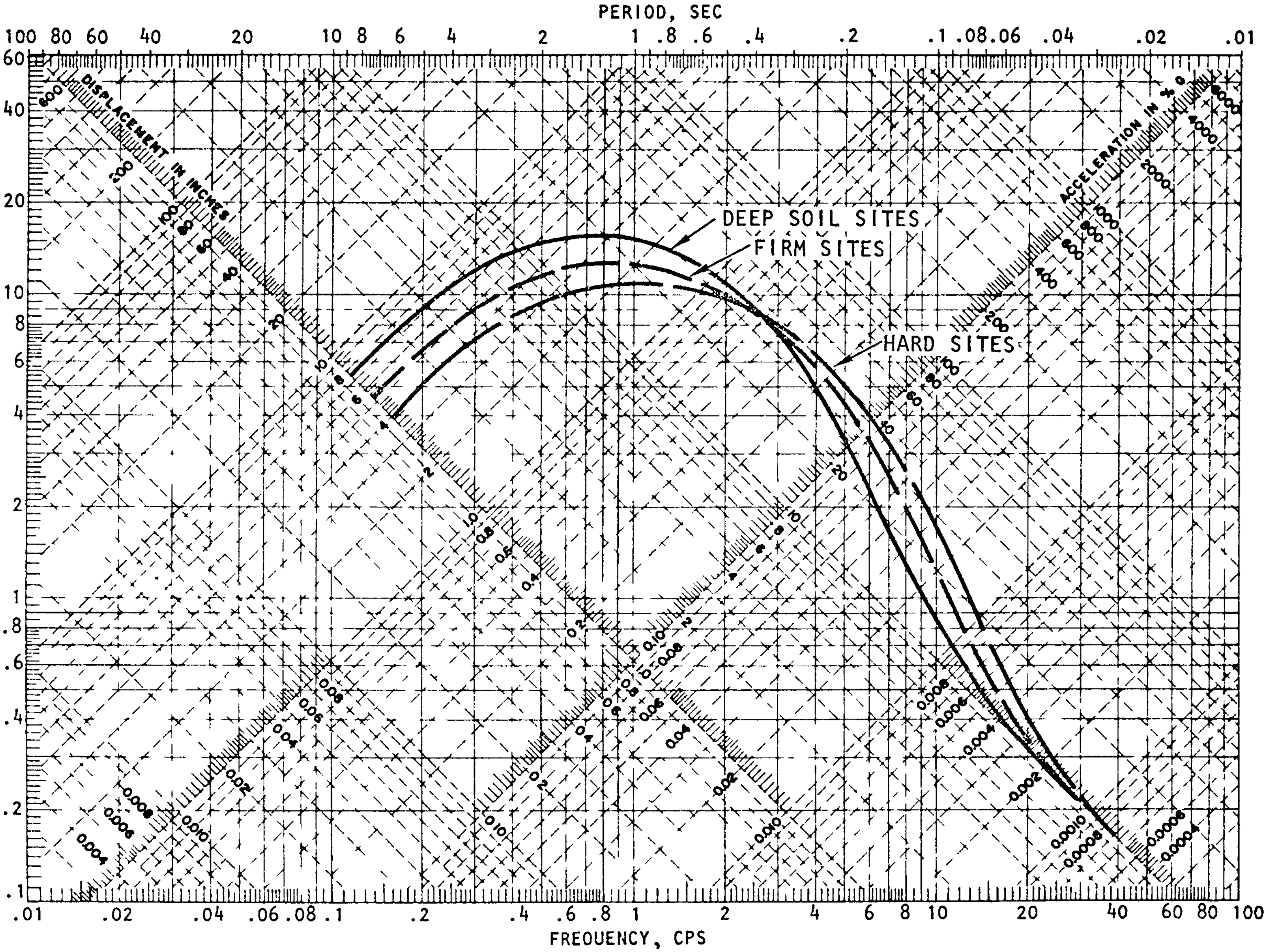


FIGURE 2-14. AVERAGE HORIZONTAL RESPONSE SPECTRA FOR DIFFERENT SITE CONDITIONS NORMALIZED TO 0.1G PEAK GROUND SURFACE ACCELERATION--2 PERCENT CRITICAL DAMPING

R-7211-13-2859

43

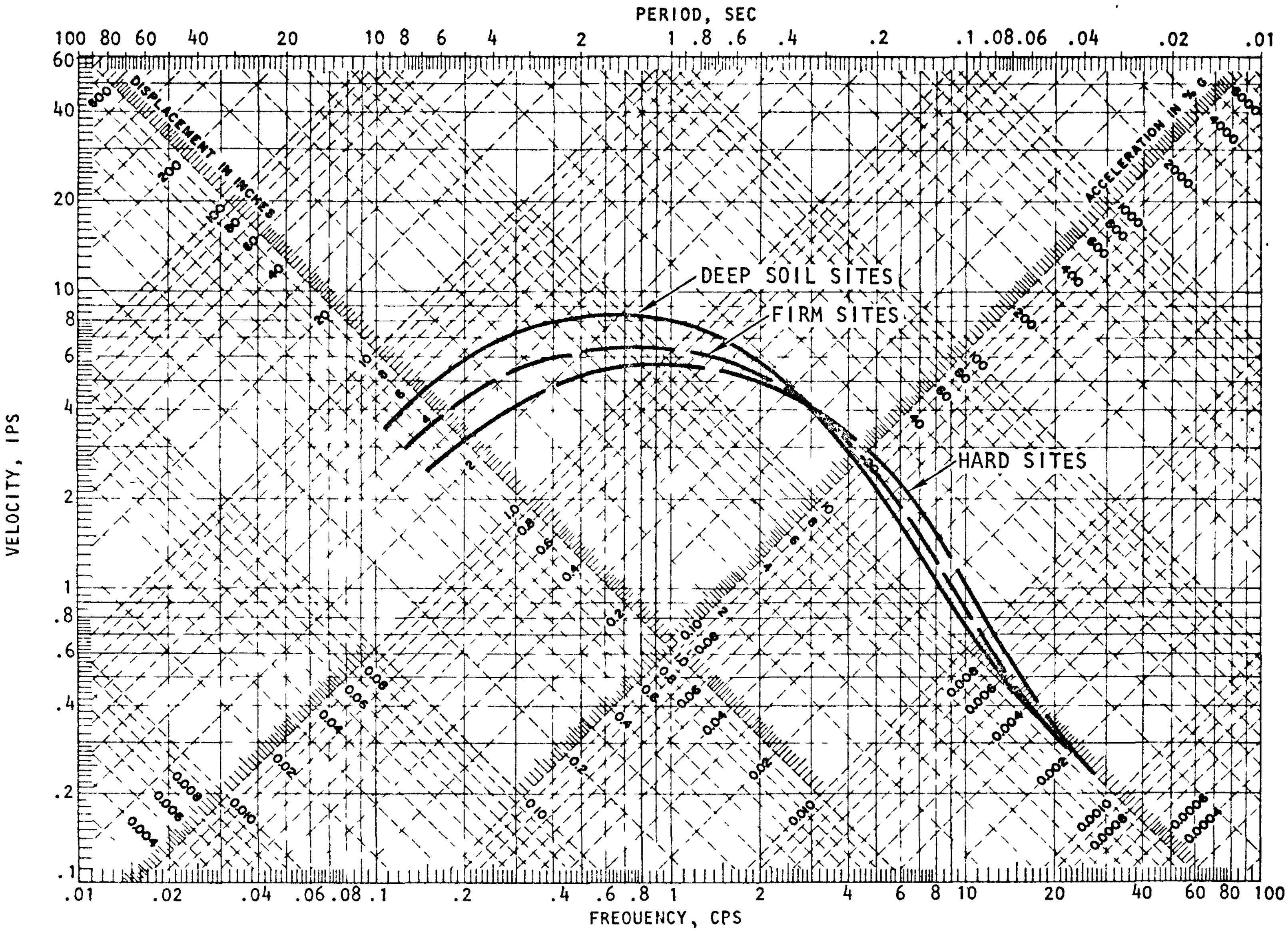


FIGURE 2-15. AVERAGE HORIZONTAL RESPONSE SPECTRA FOR DIFFERENT SITE CONDITIONS NORMALIZED TO 0.1G PEAK GROUND SURFACE ACCELERATION--10 PERCENT CRITICAL DAMPING

R-7211-13-2859

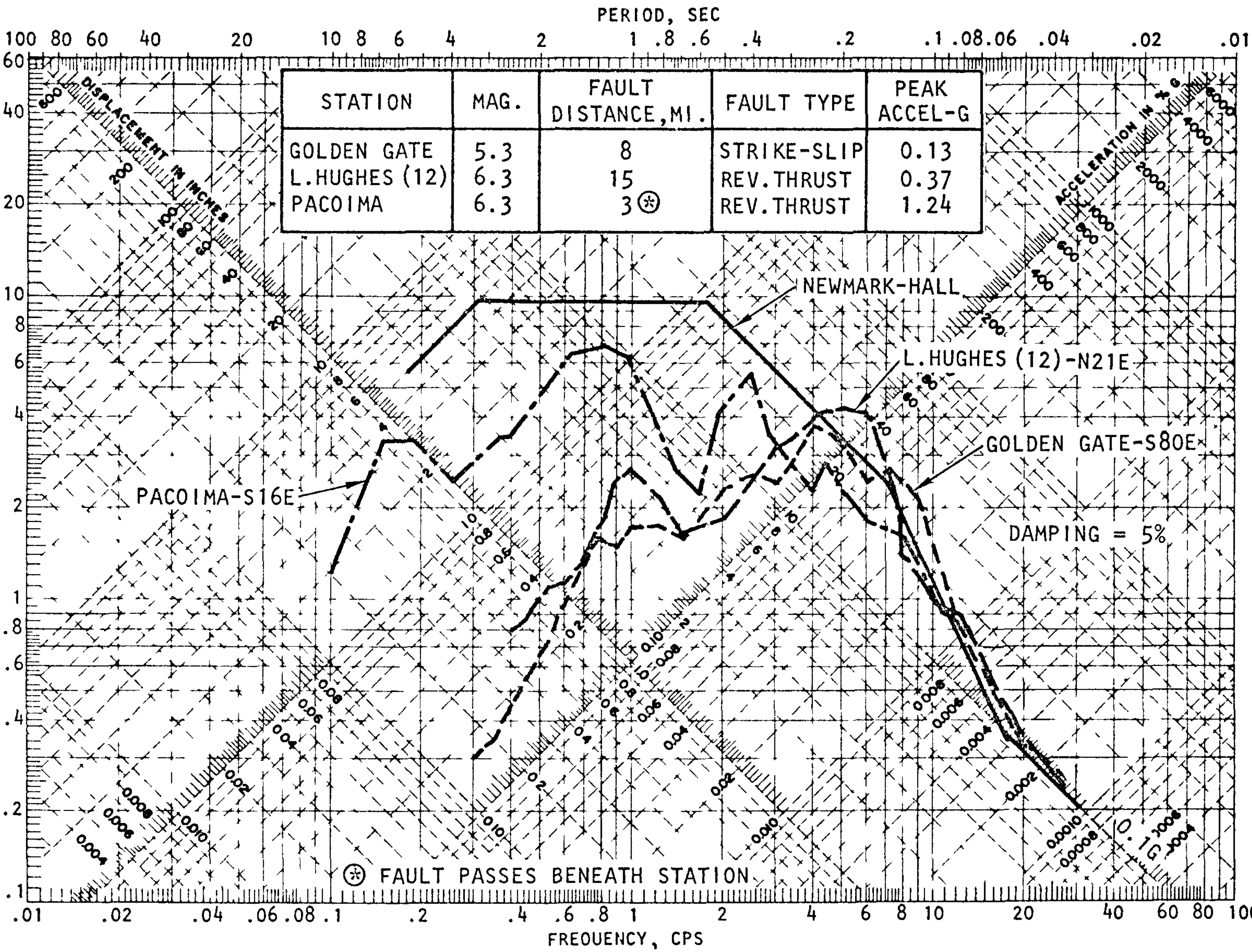
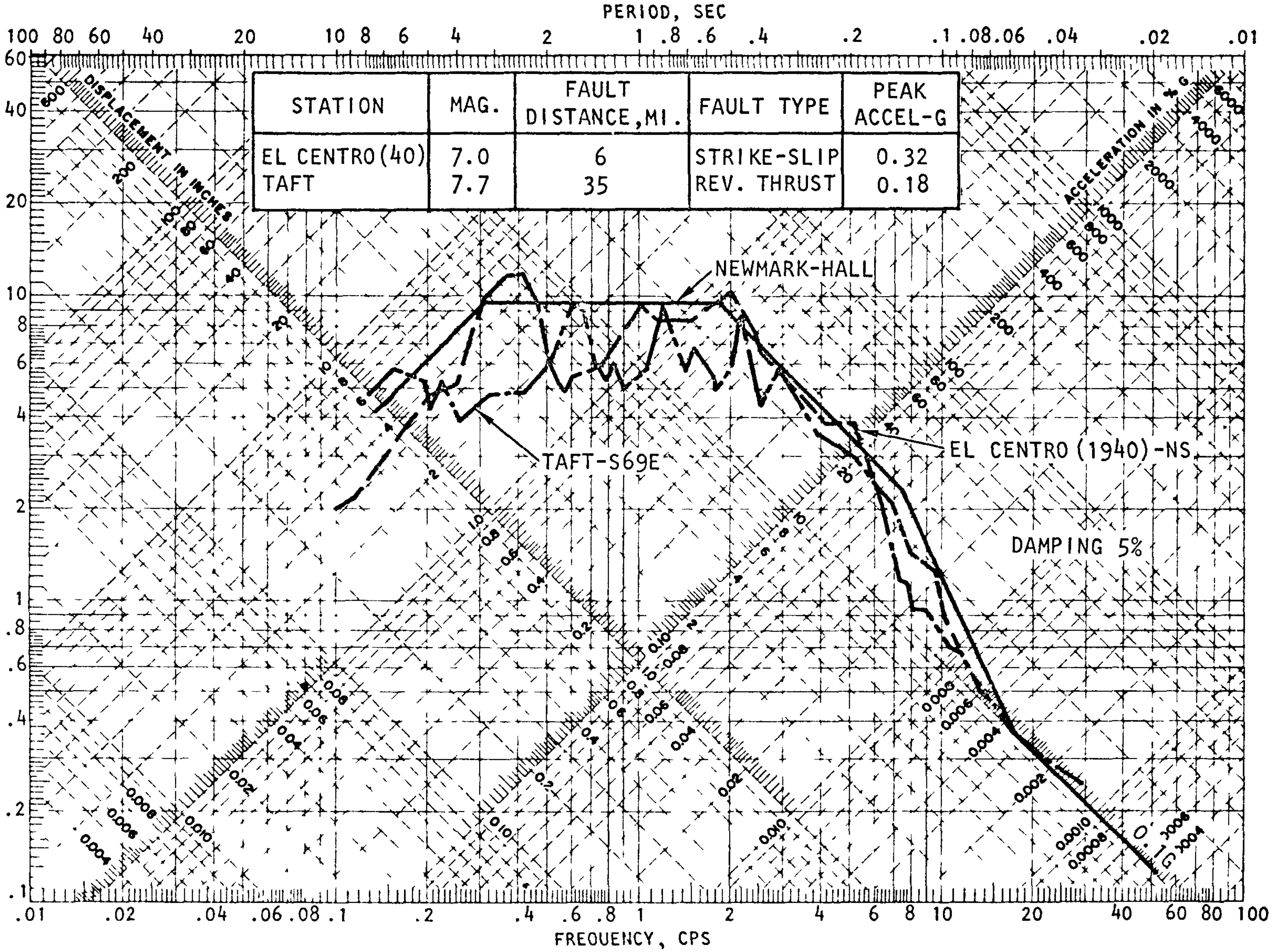
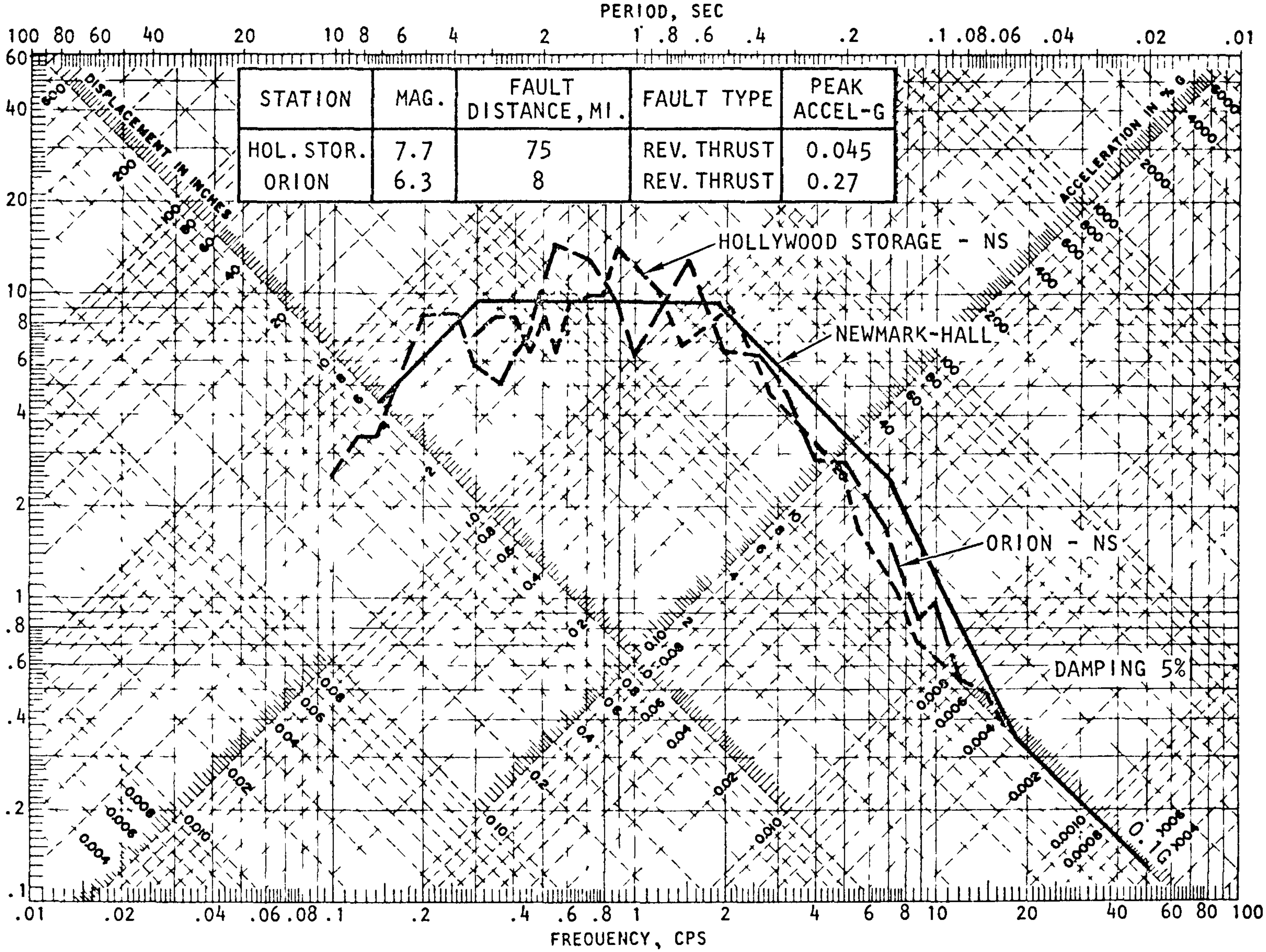


FIGURE 2-16. COMPARISON OF HORIZONTAL RESPONSE SPECTRA FROM HARD SITES NORMALIZED TO 0.1G PEAK GROUND SURFACE ACCELERATION



STATION	MAG.	FAULT DISTANCE, MI.	FAULT TYPE	PEAK ACCEL-G
EL CENTRO (40)	7.0	6	STRIKE-SLIP	0.32
TAFT	7.7	35	REV. THRUST	0.18

FIGURE 2-17. COMPARISON OF HORIZONTAL RESPONSE SPECTRA FROM STIFF SOIL SITES NORMALIZED TO 0.1G PEAK GROUND SURFACE ACCELERATION



STATION	MAG.	FAULT DISTANCE, MI.	FAULT TYPE	PEAK ACCEL-G
HOL. STOR.	7.7	75	REV. THRUST	0.045
ORION	6.3	8	REV. THRUST	0.27

FIGURE 2-18. COMPARISON OF HORIZONTAL RESPONSE SPECTRA FROM DEEP SOIL SITES NORMALIZED TO 0.1G PEAK GROUND SURFACE ACCELERATION

the Newmark-Hall spectra are considered to be near envelope spectra (say not exceeded 80 to 85 percent of the time) and to be constructed from spectra from all site profile conditions, then close agreement should be expected with hard site spectra in the high frequency range, with firm site spectra in the intermediate frequency range, and with deep soil site spectra in the lower frequency range. Since such agreement was obtained, the smooth spectra provided in Figures 2-12 through 2-15 were concluded to be indicative of the general effect of variations in site profile conditions on response spectra.

ZONE SCALING FACTORS

Before selecting a peak horizontal ground surface acceleration for scaling the normalized horizontal response spectra to zone criteria levels, consideration was given to the probable variations in response of the spectra used to derive the smooth curves as well as to the probable variations in intensity of motion within each seismic zone. Consideration was also given to the structural criteria into which the response spectra will ultimately be incorporated. The objectives were to simplify the criteria and to assess the overall factor of safety provided for design. A discussion of these factors follows.

Figure 2-19 provides a comparison between an envelope of the peak response values for the seven earthquakes considered in this study, the smooth firm site spectrum and an envelope of the smooth spectra for the three different site profile conditions, all for 5 percent of critical damping. Through a very wide range of frequencies, the envelope of peak response was found to exceed the firm site spectrum by about 50 percent, and to exceed the envelope of smooth spectra by about 35 percent. Referring to Table 2-1, the maximum peak horizontal ground surface acceleration in each zone was estimated to exceed the average peak horizontal ground surface acceleration by one-third. Therefore, if the smooth firm site spectrum is selected as an average spectrum for all site conditions, and if the average peak horizontal ground surface accelerations given in Table 2-1 are selected as scaling

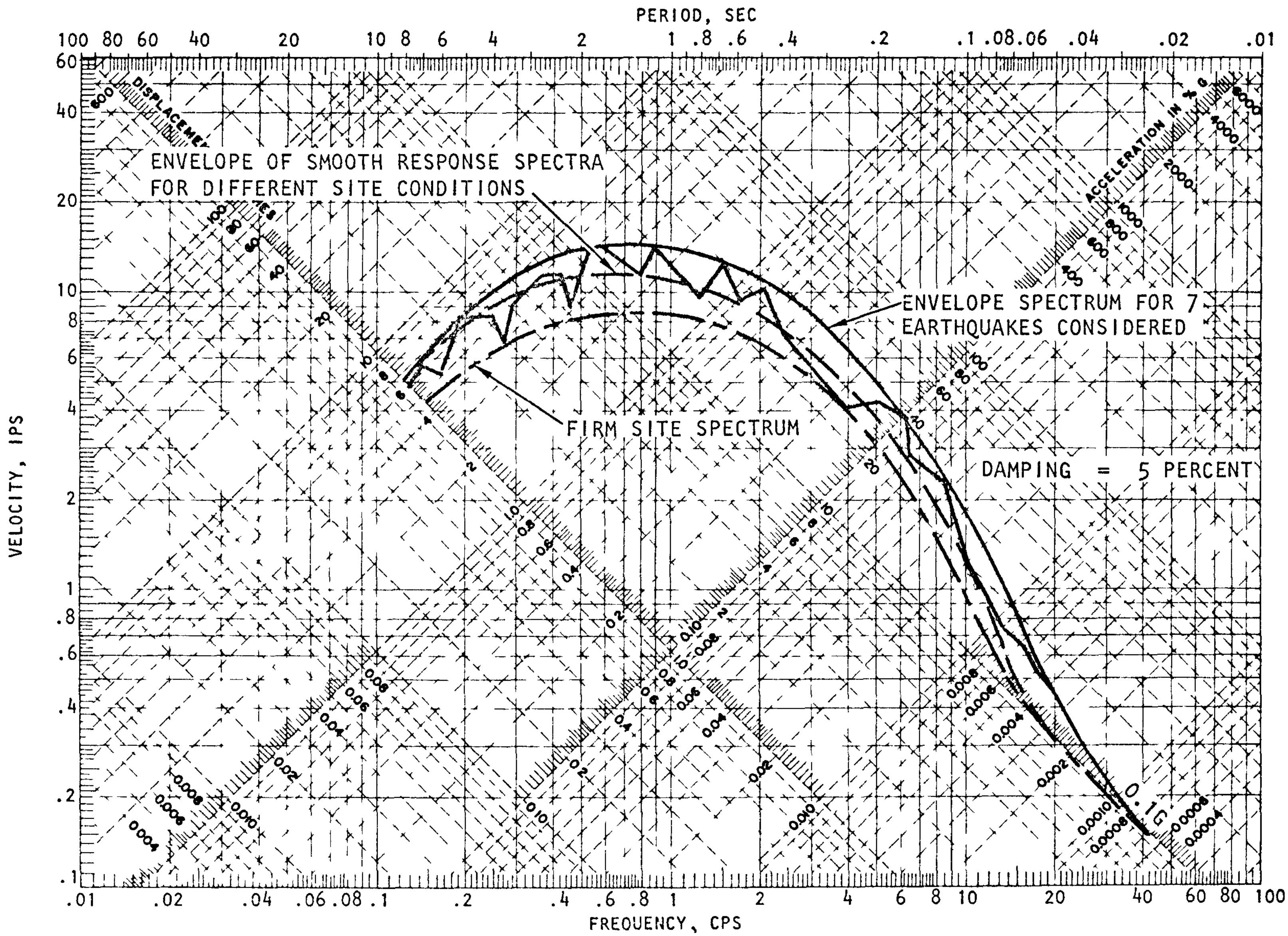


FIGURE 2-19. COMPARISON OF HORIZONTAL RESPONSE SPECTRA NORMALIZED TO 0.1G

factors, criteria spectra would result which could on occassion be exceeded by approximately 100 percent (1.5 x 1.33). If in contrast, site dependent criteria spectra are provided which consider the three general classes of hard, firm and deep soil sites, and if the same scaling factors are used, criteria spectra would result which could on occassion be exceeded by 80 percent (1.5 x 1.33). The latter procedure would provide a more conservative design but would increase the number of criteria spectra from four sets (one for each seismic zone) to twelve (three for each seismic zone). The implications of this simplification can only be evaluated by considering the overall factor of safety provided in the structural design criteria.

The structural criteria into which the ground motion criteria are to be incorporated provides a factor of safety of 3 for computed stress based on the ultimate strength of porcelain and brittle elements. This is an average factor of safety based on the seismic response being given by smooth criteria spectra. Since the seismic response based on firm site spectra and average peak horizontal ground surface accelerations as scaling factors could on occassion be exceeded by 100 percent, the factor of safety with criteria developed on this basis could be reduced to 1.5 because of excessive seismic response. In a similar manner, the factor of safety could be reduced to 1.67 because of excessive seismic response if site dependent criteria spectra were used. Since the factor of safety is also provided to account for uncertainties in analysis, design, material behavior and construction, it is essential to assure that some minimum factor of safety be retained to provide for these remaining uncertainties. For typical substation design it was concluded that a minimum factor of safety of 1.5 was adequate. On this basis, the <u>average</u> peak horizontal ground surface accelerations given in Table 2-1 were selected as scaling factors, and the normalized <u>firm</u> site spectra were selected as an average spectra applicable to all sites. However, for special installations, such as represented by the Celilo Converter Station and the Dittmer Control Center, a careful site investigation should be conducted and site dependent spectra should be developed to assure a higher factor of safety.

CRITERIA SPECTRA

Based on the discussion given in the preceding section, criteria spectra for horizontal earthquake ground motion were provided by multiplying the ordinates of the smooth normalized curves for firm sites given in Figures 2-12 through 2-15 by the ratio of the average peak horizontal ground surface accelerations given in Table 2-1 for each seismic zone over the normalization factor, 0.1 g. Criteria spectra for horizontal ground surface motion are provided in Figure 2-20 through 2-23 for all seismic zones defined in Figure 2-7.

Normalized response spectra curves were also constructed for vertical ground motion for the seven earthquakes considered above in the development of the normalized horizontal response spectra curves for hard, firm and deep soil sites. Based on this study it was concluded that the normal assumption (Reference 2-6) that the vertical response can be assumed to be equal to two-thirds the value given by the horizontal response spectra is a valid simplifying assumption for estimating vertical response. Therefore, it is recommended that vertical response spectra be scaled from the horizontal response spectra for each seismic zone by multiplying the appropriate spectra ordinates by a factor of two-thirds.

TIME MOTION RECORDS

In the analysis of complex equipment, response spectra may not provide an adequate definition of the criteria ground motion. In such cases, one or more components of earthquake acceleration scaled to give nearly equivalent response are normally used as input for the analysis. Since the normalized firm site spectra were scaled to provide the criteria spectra given in Figures 2-20 through 2-23, the El Centro and Taft records can be used to approximate the criteria spectra. Reference to Figure 2-10 will show that these records give close agreement with the criteria spectra for practically all frequencies. The analysis should be run using both records and an average of the response from both records used as an indication of the probable response of the equipment.

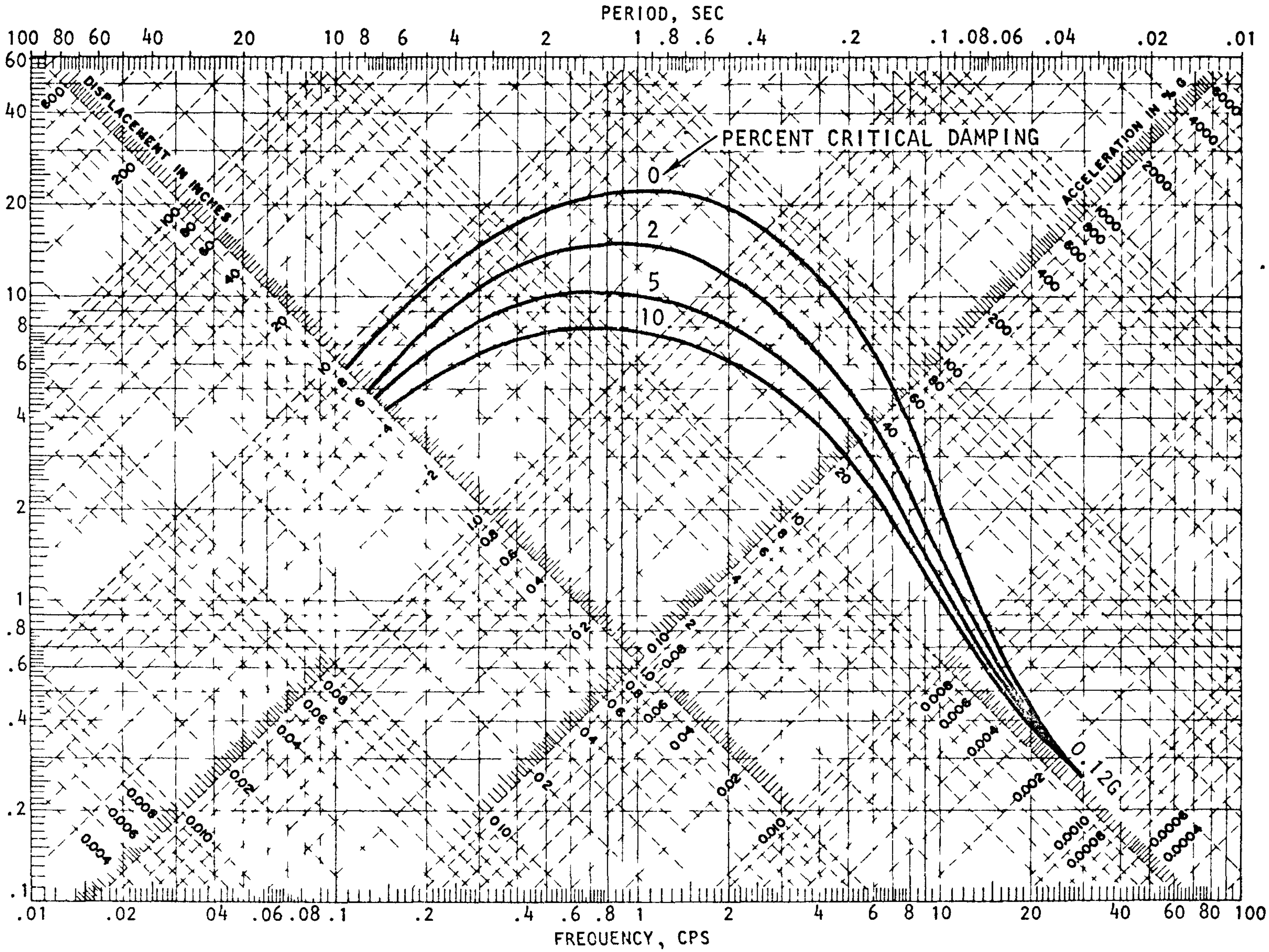


FIGURE 2-20. HORIZONTAL RESPONSE SPECTRA FOR SEISMIC ZONE A

R-7211-13-2859

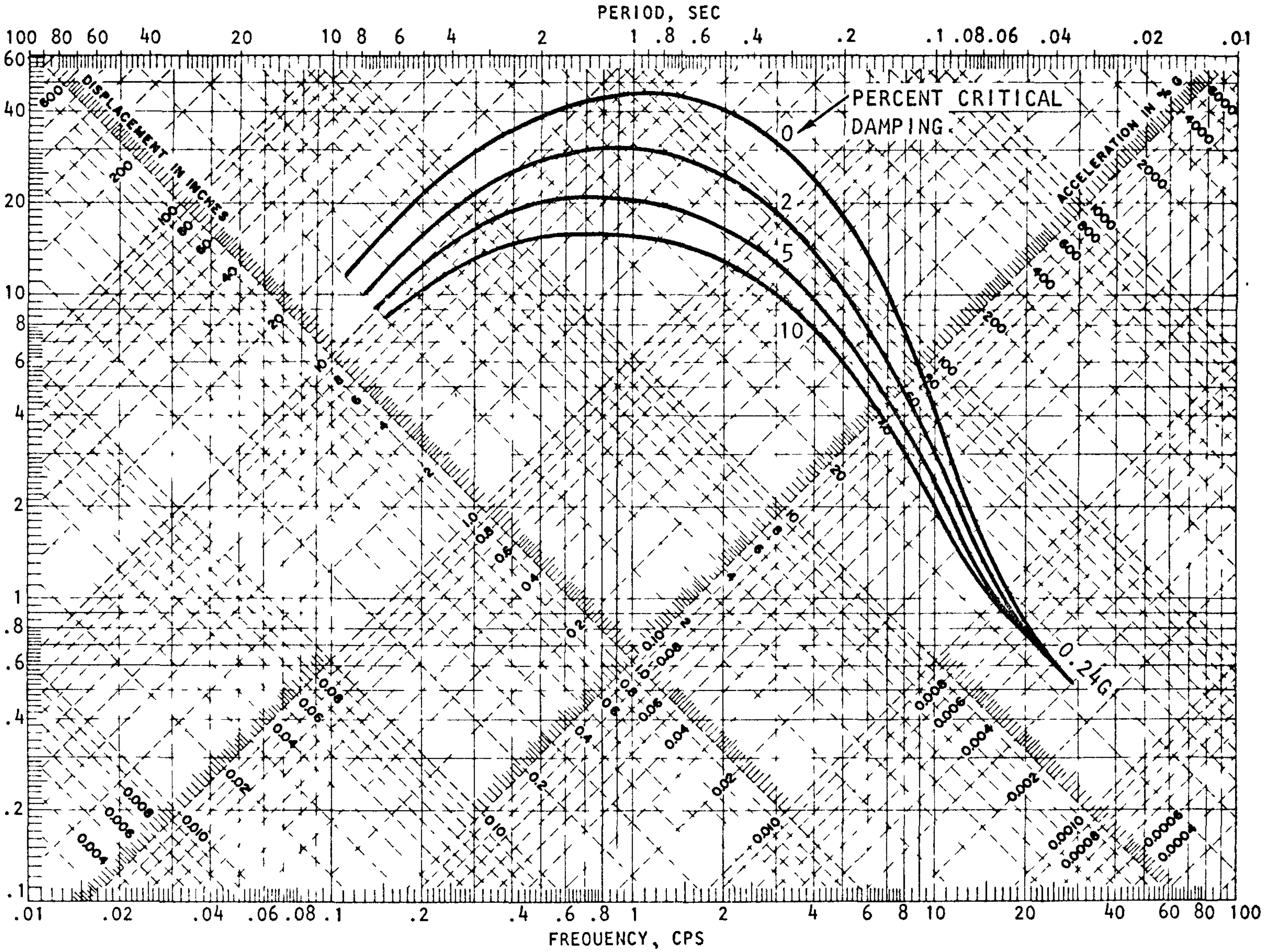


FIGURE 2-21. HORIZONTAL RESPONSE SPECTRA FOR ZONE B

R-7211-13-2859

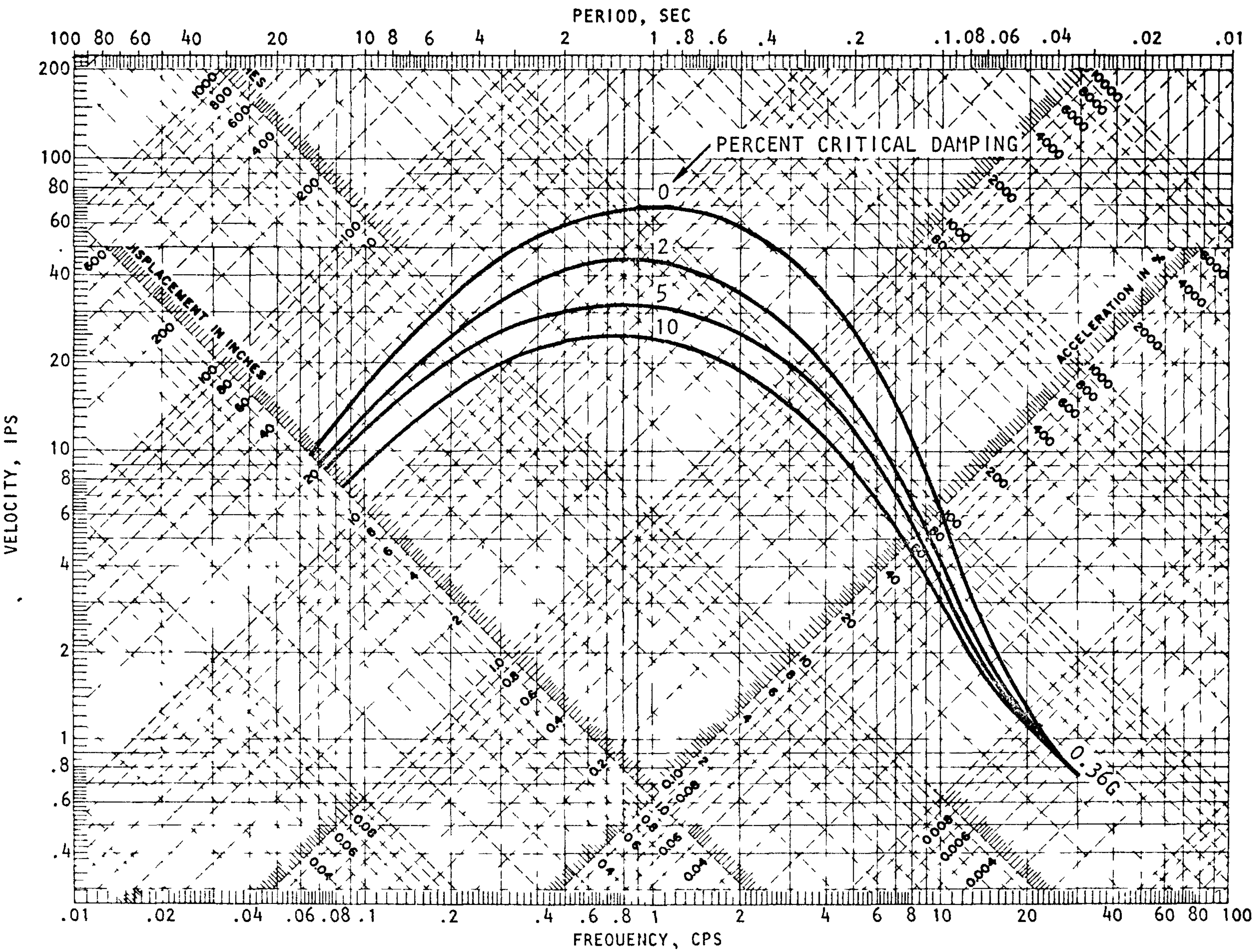


FIGURE 2-22. HORIZONTAL RESPONSE SPECTRA FOR ZONE C

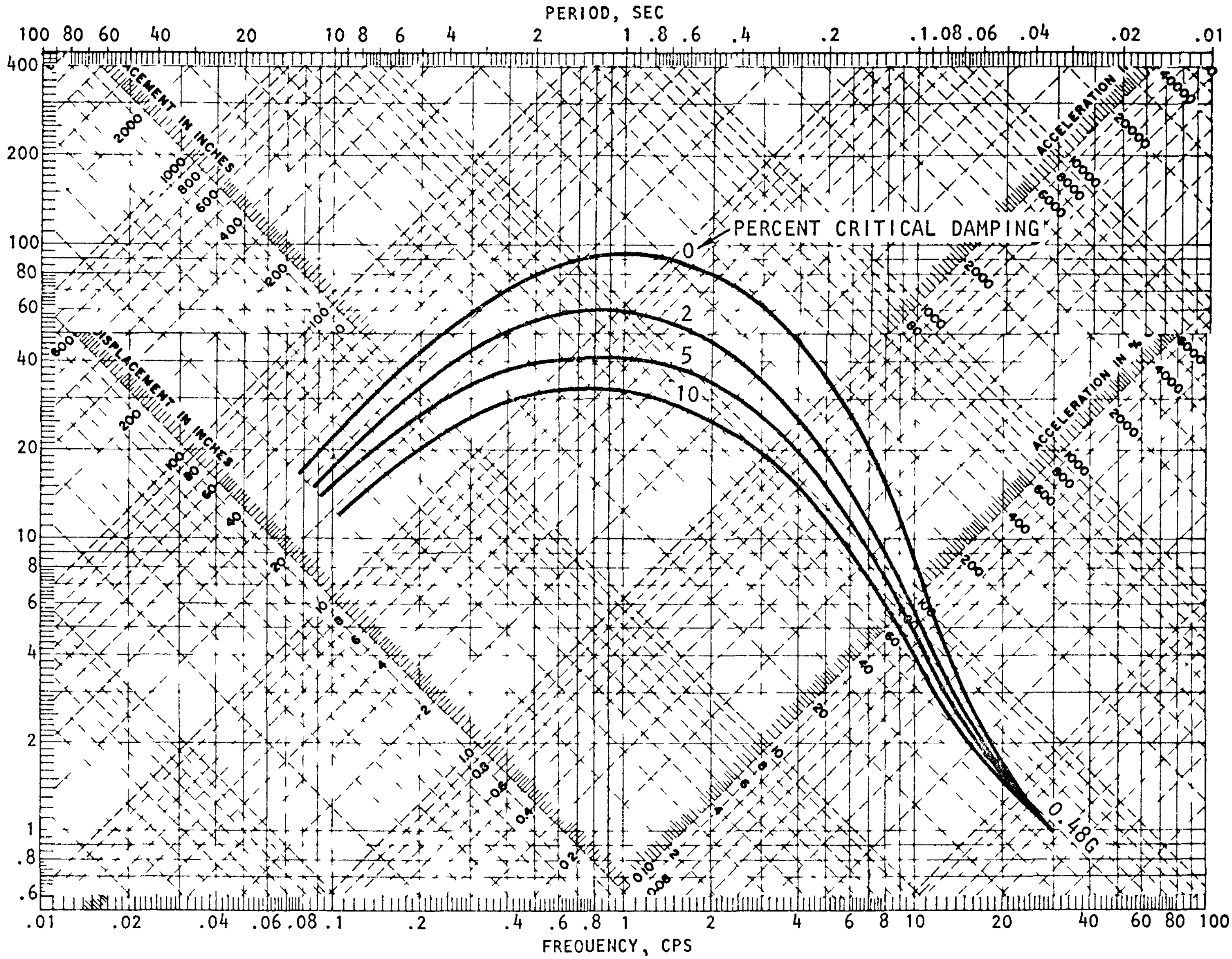


FIGURE 2-23. HORIZONTAL RESPONSE SPECTRA FOR ZONE D

R-7211-13-2859

For special cases where site dependent criteria are to be used, Figures 2-9, 2-10 and 2-11 are particularly useful in noting the range of frequencies in which the seven earthquake records used to derive the smooth, site dependent spectra have reasonable agreement with the smooth spectra. For hard sites, for example, the Golden Gate and Lake Hughes Array 12 records give good agreement with the smooth spectra for all frequencies above 4 cps. These records could, therefore, be used for analysis at hard sites if the essential frequencies of the equipment being analyzed fall within this range. In this range these records should be scaled to make the peak acceleration equal to the average peak horizontal acceleration given in Table 2-1 for the zone. For lower frequency equipment on hard sites, the Pacoima record would give better results. However, it would have to be scaled by a factor greater than the zone scaling factor to provide spectra agreement for most frequencies. The scaling factor in this case should be determined by inspection. For firm sites, the El Centro and Taft records, as noted above, could be used and would provide close agreement with the firm site criteria spectra at all frequencies. The same general comment applies to the Hollywood Storage Building and the Orion records for equipment on deep soil sites when site dependent criteria are used.

These seven sets of acceleration records in digitized form are all available and can be obtained from the Earthquake Engineering Research Laboratory of the California Institute of Technology for a nominal charge.

PROBABILITY OF OCCURRENCE OF STRONG EARTHQUAKE GROUND MOTION

A deterministic approach was used in developing the seismic regionalization map and associated ground motion criteria presented in the previous sections of this chapter. Maximum probable earthquakes were used in defining seismic zone boundaries and in establishing scaling factors for the response spectra. The criteria, if properly used, should result in modified equipment that will have more than a 98 percent probability of escaping seismic damage in future earthquakes even under the most extreme conditions. However, to effectively schedule and budget the recommended modifications, and to determine whether the modifications are economically justified, the probability of occurrence of strong earthquake ground motion of different intensity in the different seismic zones should be considered. The importance of this information is demonstrated in an example problem in Chapter 6. In order to provide information for scheduling and budgeting, the probability of occurrence of strong earthquake ground motion in the Puget Sound and Portland areas has been computed using the procedure outlined by Housner (Reference 2-9), and the basic regional seismic data on frequency of occurrence of earthquakes and intensity attenuation relationships provided in Reference 2-1. A brief review of the procedures followed and the probability relationships derived follows:

Areas Studied

The calculation of the frequency of an event such as strong earthquake ground motion in an area is based on the assumption that within the designated area, there is an equal probability of the occurrence of the epicenter of an earthquake anywhere and at anytime. Since earthquake epicenters are usually associated with certain tectonic structures which do not have uniform areal distribution, this assumption is not too well satisfied in earthquake ground motion probability studies. Great care must, therefore, be exercised in selecting the areas to be considered, and in interpreting the results. Because of the seismic activity associated with the Puget Sound and Portland areas, and because of the concentration of BPA electrical transmission facilities in these two areas, these areas were selected for study. Ultimately, the study should be extended to include additional potential epicentral areas distributed throughout the BPA service area.

Since the size of the area considered in the analysis has a significant effect on the answer, two different area sizes were considered in each study for comparative purposes. Referring to Figure 2-7, the Zone C area designated in the Puget Sound area has roughly an area of 21,000 square miles. Reference to Figure 2-2 will indicate that this area is quite similar in size and shape to the potential epicentral area designated for 6.3 magnitude earthquakes and reference to Figure 2-1 will indicate that slightly smaller epicentral areas have magnitude 7.4 earthquake potential. The plotted epicenters of past earthquakes in this area (Reference 2-1) indicates that the epicenters do have a nearly random distribution over the Zone C area, and extend to the west almost to the Zone B boundary in Figure 2-7. The larger earthquakes, however, occur near the center of the area. On this basis, an area of 21,000 square miles, considered to be the area designated by the Puget Sound Zone C in Figure 2-7, and an area of 42,000 square miles, considered to be the total area inside the Puget Sound Zone B and C areas down to approximately the 46° latitude line, were selected for the Puget Sound analysis. Since the highest ground surface accelerations are associated with the largest magnitude earthquakes (by assumption) and these earthquakes have occurred near the Center of Zone C, it should be evident that the probabilities derived for the smaller area will be too high for points near the edge of Zone C, and too low for points near the center of the area. The same statement applies to the probabilities derived for the larger area relative to their application to the Zone B area. However, the results for the larger area for the lower acceleration levels are a better indicator of Zone B probabilities than the results from the smaller Zone C area. The user of the data should recognize that the higher accelerations from either analysis could rarely occur in Zone B. Obviously, the results must be used with considerable judgement.

In selecting the areas for use in analyzing the Portland region, consideration was first given to the nearly triangular shaped potential epicentral area for magnitude 6.3 earthquakes shown for this region in Figure 2-2. Reference to the plotted epicenters (Reference 2-1) of past earthquakes indicated that this area had the most seismic activity in this region. On this basis, a triangular shaped area encompassing the magnitude 6.3 potential epicentral area shown in Figure 2-2 was selected as the smaller area for the Portland analysis. This area encompassed 7500 square miles. Considerable seismic activity in this general area has occurred in Zone B (Figure 2-7) below the 46° latitude line down to the Zone B area influenced by the Port

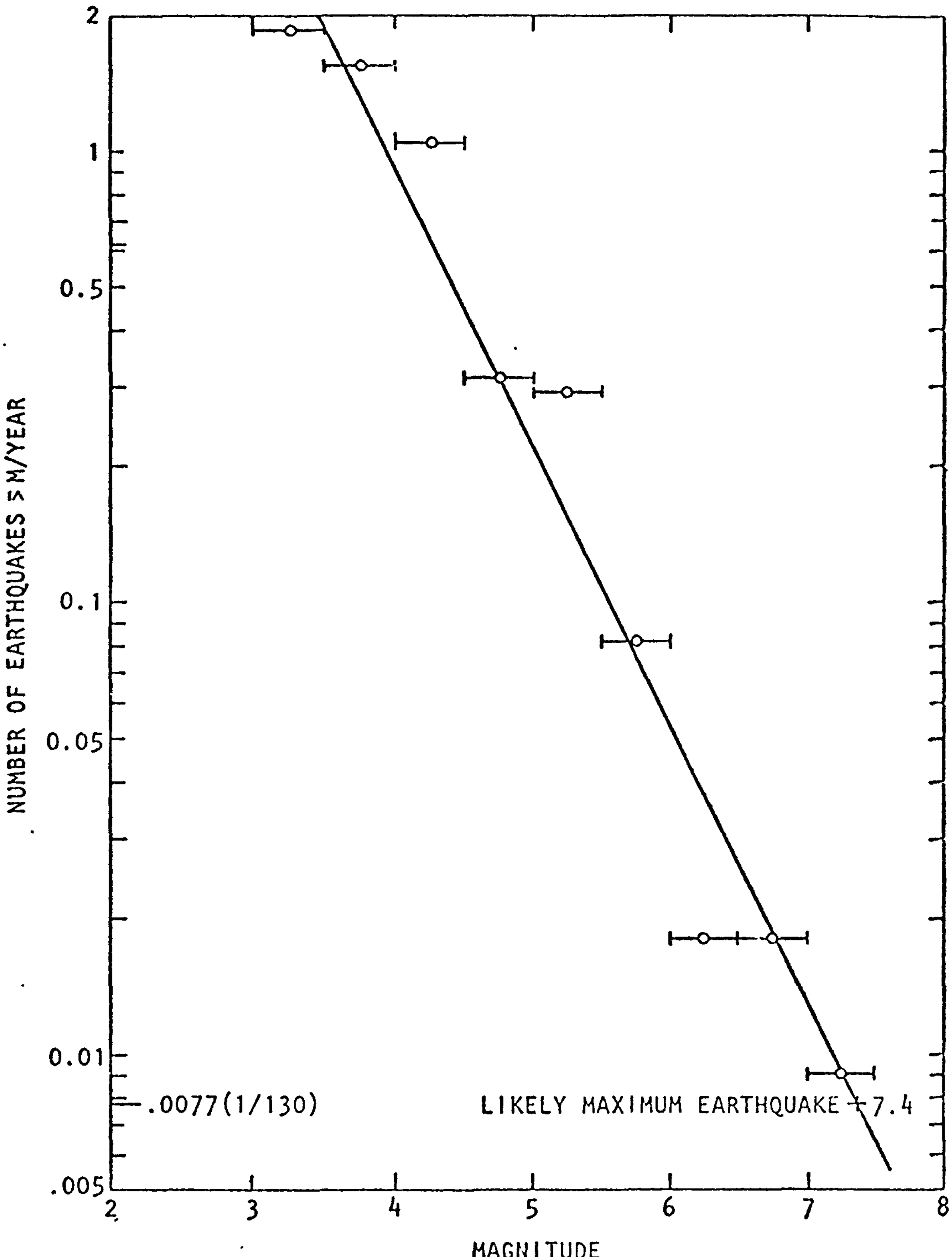
Orford epicentral area. This area of approximately 15,000 square miles was used as a second larger area for the Portland analysis. The resulting probabilities have the same general areal limitations discussed above for the Puget Sound results.

Calculation Procedure

The procedure used to calculate the probability of occurrence of strong earthquake ground motion can be explained briefly as follows for the Puget Sound area. The frequency of occurrence of earthquakes of different magnitudes in this area was assumed to be that given in Figure 2-24 (Reference 2-1). The intensity attenuation curves for the area were taken from the same reference and are given in Figure 2-25. The Gutenberg-Richter relationship between MM Intensity and peak ground surface acceleration used to define the seismic regionalization map, and previously discussed, was used to convert intensity to peak ground surface acceleration. This leads to the tabulation of data given in Tables 2-2 and 2-3.

TABLE 2-2. FREQUENCY OF OCCURRENCE OF EARTHQUAKES IN PUGET SOUND AREA

Magnitude	Number per 100 Years
4.75 - 5.25	21.
5.25 - 5.75	10.5
5.75 - 6.25	5.2
6.25 - 6.75	2.4
6.75 - 7.25	1.2
7.25 - 7.75	0.62
7.75 - 8.25	0.06



(REFERENCE 2-1)

FIGURE 2-24. FREQUENCY OF OCCURRENCE OF EARTHQUAKE IN PUGET SOUND REGION

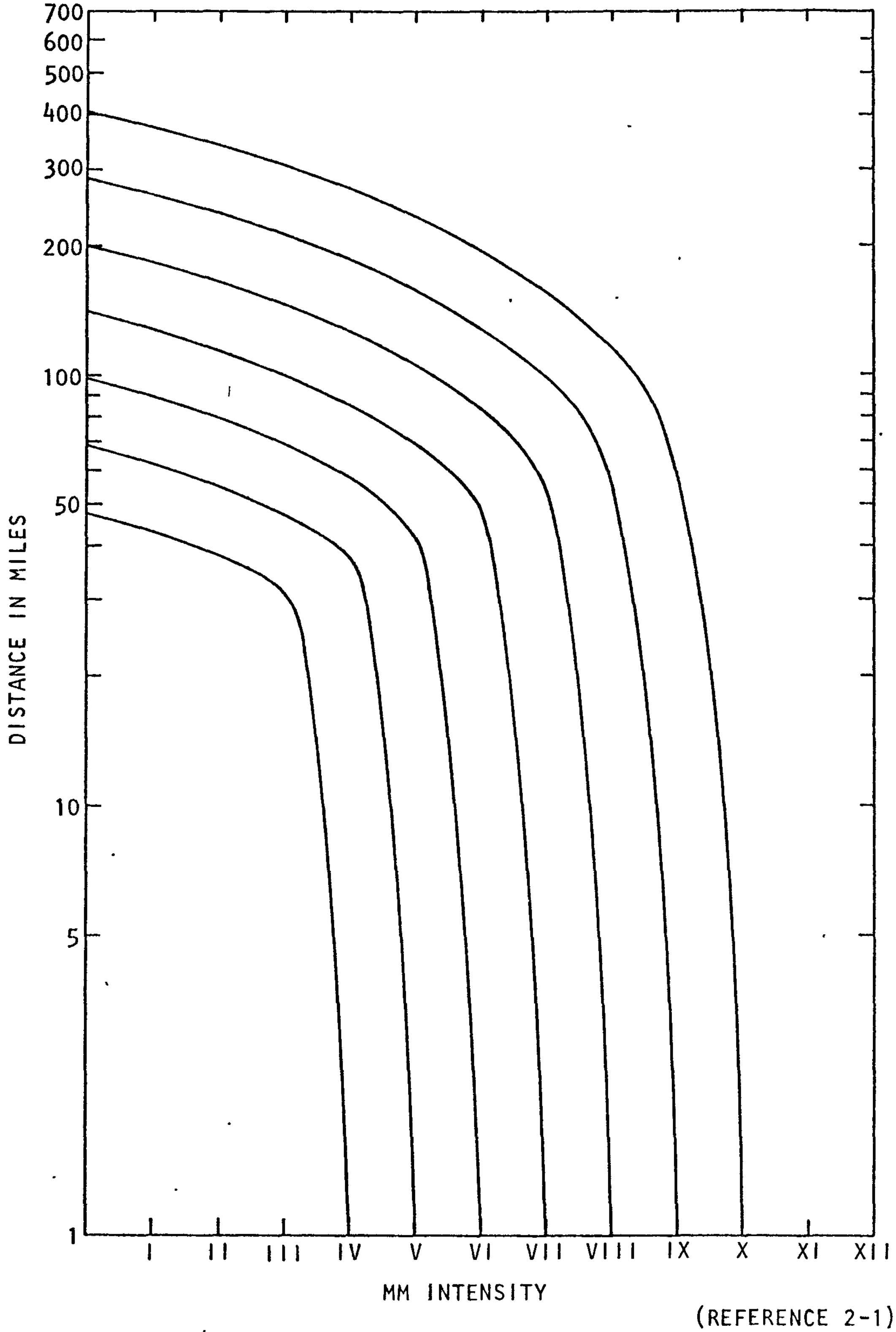


FIGURE 2-25. INTENSITY ATTENUATION CURVES FOR PUGET SOUND REGION

TABLE 2-3. AREA IN 1000 SQUARE MILES COVERED BY GROUND SURFACE ACCELERATIONS IN PERCENT GRAVITY

Acceleration Percent Gravity	Earthquake Magnitude						
	5	5.5	6	6.5	7	7.5	8
≥5	0	0.27	4.93	11.9	19.0	35.4	69.5
≥10			0.13	5.46	12.7	26.6	50.8
≥15				0.2	3.62	13.9	26.6
≥20				1.	1.71	12.5	24.56
≥25					0.31	7.71	17.34
≥30						1.41	7.93
≥35						0.76	4.93

The probability that a certain location in the region will not experience an earthquake is equal to exp (-a/A) (Reference 2-9). Here, 'a' is the area experiencing a maximum ground acceleration of say 5 percent g or more, and A is the total area under consideration. Suppose within a given recurrence interval, there are n_i number of earthquakes with magnitude M_i occurring in the region. Each magnitude M_i covers an area of a_i. Where subscript i indicates the total number of earthquakes of different magnitudes.. The probability that none of these earthquakes will be experienced in a certain location is:

$$P_{NH} = \left(e^{-a_1/A}\right)^{n_1} \left(e^{-a_2/A}\right)^{n_2} \ldots \left(e^{-a_i/A}\right)^{n_i}$$

and the probability that any of the earthquakes will be experienced is:

$$P_H = 1 - P_{NH}$$

Numbers $n_1, n_2 \ldots n_i$ can be found in Table 2-2 and $a_1, a_2 \ldots a_i$ are listed in Table 2-3. The percent probability of a given peak horizontal ground surface acceleration occurring in the Puget Sound area were then calculated for two different sized areas and are listed in Tables 2-4 and 2-5 for three different recurrence intervals.

TABLE 2-4. PERCENT PROBABILITY OF OCCURRENCE OF PEAK HORIZONTAL GROUND SURFACE ACCELERATION IN PUGET SOUND AREA OF 21,000 SQUARE MILES

Peak Acceleration Percent Gravity	Recurrence Interval, Years		
	40	80	130
≥5	86.7	98.2	99.9
≥10	60.4	84.3	95.1
≥15	24.3	42.8	59.6
≥20	19.4	35.0	50.3
≥25	11.2	21.1	31.9
≥30	2.6	5.0	8.1
≥35	1.5	2.9	4.7
≥40	1.1	2.1	3.4

TABLE 2-5. PERCENT PROBABILITY OF OCCURRENCE OF PEAK HORIZONTAL GROUND SURFACE ACCELERATION IN PUGET SOUND AREA OF 42,000 SQUARE MILES

Peak Acceleration Percent Gravity	Recurrence Interval, Years		
	40	80	130
≥5	63.6	86.7	96.2
≥10	37.1	60.4	77.8
≥15	13.0	24.3	36.5
≥20	10.2	19.4	29.5
≥25	5.7	11.2	17.5
≥30	1.3	2.6	4.1
≥35	0.7	1.5	2.4
≥40	0.5	1.1	1.7

The same procedures were then repeated for the Portland region using the earthquake frequency curves and intensity attenuation curves applicable to this region which are given in Reference 2-1. The results are tabulated in Tables 2-6 and 2-7.

TABLE 2-6. PERCENT PROBABILITY OF OCCURRENCE OF PEAK HORIZONTAL GROUND SURFACE ACCELERATION IN PORTLAND AREA OF 7500 SQUARE MILES

Peak Acceleration Percent Gravity	Recurrence Interval, Years		
	40	80	130
≥5	89.3	98.9	99.9
≥10	48.1	73.1	88.2
≥15	9.0	17.1	26.3
≥20	5.2	10.2	16.0
≥25	2.0	3.9	6.3

TABLE 2-7. PERCENT PROBABILITY OF OCCURRENCE OF PEAK HORIZONTAL GROUND SURFACE ACCELERATION IN PORTLAND AREA OF 15,000 SQUARE MILES

Peak Acceleration Percent Gravity	Recurrence Intervals, Years		
	40	80	130
≥5	67.4	89.3	97.4
≥10	28.0	48.1	65.6
≥15	4.6	9.0	14.1
≥20	2.6	5.2	8.3
≥25	1.0	2.0	3.2

Smoothed curves have been drawn in Figures 2-26 through 2-29 from the tabulated data given in Tables 2-4 through 2-7, respectively. An application of these curves to determine the zones in which a specific type of electrical equipment modification can be economically justified is demonstrated in Chapter 6.

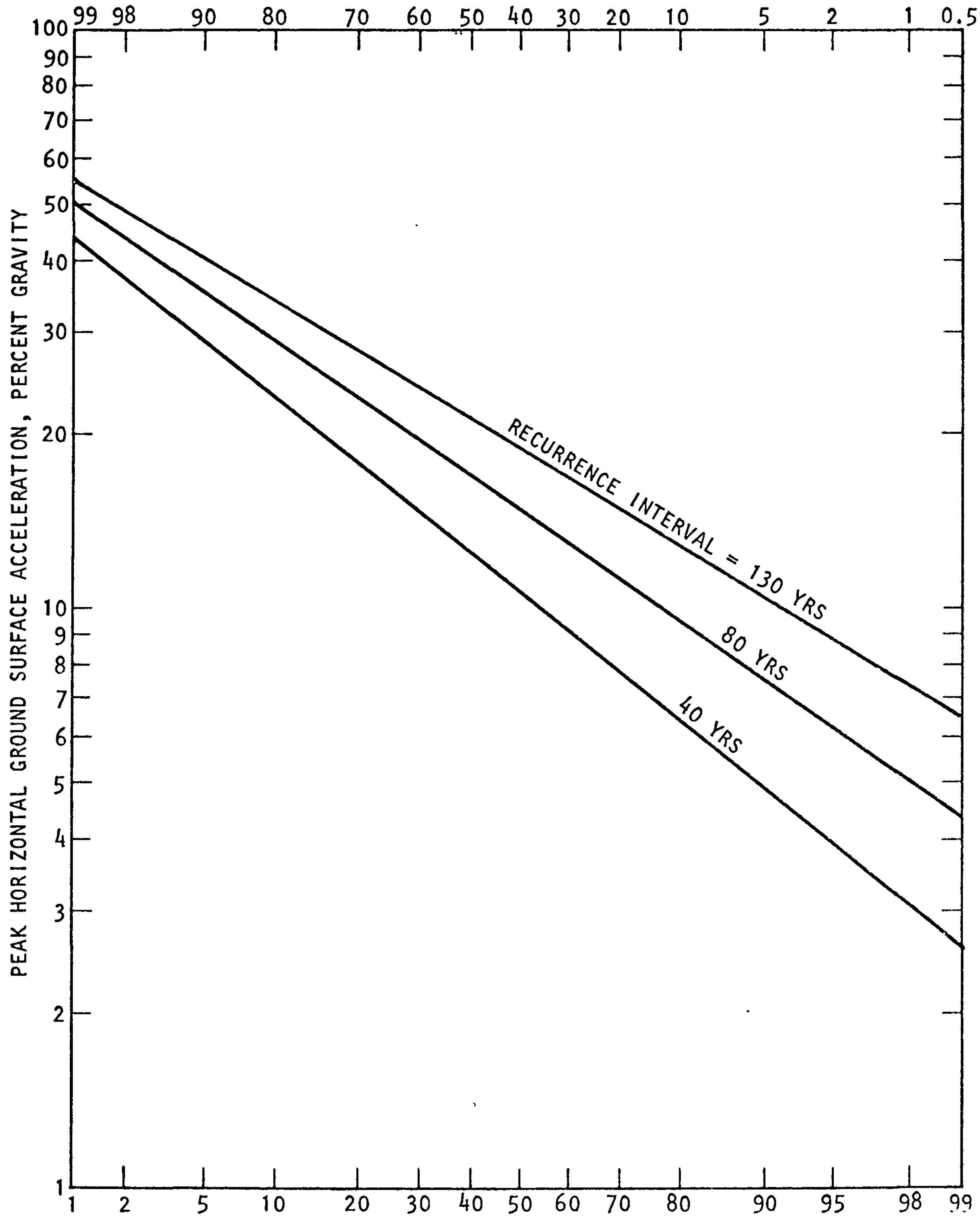


FIGURE 2-26. PERCENT PROBABILITY THAT A GIVEN SUBSTATION IN PUGET SOUND REGION WILL EXPERIENCE PEAK ACCELERATION IN RECURRENCE INTERVAL INDICATED BASED ON 21,000 MI^2 AREA

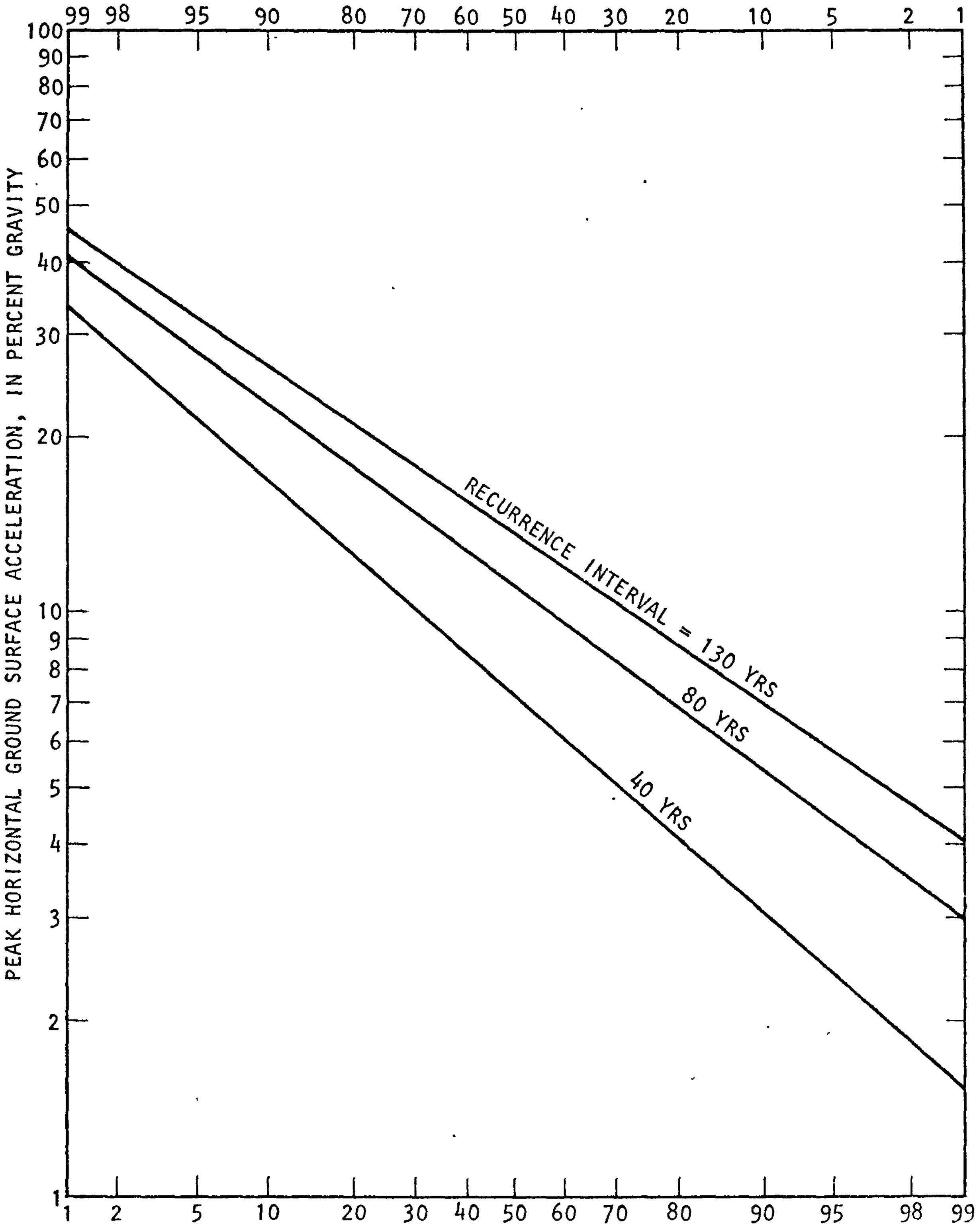


FIGURE 2-27. PERCENT PROBABILITY THAT A GIVEN SUBSTATION IN PUGET SOUND REGION WILL EXPERIENCE PEAK ACCELERATION IN RECURRENCE INTERVAL INDICATED-BASED ON 42,000 MI^2 AREA

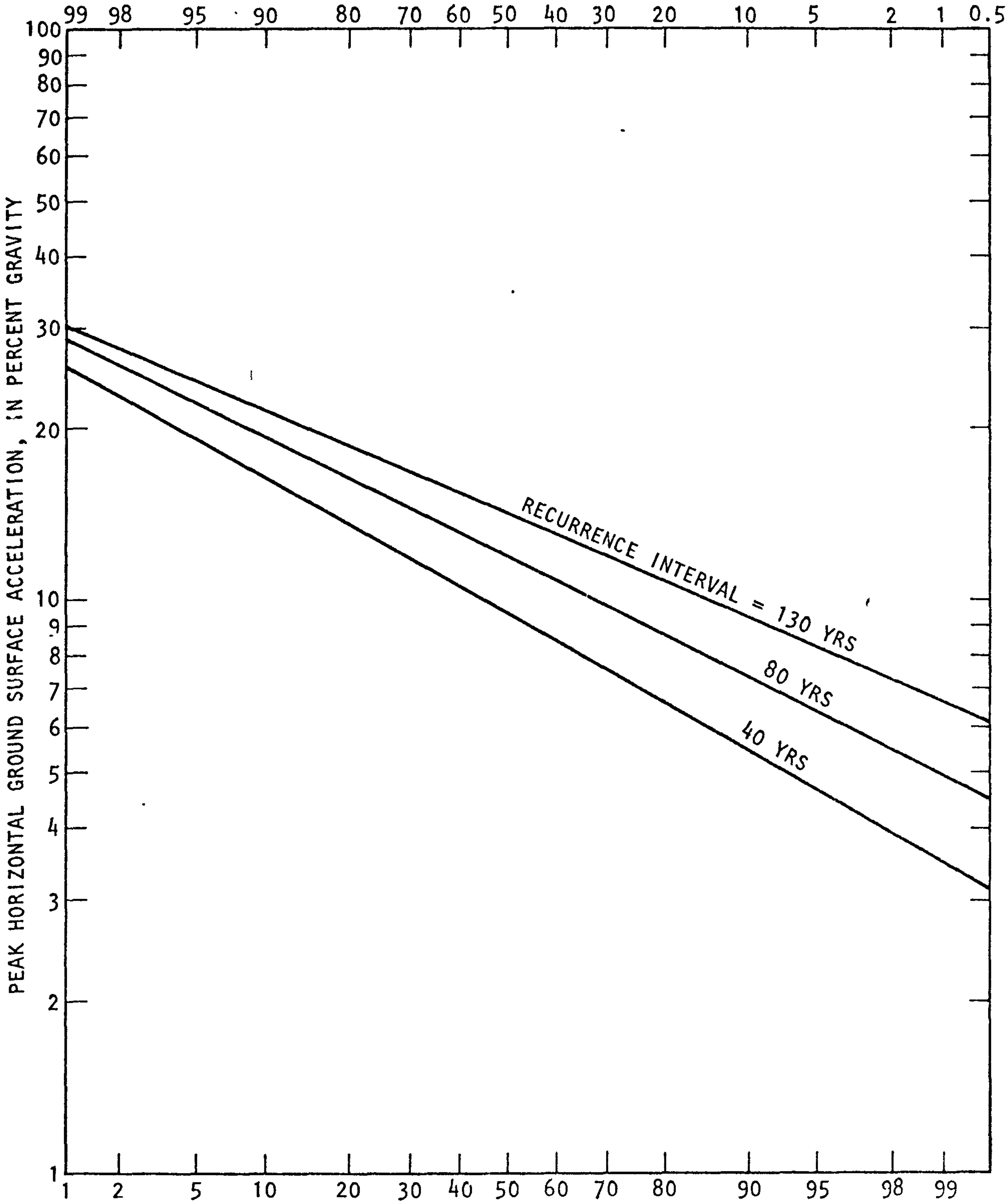


FIGURE 2-28. PERCENT PROBABILITY THAT ANY GIVEN SUBSTATION IN PORTLAND REGION WILL EXPERIENCE INDICATED PEAK ACCELERATION-BASED ON 7,500 MI^2 AREA

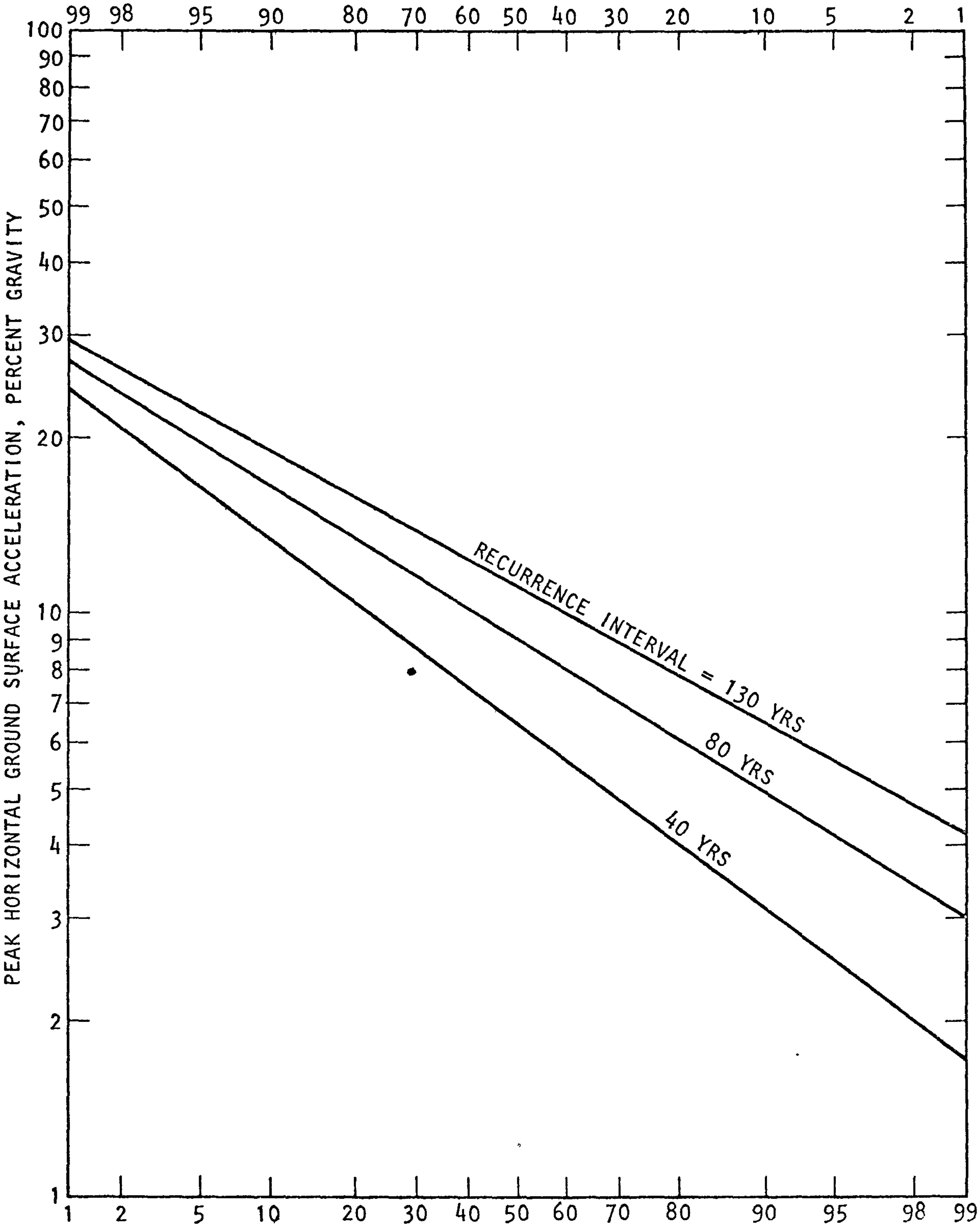


FIGURE 2-29. PERCENT PROBABILITY THAT ANY GIVEN SUBSTATION IN PORTLAND REGION WILL EXPERIENCE INDICATED PEAK ACCELERATION-BASED ON 15,000 MI^2 AREA

REFERENCES

2-1. Couch, R. D., and R. J. Deacon, *Seismic Regionalization Studies, Bonneville Power Administration Service Area,* Report No, 0-653, Shannon & Wilson, Inc., Portland, Oregon, October 1972. (Appendix E-1)

2-2. Fallgren, R. B., P. C. Jennings, and J. L. Smith, *Aseismic Design Criteria for Electrical Facilities,* Paper presented at American Society of Civil Engineers Structural Engineering Meeting, San Francisco, California, April 9-13, 1973.

2-3. *Preliminary Safety Analysis Report, Trojan Nuclear Power Plant,* Portland General Electric Company.

2-4. *Evaluation of the Geologic and Seismic Hazard of the Veterans Administration Hospital Site in Spokane, Washington,* Report No. R-7235-2435, Agbabian Associates, July 1972.

2-5. Housner, G. W., "Intensity of Earthquake Ground Shaking Near a Causative Fault," *Proceedings of the Third World Conference on Earthquake Engineering,* Aukland and Wellington, New Zealand, January 1965.

2-6. Newmark, N. M., and J. W. Hall, "Seismic Design Criteria for Nuclear Reactor Facilities," *Fourth World Conference on Earthquake Engineering,* Santiago, Chile, January 1969.

2-7. Duke, C. M., and D. J. Leeds, *Site Characteristics of Southern California Strong Motion Earthquake Stations, Part I,* Report No. 62-55, University of California, Los Angeles, Department of Engineering, November 1962.

2-8. Duke, C. M., et al., *Subsurface Site Conditions and Geology in the San Fernando Earthquake Area,* Report No. 7206, University of California, Los Angeles, Department of Engineering, December 1971.

2-9. Wiegel, R. L., Chapter 4 of, *Earthquake Engineering,* Prentice Hall, Inc., 1970.

CHAPTER 3

SEISMIC CLASSIFICATION PROCEDURE FOR EQUIPMENT AND SUPPORT STRUCTURES

INTRODUCTION

This chapter outlines the procedures used to provide a seismic classification of the electrical equipment and support structures located at the Celilo Converter Station and at the Big Eddy, John Day, Allston, and Custer substations. The objective of the classification was to identify the electrical equipment susceptible to damage from earthquake ground motion. The general procedure is illustrated by flow chart in Figure 3-1 and can be described briefly in the following manner.

The classification procedure can be logically divided into three phases. These are the following:

- Data collection and criteria formulation
- Analytical investigation
- Initial and final classification

Under data collection, information concerning foundation, support structure and equipment construction and installation was first collected and reviewed. The soils data and available test information were then combined with the equipment and support information in a data summary for each item of equipment. Classification criteria were established for earthquake ground motion, material properties, allowable stresses and factors of safety during this initial phase. A dynamic structural model of the equipment was then formulated consistent with the general overall configuration to obtain an estimate of the equipment frequencies. A preliminary analysis followed using the applicable response spectrum, damping and model to obtain an estimate of the stresses in the major components. The resulting stresses or internal forces were then compared with the classification criteria and the factor of safety computed to provide an initial classification into one of the following three categories:

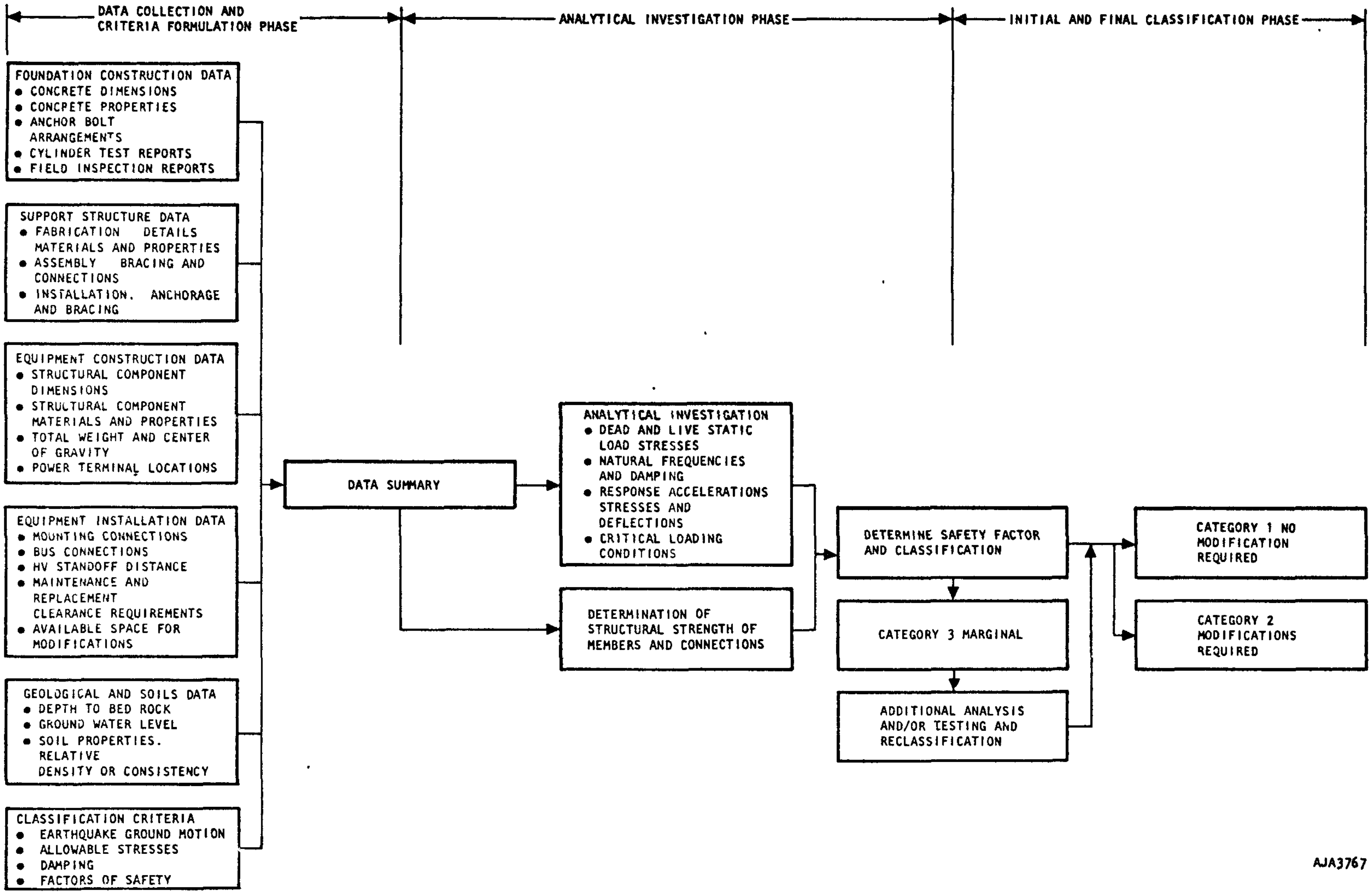


FIGURE 3-1. GENERAL CLASSIFICATION PROCEDURE

Category 1 - No modification required

Category 2 - Modification required

Category 3 - Marginal, additional investigation required

Items placed in Category 3 were subjected to a more detailed analysis before finally being placed in Categories 1 or 2.

The details of data collection are clearly enumerated in Figure 3-1 and require no additional discussion. However, documentation of the classification criteria and of the analysis and classification procedures is required and is provided in the following sections. A summary of the final equipment classification is also provided at the end of the Chapter for the stations investigated.

CLASSIFICATION CRITERIA

The horizontal earthquake ground motion used in the analyses was defined by response spectra and is provided in Figures 3-2, 3-3, and 3-4. Response to vertical earthquake ground motion was assumed to be equal to two-thirds of the horizontal response. For complex analyses, time motion records from previous earthquakes were used which provided response equal to or greater than the criteria spectra for the range of frequencies defined by the natural frequencies of the structure being analyzed. The ground motion criteria in Figures 3-2, 3-3 and 3-4 were developed from a detailed consideration of the sites involved and was done prior to the formulation of the general criteria in Chapter 2.

The material design properties, allowable stresses and/or factors of safety required for classification were in accordance with the following applicable codes or specifications unless otherwise noted:

a. <u>Steel</u>. The applicable specifications were (1) AISC "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings," 7th Edition and (2) AISI "Specification for the Design of Light Gage Cold-Formed

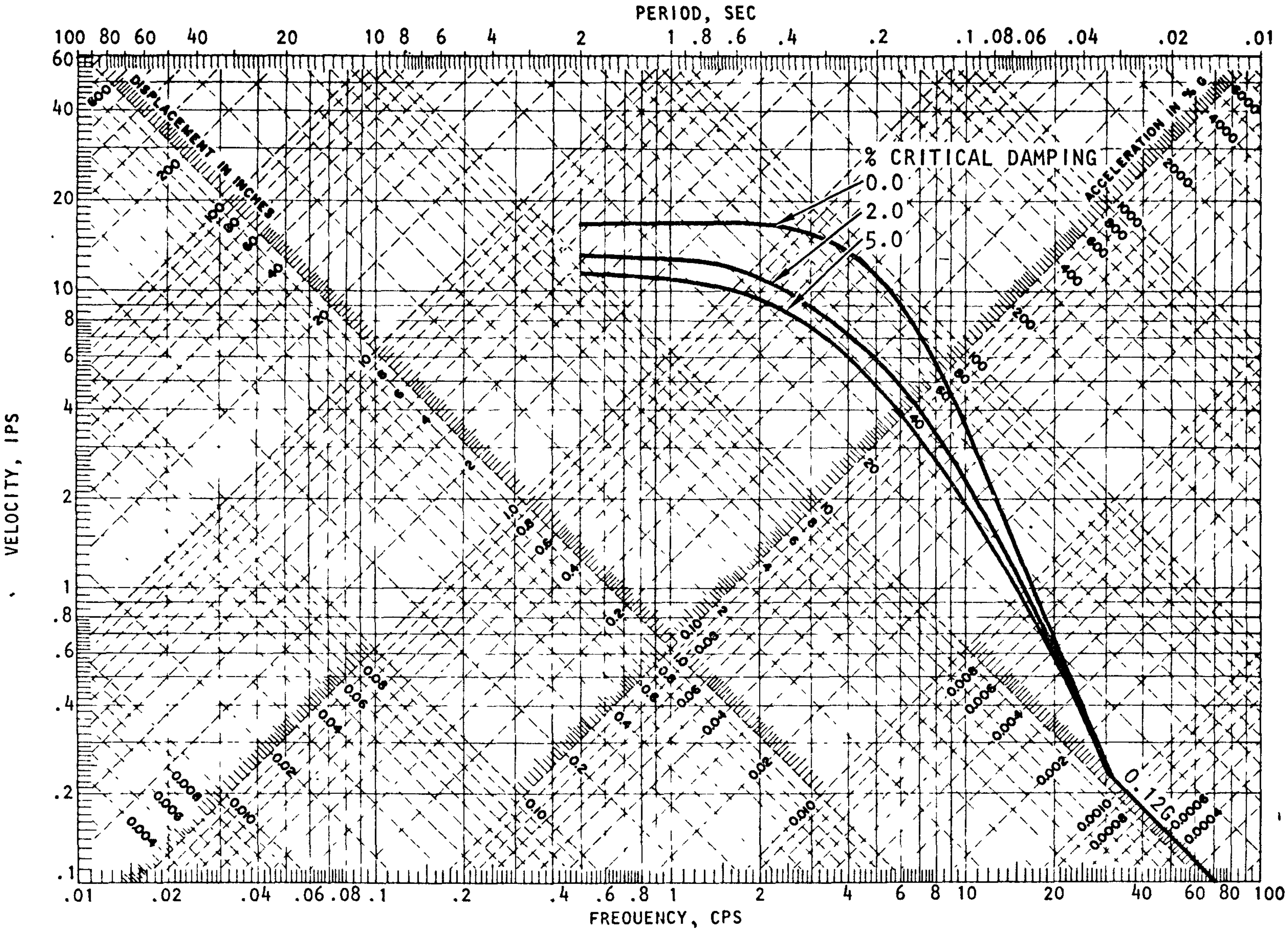


FIGURE 3-2. DESIGN RESPONSE SPECTRA FOR HORIZONTAL GROUND MOTION: CELILO CONVERTER STATION, BIG EDDY SUBSTATION, AND JOHN DAY SUBSTATION

R-7211-13-2859

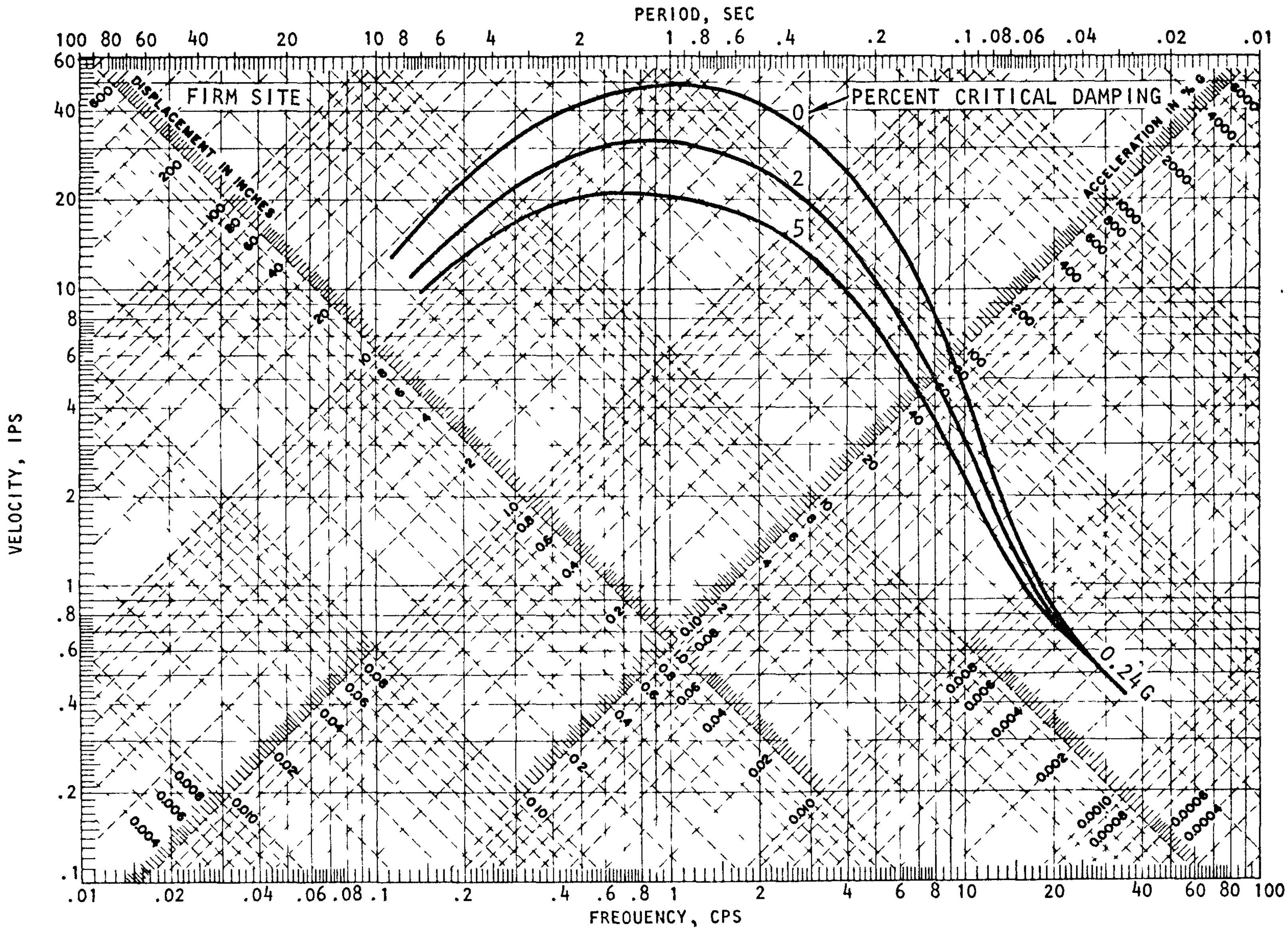


FIGURE 3-3. DESIGN RESPONSE SPECTRA FOR HORIZONTAL GROUND MOTION AT ALLSTON SUBSTATION

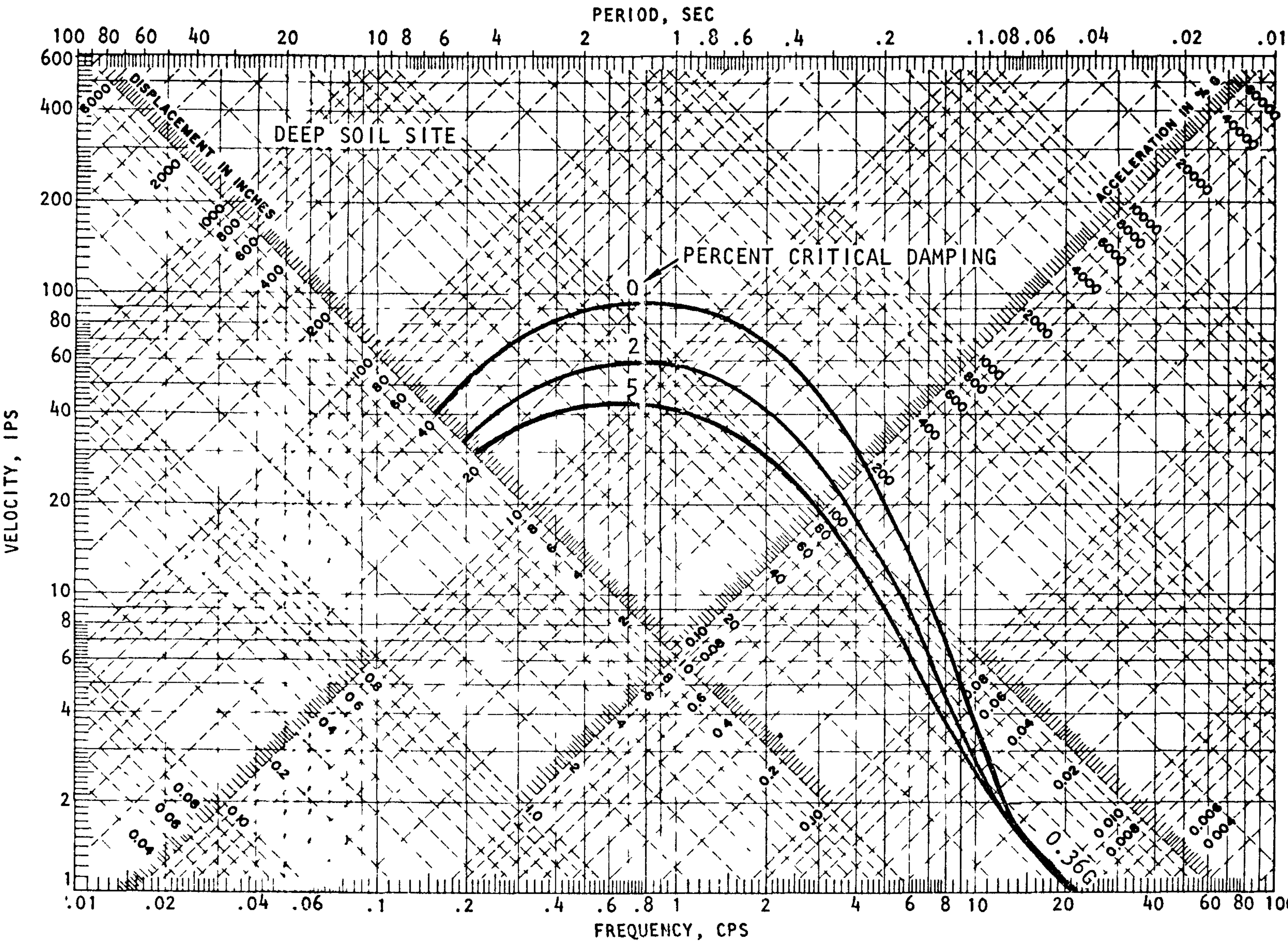


FIGURE 3-4. RESPONSE SPECTRA FOR HORIZONTAL EARTHQUAKE GROUND MOTION AT CUSTER SUBSTATION

Steel Structural Members," and (3) UBC, 1970 Edition, Chapter 27. The safety factors incorporated in the AISC and AISI recommended allowable design stresses were utilized, except that uninspected welds were evaluated at 1/2 of recommended design stresses.

b. Structural Aluminum. The applicable specifications were (1) ASCE Paper No. 3341, "Suggested Specifications for Structures of Aluminum Alloy 6061-T6 and 6062-T6," (2) ASCE Paper No. 3342, "Suggested Specifications for Structures of Aluminum Alloy 6063-T5," and (3) UBC, 1970 Edition, Chapter 28. The safety factors incorporated in the recommended allowable design stresses noted in these sources were utilized except that welds for which the inspection records were not traceable were evaluated at 1/2 of recommended design stresses.

c. Concrete. The applicable specifications were (1) ACI "Standard (ACI 318-71) Building Code Requirements for Reinforced Concrete," and (2) UBC, 1970 Edition, Chapters 26 and 29. Existing concrete, for which the design ultimate stress data was unavailable, was assumed to have an ultimate compressive strength, f'_c, of 2000 psi.

d. Metal Castings. The applicable ASTM specifications were consulted to establish the minimum tensile stress. A nominal safety factor of 3.0 based on the proportional limit was required for classification in category (1).

e. Ceramic Insulators. The ultimate mechanical strength of insulators was determined in accordance with the applicable ASA standards or manufacturers' test data. The bending moment capacity of insulators was based on the ultimate cantilever load. The effective bending modulus (EI) of insulators was estimated from the cantilever load deflection plot when available from the manufacturer (see Reference 3-1). Ceramic insulators were evaluated as complete units, including end fittings, where adequate manufacturers' test data were available for such assemblies. Insulators for which adequate test data were not available were evaluated by computing the ultimate strengths from the material and section properties of the insulators and comparing the results with the computed properties of insulators which have been tested. The design ultimate strengths of the insulators were then adjusted to reflect the same relation as that between the computed and test values of ultimate strength for the tested insulators.

A nominal safety factor of 3.0 based on ultimate values was required for classification in category (1).

f. Soils and Foundations. Soil-structure interaction was not a substantial consideration except for equipment-support structure-foundation items which were treated as rigid bodies. For these items, the following assumptions were made:

- The horizontal shear transfer between soil and foundation was assumed to act only as friction between the base of shallow foundation slabs and supporting soil. The effect of embedment was not considered.

- Since the friction coefficient between the foundation and supported equipment is affected by horizontal and vertical motion, only positive anchorage such as anchor bolts was considered to provide shear transfer between the foundation and the equipment. The friction force between equipment and foundation due to preloading the anchor bolts was used in addition to the direct shear strength of inserts to resist horizontal loading.

- For classification in category (1), M_{DL}, the dead load moment of stability of the equipment was required to provide a nominal safety factor of 3 (i.e., $M_{DL}/M_0 \geq 3$) against seismic overturning, M_0, for rigid equipment. The moment of stability also included resisting moments from anchorages or embedments. Such anchorages were designed for the corresponding loading with a factor of safety of 3 based on yield. Foundations for rigid equipment, which formed a short extension of the rigid equipment, were designed for a factor of safety against overturning of 3.0 considering the combined equipment-structure system.

- Spread foundations of rigid equipment as well as spread footings of non-rigid equipment configurations were analyzed and/or designed assuming triangular distribution of soil base pressure, for earthquake-induced lateral inertial loads combined with vertical dead loads. In this case, the maximum soil bearing stress under the combined loading was required not to exceed two times the normal allowable bearing stress under static loading conditions but not more than 6 KSF in any event. The loaded area of the foundation was required to be not less than 2/3 of the total base area.

Damping (expressed as percent of critical damping) in electric power apparatus generally ranges from 0.5 percent to 7.0 percent. Figure 3-2 indicates the very large effect of damping on response. The only reliable method of determining damping is by actual testing of equipment as installed. In the absence of test data, conservatism was used in choosing damping values. The following list provides conservative estimates of damping values, based upon testing performed by utilities on typical equipment configurations:

a.	Steel or aluminum supported equipment (working stress range)	1 percent
b.	Insulator stacks	2 percent
c.	Insulator supported equipment frames	1-2 percent
d.	Foundation-soil interaction (Estimate including both internal and radiation damping)	5-10 percent

Where available, test data for similar equipment was used to establish the damping values used in analysis.

Both vertical and horizontal response was considered in the analysis. The combination of vertical and horizontal response which gave the most severe combination of stress was used. The horizontal earthquake component was oriented to provide maximum stresses in the structure.

ANALYSIS PROCEDURE

In general, most electrical power equipment can be placed in one of the model classes illustrated in Figure 3-5. The basic configuration and weight distribution determine the particular model type. Analysis procedures for the equipment models illustrated are discussed in subsequent paragraphs.

As far as feasible, each equipment/support unit, including its foundations, was treated as a discrete structure. For these typical cases it was assumed that connecting power cables or bus did not impose significant loads or restraint on the units under investigation. If the existing installation made this assumption invalid, recommendations were made for the provisions of flexible connections to the bus or a more extensive analysis was performed which included the bus and power cable, their support structure, and the dynamic interaction of the interconnected units.

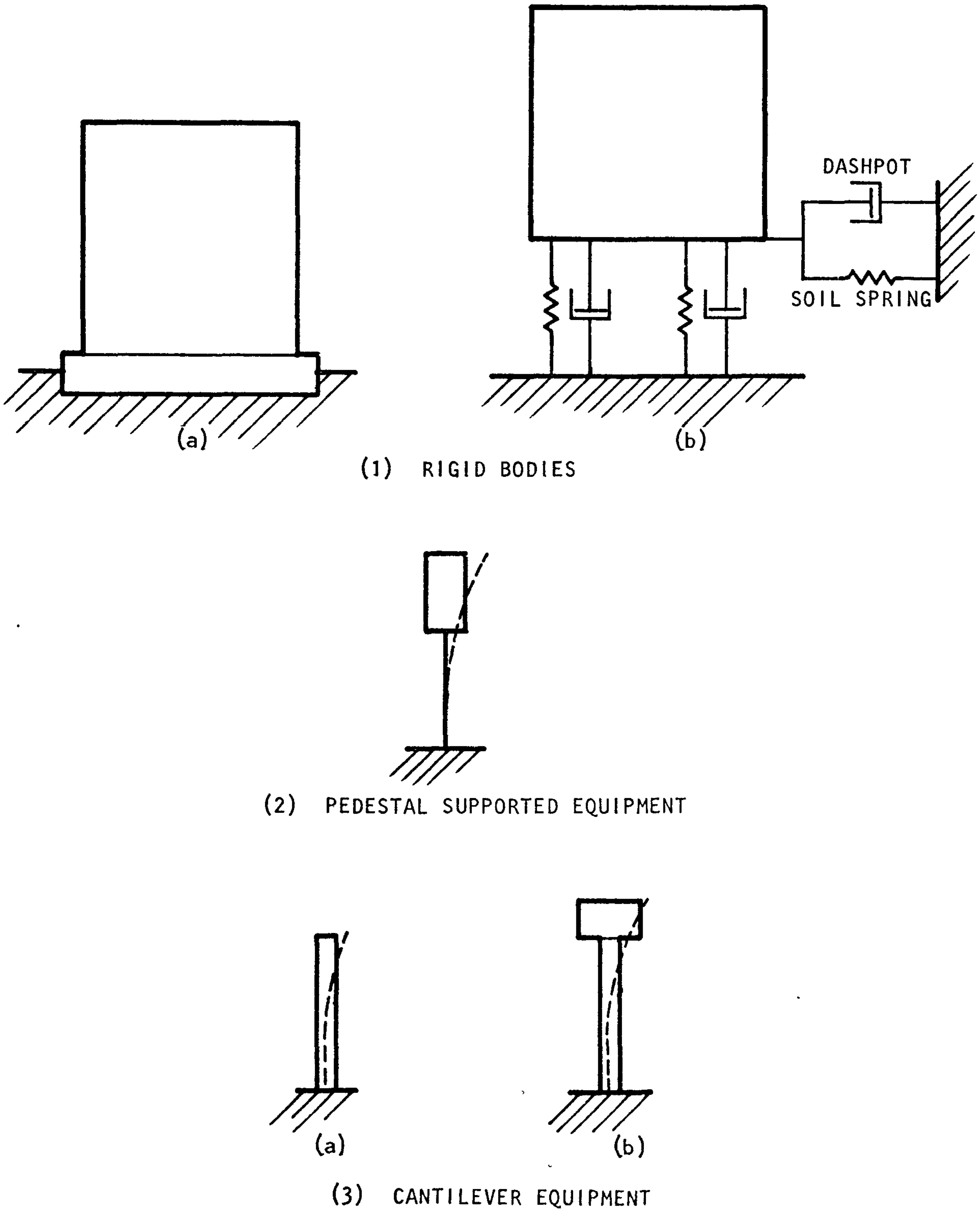
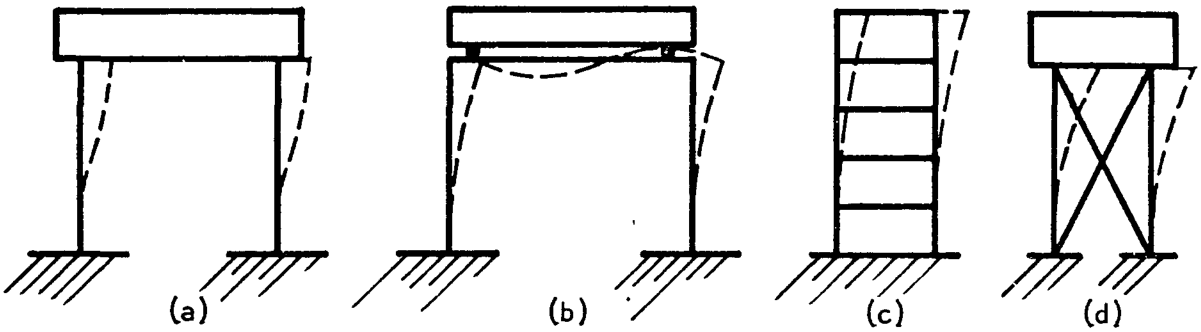
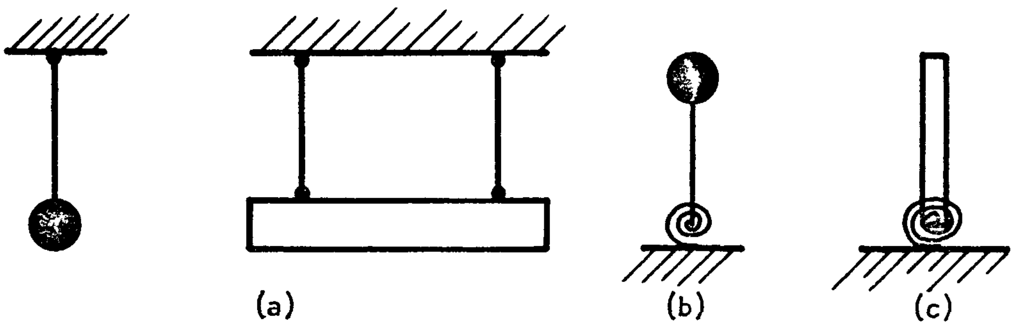


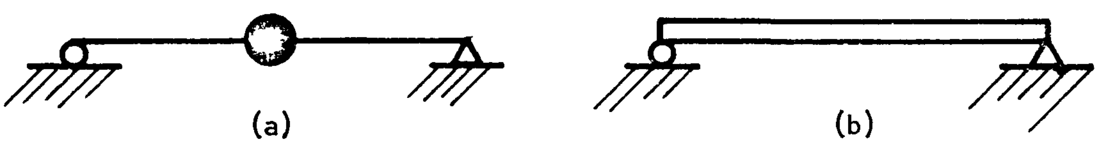
FIGURE 3-5. EQUIPMENT MODEL CLASSES



(4) FRAME SUPPORTED EQUIPMENT

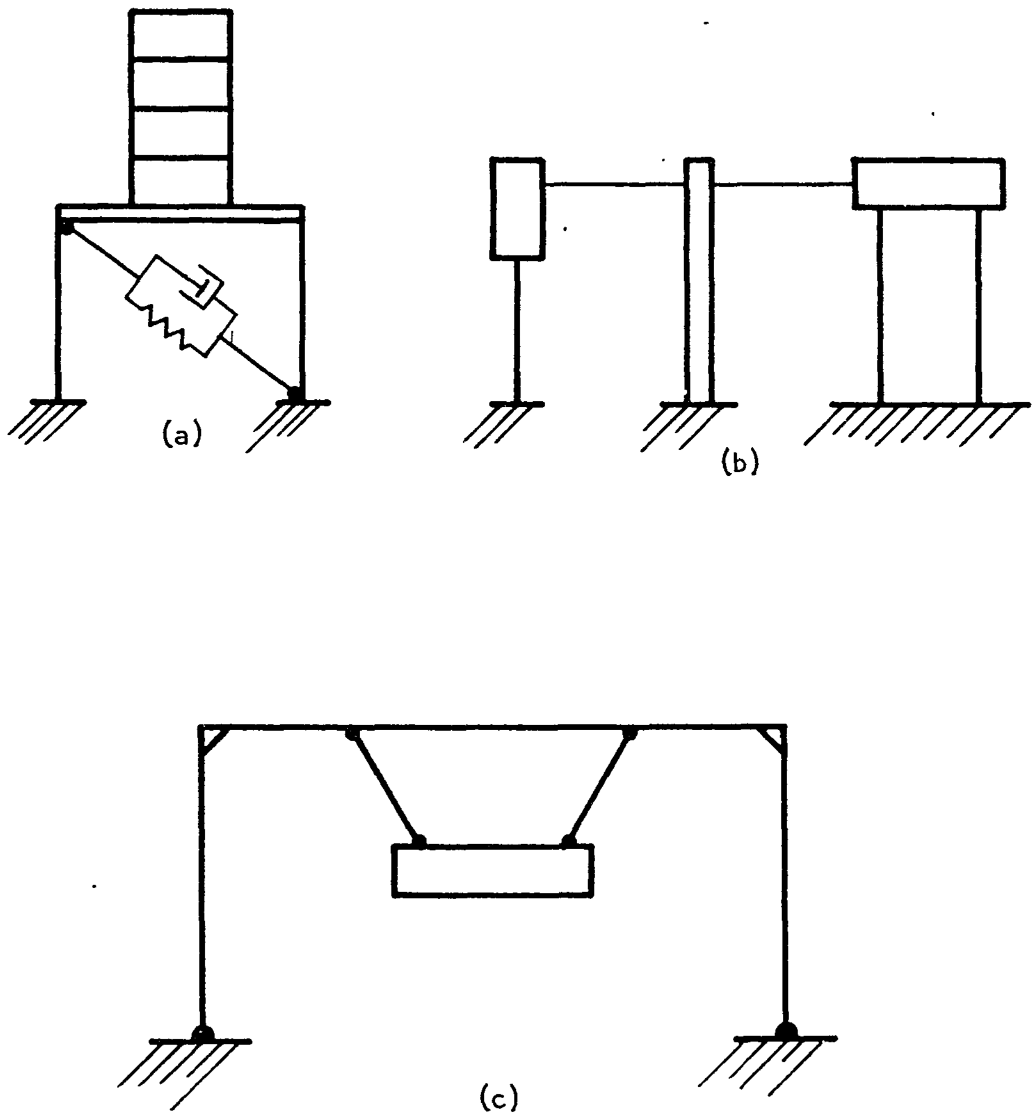


(5) PENDULAR SUPPORTED EQUIPMENT



(6) BEAM OR BEAM SUPPORTED EQUIPMENT

FIGURE 3-5. (CONTINUED)



(7) COMPLEX EQUIPMENT--COMBINATIONS OF 1 - 6

FIGURE 3-5. (CONTINUED)

For many items of equipment a simple approximate calculation determined the fundamental frequency of vibration within reasonable limits of accuracy. Then, using the design response spectrum associated with the assumed model, accelerations and stresses at critical points in the model were calculated with reasonable accuracy. For items which had either a complex configuration, interconnection with other equipment, poor connection details, or nonlinear response, a simple model could not yield reliable information and more refined models were required.

A brief discussion of the equipment models follows:

a. Rigid Equipment. Rigid equipment was defined as items having an estimated natural frequency of vibration greater than 30 cps. For equipment in this class a frequency estimate of the combined equipment-foundation system, considering the foundation to be supported by soil-spring-dashpots, was made in accordance with Reference 3-2.

   For soil sites a surface shear wave velocity of 400-700 ft/sec was assumed. The unit weight of the soil was taken as 100-125 lb/ft^3, and Poisson's ratio was assumed to be equal to 0.35-45.

   The amplified equivalent lateral force due to the foundation-soil interaction was used to determine overturning and base shear resistance.

   Examples of rigid equipment considered were converter transformers, oil circuit breakers, and filter reactors.

b. Pedestal Support Equipment. The pedestal equipment model represented cases where heavy, stiff, equipment was supported on a relatively light, flexible post-type, support structure fixed to a foundation. This model assumed that the equipment was very stiff compared to the support. Both translational and rotational inertia of the equipment as a rigid body were considered in the model. This model was used in estimating the natural frequency for cases where the equipment bending mode natural frequency was greater than twice the frequency computed for the system considering the lumped mass of the equipment and the stiffness of the pedestal only. Examples of pedestal equipment were potential transformers, coupling capacitor potential devices, and isolating transformers.

c. Cantilever Equipment. The cantilever model assumed that the weight and stiffness of the equipment were distributed along the equipment length. Examples of cantilever equipment were lightning arresters, insulator stacks, and valve damping capacitors.

d. Frame Supported Equipment. The frame model was applied to cases where heavy equipment was supported on rigid frame or braced frame support structures. Both single and multilevel configurations were considered. This model was used in estimating the natural frequency of equipment support systems for cases where the equipment natural frequency was greater than twice the frequency computed from the combined equipment and frame model. The valve platforms and AC capacitor racks were examples of frame supported equipment.

e. Pendular Supported Systems. The pendular model was applied to items of equipment supported as a simple pendulum in either the hanging or inverted configuration. It should be noted that this type of system becomes nonlinear if vertical motion is considered. An example of pendular supported equipment was the RI filter reactor.

f. Beam or Beam Supported Equipment. The beam model was applied to equipment with multiple supports. Equipment with weight and stiffness uniformly distributed as well as beams carrying heavy equipment were modeled as beam supported. An example of beam type equipment was tubular bus.

g. Complex Items. Complex items were combinations of two or more of the above models which in some cases required 3-D computer analysis for classification. Items in this category included current dividers and air power circuit breakers.

CLASSIFICATION PROCEDURES

The internal stresses or resultant forces computed from the analysis were compared to the allowable or ultimate stresses of the critical component and the factor of safety computed. When allowable stresses were used, the factor of safety was calculated as follows:

$$\text{F.S.} = \frac{\text{(allowable force)} \times \text{(code safety factor)}}{\text{computed force}}$$

When ultimate stresses were used, the factor of safety was calculated as follows:

$$\text{F.S.} = \frac{\text{ultimate force}}{\text{computed force}}$$

The computed factors of safety were compared to the required safety factors given in the classification criteria and classification was then made to one of the three categories as follows:

Category 1: $R \geq 1.05$

Category 2: $R \leq 0.95$

Category 3: $0.95 < R < 1.05$

Here, R is defined as the ratio of the computed safety factor to the required safety factor. Category 3 items were reviewed and more refined analysis provided for final classification into Category 1 or 2 as previously noted. A flow chart of this classification procedure is provided in Figure 3-6.

FINAL SEISMIC CLASSIFICATION

Items subjected to investigation and seismic classification were, in general, selected by BPA and were listed in Exhibit I of the basic contract. However, additional items were included in the study after an on-site inspection and drawing review. A summary of the final seismic classification assigned to the equipment and support structures investigated at the Celilo Converter Station and at the Big Eddy and John Day substations is provided in Table 3-1. A similar summary for the Allston and Custer substations is provided in Table 3-2. Design modification concepts and sketches for Category 2 items are provided and discussed in Chapter 4.

REFERENCES

3-1. Borhaug, J. E., et al., "The Response of Substation Bus Systems to Short Circuit Conditions, Part II: Measurements on the Transverse Vibration of Selected Station Post and Pin-Cap Insulators," *IEEE Trans. Power Apparatus and Systems,* Vol. 90, 1971, pp. 1706-1711.

3-2. Richart, F. E., et al., "Vibrations of Soils and Foundations," Prentice-Hall, 1970.

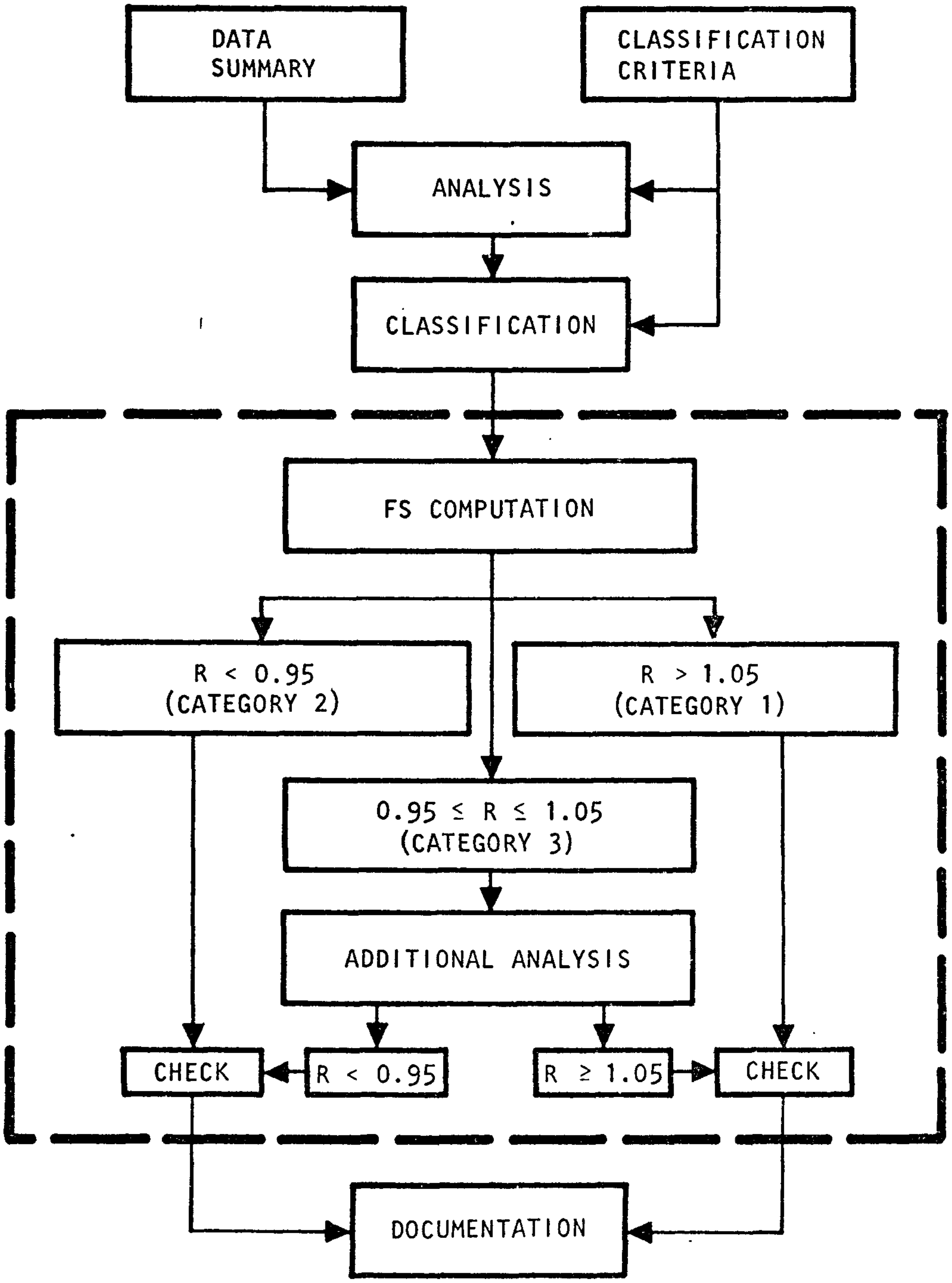


FIGURE 3-6. CLASSIFICATION PROCEDURE

TABLE 3-1. FINAL SEISMIC CLASSIFICATION FOR CELILO CONVERTER STATION AND BIG EDDY AND JOHN DAY SUBSTATIONS

Area/Equipment Item	Seismic Classification Category
Bridge Room--Celilo	
Valve platforms	1
Current dividers	2
Bushing reactors	1
Anode reactors	1
*Isolating transformers	1
Wall bushings	1
*Cathode bus supports	1
*Anode bus supports	1
Valve Damping Yard--Celilo	
Valve damping resistors	1
Valve damping capacitors	2
AC lightning arrestors	
L/L	1
L/G	
650 bil	1
900 bil	2
1300 bil	2

SYMBOLS
1 = Category 1--no modifications required
2 = Category 2--modifications required
3 = Category 3--marginal, additional analysis required
NR = Modifications not required
* = Indicates items not listed in Exhibit I

TABLE 3-1. (CONTINUED)

Area/Equipment Item	Seismic Classification Category
Valve Damping Yard--Celilo	
DC lightning arrestors	
650 bil	1
900 bil	2
1300 bil	2
Load break switches	1
*Disconnect switches	1
Converter transformers	2
Power circuit breakers (230 kv)	2
*230 kv disconnect switches	1
Wall bushings	1
*Bus and bus supports	1
DC Yard--Celilo	
Coupling capacitors	1
*Line traps	1
400 kv disconnect switches	1
DC line arresters	2
DC surge capacitors	1
1300 bil transductors	1
Voltage dividers	2
6th harmonic filter capacitors	2
20th harmonic filter capacitors	2
*DC filters	1

TABLE 3-1. (CONTINUED)

Area/Equipment Item	Seismic Classification Category
DC Yard--Celilo	
*DC smoothing reactors	2
RI and neutral bus filter reactors	1
Bus and bus supports	1
*Load break switches	1
*Disconnect switches	1
DC SR Lightning arrester	2
AC Yard--Celilo	
PT grid timing	1
Load break switches	1
PF capacitors	2
Power circuit breakers (HSF6)	2
HP filter capacitors	2
HP filter reactors	2
HP filter resistors	2
11th and 13th Harmonic filter capacitors	2
11th and 13th Harmonic filter reactors	2
AC lightning arresters	1
*Disconnect switches	1
*Bus and bus support	1

TABLE 3-1. (CONTINUED)

Area/Equipment Item	Seismic Classification Category
Voltage Conditioning Room--Celilo	
High current anode reactors	1
Copper ring and suspension system	1
Converter Station Building--Celilo	
General steel	1
Stone work	2
Air conditioner mounts	1
Current divider supports	1
Degassing Transformer Yard--Celilo	
*Oil conservator	2
Degassing reactor	1
Spare Storage--Celilo	
*Degassing transformers	2
*Degassing reactors	2
*HP filter reactors	2
*11th and 13th filter reactors	2
*5th and 7th filter reactors (Big Eddy)	2

TABLE 3-1. (CONTINUED)

Area/Equipment Item	Seismic Classification Category
Spare Storage--Celilo	
*1300 bil transducers	2
*Voltage dividers	2
*Isolation transformers	2
*Lightning arresters (parts)	2
*DC reactors	2
*Converter transformers	2
*Oil tanks	2
Big Eddy	
5th and 7th Harmonic filter reactors	2
5th and 7th Harmonic filter capacitors	2
230 kv Power circuit breakers	2
John Day	
500 kv Power circuit breakers	2

TABLE 3-2. FINAL SEISMIC CLASSIFICATION FOR ALLSTON AND CUSTER SUBSTATIONS

	Seismic Classification Category
Allston Substation	
500 KV Yard	
APCB/CT	2
American Elin PT	2
*Fuji Transformers	2
*Lightning Arrester	2
*Disconnect Switches (ITE)	2
*Bus Supports	1
*Dead End Towers	1
Control Building and Station General	
*Building	2
*Control Room Ceiling	2
Microwave Tower	1
Custer Substation	
500 KV Yard	
APCB/CT	2
Toshiba-Mitsui PT	2
Pantograph Switch A/C	2

SYMBOLS
- 1 = Category 1--no modifications required
- 2 = Category 2--modifications required
- 3 = Category 3--marginal, additional analysis required
- NR = Modifications not required
- * = Indicates items not listed in Exhibit I

TABLE 3-2. (CONTINUED)

	Seismic Classification Category
Custer Substation (Continued)	
500 KV Yard (Continued)	
Lightning Arresters-GE	2
*Transformers-Brown/Boveri	2
*Disconnect Switches (ITE)	2
*Dead End Tower	1
*Bus Supports	1
230 KV Yard	
*PCB Penn/ITE	2
*Disconnect Switches (ITE)	2
*Dead End Tower	2
Station General and Control Building	
*Control Room Ceiling	1
Microwave Tower	1

Note: The classification for the Custer Substation is based upon the assumption that foundation instability does not occur. However, this site is considered susceptible to liquefaction under strong earthquake ground motion.

CHAPTER 4

DESIGN MODIFICATIONS FOR ELECTRICAL EQUIPMENT AND SUPPORT STRUCTURES

INTRODUCTION

This chapter provides a discussion of design modification concepts and design sketches for equipment and support structure modifications. The modifications are for items listed in seismic classification Category 2 at the Celilo converter station and at the Big Eddy, John Day, Allston and Custer substations. These items are listed in Tables 3-1 and 3-2 of Chapter 3. The recommended design modifications are preceded by a discussion of the general approach followed in developing the modification concepts.

GENERAL APPROACH

There are four potentially feasible basic methods of increasing the seismic resistance level of existing equipment-support combinations:

- Minimize the system response to seismic excitation by changing the natural frequency, increasing the damping, or shock isolation
- Reinforce the existing support structure
- Install new structural supports to brace existing structures
- Replace the existing support structure with redesigned structures

The last three methods may stiffen the structures sufficiently to change the natural frequencies, and it will usually be necessary to reinvestigate the structure's seismic response. Any logical choice among these four methods of modification will be influenced by consideration of relative direct cost, relative shutdown time required for the work, and relative confidence in the adequacy of the end product. The confidence level may be the controlling consideration, since no modification can be justified without assurance that the resistance of the modified structure to seismic ground motion will fully satisfy the design criteria. A general discussion of each of these methods follows.

Measures to Minimize System Response

Response to seismic excitation is largely determined by the natural frequency and damping of the equipment. As an example, using the response spectra for the Celilo Converter Station (Figure 3-2, Chapter 3), a reduction of natural frequency from the range of 2 to 4 cps at 2 percent damping to the range of 0.5 to 1 cps at 5 percent damping will reduce acceleration response from 0.45 g to a range of 0.10 g to 0.18 g.

In cases where the existing equipment-support arrangements have low damping, moderately low natural frequencies and moderate overstress conditions, reducing the natural frequency and increasing damping may be the simplest and most economical modification that can be applied to increase the resistance to seismic ground motion. At initial natural frequencies lower than about 0.5 cps relative displacement rather than acceleration is likely to be the critical response, and an increase in the natural frequency may be beneficial. At frequencies between 4 cps and 10 cps an increase in damping is effective in reducing response but varying the natural frequency sufficiently to be beneficial is difficult.

Reduction of natural frequency is most directly accomplished by reducing system stiffness through introduction of spring elements in the primary support structure, although, in some cases, increasing the supported mass may be a practical approach. If spring elements are introduced at all support points and are "soft" enough to provide nearly all of the deflection in the support structures, the result is a shock isolated system.

The most practical damping methods for the damping and frequency ranges of interest are hysteretic damping and friction damping. Hysteretic damping is an inherent property of elastomeric springs, but varies with the kind of elastomer, the specific compound and the type of stress in the element (i.e., bending, shear, or compression), and cannot be described separately from the spring characteristics. Other types of hysteretic elements are available, utilizing the cyclic plastic deformation of materials.

Friction damping can be introduced by separate damping units. For most of the potential applications to BPA facilities, friction damping incorporated in the spring assemblies is probably desirable. Considerable damping can be provided by parallel stacked Belleville spring washers which have a rubbing action as the stack is deflected.

Reinforcement of Existing Support Structures

Where it appears that the extent and cost of the modifications required are reasonable and the shutdown time is not excessive, in-place reinforcement may be the preferred modification method for all existing structures. Such modifications include connection reinforcement and limited additional bracing. However, the use of welding to reinforce connections in existing aluminum structure should be avoided due to the difficulty of obtaining qualified welders and adequate quality control for field welding of aluminum alloys.

Similarly, foundations which require modification to provide adequate anchorage for the supported equipment should be reinforced rather than being replaced wherever practical.

It should be noted that confidence level in the final product is very important. Where the original structure design is so indeterminate as to limit confidence in a modified or reinforced structure, the in-place modification is not warranted, and the structure should be replaced, or supported by new structural supports.

Use of Additional Structural Supports to Brace Existing Structures

Where the necessary horizontal and/or vertical clearances are available, and the required access space for equipment maintenance and replacement is adequate to permit additional supports, the use of an external structure to provide additional lateral bracing for existing equipment support structures has been considered. In most cases high voltage insulation will require that such additional braces consist, at least in part, of insulator assemblies. There are two methods of bracing which appear potentially useful.

a. Insulator stacks used directly as diagonal guys or braces. This arrangement has the disadvantage of taking up a relatively large amount of yard space at ground level.

b. Overhead insulator columns, braces or diagonal suspension strings supported by new steel framing which spans the entire equipment group. This has the advantage of occupying a minimum of additional ground level space in the immediate vicinity of the existing equipment. However, clearances for overhead power lines and connection drops present a problem.

Replacement of Existing with New Support Structure

Replacement with new structures may be cost-effective for cases in which extensive modifications would be required to make existing equipment support structures adequate to resist the criteria seismic ground motions and in which the feasible measures available to minimize structure response does not provide adequate structural response attenuation. Where the principal modifications required occur in the lower tier of insulator supported frames, the most direct method of modification was in some cases found to be replacement of existing insulators with heavier insulators. However, complete replacement of the entire support structure was not often found feasible.

DESIGN MODIFICATION CONCEPT DEVELOPMENT

Each of the equipment support structures classified in Category 2 was reviewed to determine the most direct and cost-effective method of providing the required seismic resistance. Various modification concepts developed by others, as well as by AA, were studied and presented to BPA for review and approval. Design sketches which have been reduced and are presented in this chapter, were then prepared. Final construction drawings and specifications will be prepared by BPA as the modifications are to be implemented. A discussion of the studies associated with the modification concepts for the different items of equipment is provided in the remaining sections.

CURRENT DIVIDERS AND ANODE REACTORS

The current dividers and anode reactors are suspended from the valve hall roof at the Celilo converter station with insulator hangers to form a three-dimensional pendular system which can experience large displacements during seismic excitation. During the 1971 San Fernando earthquake, the same type of suspension system failed on all 42 current dividers at the Sylmar converter station. The damage caused by the falling equipment resulted in the highest cost and longest downtime for repairs of all the equipment damaged by the earthquake. Assurance of the satisfactory performance of this suspension system at Celilo was, therefore, a very important part of the design modification study and considerable effort was expended on the analysis of this system. A summary of the results of this study with recommendations follow. A more detailed discussion of the analyses performed on the current divider suspension system is provided in Appendix A-1.

An analysis was provided of the current divider and anode reactor support system in the initial Assessment Study (Reference 4-1) which was based upon a linear elastic model. This required certain simplifying assumptions which were provided because of time limitations. Also, between the time of completion of that study and the initiation of the design modification study, experimental field data became available on the building and suspension system response which could be used to improve the reliability of this model. Following the San Fernando earthquake shock on February 9, 1971, an accelerograph was installed (by others) on the roof of the valve hall at Sylmar and the actual fundamental frequency of the building was determined during an aftershock. (The valve halls at Celilo and Sylmar are similar structures.) The frequencies of the current divider and anode support system were also determined by tests conducted by the Los Angeles Department of Water and Power. Based on this information, the stiffness of the model was modified from that initially used to provide the measured fundamental frequency of the building.

A decision was also made at this time to divide the basic linear elastic model into two separate models which are shown in Figure 4-1. The first model was a modified building model and the second was an expanded model of a roof segment (1 bay) with a current divider and anode reactor. The modified building model was analyzed for an earthquake ground motion consistent with the response spectra given in Figure 3-2 of Chapter 3. Three components of the Helena 1935 earthquake ground motion were used as input in contrast to the C-1 artificial record used in the Assessment Study analysis. The latter record, since all three components were in phase, probably gave a more conservative stronger response. The computed lateral response of the building roof was then used as input motion to the expanded roof/current divider model. The vertical input motion for the model was assumed to be the vertical ground motion (i.e., the building acted as a filter for the horizontal motion but not the vertical input motion). The analysis procedure is shown schematically in Figure 4-1.

The results of the linear elastic analysis indicated that the suspension hangers for the current divider would not experience compression for the criteria ground motion. Thus, the results of this analysis gave lower post insulator hanger stresses than the Assessment Study and indicated that a concept in which the suspension insulators would be replaced with post insulators would perform well. However, replacing the suspension insulator

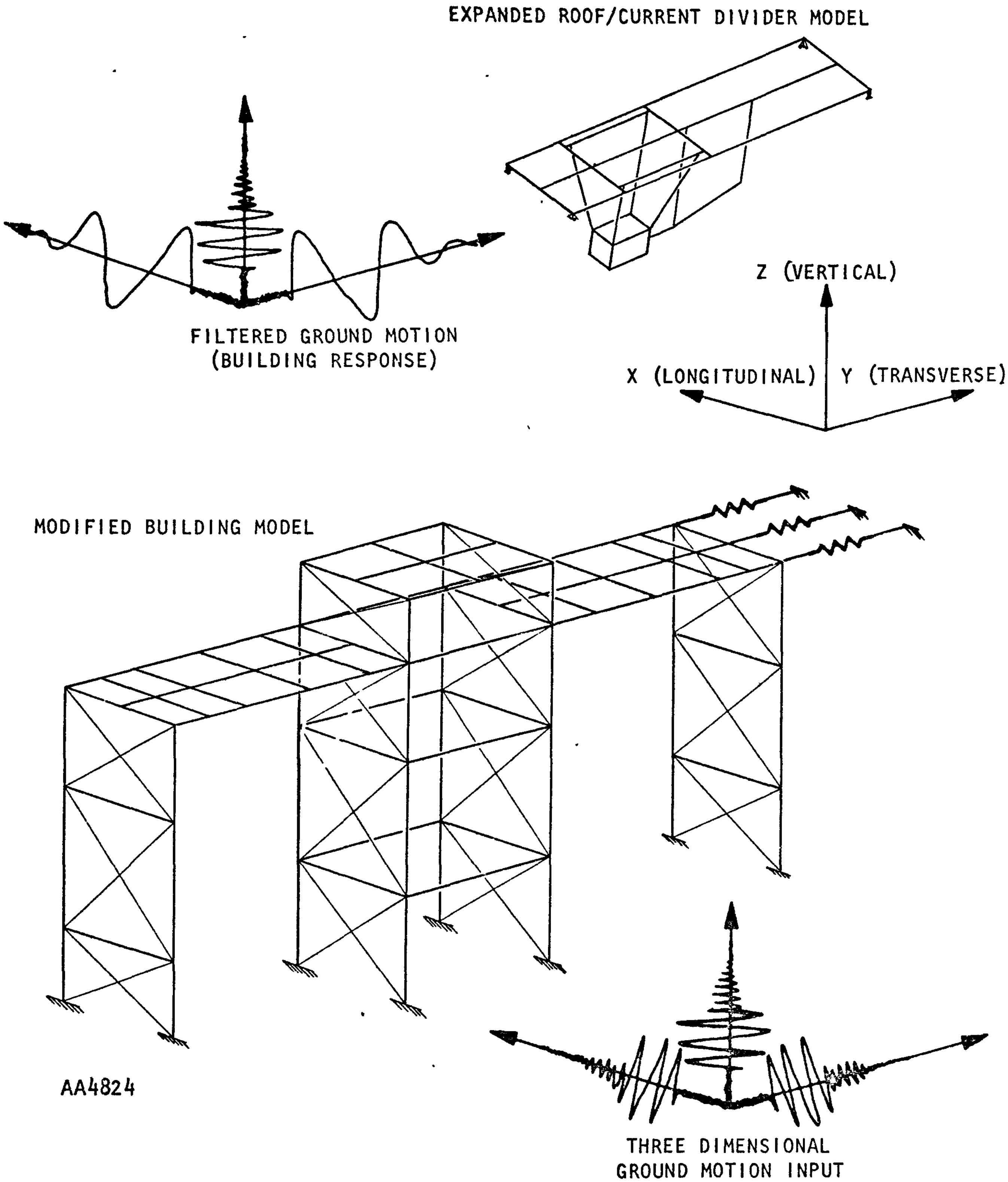


FIGURE 4-1. MODIFIED LINEAR MODELS

hangers would be a very expensive modification. In addition, the nonlinear effect of vertical excitation on the pendular system was unknown. For this reason, additional analysis was provided which included the nonlinear behavior of the present suspension insulator hangers in order to determine with confidence that the present suspension system would not experience impact under the criteria ground motion.

Both two- and three-dimensional nonlinear models were developed for this final analysis which incorporated the unloading capability of the present suspension hanger system. The results of the three-dimensional analysis indicated that the suspension hanger system did in reality experience several periods of unloading for the criteria ground motion, but impacting of the current divider on the unloaded hanger did not occur and the resulting peak hanger force did not exceed 5000 lbs. The displacement time history of a point on the current divider near a valve connection link was also computed and the estimated upper bound of the extension and contraction required in the current divider valve connection link was estimated to be ±4 in.

Since the present insulator hangers have a 15,000 lb rating, the factor of safety for the criteria ground motion is 3.0 and no revision of the suspension system is recommended. However, the current divider valve connecting links should be modified to allow for ±5 in. of relative extension or compression. This value is 25 percent greater than the upper extension bound computed and provides a factor of safety for displacement.

AIR POWER CIRCUIT BREAKERS

Air Power Circuit Breakers (APCB's) have been classified in Category 2 at Big Eddy, John Day, Allston and Custer substations. Since APCB's have been provided of different makes, and three different strengths of earthquake ground motion have been specified for these four substations, a more detailed classification is provided in Table 4-1. The range of natural frequencies for all APCB's reviewed was 1-3 cps and the factors of safety against collapse were all less than 3.0 (required for porcelain). If all APCB's were evaluated for the strongest ground motion, the factor of safety against collapse for all

TABLE 4-1. CLASSIFICATION OF AIR POWER CIRCUIT BREAKERS

ABCB	Frequency Estimate, cps	Damping Estimate (% Critical Effective Damping)	Allowable Static Lateral Load	Factor of Safety Against Collapse			
				Big Eddy 0.12 g ⊛	John Day 0.12 g ⊛	Allston 0.24 g ⊛	Custer 0.36g ⊛
500 KV Merlin-Gerlin	2.9	2%	0.15 g	1.1			<0.4
500 KV Delle-Alsthom (Cogenel)							
4 support insulators per column	1.7	2%	0.30 g			1.1	
5 support insulators per column w/guys	2.6	2%	0.36 g	2.6			
500 KV GE (500-2Y)	1.9	2%	<0.20 g		<1.67 (<1.0*)		
220 KV GE	1.7	2%	0.20 g	1.6			
220 KV Allis-Chalmers	1.9	2%	0.21 g	1.6			
(220 KV Oerlikon** minimum oil PCB)	1.1	2%	0.62 g			1.8	

⊛ Peak ground acceleration
* Operational factor of safety
** Included due to configurational similarity to APCB

APCB's would be less than 1.0. The APCB's have been designed to function primarily as electrical mechanisms and not as structures capable of resisting lateral inertia loading. In some designs, the porcelain support column must also function as a pressure vessel which causes initial stresses within the porcelain. Thus, the seismic fragility of the APCB should not be unexpected. Due to the fragility and operational importance of the APCB, modifications to insure operational survival in the specified seismic environment is required. The following modification concepts were considered:

1. Change of configuration
2. Strengthening of support columns
3. Incorporation of damping devices in the APCB
4. Isolation

Concept 1, which would change the configuration of the APCB (for example, introduce guys to brace the support columns), is desirable from a structural viewpoint but electrically undesirable due to the introduction of multiple leakage paths. In addition, a new configuration must be carefully analyzed to insure that dynamic amplication effects are reduced.

Concept 2, which provides for strengthening the support columns, is also a desirable structural modification. However, the support columns are specialized, costly porcelain elements which are not off-the-shelf replacement items. Concepts 1 and 2 would both require electrical and dynamic testing for validation. In addition, the APCB would have to be dismantled for modification.

Concept 3, which would introduce damping devices in the APCB, has been successfully implemented and dynamically tested by European APCB manufacturers for South American utilities. This concept is very desirable to attenuate the dynamic response of the APCB, however, this concept can only be implemented by the manufacturer who has intimate knowledge of the mechanical design details. The incorporation of an auxiliary mass damper in the APCB design is an alternate means of controlling the equipment response which can also only be implemented by the manufacturer.

Concept 4, which would isolate the APCB from ground motion, was found to be the least-costly modification alternative for existing breaker installations. This concept would involve the placement of the APCB upon a support platform which would be isolated or heavily damped to attenuate the APCB response. This concept is, therefore, recommended.

The feasibility of two seismic isolation concepts were examined in detail for the APCB's listed in Table 4-1 and will be briefly summarized here. A more detailed discussion of the analysis of these two concepts is provided in Appendix B-1. The basic two degree-of-freedom (DOF) model used in the design studies is shown in Figure 4-2. The model allows freedom in choosing three support parameters, M_p (platform mass), K_I (isolation stiffness), C_I (effective isolation damping), to achieve attenuation of the equipment response. Previous studies have indicated that the greatest reduction to APCB seismic response is given by a low frequency, damped support system. However, the displacement response of such a system can be several inches due to the required flexibility of the support. It should be noted that the large displacement response (stroke) is required in order for a damping device of realistic dimensions to achieve the required damping level. The simplest type of flexible isolation is a platform supported by rocker arms forming a pendular system. This type of system is shown in Figure 4-3. An alternative support system is shown in Figure 4-4. In this case, the supports are elasto-plastic elements which form a hysteretic two degree-of-freedom system.

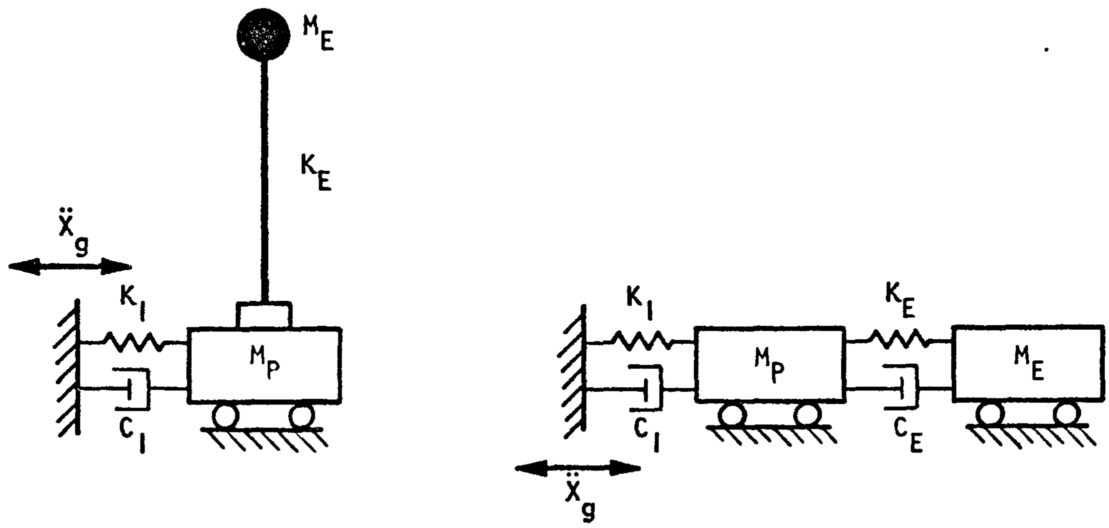


FIGURE 4-2. BASIC EQUIPMENT ISOLATION MODEL FOR DESIGN STUDIES

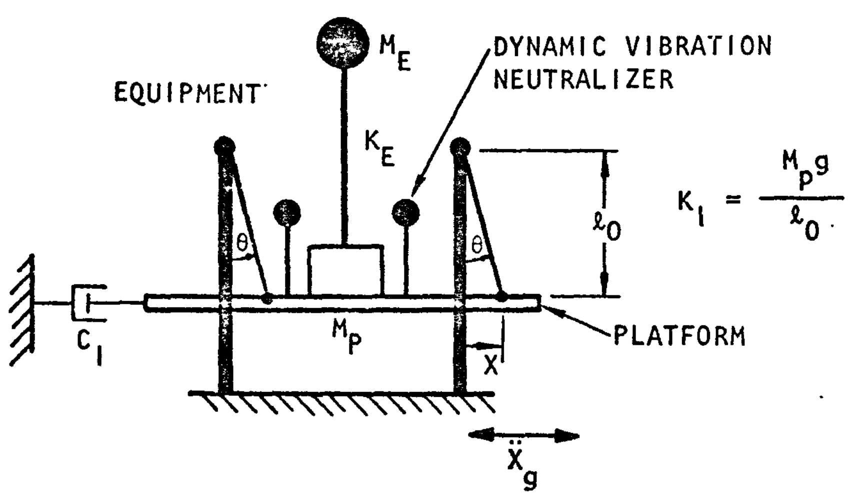


FIGURE 4-3. PENDULAR ISOLATION CONCEPT

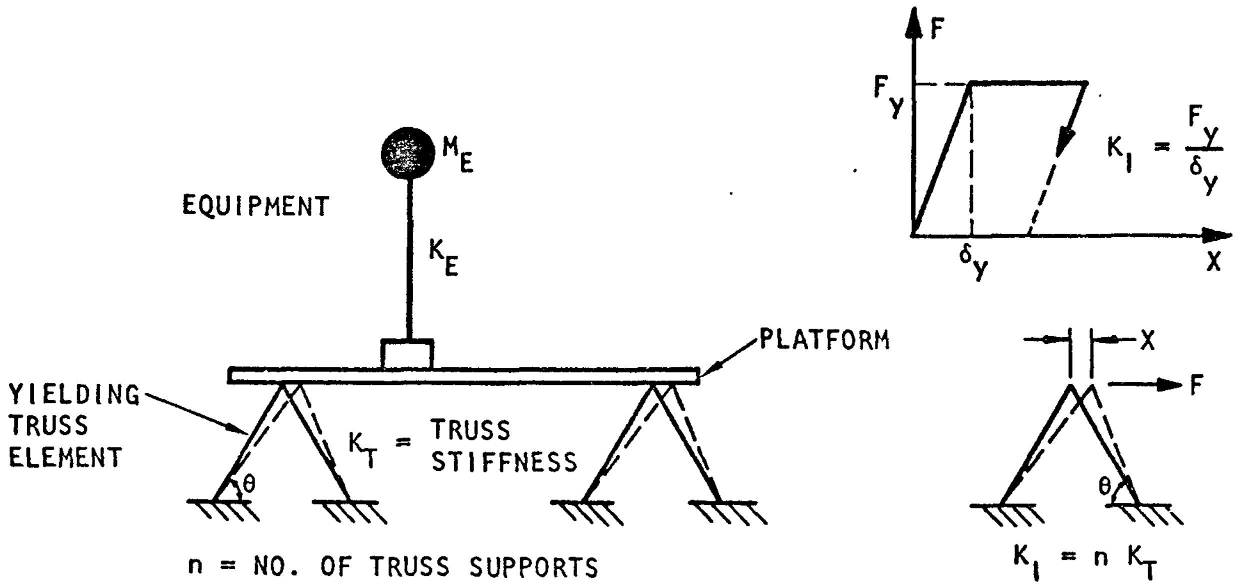


FIGURE 4-4. YIELDING SUPPORT ISOLATION CONCEPT

The pendular platform system is applicable to all APCB configurations, since the entire APCB and structural supports would be placed upon the platform. The yielding platform system is applicable only to APCB configurations which do not have integral structural supports. The existing structural supports would be replaced by a yielding structural support. One APCB configuration listed in Table 4-1, cannot utilize the yielding support system.

Attenuation of the vertical response of the APCB has not been indicated in either concept as it is probably not required. However, vertical attenuation can be provided with the pendular system. Since the isolation concepts proposed require a development program which will include model studies and verification tests of the prototype, the need for vertical isolation should be determined during the test phase of this program.

The concept of pendular isolation is shown in Figure 4-5. An entire three-phase APCB, including CT, would be placed upon a steel frame platform and suspended from rocker arms. The arms would be hung from portal or T-frame ground supports. Heavy duty truck U-joints could be utilized for the rocker pins. The detail design of the support frame is not shown, as this can be accomplished with normal structural design practice. The only requirement for the frame is that adequate vertical stiffness be maintained to eliminate vertical or platform amplification. It should be noted that additional rocker arms and supports can be added to the platform since the system response is a function of the rocker arm length. Thus, the platform can be supported at several points in order to shorten the platform spans. A typical 500 KV APCB/CT isolation platform would be 50 ft x 50 ft in plan and weigh 15,000 lb. The rocker arm would be 8 ft long and minimal platform ground clearance is required. A small friction damper would be added to prevent wind excitation. The fundamental frequency of a typical platform system would be 0.2 cps which would attenuate the APCB response to 0.10 g for the highest intensity ground motion in the BPA service area. However, the displacement response of the platform would be ±10-20 in. The bus attachments would have to accommodate this motion. A flexible bus drop-in as shown in Figure 4-5 is recommended.

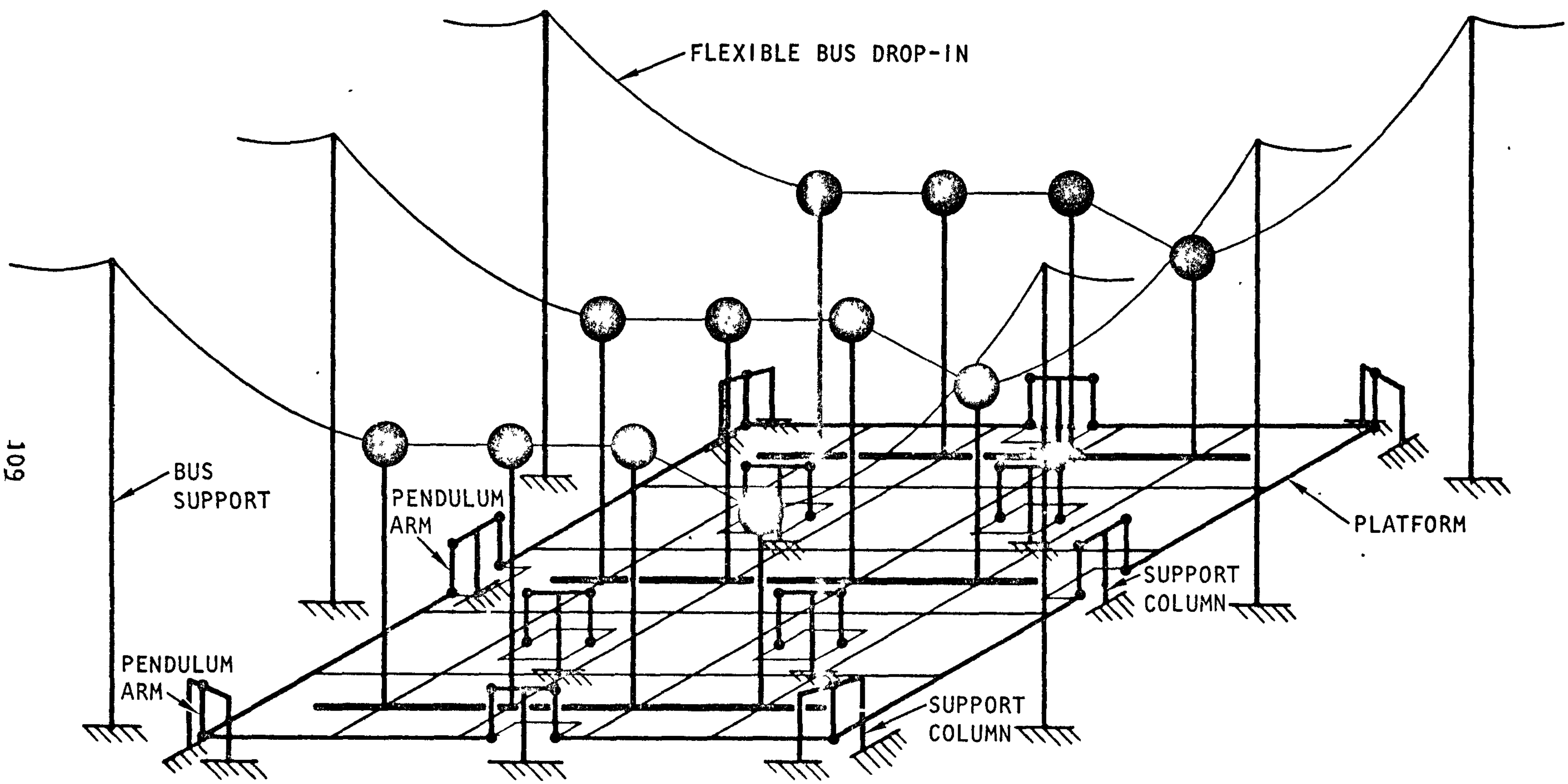


FIGURE 4-5. TYPICAL THREE-PHASE APCB/CT ISOLATION PENDULAR PLATFORM CONCEPT

R-7211-13-2859

One way of reducing the displacement of the platform to earthquake excitation would be to increase the equivalent viscous damping in the pendulum arms through the introduction of damping devices. An effective damper should have nonlinear characteristics. Since the excitation is transient, the response of the system becomes a complex process for which few analytical results are available. The amount of damping should be determined experimentally during the development program. Additional discussion of the analyses performed in the study of this concept is provided in Appendix B-1.

The concept of a yielding support is shown in Figure 4-6. An entire three-phase APCB, including CT, would be placed upon a steel frame platform supported on yielding tripod supports. Each truss element of the tripod would be an elasto-plastic device. Such devices are commercially available. The detail design of the support frame is not shown, as this can be accomplished in accordance with normal design practice. The platform should have adequate vertical stiffness to eliminate vertical amplification. The platform support points can be placed directly under the breaker columns, thus eliminating loaded beam spans. The tripod supports would replace the existing structural supports of the APCB which are not integral units. The tripod-platform connection details would be designed to minimize bending or frame effects in the tripod. This concept does not introduce additional support flexibility but instead introduces hysteretic damping to attenuate the APCB response. The concept, in effect, isolates the platform and APCB from shear forces which exceed the yield level of the support tripods. Thus, this concept can only be utilized when the allowable static lateral load level of the APCB is greater than or equal to that induced by the peak ground acceleration. Additional discussion of the analyses performed in the study of this concept are provided in Appendix A-2.

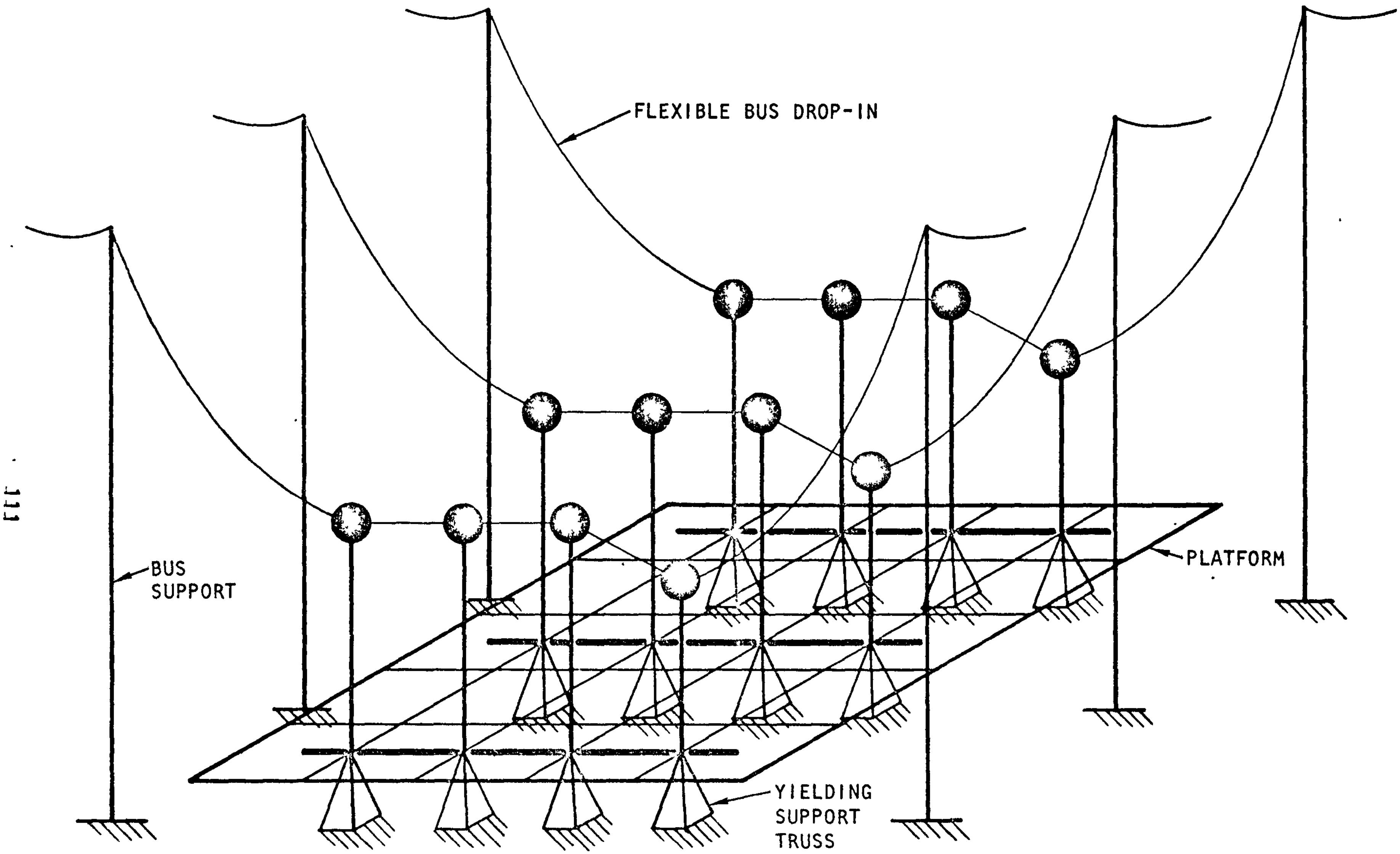


FIGURE 4-6. TYPICAL THREE-PHASE APCB/CT ISOLATION YIELDING PLATFORM CONCEPT

R-7211-13-2859

All APCB's reviewed require modification. These include the 500 KV Merlin-Gerlin, GE, and Delle Alsthom and the 230 KV GE, Allis-Chalmers, and Oerlikon APCB's. It is recommended that development studies be initiated on the pendular isolation concept, This should consist of a scaled model study to verify the parameters of the model, and to determine if vertical isolation is required. A prototype of the concept should then be designed in detail, fabricated, and proof tested to validate the concept. Operational validation tests of the APCB should also be performed with the prototype. The estimated cost of fabrication and installation of the isolation concept proposed above is about $15,000 for each 500 KV 3-phase APCB. This assumes that a large number of APCB's will be isolated and does not include the cost of development and testing of models and prototypes.

CONVERTER TRANSFORMERS, DC SMOOTHING REACTORS, OIL AND HSF6 CIRCUIT BREAKERS AND HIGH PASS FILTER REACTORS

These items of heavy electrical equipment were classified as rigid body equipment for purposes of analysis. Such items are normally susceptible to damage from earthquake ground motion only from the standpoint of sliding or possibly overturning. The potential earthquake damage can be nullified by properly anchoring the equipment to the foundation provided the foundation is adequate. The investigation indicated that all foundations for these items of equipment were adequate. However, the connections between the equipment and the foundations were all found to be inadequate. The modifications recommended, therefore, consists only of providing adequate connections. Details of anchorage connections for these four types of equipment are provided in Figures 4-7 through 4-10 and in Figure 4-25.

VOLTAGE DIVIDERS

The voltage dividers at Celilo were placed in Category 2 because of potential overstressing of the support frame attachment. These items of equipment were modeled as cantilever elements for analysis. The modification required is simple and requires only a new support frame and foundation attachment which are shown in Figure 4-11.

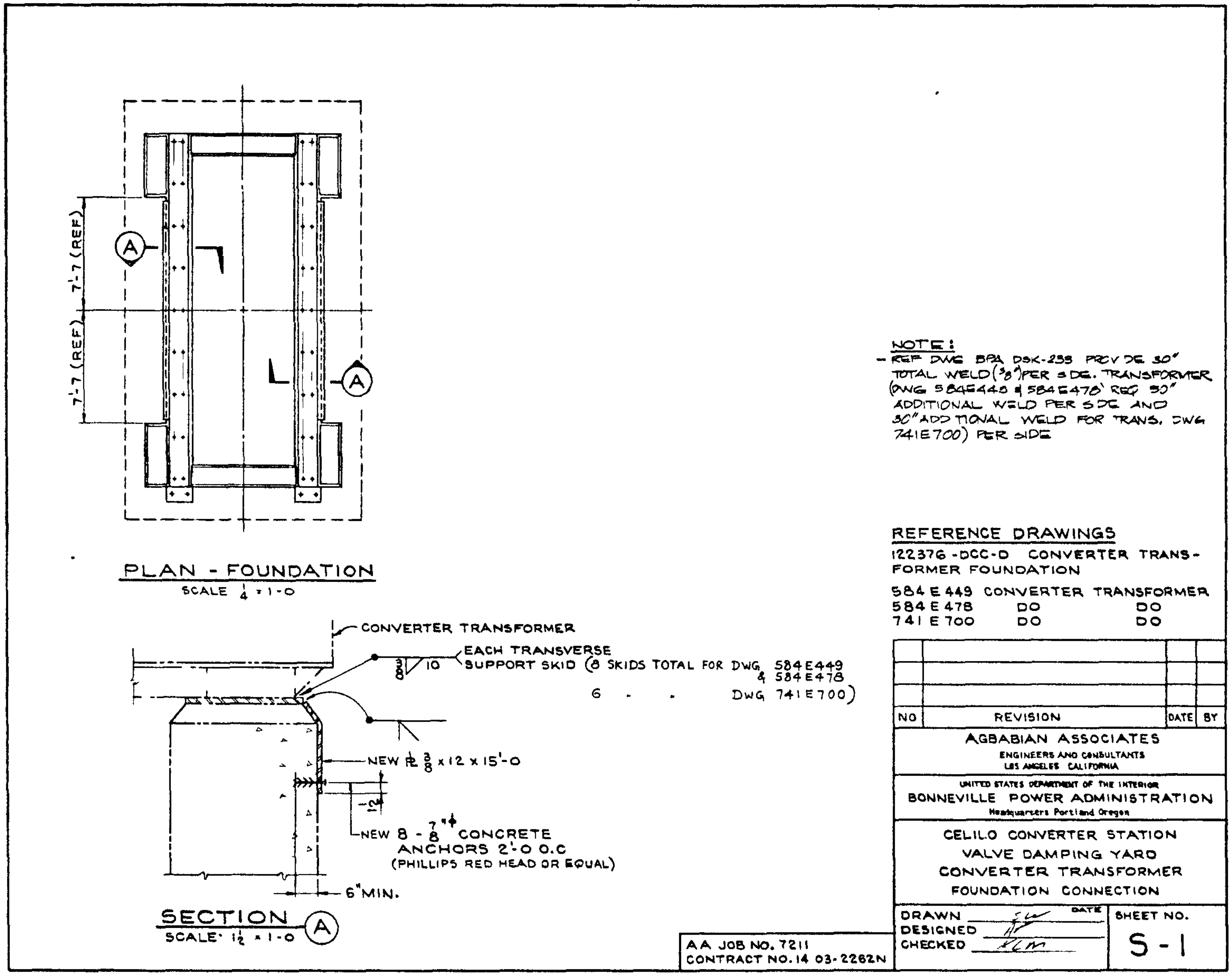


FIGURE 4-7. CONVERTER TRANSFORMER FOUNDATION CONNECTION MODIFICATIONS

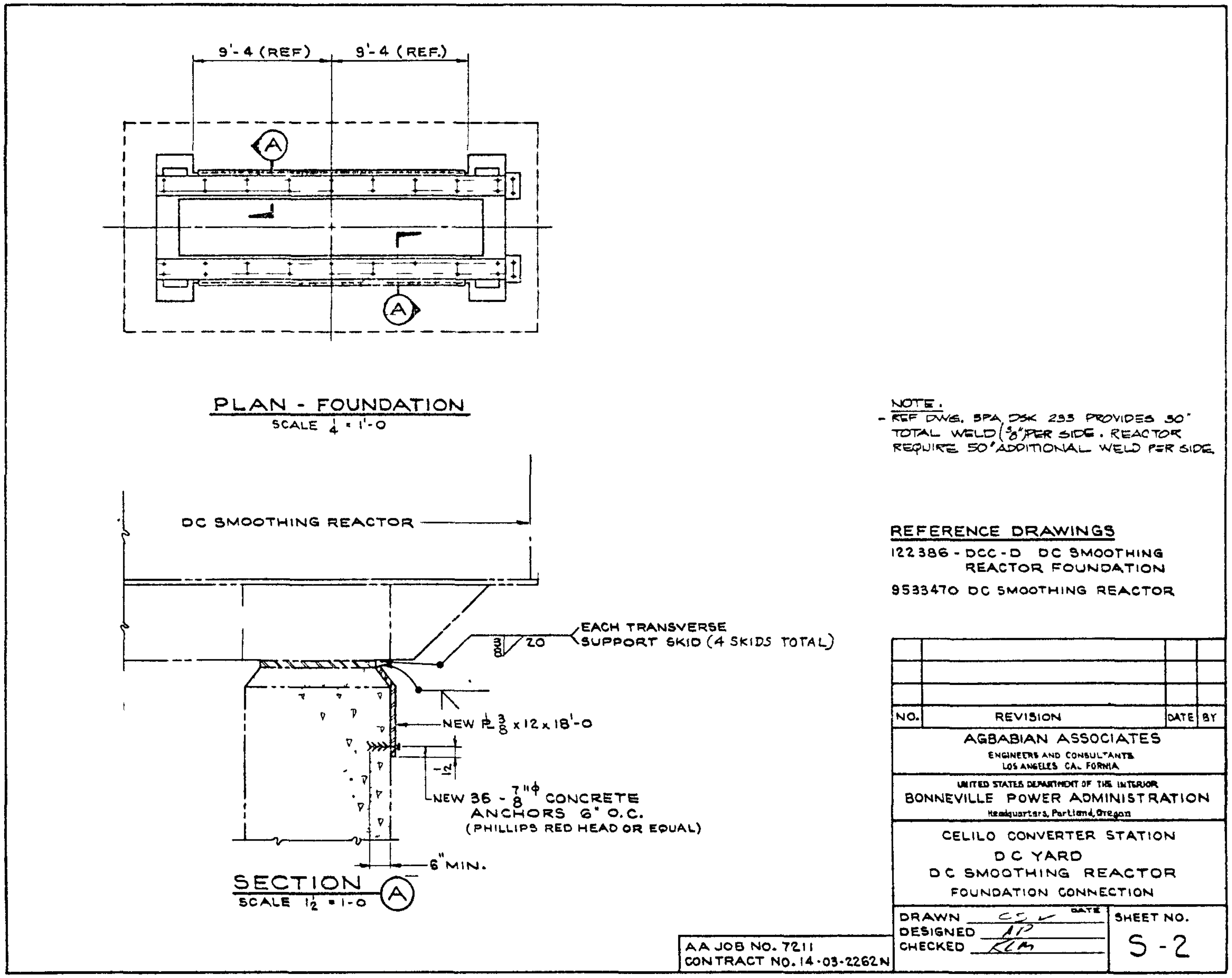


FIGURE 4-8. DC SMOOTHING REACTOR FOUNDATION CONNECTION MODIFICATION

R-7211-13-2859

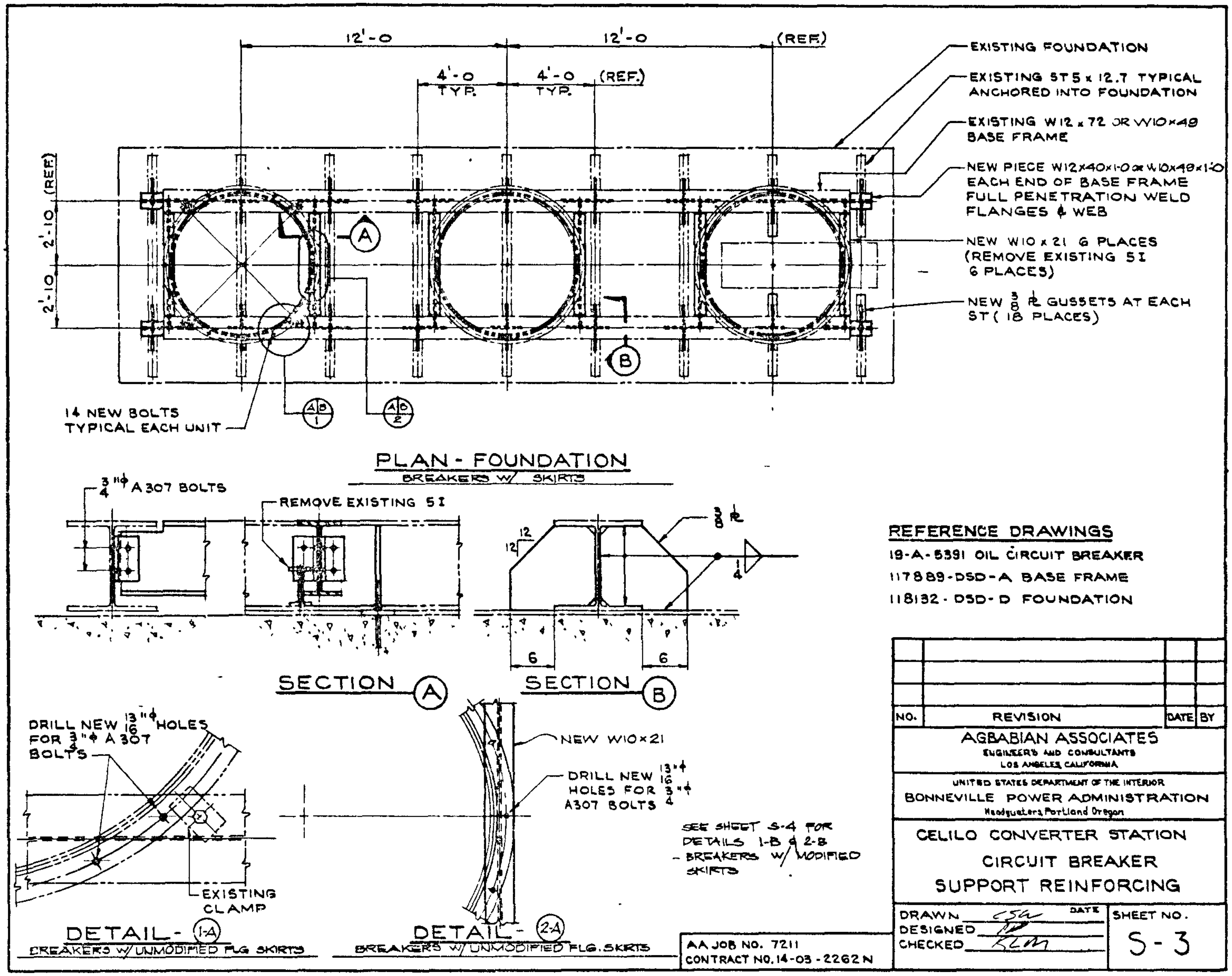


FIGURE 4-9. CIRCUIT BREAKER FOUNDATION BASE FRAME MODIFICATIONS

R-7211-13-2859

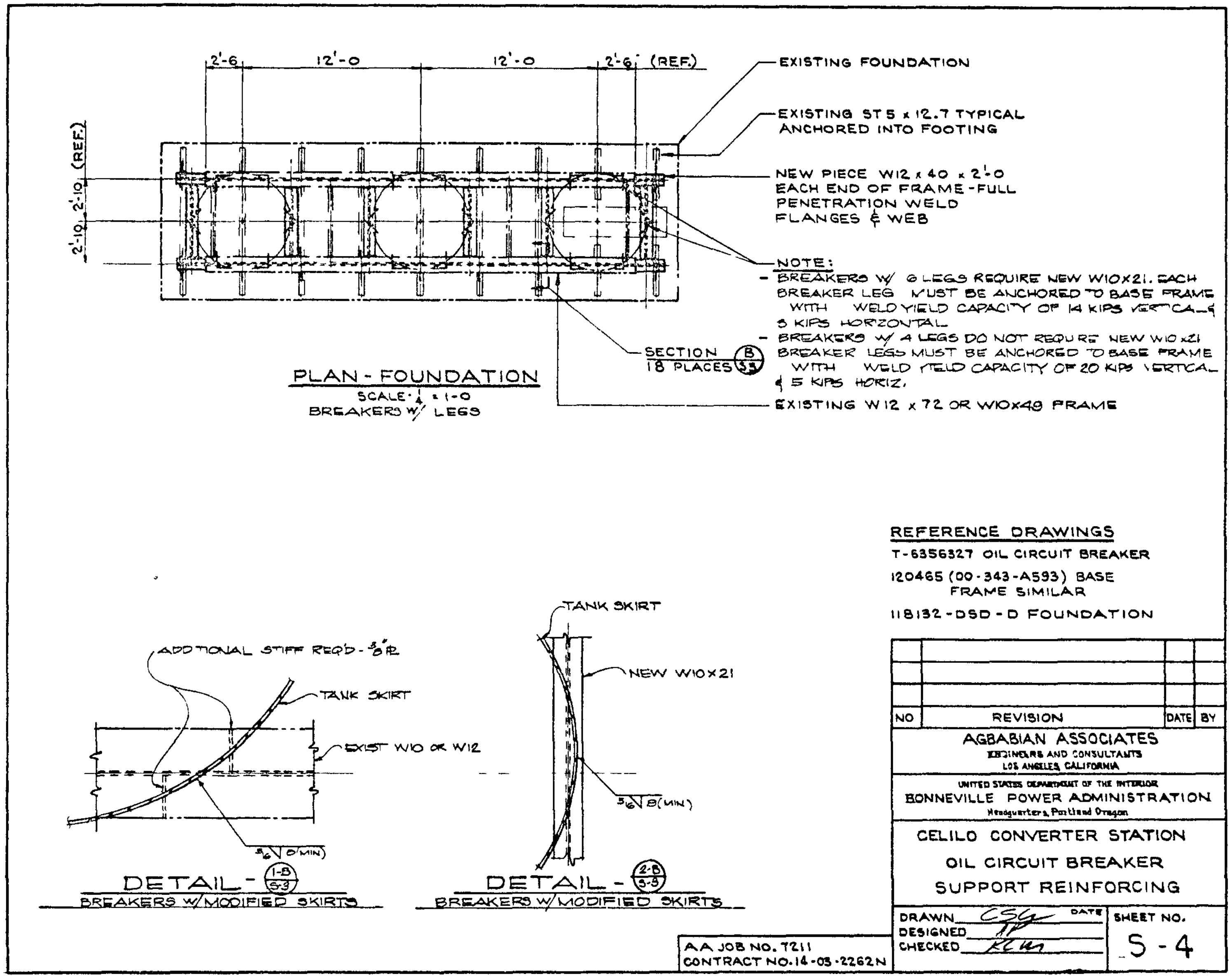


FIGURE 4-10. CIRCUIT BREAKER FOUNDATION BASE FRAME MODIFICATION

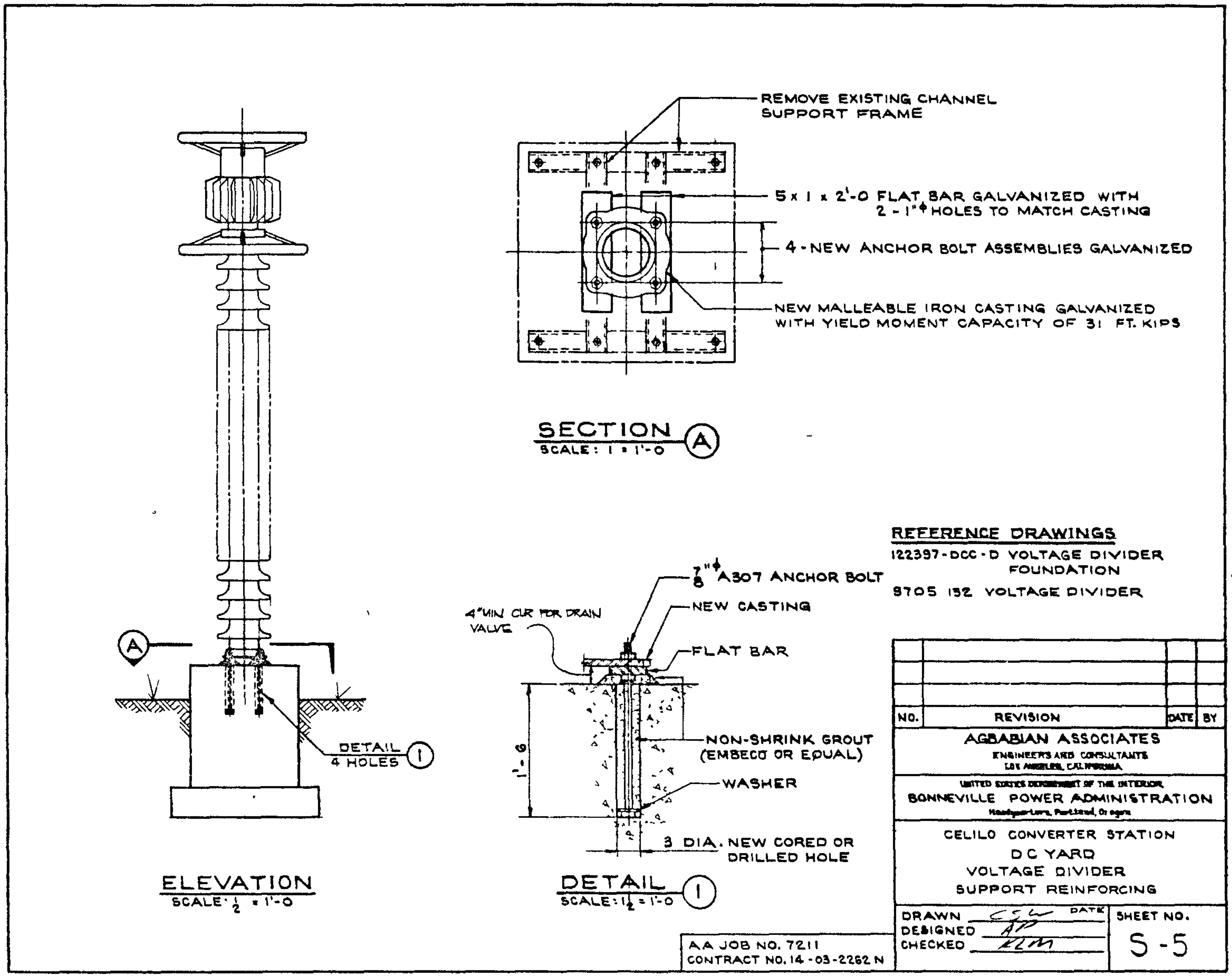


FIGURE 4-11. VOLTAGE DIVIDER SUPPORT MODIFICATION

R-7211-13-2859

LIGHTNING ARRESTERS - Celilo Converter Station

The lightning arresters investigated at Celilo are listed in Table 4-2 by location and type. Those placed in Category 2 require modification. The lightning arresters were modeled as complex cantilever structures to simulate the restraining effect of the bus and bus supports. The resulting system was analyzed with a three-dimensional modal analysis.

TABLE 4-2. LIGHTNING ARRESTER CLASSIFICATION

Location	Number of Items	Type	Classification Category
Valve Damping Yard	18	AC Line to Line	1
Valve Damping Yard	6	AC Line to Ground 900	2
Valve Damping Yard	6	AC Line to Ground 1300 bil	2
Valve Damping Yard	2	DC Line to Ground 650 bil	1
Valve Damping Yard	2	DC Line to Ground 900 bil	2
Valve Damping Yard	2	DC Line to Ground 1300 bil	2
Valve Damping Yard	3	Converter Transformer Neutral	1
Valve Damping Yard	18	Converter Transformer Line to Ground	1
DC Yard	2	DC Line Arresters	2
DC Yard	2	DC SR Arresters	2
AC Yard	2	AC Arresters	1

The lightning arresters classified in Category 2 did not have an adequate factor of safety against porcelain bending failure. Only one concept was considered for design modification. This concept is essentially the same as used at Sylmar (Reference 4-2) and consists of adding Belleville springs at the base connection of single porcelain arrester stacks. The addition of the Belleville springs not only reduces the natural frequency of the system, but also reduces the bending stress at the base attachment point.

Some bus connection modifications are also required. As this concept incorporates provision of adequate resistance with simplicity and low cost, no other concepts were developed. The design details of the arrester modifications are shown in Figures 4-12 through 4-16.

LIGHTNING ARRESTERS - Allston and Custer Substations

The 500 KV lightning arresters at both Allston and Custer Substations were classified in Category 2 due to inadequate factor of safety against porcelain bending failure. The recommended modification concept is to replace the existing structural pedestal supports at both stations with the "low profile" lightning arrester pedestal as shown on BPA dwg. 142568-DSD-A. The specified structural material is ASTM A618 structural tubing. The Belleville spring detail shown in Figure 4-12, is required at the base of the single porcelain arrester stack. The pedestal base connection and foundation should be designed to withstand an overturning moment of 325,000 lb-in.

VALVE DAMPING CAPACITORS

There are six valve damping capacitors in each valve damping yard at Celilo for a total of thirty-six that require modification. These items were modeled as simple cantilever elements for analysis. The connecting bolts in the base-channel-frame connection were found to be overstressed. Figure 4-17 indicates the modification required. Note that a general inspection of all bolted connections is required in addition to the replacement of base connection bolts to insure that all bolts have adequate thread grip. Belleville springs are also required.

DC FILTER CAPACITOR RACKS

The 6th and 20th harmonic filter capacitor racks were investigated at Celilo and were found to require modification. These racks were modeled as frame supported equipment. The diagonal bracing provided by insulator strings and the main platform upon which the capacitors are supported were found to be inadequate. The following three concepts were investigated:

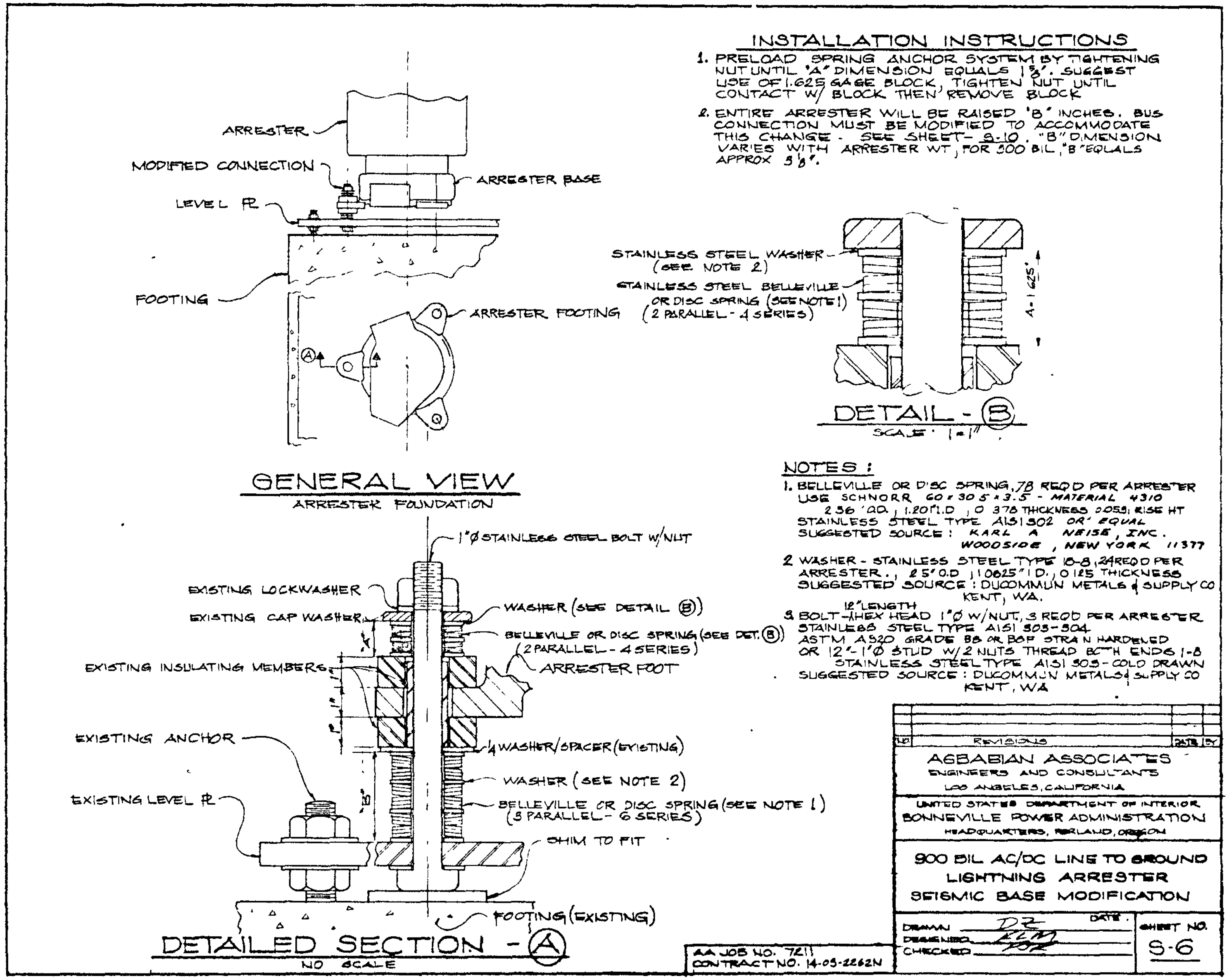


FIGURE 4-12. 900 BIL AC/DC LINE TO GROUND LIGHTNING ARRESTER BASE MODIFICATION

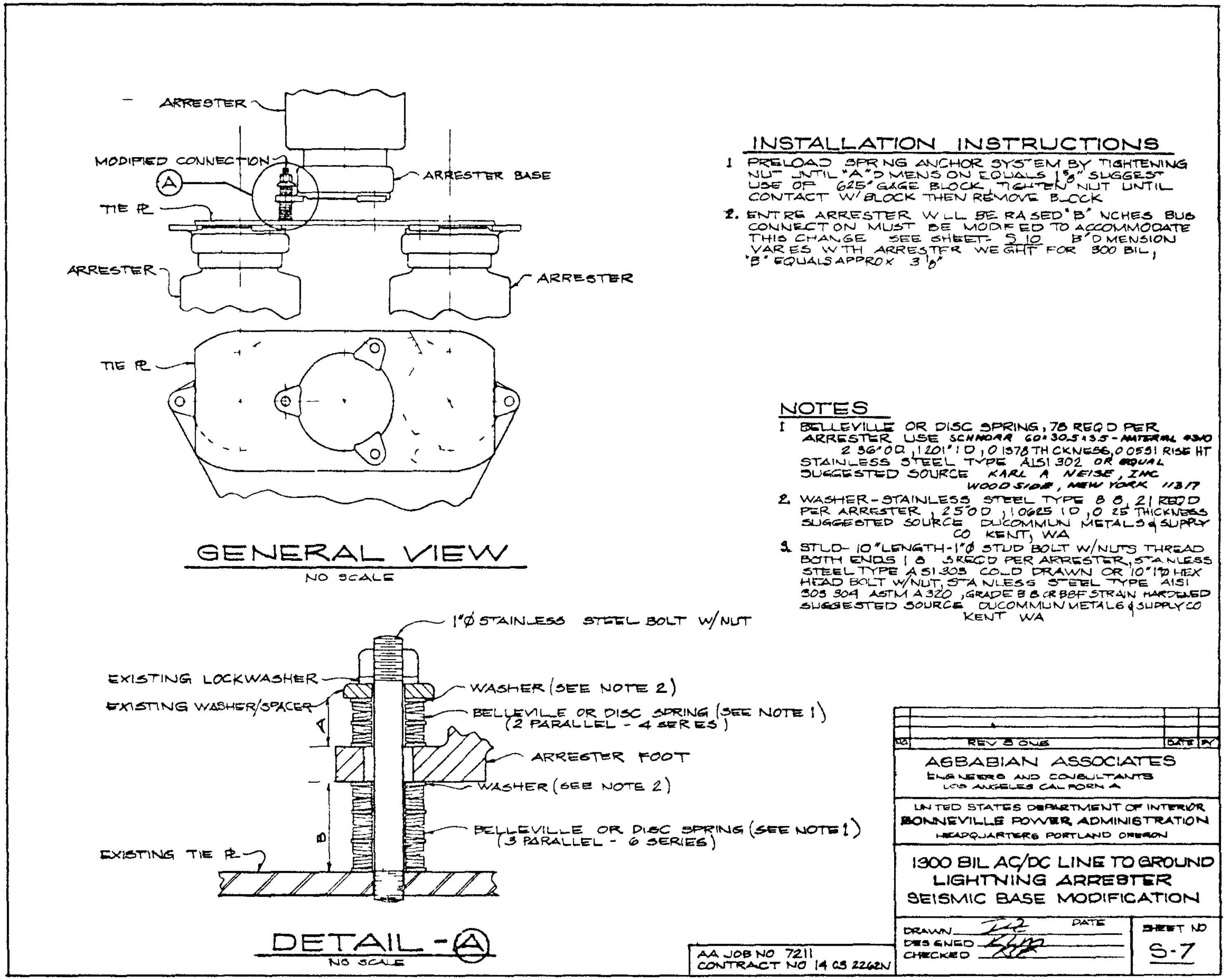


FIGURE 4-13. 1300 BIL AC/DC LINE TO GROUND LIGHTNING ARRESTER BASE MODIFICATION

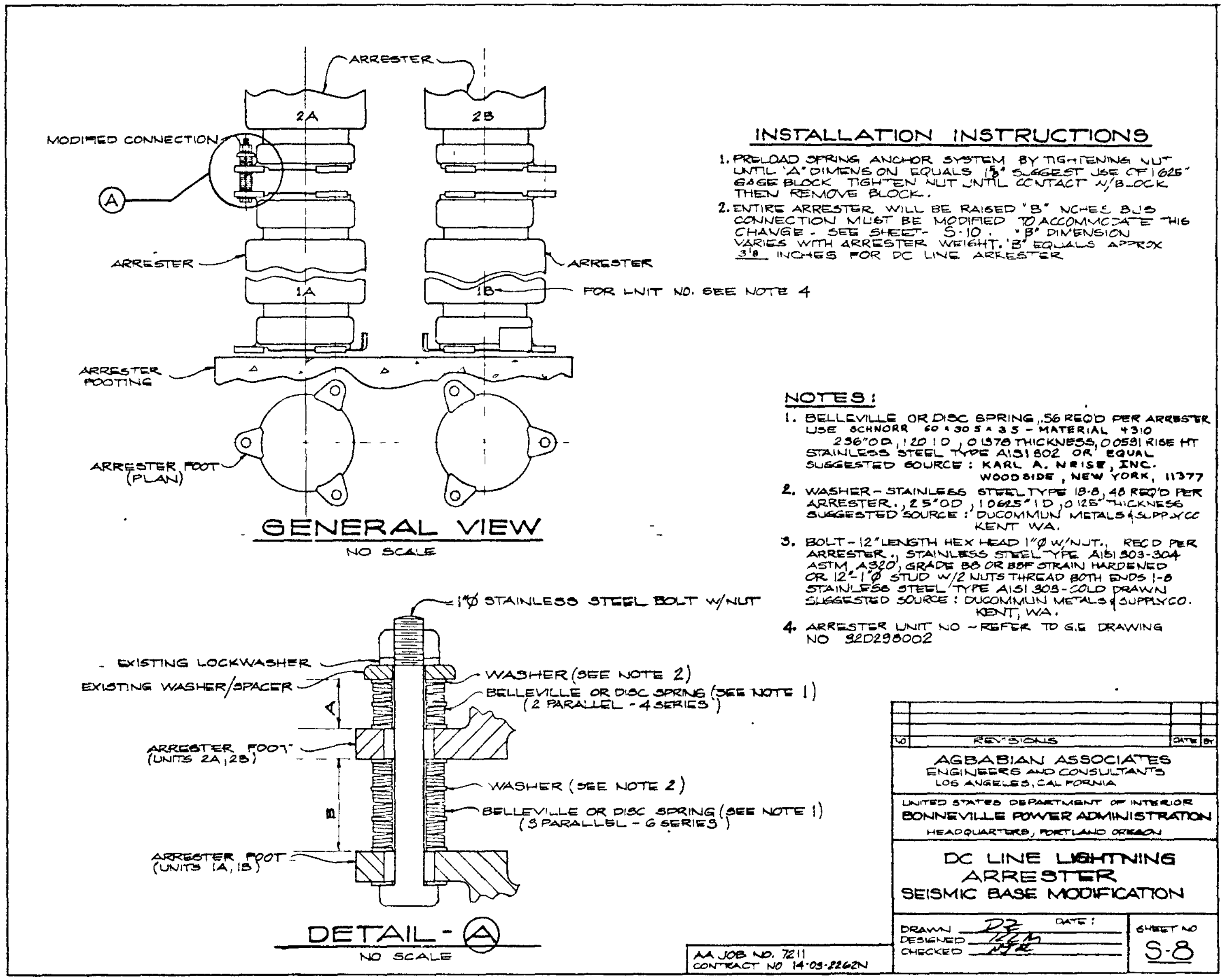


FIGURE 4-14. DC LINE LIGHTNING ARRESTER BASE MODIFICATION

R-7211-13-2859

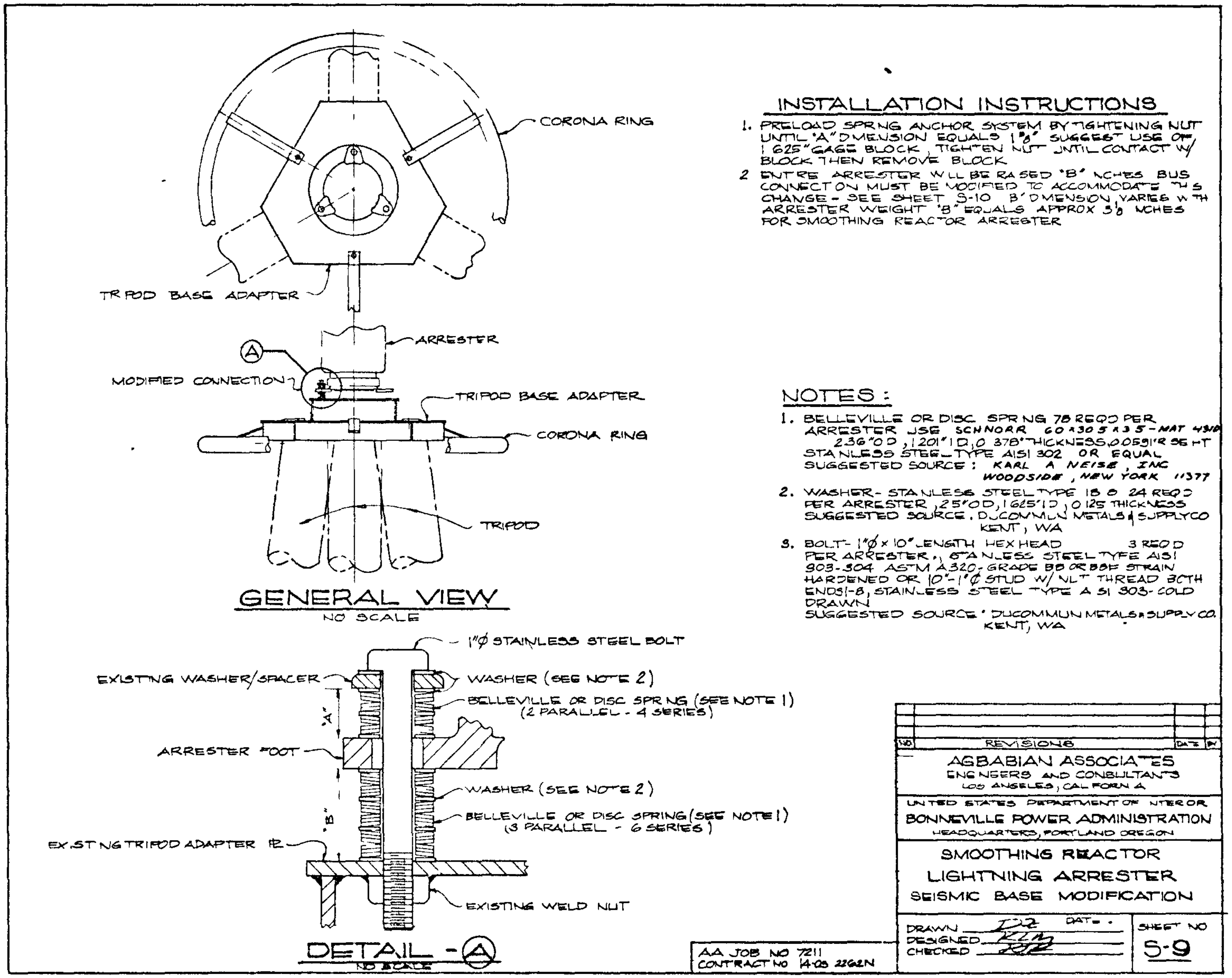


FIGURE 4-15. SMOOTHING REACTOR LIGHTNING ARRESTER BASE MODIFICATION

R-7211-13-2859

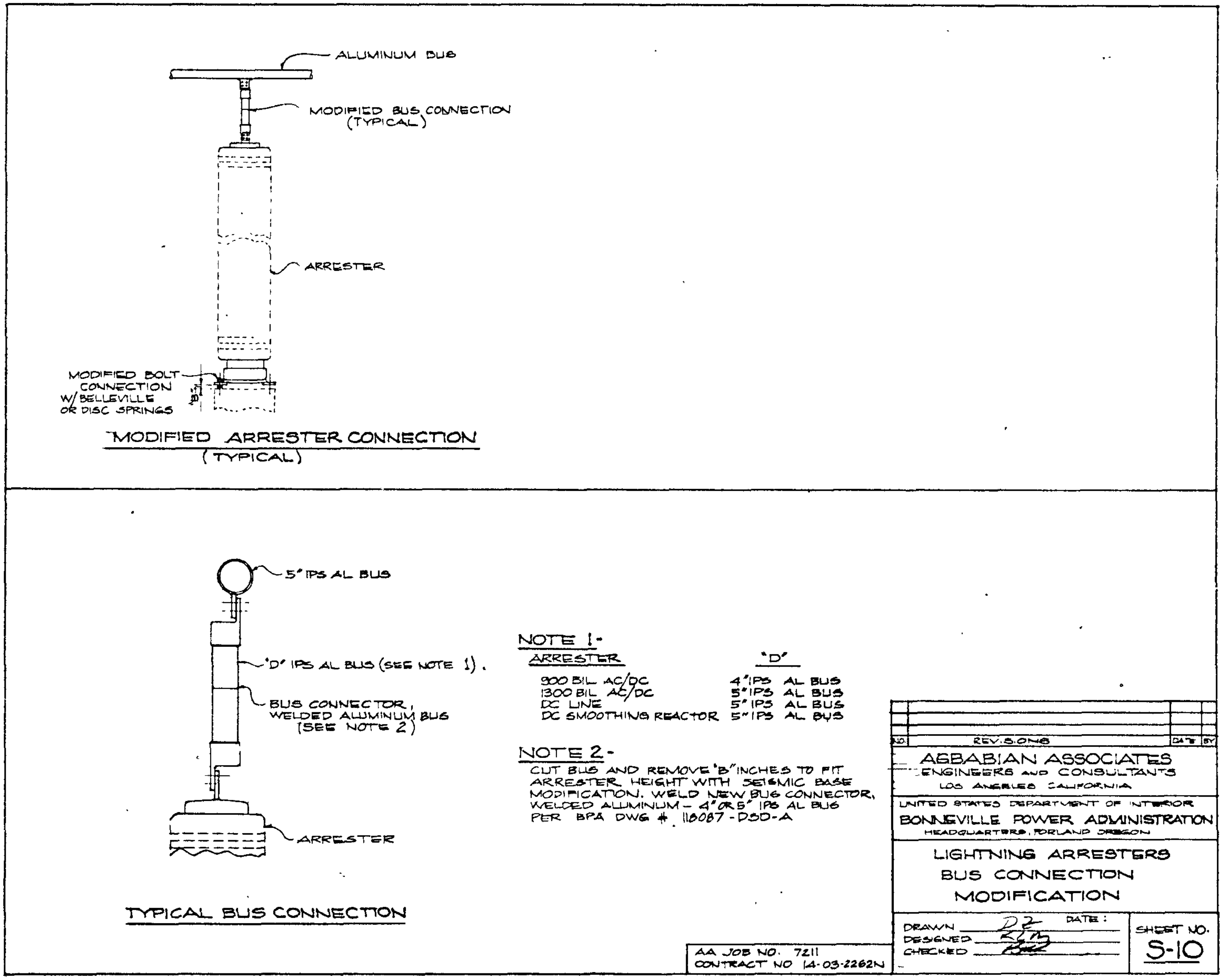


FIGURE 4-16. LIGHTNING ARRESTER BUS CONNECTION MODIFICATION

R-7211-13-2859

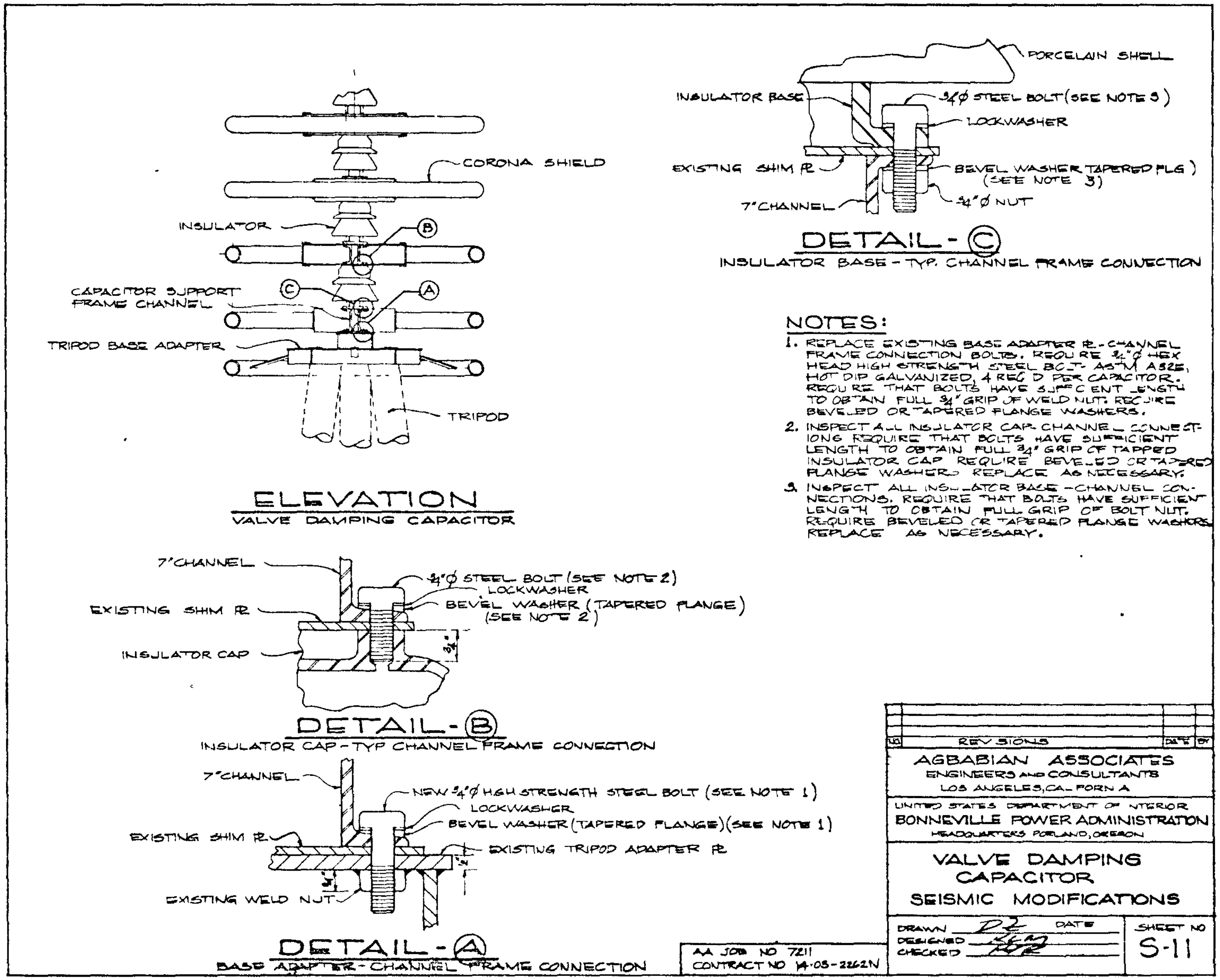


FIGURE 4-17. VALVE DAMPING CAPACITOR MODIFICATIONS

Concept 1--Strengthening Existing Base

The first concept consisted of strengthening the existing base section including the main platform upon which the capacitors are supported. Several new aluminum members would be required, and several existing connections would have to be upgraded. Existing diagonal members (in a vertical plane) for bracing the capacitor platform would be replaced with stronger diagonals; additional diagonals would also be required. The main advantage of this concept is its relative economy and lack of basic configuration change.

Concept 2--New Steel Platform (2nd Level)

This scheme was similar to Concept 1 as new diagonal bracing insulators would be required; however, rather than rework the existing platform above the base post columns, a new steel platform frame would be provided. This scheme would be considerably more expensive than Concept 1, but an advantage would be that greater seismic resistance could be attained,

Concept 3--Add Insulator Columns

This scheme required installing additional station post insulator columns to the platform to provide moment resisting rigid frames for lateral resistance in lieu of the diagonally braced system currently employed. This scheme required so many new insulator columns, with additional connections, that the cost was very high. The scheme, therefore, was not recommended.

Based upon a cost comparison, Concept 1 was selected as the preferred concept. Details of this concept are provided in Figures 4-18 and 4-19.

HIGH PASS FILTER RESISTORS

The high pass filter resistors at Celilo were placed in Category 2 due to an insufficient factor of safety against porcelain failure. Three feasible modifications were considered. The first concept incorporated use of additional guy cables with suspension insulators at the level of the change in support insulators, about 80 in. above the base. Three new insulated guy cables with turnbuckles would be attached to a new adapter ring and anchored to dead men at the ground. This concept is recommended because of its economy.

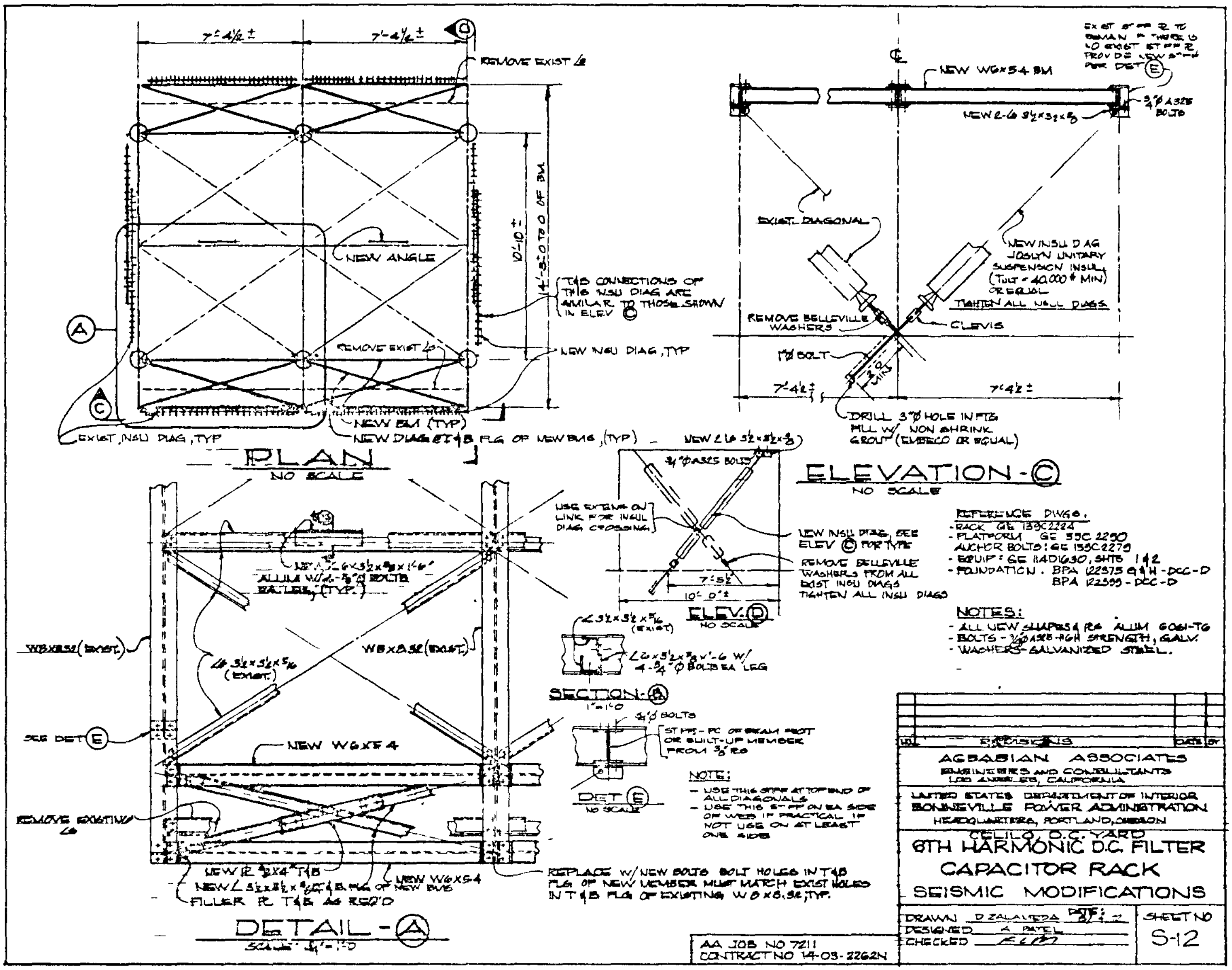


FIGURE 4-18. 6TH HARMONIC DC FILTER CAPACITOR BASE FRAME MODIFICATIONS

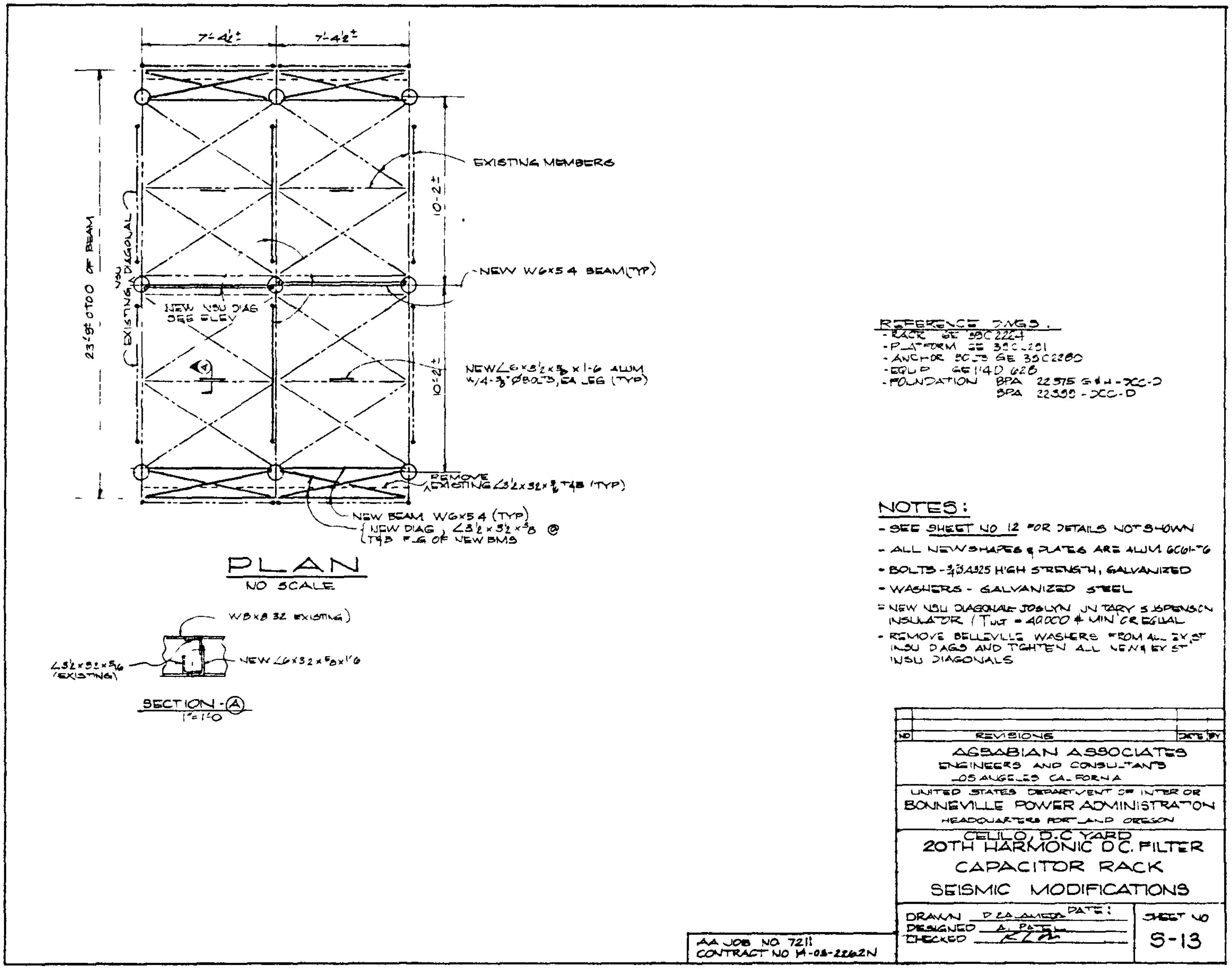


FIGURE 4-19. 20TH HARMONIC DC FILTER CAPACITOR BASE FRAME MODIFICATIONS

R-7211-13-2859

The most significant drawback to this concept is that the guys take up additional yard space which could interfere with maintenance operations.

The second concept was most direct and involved replacement of existing insulators with stronger and shorter units to provide adequate seismic resistance. This system has the advantage of generally utilizing the present configuration, but it is more costly.

The third concept would incorporate modifications according to those provided at Sylmar. This scheme essentially consists of a lower stack of insulators requiring no guys which involves a complete change to the existing configuration. This concept is acceptable except for the relatively high cost.

Concept details for the preferred concept, Concept 1, are provided in Figure 4-20.

HARMONIC FILTER REACTORS

There are twelve harmonic filter reactors in total at Celilo and Big Eddy, six of the fifth and seventh harmonic reactors and six of the eleventh and thirteenth harmonic reactors. These items were modeled as rigid body equipment for analysis. All of these items were found to require foundation modifications as well as modification to the connection between the equipment and foundation. The major problem with the harmonic filter reactors is due to a relatively high center of gravity and a corresponding tendency to overturn the reactor and its foundation under lateral loads. Three foundation modification concepts were studied and are described below.

Concept 1 utilizes anchor piles to increase the stability of the reactors and their footings. At each of the four corners of each foundation a 14 ft pile would be placed and connected to steel channel frames. The steel channel frames would be placed along the four edges of existing foundation pads with spaced connections to the concrete. This concept is schematically illustrated in Figure 4-21. A previous foundation report for Celilo indicated soil material for which no apparent difficulties in using

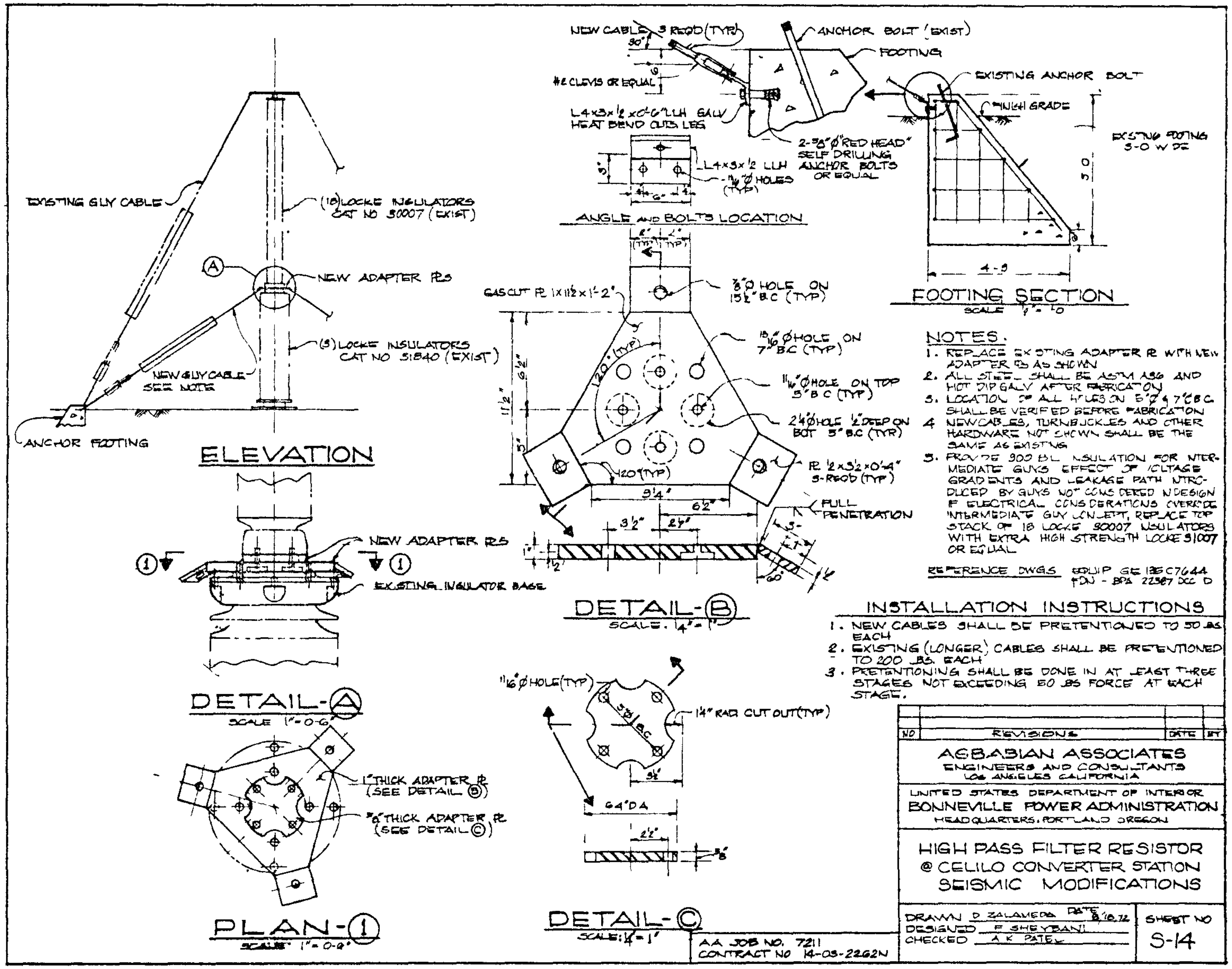


FIGURE 4-20. HIGH PASS FILTER RESISTOR MODIFICATIONS

R-7211-13-2859

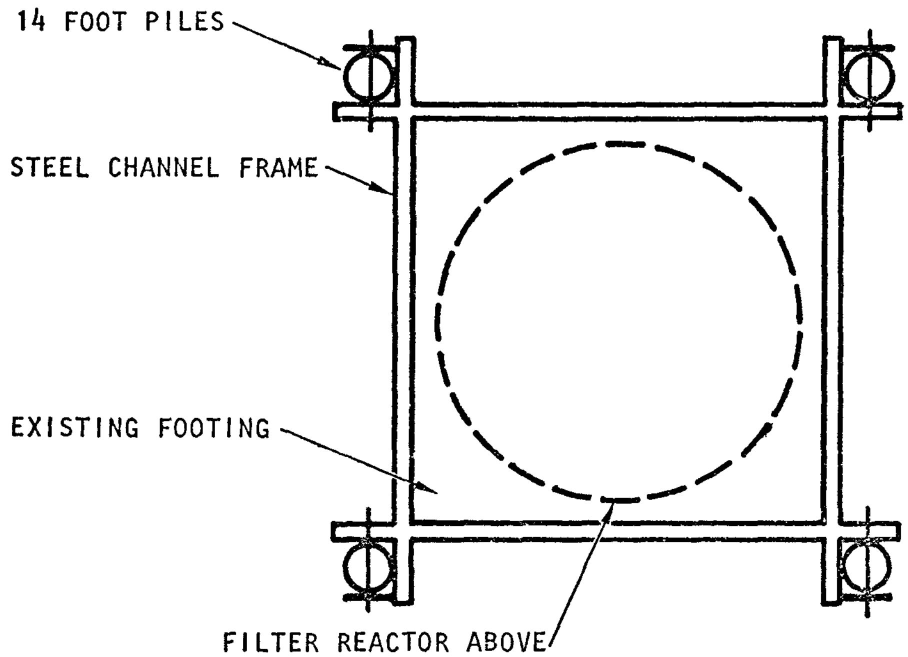


Figure 4-21. SCHEMATIC PLAN--CONCEPT 1--PILE ANCHORAGE CF EXISTING FOOTING

piles of short length is anticipated. The piles should be drilled in place concrete piles, steel pipe or H-sections, or treated timber piles capable of resisting uplift forces. This concept has the advantages of lowest estimated cost, least down time and minimum disturbance to the electrical system. It is envisioned that the channels could be attached and the piles could be placed with the filter reactors in place.

Redesign Concept 2 is shown schematically in Figure 4-22. It consists of pouring additional concrete around the periphery of each existing footing and anchoring it to the existing foundation by means of prestressing strands placed to extend through the new and existing sections of the footing in each direction.

Redesign Concept 3 requires replacement of the existing foundation with a new foundation. It would require removing the reactors from their existing position temporarily and replacing them on the new pad. This would require considerable shutdown time. As with the other two concepts, positive mechanical anchorage to the footings from the filter reactor body would be required.

After review, Concept 1 which involves anchorage of the existing footings utilizing anchor piles was selected as the preferred concept because of least cost and minimum interference with existing operations. The design sketches for the selected concept are shown in Figures 4-23 through 4-25.

AC FILTER CAPACITOR RACKS

The AC Filter Capacitor Racks requiring modification included those in the power factor correction, high pass filter, and 11th and 13th harmonic filter areas of the AC yard at Celilo, and the 5th and 7th harmonic filter areas at Big Eddy. Each rack configuration was modeled as a three-dimensional frame structure and analyzed by a dynamic modal analysis. The post insulators and aluminum welds were found deficient in these racks. As capacitor racks are such universally utilized items of equipment, several design modification schemes were considered. A listing of the concepts and a summary of the advantages and disadvantages of each is provided in Table 4-3. A brief description of the various concepts follows.

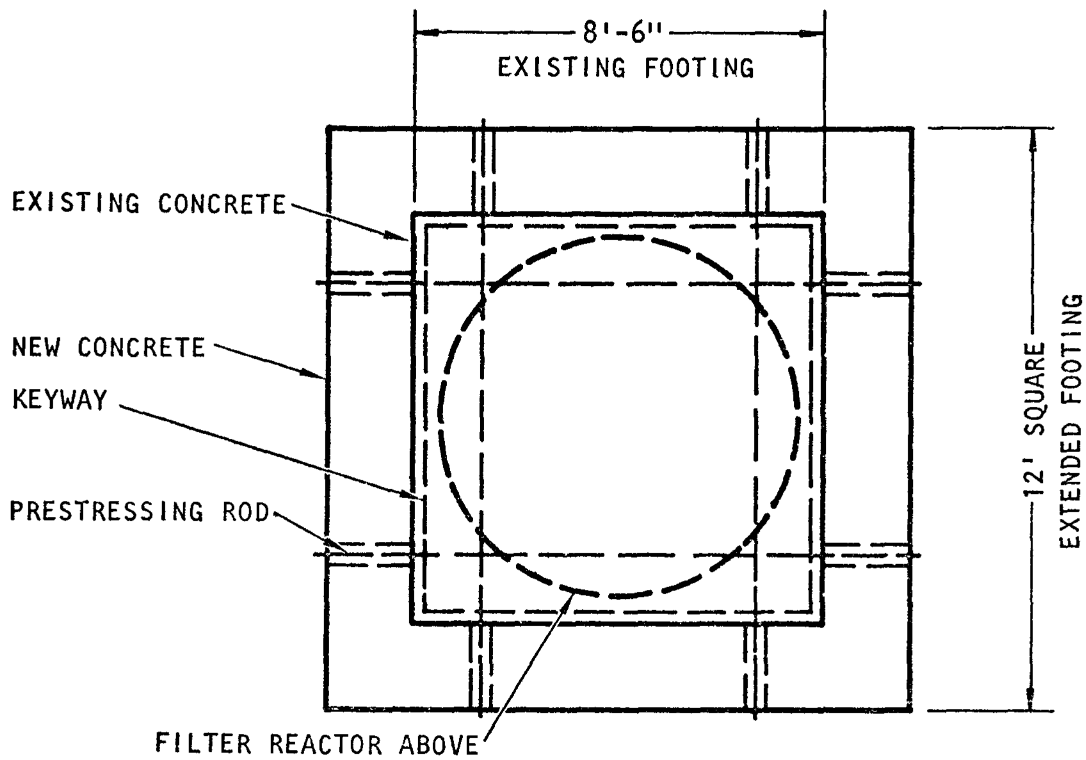


Figure 4-22. SCHEMATIC PLAN--CONCEPT 2--ENLARGE EXISTING FOOTING AND PRESTRESS

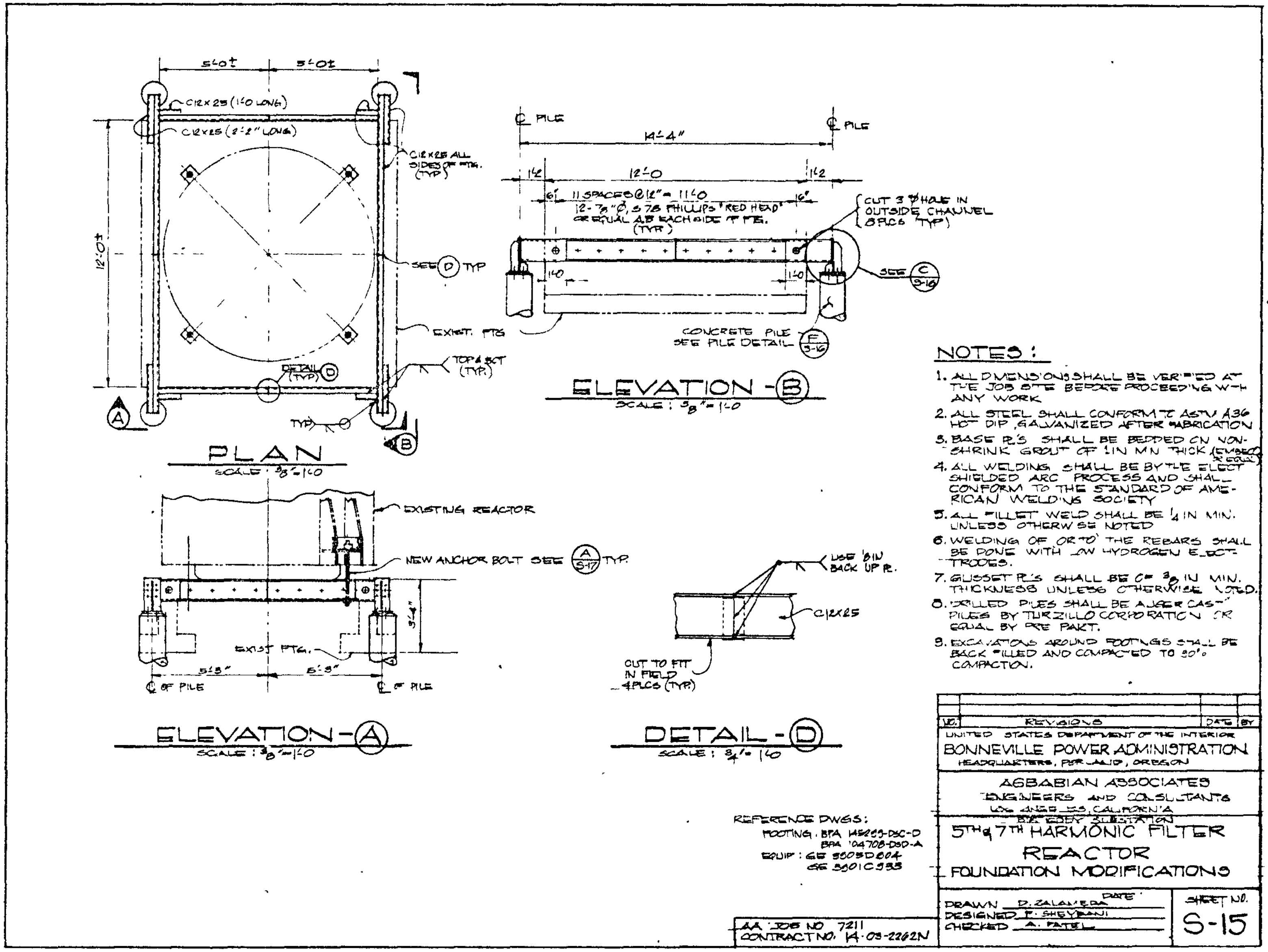


FIGURE 4-23. FIFTH AND SEVENTH HARMONIC FILTER REACTOR FOUNDATION MODIFICATION

R-7211-13-2859

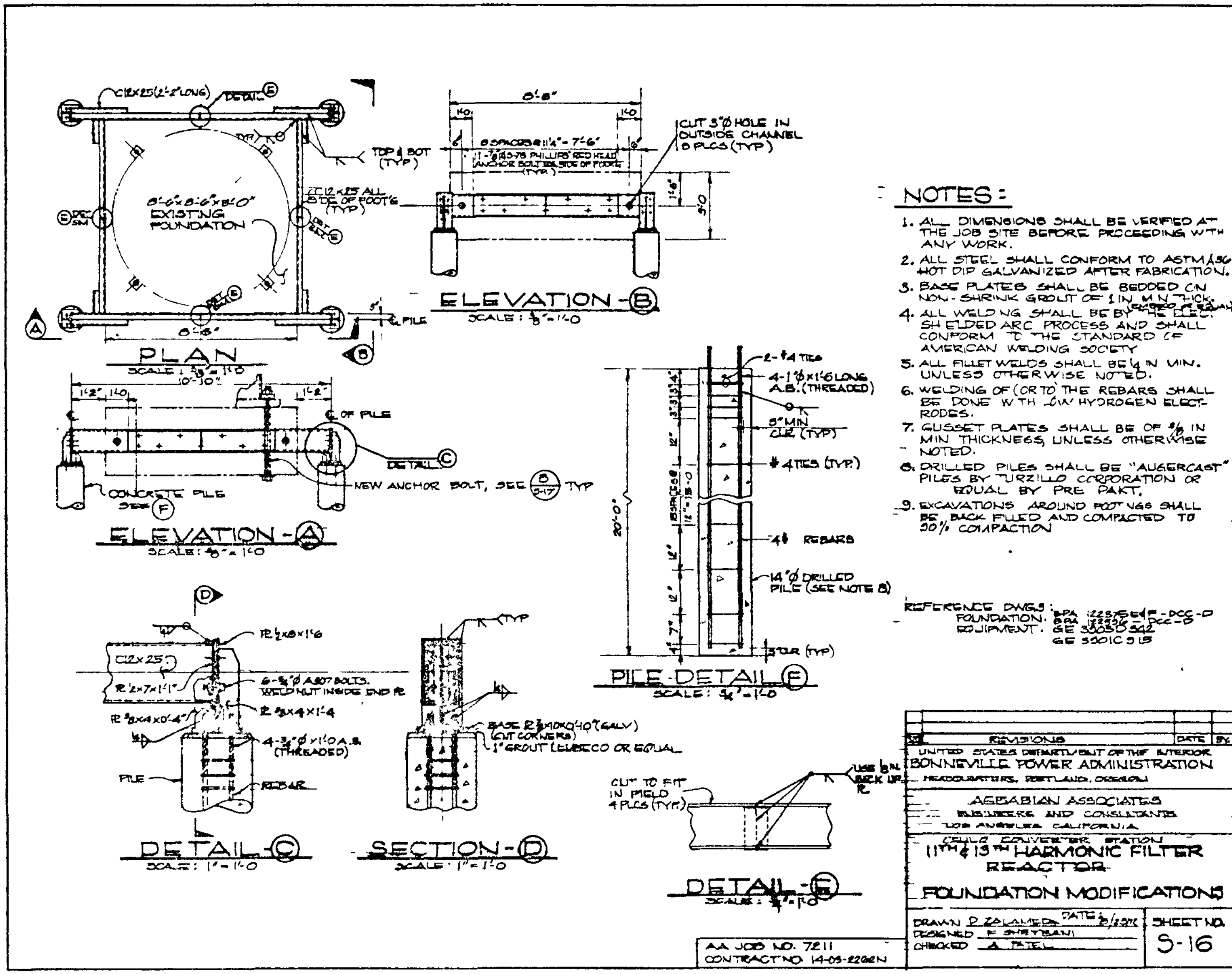


FIGURE 4-24. ELEVENTH AND TWELFTH HARMONIC FILTER REACTOR FOUNDATION MODIFICATION

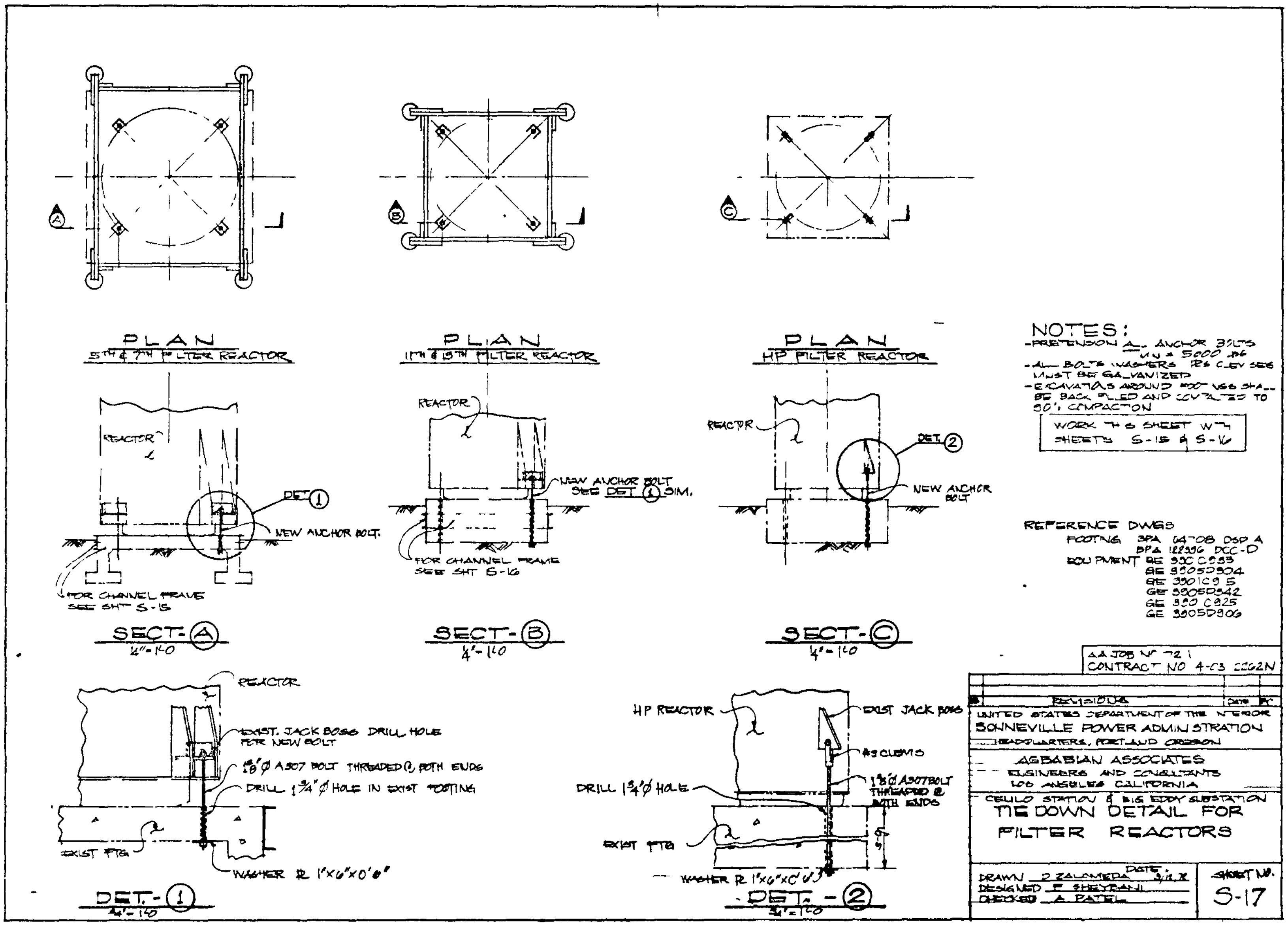


FIGURE 4-25. FILTER REACTOR ANCHORING DETAILS

TABLE 4-3. A.C. CAPACITOR RACK CONCEPT EVALUATION

No.	Concept	Rating Factor: Seismic Resistance	Cost	Electrical Function	Maintenance Access	Complexity	System Down Time	Rating
1	In Place Strengthening		(-)					-
2	New Aluminum Rack			(+)	(+)	(+)		4
3	New Steel Rack	(+)			(+)	(+)		3
4	Overhead Support System		(+)		(+)	(+)	(+)	1
5	Adjacent Support System			(-)	(-)			-
6	Horizontal Shock Isolation		(+)		(+)		(+)	2
7	Partial Isolation of Base	(-)	(+)		(+)			-

<u>Note</u>: A plus (+) under rating factors in this table indicates special merit of the concept considering the given factors, a blank indicates no special merit, and a negative (-) indicates sufficient disadvantage for disqualification, i.e., the concept is questionable or not workable. An overall rating is given the workable concepts in the last column of the table.

R-7211-13-2859

Concept 1--In-Place Strengthening

Because of the relatively high stresses induced in the members of the capacitor racks in the present configuration under the design earthquake loading, necessary modifications would be quite extensive. New foundations would be required. Reworking of most existing connections would be required and some new members would be needed. The extent of modifications would require disassembly, strengthening and reassembly for each rack. This was judged to be more costly than provision of a new rack of aluminum or steel. Thus, this concept was discarded and received no further attention.

Concept 2--New Aluminum Racks

An aluminum rack preliminary design was prepared of a similar configuration to the existing racks. This design involved stronger aluminum framing members, porcelain posts and connections. Existing foundations would also require modification. The major disadvantage to this concept was the high cost. The main advantage was that there would be no change from the existing configuration.

Concept 3--New Steel Racks

A preliminary design for a steel rack of similar configuration to the existing aluminum racks was made. The major advantage of this concept is that it provides maximum seismic resistance but at less cost than aluminum racks (Concept 2). The major disadvantage of this design is that the cost may be relatively high compared with other potentially workable concepts, and electrical properties may be changed sufficiently to affect performance.

Concept 4--Overhead Support System

In this concept the existing racks would be supported from the top using a steel frame and fiberglass or porcelain insulator posts or string assemblies as struts and/or guys. The supports would provide a redistribution of member forces from that for the original cantilever structure. The forces would be reduced to acceptable levels. The struts or guys of insulator material would be supported from above by steel beams, the height of which would be determined by the minimum insulation requirement for the filter capacitor yard. The steel beams acting as a roof frame over an entire filter capacitor yard would be supported by steel columns on concrete footings adjacent to the perimeter of the yard. The principal advantages of this concept are the relatively low cost and minimum interference with the capacitor operations. The majority of construction could be off-site fabrication, and only a few days of shut-down would be required while providing the top connections. The main disadvantage would be the hazard of working in the space between existing transmission lines and the capacitors, or the shutdown of the transmission lines during erection and connection. As in all of the concepts which utilize the existing racks, a thorough inspection is required of existing rack connections to assure their adequacy. Modifications should be made, where any deficiencies are noted. Also, a few member connections will require modification because of high calculated stress levels under the design earthquake loading.

Concept 5--Auxiliary Supports for Existing Racks

This concept would provide additional support of existing racks utilizing framing immediately adjacent to each rack. The intrusion into the electrical clearance space could possibly be overcome by utilizing insulator posts without conductive joints or components. However, the additional drawback of interference with maintenance access space was considered serious enough to discourage further development of this concept with any details.

Concept 6--Horizontal Shock Isolation of Existing Racks

This concept consists of mounting each existing rack on a horizontally shock isolated steel platform. This platform in turn would be mounted on the existing footings which would require minor modification. The shock isolation would be effected by providing a relatively friction-free horizontal surface between the platform and existing footings, utilizing teflon pads. The lateral motions would be restrained by relatively soft isolator springs. The major advantage to this concept would be the relatively low cost but the most serious drawback would be the care required in design and installation to assure workability. One significant factor in this concept is that up to 4-1/2 in. of relative lateral displacement would be expected when the system is subjected to the design earthquake motions.

Concept 8--Partial Isolation of Base Using Belville Springs

This concept would utilize Belville springs located at the top and bottom of the base post insulator columns. This concept would lower the natural frequency of the system mainly in the rocking mode. Analysis has shown that the frequency reduction was not sufficient to prevent overstress of existing members for the design earthquake loading, and that the concept has inherent instability. Therefore, this concept received no further consideration.

After carefully comparing each concept, Concept 4 which utilizes an overhead support system was selected as the most viable modification. Details of the overhead support system are provided in Figures 4-26 through 4-31.

SPARE EQUIPMENT AND OIL STORAGE

The proper storage of spare equipment is very important. If a seismic event occurs which renders some equipment inoperable, then the spare equipment must not be similarly damaged. The recommended tiedown details for spare equipment at Celilo is shown in Figure 4-32. Oil tanks must also have sufficient anchorage to prevent lift-off of the tank. The anchorage requirements are also shown in Figure 4-32.

DEGASSING TRANSFORMER OIL CONSERVATOR

Equipment supported upon elevated platforms must be properly braced for amplified lateral loading. The bracing required for the oil conservator in the degassing yard is shown in Figure 4-33.

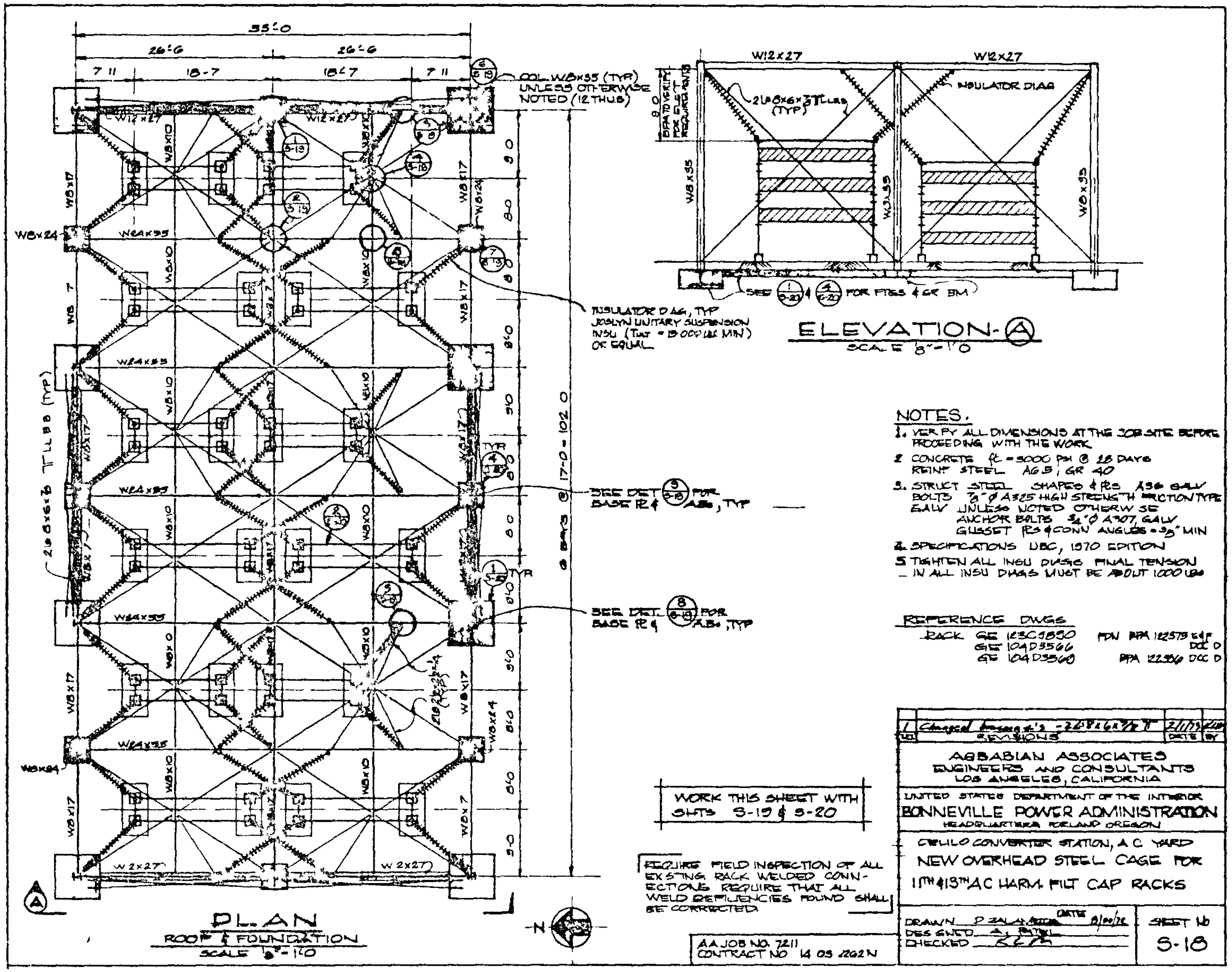


FIGURE 4-26. 11TH AND 13TH AC HARMONIC FILTER CAPACITOR RACK OVERHEAD SUPPORT FRAME

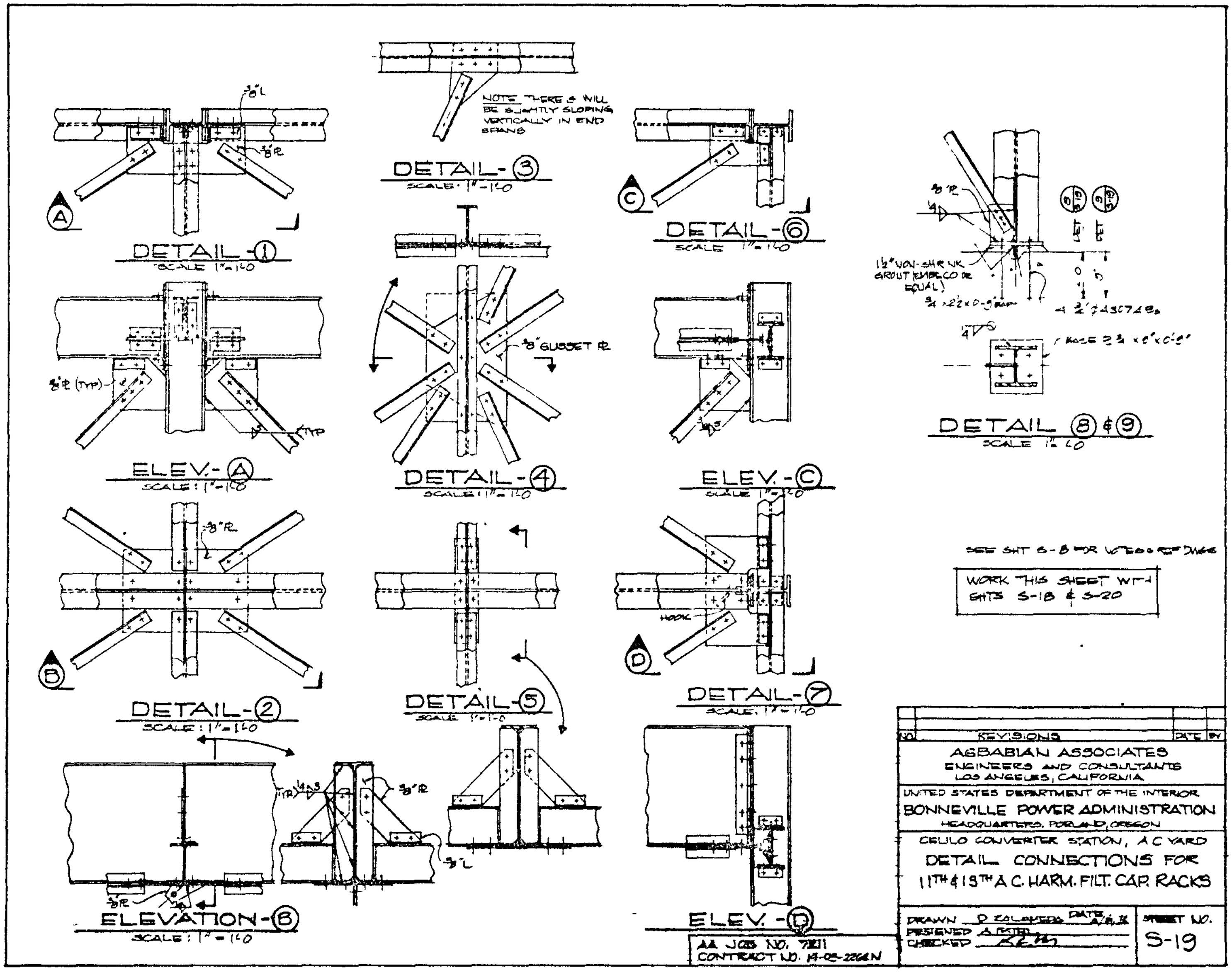


FIGURE 4-27. 11TH AND 13TH AC HARMONIC FILTER CAPACITOR RACK OVERHEAD SUPPORT FRAME DETAILS

R-7211-13-2859

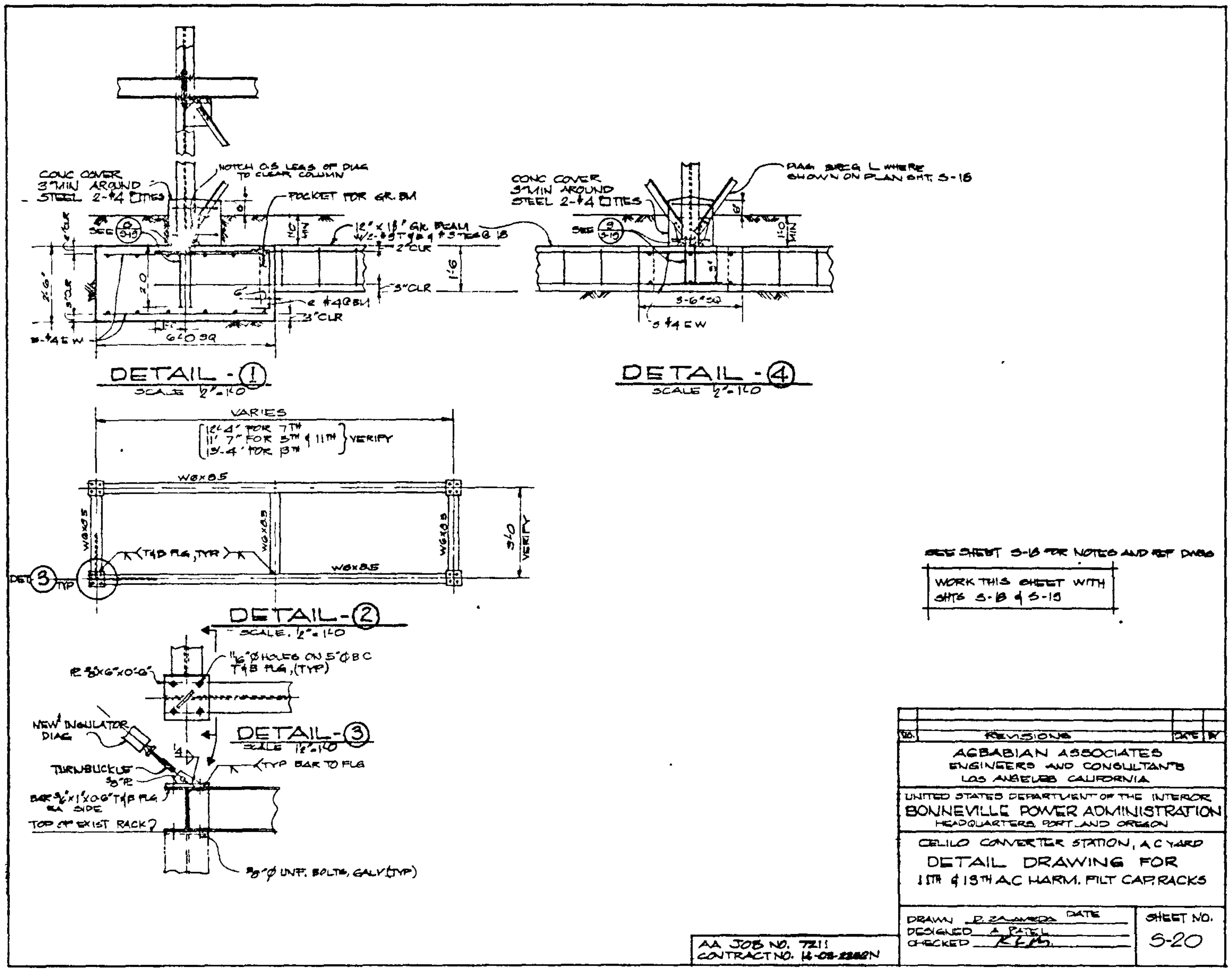


FIGURE 4-28. 11TH AND 13TH AC HARMONIC FILTER CAPACITOR RACK OVERHEAD SUPPORT FRAME DETAILS

R-7211-13-2859

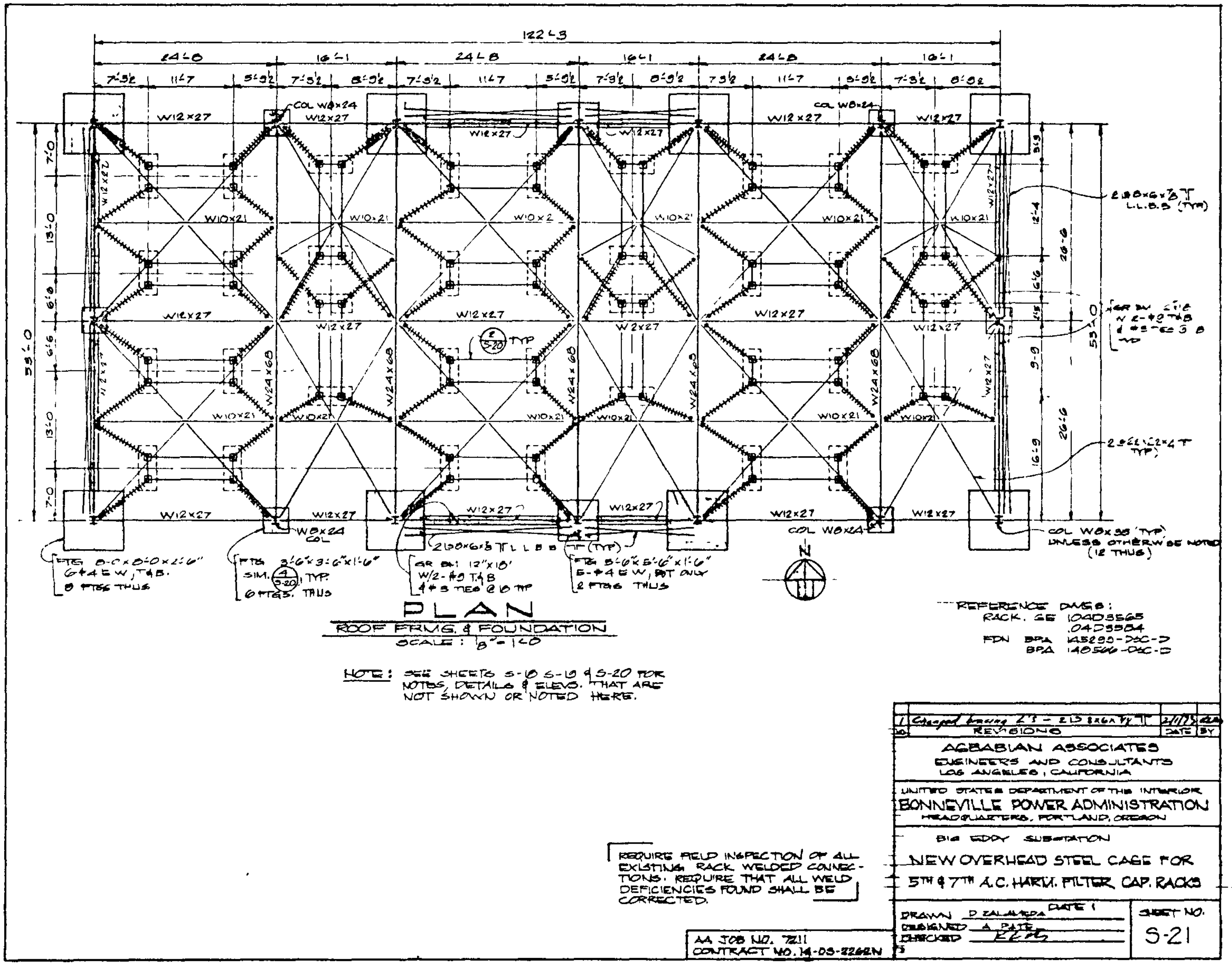


FIGURE 4-29. 5TH AND 7TH AC HARMONIC FILTER CAPACITOR RACK OVERHEAD SUPPORT FRAME

R-7211-13-2859

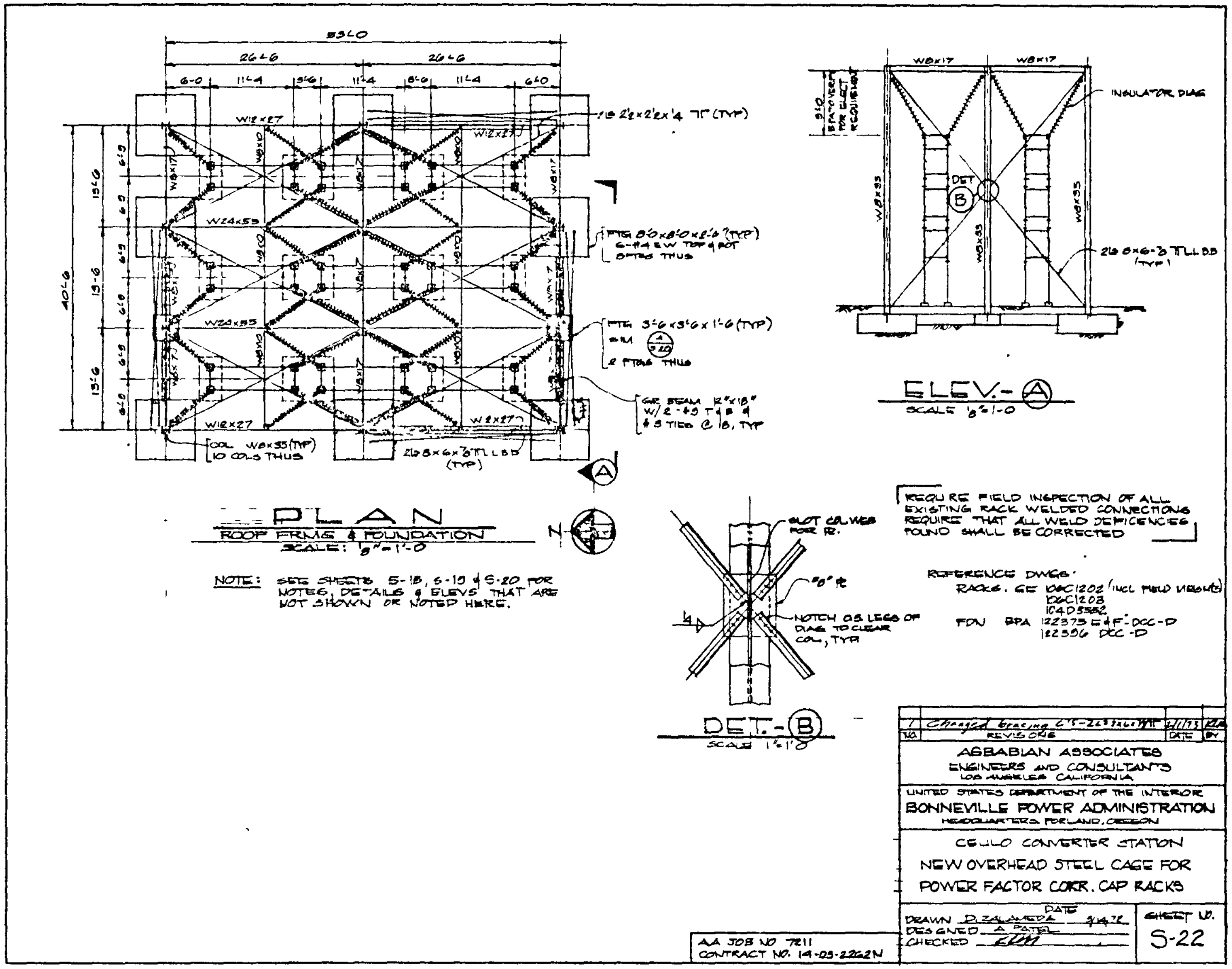


FIGURE 4-30. POWER FACTOR CORRECTION CAPACITOR RACKS OVERHEAD SUPPORT FRAME

145

R-7211-13-2859

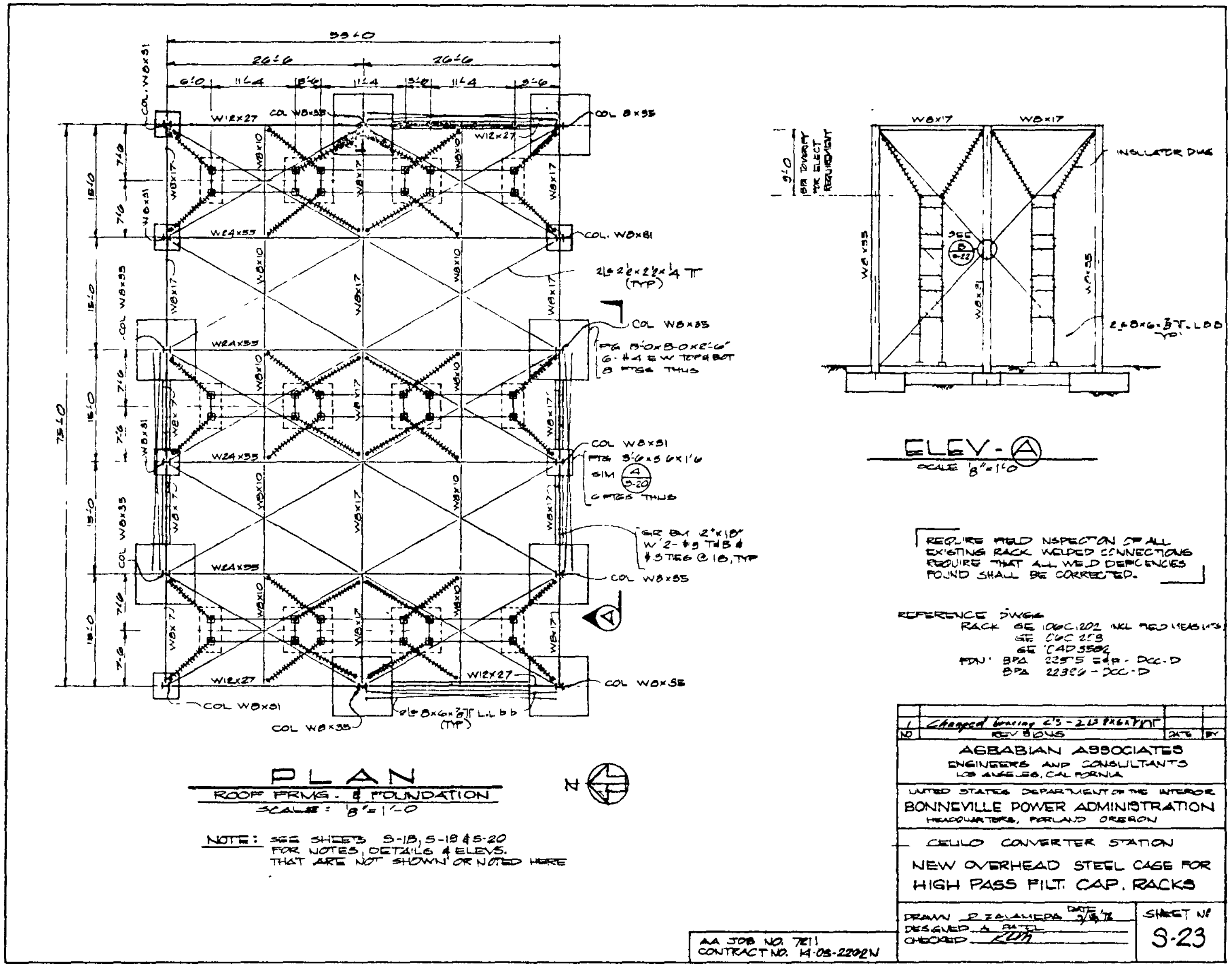


FIGURE 4-31. HIGH PASS FILTER CAPACITOR RACK OVERHEAD SUPPORT FRAME

R-7211-13-2859

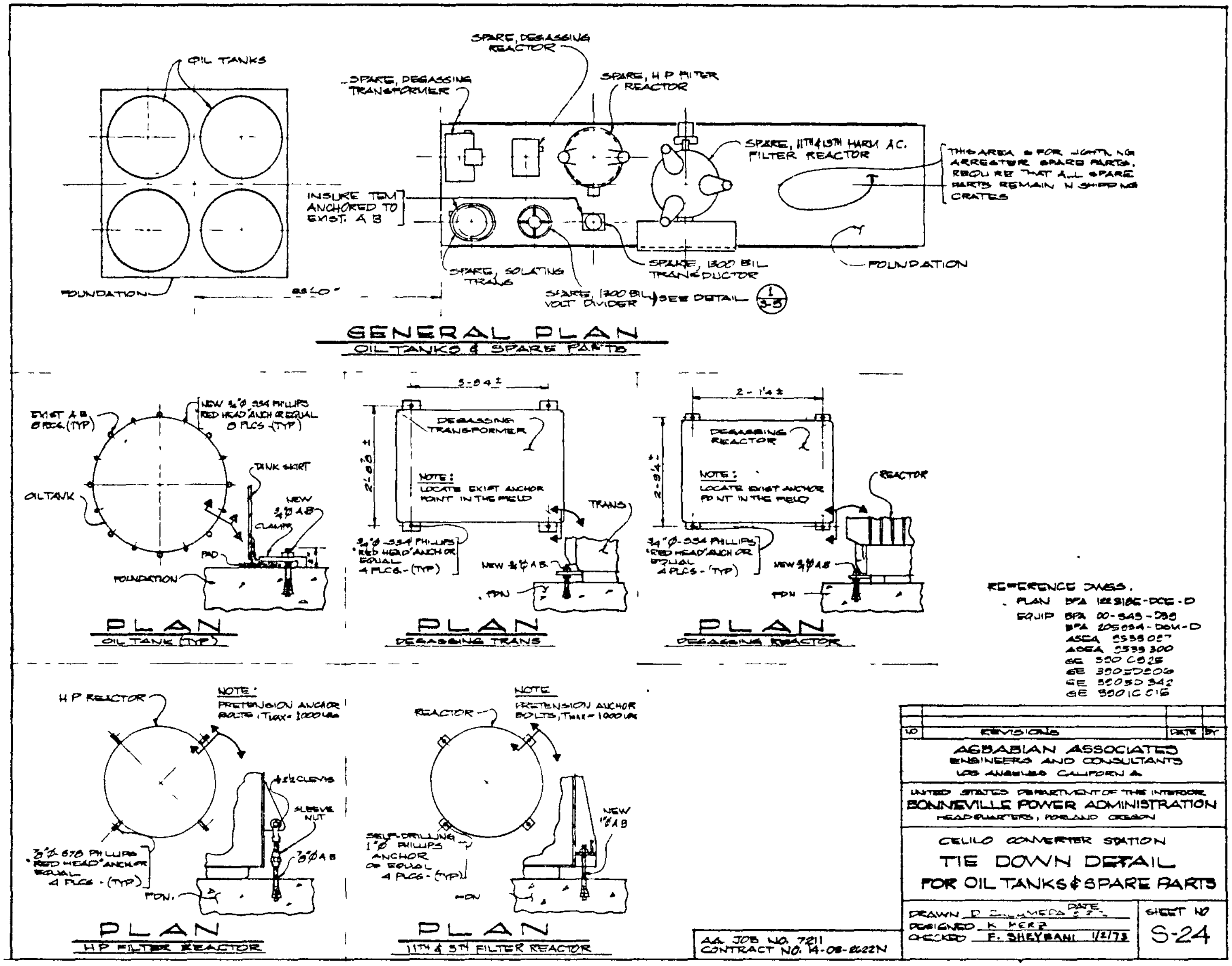


FIGURE 4-32. SPARE PART STORAGE AND OIL TANKS MODIFICATIONS

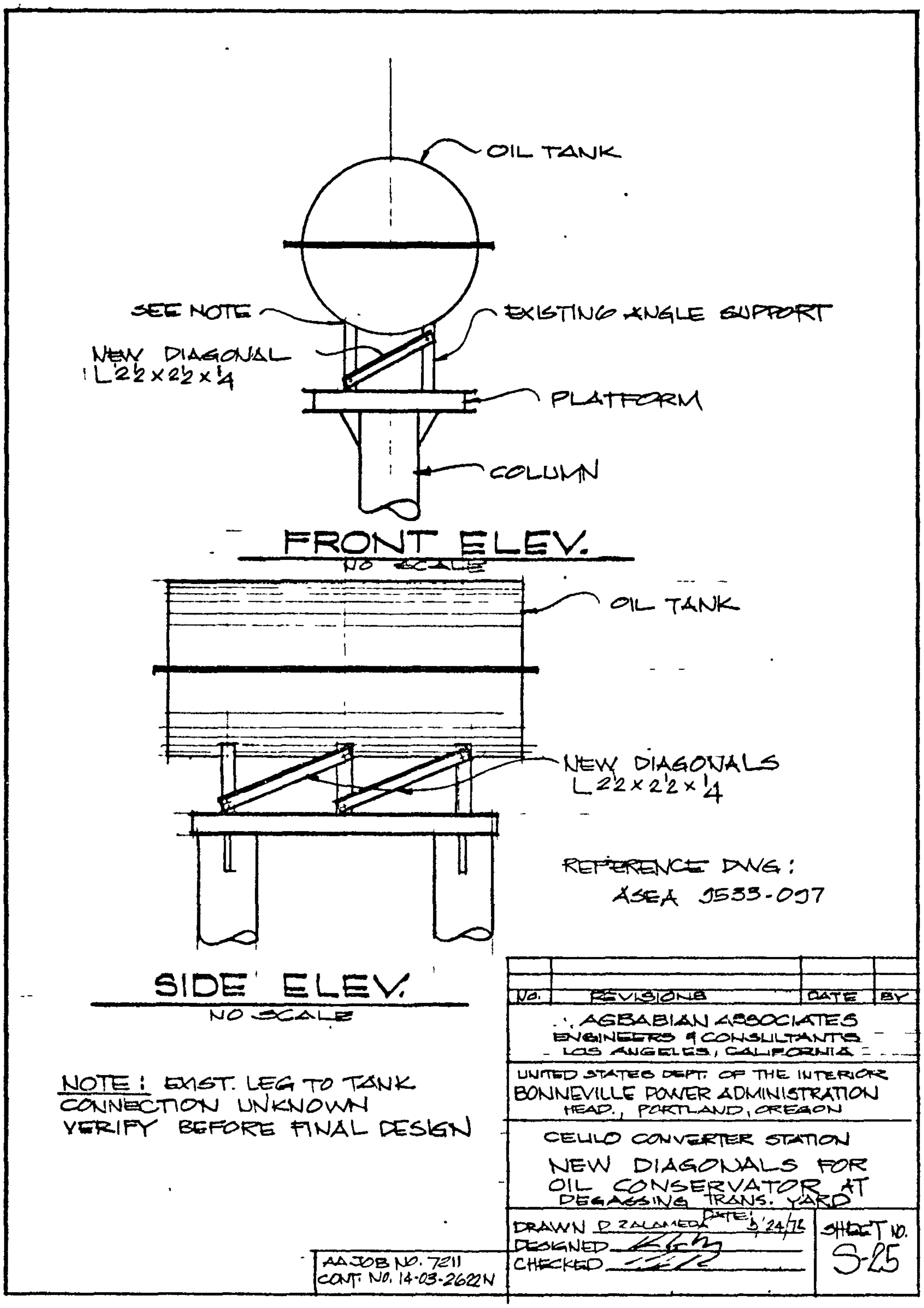


FIGURE 4-33. DEGASSING TRANSFORMER OIL CONSERVATOR MODIFICATION

PRECAST CONCRETE CANOPIES OF CELILO TERMINAL BUILDING

The precast concrete canopies of upper roof, lower roof, and visitors' gallery at the second floor of the terminal building at Celilo were investigated for gravity and earthquake loads. The estimated vertical frequency is 7.5 cps and the horizontal frequency is 5.0 cps. Based on these frequencies and using the criteria spectra for Celilo, the vertical and horizontal accelerations, would be 0.30 g and 0.48 g, respectively. However, because of the difficulty in estimating the stress concentrations that developed in the connections, higher design accelerations should be used. For example, the Uniform Building Code, 1970 Edition, Table 23-I, is quite restrictive regarding exterior ornamentation connections. For Zone 2, the UBC requires that exterior ornamentations be designed for an acceleration of 0.5 g from any direction and that connections should be designed for an acceleration of 1 g in any horizontal direction. Therefore, in the classification of precast concrete canopies, twice the computed acceleration, or an acceleration of 1 g, was used in the analysis of the connections. The results are listed in Table 4-4. It is important to note that structural members for the lower roof and visitors' gallery were calculated to have a factor of safety in bending for dead load alone of less than unity. Some connections at all three locations were found to be inadequate.

TABLE 4-4. FACTORS OF SAFETY FOR PRECAST CONCRETE CANOPIES

	Vertical Gravity Load Only		Horizontal and Vertical Seismic Loads	
Location of Canopy	F.S.	Category	F.S.	Category
Upper Roof	1.8	1	<1.0	2
Lower Roof	<1.0	2	<1.0	2
Visitors' gallery	<1.0	2	<1.0	2

Two possible modifications are feasible for the upper roof canopy, the lower roof canopy and the visitors' gallery canopy, which are the following:

a. Remove existing roof, install horizontal and vertical bracing between outlookers, install new strongback beams on top of existing precast units and rehang the precast units.

b. Remove existing precast units and replace them with light-weight fiberglass units that have desired concrete-like finish.

Based upon a careful study, it was concluded that it would be easier, quicker and less costly to replace the precast canopies with fiberglass canopies with the necessary minor modification in the supporting structure. Therefore, this modification was recommended.

PANTOGRAPH SWITCH - Custer Substation

The 500 KV pantograph type switch located at Custer Substation is basically a pedestal type item of equipment. This item of equipment was classified in Category 2 due to an inadequate factor of safety against porcelain bending failure. The present switch support is 2 stacks of EHV Station Post Insulators (Reference BPA Drawing No. 208911D-DSD-F), consisting of 4 post insulators each. The base insulator of each stack should be replaced with an equivalent insulator with cantilever moment capacity of 300,000 lb-in. (the present unit has a specified moment capacity of 208,000 lb-in.).

POTENTIAL TRANSFORMERS - Allston and Custer Substations

The 500 KV potential transformers located at both Custer and Allston Substations are porcelain cantilever type equipment with natural frequencies in the range of 4-5 cps. These items of equipment were placed in Category 2 due to inadequate foundation anchorage. For the PT located at Allston Substation, the existing anchor configuration and foundation need to be modified to resist an overturning moment of 442,000 lb-in. at the equipment base. A modification concept, which utilizes additional channel anchors and adds a ring of reinforced concrete around the existing foundation, has been prepared. For the PT located at Custer Substation, the equipment anchorage and foundation need to be modified. A modification concept which utilizes drilled piles to stabilize the equipment and foundation and to resist an overturning moment of 990,000 lb-in., has been prepared. However, it is recommended that a soils investigation be conducted at the Custer site before any foundation modifications are finalized.

ADDITIONAL ITEMS OF EQUIPMENT - Custer and Allston Substations

The items of equipment recommended for study and not listed in Exhibit I, were classified as indicated in Table 3-2 of Chapter 3. The items of equipment placed in Category 2 have, in general, foundations that do not meet the classification criteria. Items recommended for study but not classified are listed below.

Allston and Custer

- 230 KV PT
- 230 KV Bus Supports
- Battery Racks
- Oil Tanks
- Tertiary Racks

REFERENCES

4-1. *Assessment of Earthquake Resistant Design of AC-DC Converter Stations, HV-DC Pacific Intertie*, Agbabian-Jacobsen Associates, R-7119-1984, August 1971.

4-2 Letter Communication from General Electric Corporation to Bonneville Power Administration relative to modification to improve the earthquake withstand capabilities of the Sylmar Station, dated December 10, 1971.

CHAPTER 5

SEISMIC CLASSIFICATION PROCEDURES AND DESIGN MODIFICATIONS FOR DITTMER CONTROL CENTER

INTRODUCTION

The Dittmer Control Center is a vital command and communication facility requiring a different approach for earthquake assessment than that used for substation review. Because of the interdependence of subsystem elements, and a limitations on contract scope, an item-by-item analysis was not considered a realistic or practical approach. An alternate approach of assessing equipment and structures by generic class (e,g,, concrete structures, heavy equipment, metal racks, etc.) was adopted in order to assess the greatest number of items as possible within the time available. Generic classes of potentially earthquake-sensitive items were then analyzed by order of their priority of importance to the Dittmer control operation. An explanation of review priorities, preliminary seismic classification categories and a discussion of each generic class is provided in the following sections of this chapter.

EQUIPMENT REVIEW PRIORITIES

Items initially considered for investigation and seismic classification were selected by Bonneville Power Administration (BPA) and listed in Exhibit I of the basic contract (Reference 5-1). This list identified the building power equipment (primary power, emergency power, and uninterruptible computer power) and computer equipment (cabinets and consoles) as critical items. In addition to these items, other equipment and structures were also included after a site visit and plan review. Priorities for classification were then established based on three considerations concerning the effect of earthquake damage on Control Center operation. These considerations were the following:

a. Operational Criticality. How much effect would damage or failure of the particular item have on the overall immediate operation of the Control Center?

b. System Redundancy. Is the component or subsystem backed-up by equivalent units or alternate subsystems, so that earthquake damage would cause only a certain amount of inconvenience in operating function?

c. Shock Sensitivity. How susceptible is the item to earthquake damage, regarding its tie-down design and nature of internal components? Items free-standing and in danger of tip-over, or designed with brittle or acceleration-sensitive internal components would have a high rating.

Based on these considerations, equipment and structures were grouped into three priorities for classification to establish the order and depth of critical review that could be applied within the scope of the contract. Subsystem descriptions were based upon the Reference 5-2 document supplied by BPA. A listing of the items placed in each priority follows. Items identified with an asterisk (*) were included in the Exhibit I list.

PRIORITY I ITEMS

Structures and equipment which are crucial to the operation of the Control Center, are provided with minimal redundancy and are designed in such a way that earthquake damage is likely, were assigned Priority I. Such items were analyzed first insofar as contract funding would permit to determine design adequacy.

Structures

a. Microwave Tower. A reinforced concrete structure which a preliminary analysis indicated had a deficient factor of safety.

b. Control Center Building Frame. A reinforced concrete structure with a basement and two stories above grade. Preliminary analysis indicates a deficient factor of safety.

c. Prefabricated wall panels and overhead panels of the main building which could be dangerous to personnel due to connection failures.

Operational Equipment--Primary Communication

a. Data Acquisition Receiver Racks, LFC Subsystem. Tall metal frames supporting critical electronic equipment panels used for raw input data receival and conditioning for computer analysis.

b. Microwave Equipment Racks and Battery Racks. Metal frames supporting electronic panels and wiring or critical battery arrays for continuous power.

c. Group Display Boards. Large panels fastened to the walls, required for dynamic decision making.

PRIORITY II ITEMS

Equipment and subsystems which are important to Control Center operation but could be temporarily replaced by other subsystems or by additional manpower were assigned Priority II. Priority II equipment is *nonredundant and possibly* shock sensitive.

Building Support Equipment

a. Basement Air Conditioning for Computers. Heavy mechanical units supplying cooling air to basement personnel and equipment, including computers.

b. Overhead Lighting Fixtures in Display Room. Flexible frames supporting large lighting panels of the type that sustained heavy damage during the Sylmar earthquake. The fixtures are suspended in a manner which allows considerable lateral motion.

c. Main Power Equipment. Transformers and control panels which supply primary building power from standard utility lines.

Operational Equipment--Data Processing

*a. Free-Standing Computer Consoles (RODS, SCADA I). Equipment standing on raised floors without tie-downs. In danger of tip-over from earthquake ground motion. Such an effect would cause extensive internal damage to most computer and peripheral consoles.

*b. Memory Disks or Drums (Fixed or Moveable Head of RODS, SCADA I, LFC data acquisition). These consoles or components, even when tied-down, are acceleration-sensitive, and could easily malfunction or fail in an earthquake.

c. Acquisition Mini-Computers. Small electronic cabinets which are designed to receive digital data from several powerhouse locations and concentrate the data for computer analysis.

d. Output Consoles with Mercury Relays (ILDS and Portland Area S/C). These components are typically acceleration-sensitive, and could generate false signals during an earthquake.

*e. Uninterruptible Power Supply (UPS) for Computers. Subsystem of support equipment consisting of battery racks and control panels to supply continuous voltage computer power.

PRIORITY III ITEMS

Equipment and subsystems which are secondary to Control Center operation (e.g., emergency backup units) or are highly redundant, or are extremely rugged are included in Priority III group. Only a qualitative review was given to these items.

Building Support Equipment

a. Penthouse Air Conditioning Heavy Units. Mechanical equipment which provides air conditioning for personnel on the first and second floor.

*b. Emergency Generator, A small diesel-engine generator is provided to supply building power in case of primary building power loss, The equipment is extremely compact and rugged, and is judged to be secondary to Control Center operation due to its redundant operation,

Operational Equipment

Secondary data acquisition, internal communication and auxiliary electrical equipment which is rugged and noncrucial to the facility operation, These items were not classified,

a. Internal Telephone and Ross Cabling, Rugged and extremely redundant equipment,

b, Consoles and Cabling to Monitor Station-Status, Hard-wired solid state electronics which are inherently shock resistant, and are bolted to the floor, Units are only secondary to Control Center operation, and could be temporarily replaced by telephone communications,

c, Emergency VHF Communications Equipment, Portable transmitter/receiver unit for emergency communication with other BPA substations, Equipment is redundant to the microwave and telephone subsystems,

SEISMIC CLASSIFICATION CATEGORIES

After analysis, each type of equipment was assigned a seismic classification according to its ability to withstand the criteria earthquake ground motion. Categories used were the same as defined in Chapter 3.

Category 1	No modification required
Category 2	Modifications required
Category 3	Marginal, additional analysis required

EARTHQUAKE GROUND MOTION CRITERIA

The earthquake ground motion used for seismic classification was developed from a detailed consideration of the site and was done prior to the formulation of the general criteria in Chapter 2. The Dittmer Control Center is located in Zone B in the Portland-Williamette River potential epicentral area. A peak horizontal ground surface acceleration of 0.24 g was used which is comparable to Zone B criteria given in Chapter 2. Because of the importance of this facility, site-dependent spectra were developed and are given in Figure 5-1.

CLASSIFICATION SUMMARY

Based upon the above criteria, a classification of the equipment and structures was conducted for susceptibility to seismic damage. A summary of the results of the classification study is provided in Table 5-1. Discussion of the classification study and of the design modification concepts follows in the remaining sections of this chapter.

COMPUTER CABINETS AND DISPLAY CONSOLES FOR RODS AND SCADA 1 SUBSYSTEMS

Classification for Earthquake Design Adequacy

The computer cabinets and display consoles are subject to three potential types of earthquake damage as a result of their free-standing support on a raised floor:

1. Support-foot lift-off and potential unit tip-over

2. Support-foot sliding and potential adjacent impact or electrical cable damage.

3. Acceleration damage to fragile electromagnetic components.

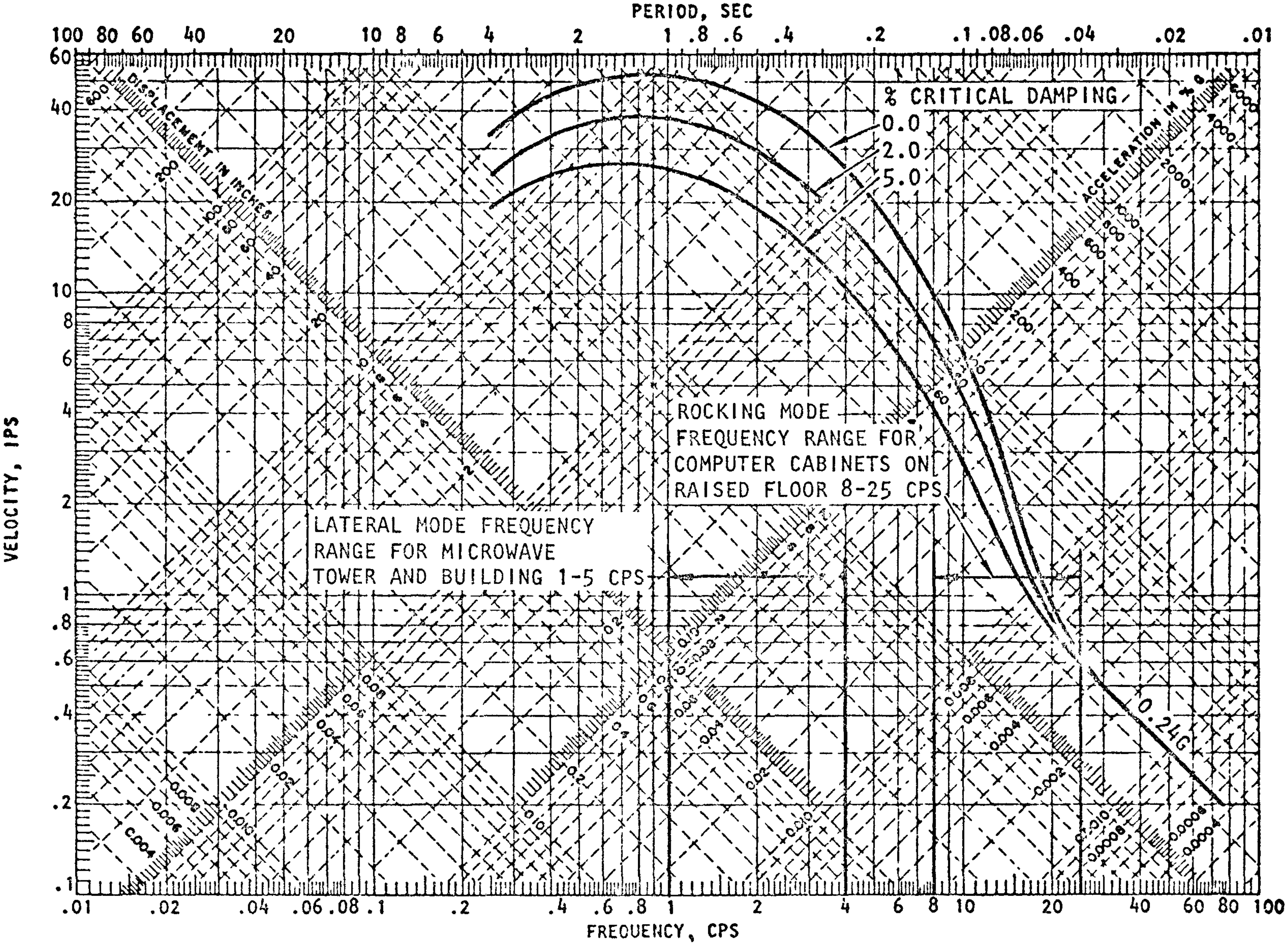


FIGURE 5-1. HORIZONTAL RESPONSE SPECTRA CURVES FOR EQUIPMENT AT DITTMER CONTROL STATION; USE 2% CURVE FOR EQUIPMENT ASSESSMENT

TABLE 5-1. SUMMARY OF SEISMIC CLASSIFICATION CATEGORIES FOR DITTMER CONTROL CENTER EQUIPMENT AND STRUCTURES

Item	Seismic Classification
Priority I Items:	
Microwave Tower Structure	2
Control Center Building Frame	2
Precast Concrete Panels	2
Electronic Equipment Racks, LFC and MW	1
Battery Racks for Microwave and UPS Systems	2
Group Display Boards	3
Priority II Items:	
Basement Computer Air Conditioning	2
Overhead Lighting in Display Room	2
Main Power Equipment	2
Computer Units	
*Equipment Group A	2
*Equipment Group B	1
*Equipment Group C	1
Uninterruptible Power	2
Priority III	
Penthouse Air Conditioning	2
Emergency Generator	2

*See Table 5-2 for listing of equipment in Groups A, B and C.

The second and third failure mechanism are of a low-probability nature and do not present a major threat of extensive and permanent damage to internal components or cabling. Most of the units are designed with sufficiently long electrical cables and with rugged solid state electronics so that sliding and shock loading are not significant problems. The first failure mechanism, however, is of major concern, and must receive careful attention. Unit tip-over, especially due to large lateral loads on units with high centers of gravity, represents a very real problem. Approximately 80 out of 131, or 61 percent of the cabinets and consoles have been classified in Category 2, requiring modifications, because of tip-over danger. Another 14 of the cabinets were initially classified as Category 3, questionable, because of low factor of safety for tip-over or known fragile electromechanical components. Many of the units not identified in Category 2 are in some danger of tip-over but are of a peripheral or auxiliary nature as far as the primary computation function is concerned, and do not require redesign.

A summary of the investigation is presented in Table 5-2 below, for three groups of equipment: (A) Main System, (B) Disk Memory, and (C) Display Consoles.

TABLE 5-2. CLASSIFICATION OF COMPUTER UNITS

Model No. and Description		Equipment Group	Quantity	Over-turning Factor	Seismic Classification Category
RODS COMPUTE	ME10 Core Memory; Mag Tape	A	19	0.7	2
	KA10 Airthmetic Processor	A	2	1.4	2
	DT04/DF10 Buffer; Con.	A	20	0.6	2
	PDP-11 Comp.; Other Aux.	A	28	0.4	2
	RD10 Fixed Head Disk	B	10	1.8	1
	RP02 Disk Pack Drive	B	4	2.4	1
SCADA COMPUTE	- CSU GE Pack 4010	A	2	0.7	2
	- Digital I/O; ASU	A	3	0.6	2
	- Magnetic Tape; Aux.	A	6	0.6	2
RODS & SCADA OTHER FUNC.	Display Consoles	C	24	>3.	1
	Peripheral and Aux.	-	13	>3.	1

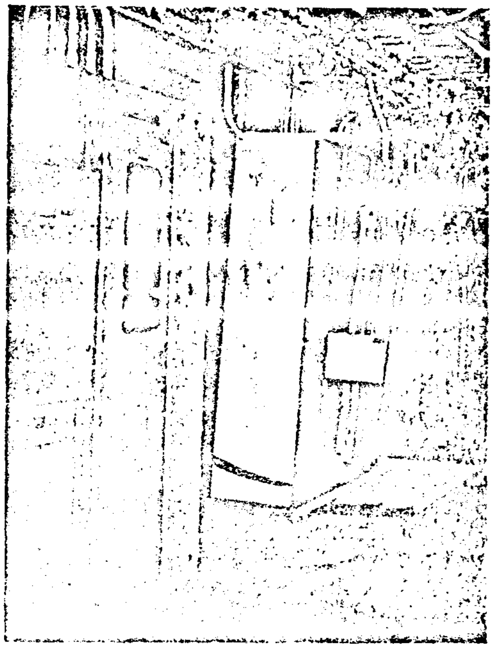
Discussion

The primary computer cabinets of RODS and SCADA 1 and particularly the auxiliary data acquisition and display cabinets of RODS, have been classified in Category 2 requiring modification, because of their low factor of safety for tip-over (below 1.5 in all cases). This situation is the result of the large weights and high centers of gravity of the cabinets. Sliding of the cabinets on the raised floor surface may represent a load-limiting device which prevents tip-over, but this mechanism cannot be counted on due to the variability of friction.

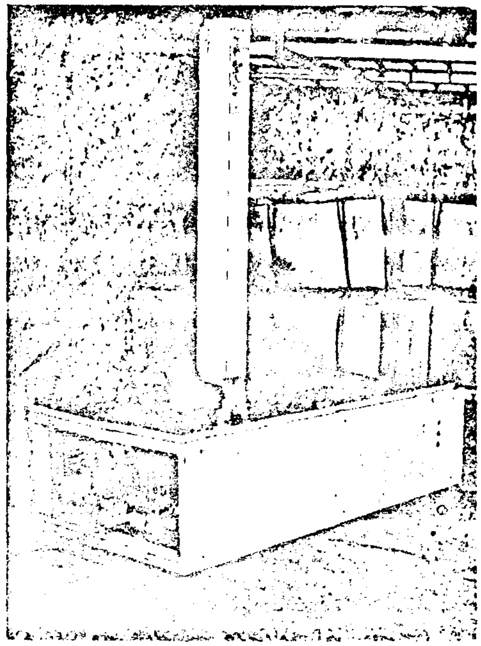
For cabinets with a marginal factor of safety for tip-over (between 1.8 and 2.8), and with fragile internal components, more investigation was required. The RODS Fixed Head Disk and Disk Pack Drive Units (Models RD10 and RP02, respectively) fell within this criterion and were initially placed in Category 3. Detailed vendor data were obtained and reviewed before these units were placed in Category 1. The required factor of safety for a rigid body on a flexible base is 3.0, but for units with structure modal frequencies below 20 cps, such as the disk memory units, only a factor of safety of 2.0 was considered necessary. The display consoles located in the basement Dispatch Room are low profile rugged units with factors of safety above 3.0, and are therefore, in no danger of tip-over or impact shock damage. The entire group of display consoles for the RODS and SCADA 1 subsystems were, therefore, classified in Category 1.

The evidence at Sylmar (Reference 5-3) indicates that the computer cabinets moved around considerably, but did not tip over during the February 1971 earthquake. However, differences in the floor and cabinet structural designs, and in the earthquake characteristics between Sylmar and Dittmer are sufficiently great that verification against tip-over failure cannot be assured. Only a limited number of cabinets were in place at Sylmar, and most of these were relatively low profile so statistical evidence is lacking regarding cabinet stability. Several other types of rigid items did tip over at Sylmar and the failure mode must, therefore, be considered a real and present danger at Dittmer.

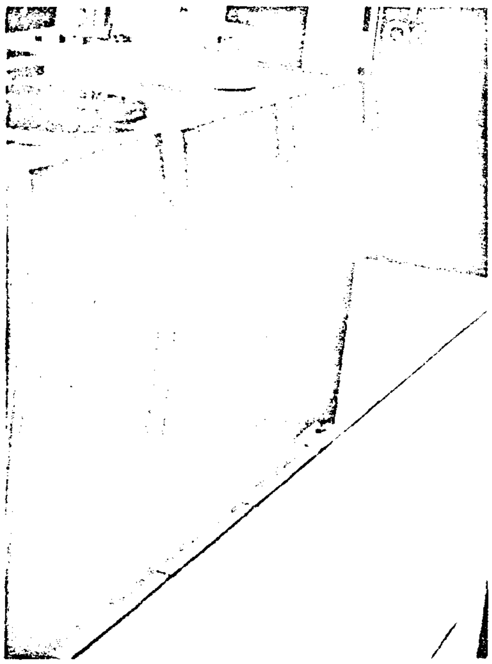
Photographs of typical tall cabinets subject to tip-over are included in Figure 5-2. Detailed discussion and analysis are included in Reference 5-4.



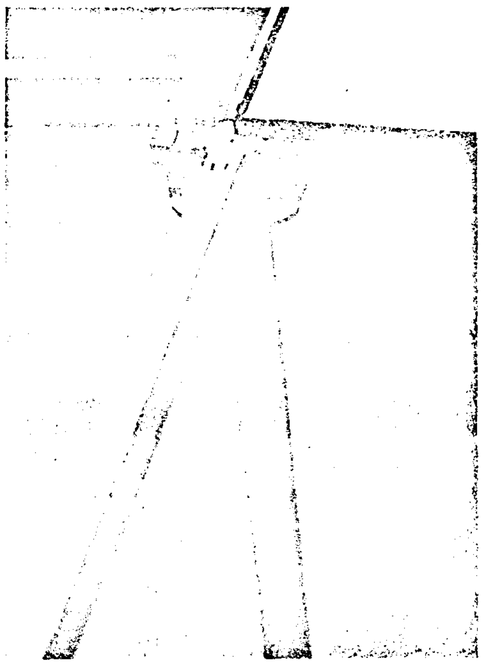
(a) SYLMAR CONTROL PANEL, WARPED



(b) SYLMAR CABINET, TOPPLED



(c) DITTMER COMPUTER CABINETS



(d) RAISED FLOOR DETAIL

FIGURE 5-2. PHOTOGRAPHS OF FREE-STANDING CABINETS SUBJECT TO TIPOVER

Modification Concepts

The main computer cabinets of the RODS and SCADA I system are tall, heavy units free standing on the raised floor and are in danger of tip-over. These units, designated as Group A in the preceeding section, were classified Category 2, and require structural modifications to ensure stability under lateral loading.

Six potential simple fixes could be implemented to prevent support foot lift-off and potential unit tip-over of the Group A computer cabinets during a severe earthquake:

a. Secure cabinet directly to the pedestals supporting the raised floor. This will require spreader brackets from the cabinet corners to the pedestals. The spreader brackets may be fastened under the raised floor to avoid protrusions.

b. Secure cabinet to the floor panels and bolt the panel to the corner pedestals to prevent panel lift-up. Panel bolt-down may not be required if the effective width of the panels and cabinet as a unit is sufficiently large.

c. Fasten adjacent cabinets together, using an external framework, arranging the cabinets in an L-shape to provide large effective base width.

d. Fasten adjacent cabinets together and fasten cabinets to the floor instead of arranging in L-shape. Requires additional pedestals under the frame corners or spreader brackets bolted to existing pedestals.

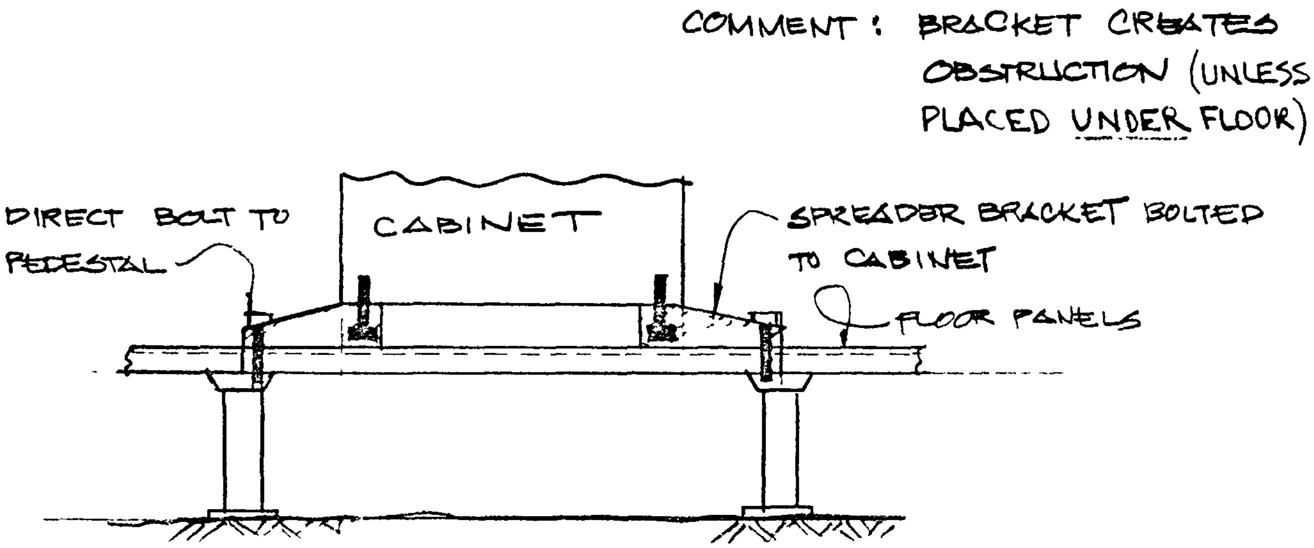
e. Shock isolate the cabinet so that it does not experience floor motion. This method is most effective for shock-sensitive memory disk units.

f. Shock isolate the floor panels at the support pedestals by installing springs between the pedestal cap fitting and pedestal rod. This method could provide shock isolation for an entire floor section.

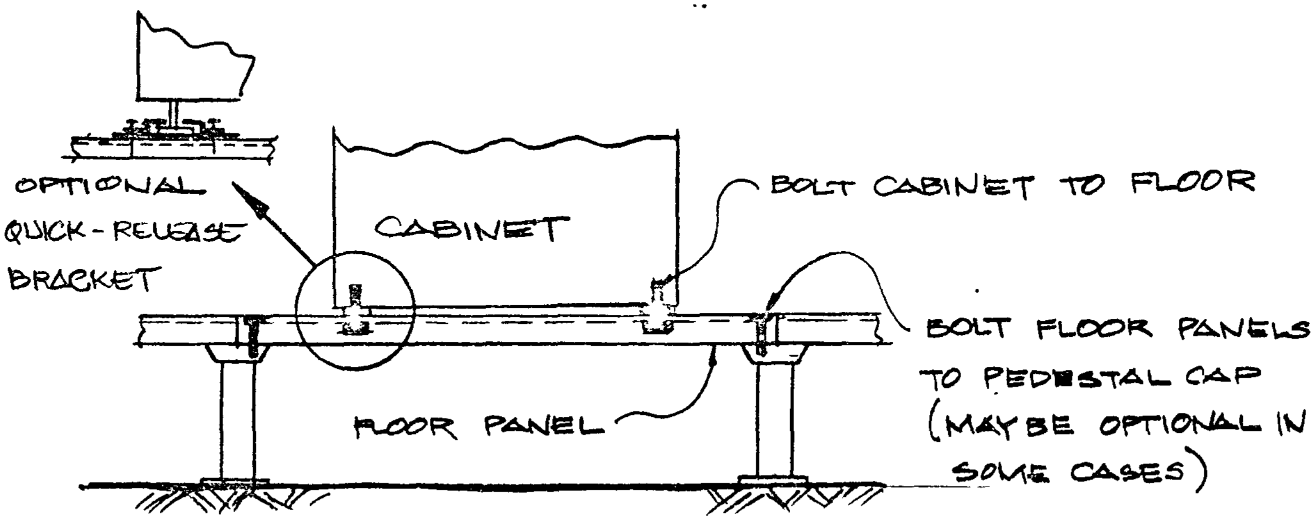
These potential redesign methods are shown schematically in Figure 5-3. Several factors must be considered in selecting the best concept for implementation. Assuming that each design could provide practical insurance of earthquake stability, care must be exercised to maintain cabinet grounding, ease of unit replacement, minimal down-time during modification, and minimal initial cost. A comparison of the rating factors affecting the final selection of the appropriate concept for each computer cabinet design is presented in Table 5-3 to provide a quantitative comparison of the various design features. A high rating for each of the four design considerations will indicate a favorable concept, with points assigned according to the relative importance of each.

TABLE 5-3. COMPUTER CABINET REDESIGN CONCEPT EVALUATION FOR EARTHQUAKE TIP-OVER STABILITY

No.	Concept Description	Rating Factor (High is Favorable)				Overall Rating
		Electrical Grounding	Instal-lation Down-Time	Unit Maintenance	Initial Cost	
1	Direct Tie-down	0	10	10	25	45
2	Indirect Tie-down	5	15	20	40	80
3	Bolt Together, L-shape	10	20	5	30	65
4	Bolt Together, Ends down	5	10	5	20	40
5	Cabinet Isolation	10	5	30	10	55
6	Floor Isolation	10	0	30	/0	40
POSSIBLE POINTS/CATEGORY		10	20	30	40	100

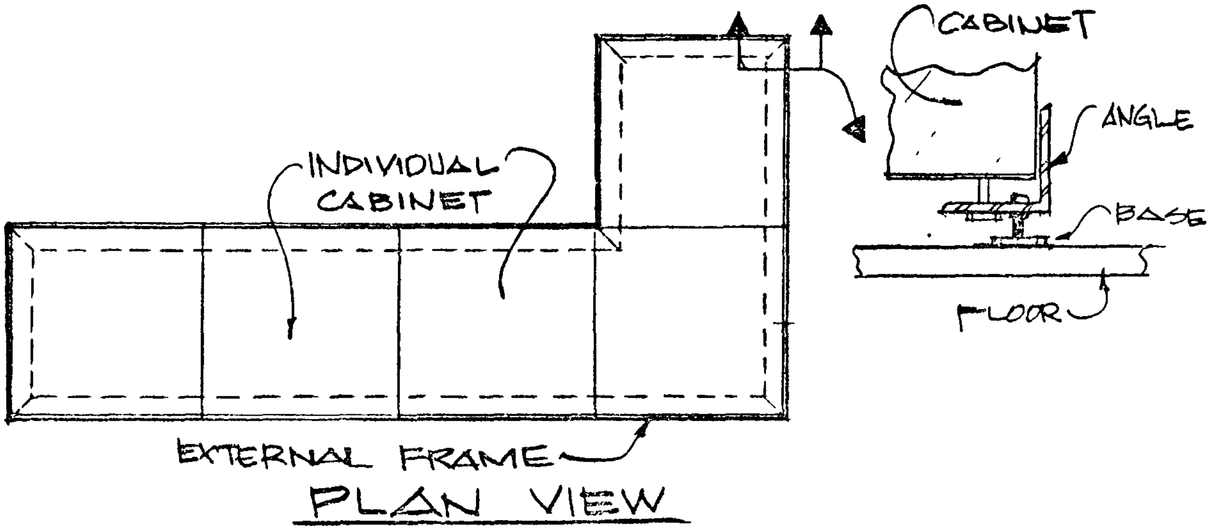


(a) REDESIGN CONCEPT NO. 1--DIRECT BOLT-DOWN TO PEDESTAL

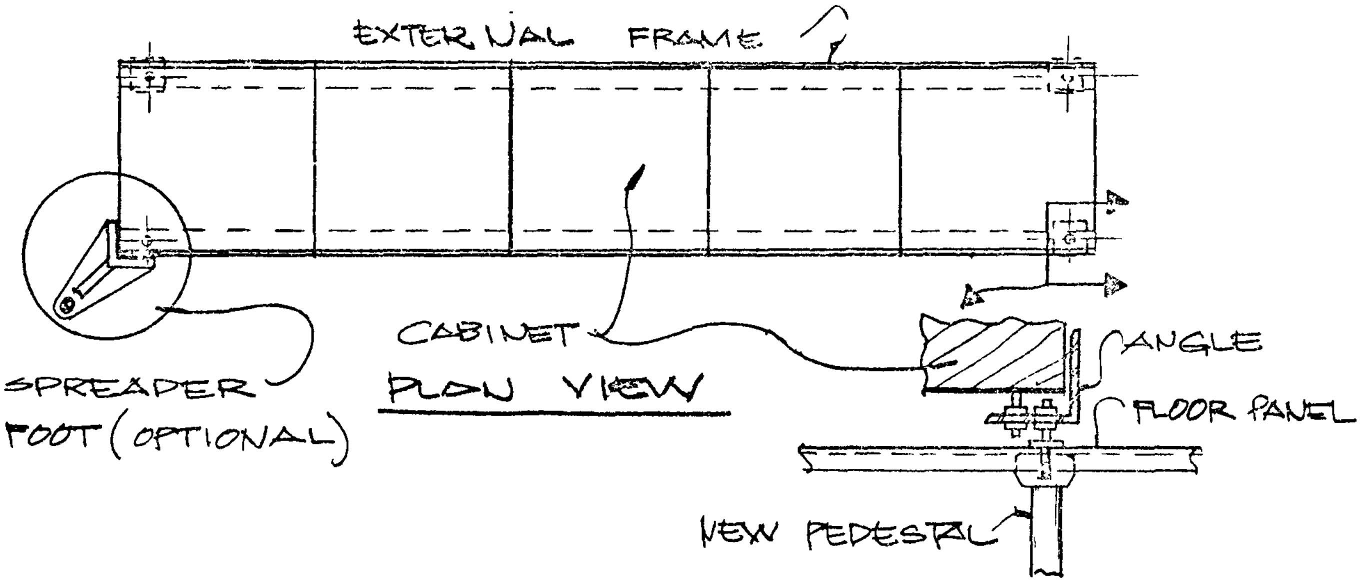


(b) REDESIGN CONCEPT NO. 2--INDIRECT BOLT-DOWN (CABINET TO PANEL TO PEDESTAL)

FIGURE 5-3. COMPUTER CABINET ATTACHMENT REDESIGNS

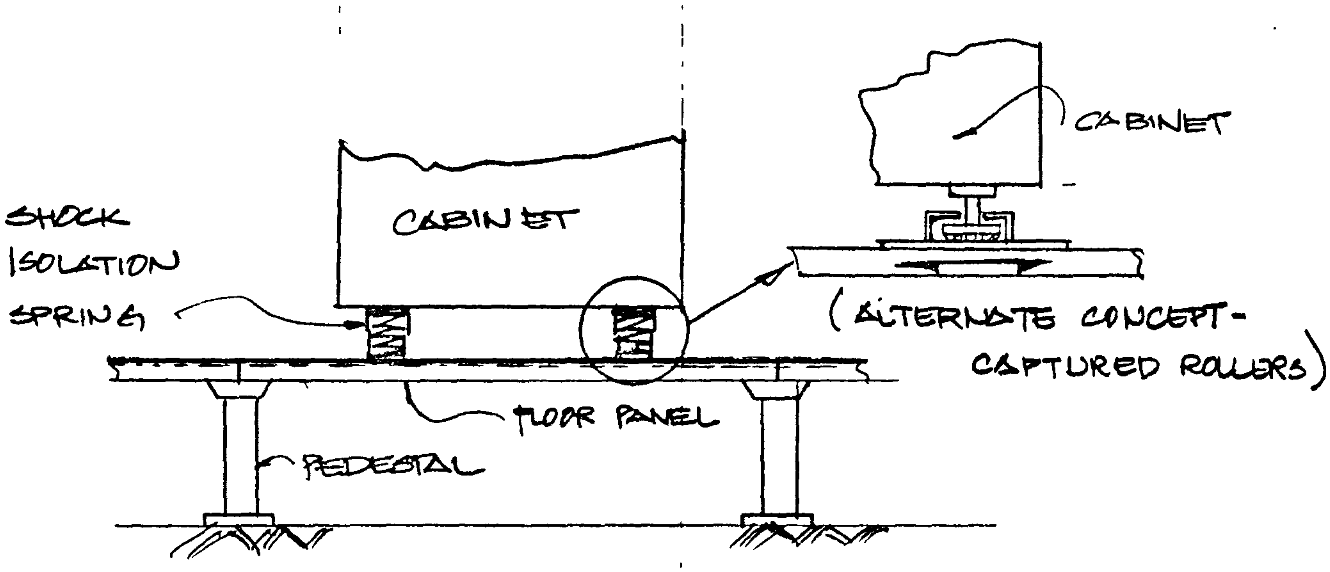


(c) REDESIGN CONCEPT NO. 3--CABINETS BOLTED TOGETHER IN L-SHAPE

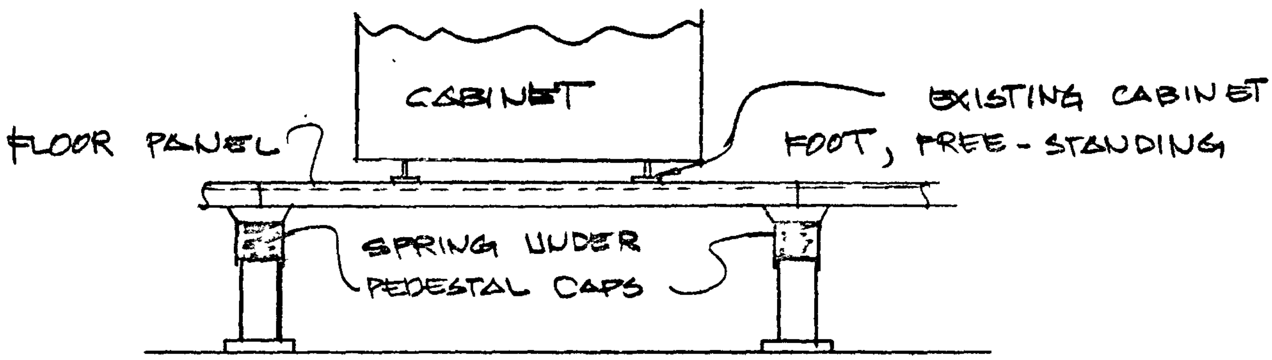


(d) REDESIGN CONCEPT NO. 4--CABINETS BOLTED TOGETHER;
END CABINET BOLTED DOWN

FIGURE 5-3. (CONTINUED)



(e) REDESIGN CONCEPT NO. 5--SHOCK ISOLATE INDIVIDUAL CABINETS



(f) REDESIGN CONCEPT NO. 6--SHOCK ISOLATE FLOOR AT PEDESTAL CAP FITTINGS (VERTICAL ONLY)

FIGURE 5-3. (CONTINUED)

The most favorable design is Concept No. 2, utilizing indirect tie-down. Electrical grounding would be a slight problem with some form of insulation shield required to surround each fastener tieing down a cabinet foot. Maintenance of the cabinet would also be a problem since the support foot would have to be unbolted to remove the unit. Neither of these difficulties is considered severe, and since system down-time and installation costs are both at a minimum, the concept appears optimal.

Bolting the units together in an L-shape, Concept No. 3, is the second most favorable design due to low cost (if cabinet modifications are minimal) and negligible down-time. Maintenance could pose a severe problem, however, since the frame tieing the cabinets together would have to be unfastened from the cabinets to allow for unit replacement. Any cabinet design changes which modify the fastener pattern would require a frame redesign. Concept 4, with a frame bolting the units together, and additional pedestals to bolt the end units down to the subfloor is favorable from an overall cabinet layout standpoint, but poses serious grounding and maintenance problems. Similarly, the direct bolt-down of each cabinet to the adjacent floor pedestal, Concept No. 1, poses grounding problems and even more serious maintenance problems since the spreader bracket at each cabinet corner is a special design.

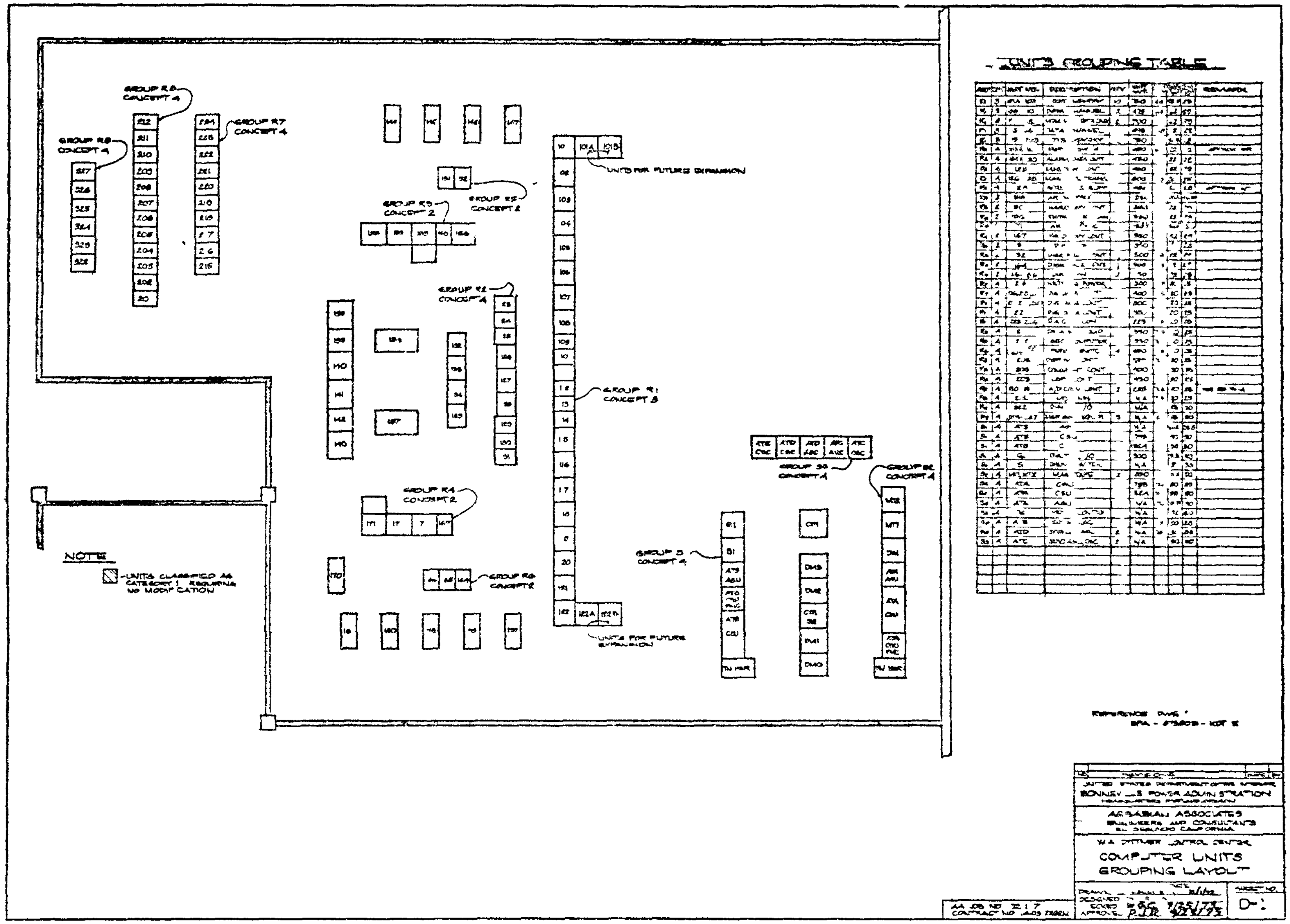
Shock isolation of the individual cabinets (Concept No. 5) or of an entire floor section (Concept No. 6) is favorable from a cabinet modification and maintenance standpoint, but would be costly to implement.

Recommended Design Modifications

The tall, heavy memory and processing cabinets of the RODS and SCADA 1 subsystems were classified in Category 2, requiring modification because of tip-over danger in a severe earthquake. Alternate design concepts were compared with principal attention given to direct bolt-down of separate cabinets or the bolt-together of groups of cabinets. This section with attached drawings presents the recommended modification and associated structural member sizes for each group of cabinets as they are actually located on the basement raised floor.

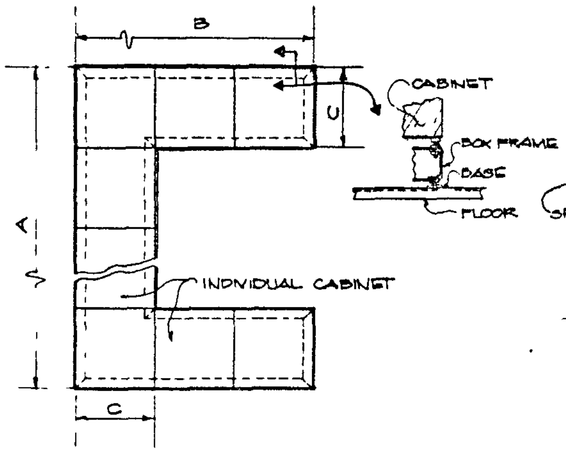
A total of nine groups of RODS cabinets and three groups of SCADA 1 cabinets, designated R1 through 9 and S1 through 3 on the D-1 drawing entitled, "Computer Units Grouping Layout," have been identified for modification. The D-1 drawing is reproduced in Figure 5-4(a) for reference. Modification Concept No. 2 (described in the preceding section) consisting of direct bolt-down of individual cabinets, has been recommended for Groups R3 through 6. These cabinets of the RODS system include the Arithmetic Processors and Controllers which are located next to the Fixed Head Disk units. Redesign Concept No. 3, consisting of bolt-together framing of cabinets arranged in a C-configuration for stability, is recommended for Group R1, the RODS core-memory units. The other cabinet groups of the RODS and SCADA I subsystem are arranged in a line, and are to be modified utilizing Concept No. 4. A frame is used to bolt adjacent cabinets together, and the ends of the frame are bolted down to the floor. Additional pedestals should be installed under the raised floor at the location of the frame support feet to carry local vertical loads.

The three modification concepts utilized to bolt the cabinets down are shown on the D-2 drawing entitled, "Design Concepts for Computer Seismic Loads," which is reproduced in Figure 5-4(b). Since the cabinets arranged in a common line are typically bolted together by the supplier, additional external framing may not be required (see Note 1 of Drawing D-2). However, if the supplier-provided framing is not strong enough to carry the induced loads from an earthquake lateral excitation, an added external box-frame as shown on the D-2 drawing will be required. The frame, designed to carry loads in

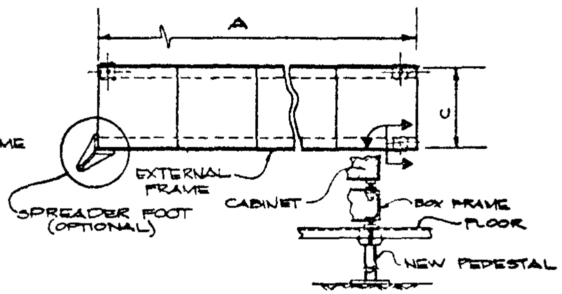


(a) CABINET LAYOUT DRAWING

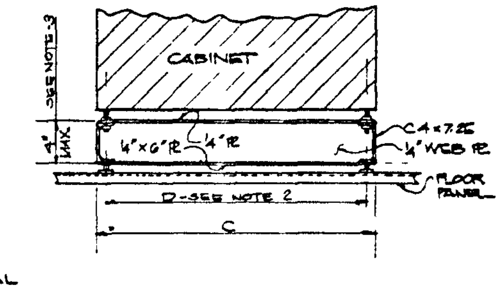
FIGURE 5-4. COMPUTER CABINET REDESIGN DETAILS



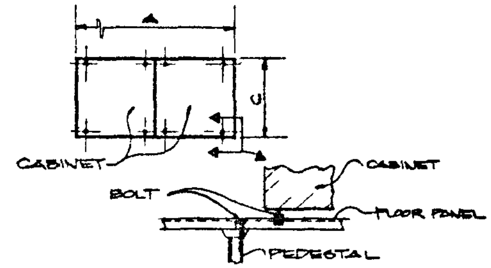
PLAN VIEW
CONCEPT NO 3



PLAN VIEW
CONCEPT NO 4



BOX FRAME SECTION
(OPTIONAL SEE NOTE 1)



PLAN VIEW
CONCEPT NO 2

NOTES:

1. MOST OF INDIVIDUAL CABINETS LOCATED IN A COMMON LINE ARE BOLTED TOGETHER. IF CONNECTION STRENGTH IS NOT SUFFICIENT FOR EARTHQUAKE DESIGN THE OPTIONAL BOX TYPE FRAME IS TO BE PROVIDED.
2. DISTANCE OF BOLT CENTER TO CENTER (D) WAS BASED ON THE MAXIMUM OF THE CABINET WIDTHS (C) DIMENSION (D) SHALL BE DETERMINED UPON INSTALLATION FOR BOX TYPE FRAME CONCEPT.
3. DEPTH OF BOX TYPE FRAME IS TO BE 4 IN MAXIMUM, BASED ON WORSE CASE STRENGTH REQUIREMENTS IF EXISTING CABINET FRAMING IS SUFFICIENTLY STRONG TO CARRY A LARGE PORTION OF BENDING LOADS, OR IF A PARTICULAR CABINET GROUPING IS LIGHTWEIGHT A SMALLER BOX-FRAME DEPTH WILL BE REQUIRED.
4. CONCEPTS DISCUSSED IN AA MEMO 7211-7-2524, DATED OCTOBER 30, 1972

GROUPING DIMENSIONS

GROUP	CONCEPT	UNITS	DIMENSION (IN)			
			A	B	C	D
R	3	26	652	93	29	
R_2	4	9	222		28	
R_3	2	3	1405		49.5	
R_4	2	2	185		49.5	
R_5	2	2	44		29	
R_6	2	3	66		29	
R_7	4	10	200		25	
R_8	4	12	240		25	
R_9	4	6	139		30	
S	4	5	183		30	
S_2	4	6	2155		30	
S_3	4	5	150		30	

NOTE SEE SHEET D-1 FOR GROUPING DESCRIPTIONS

REFERENCE DWG:
BPA-1073808-KDT-E

NO.	REVISIONS	DATE	BY

UNITED STATES DEPARTMENT OF THE INTERIOR
BONNEVILLE POWER ADMINISTRATION
HEADQUARTERS PORTLAND, OREGON

AGBABIAN ASSOCIATES
ENGINEERS AND CONSULTANTS
EL SEGUNDO, CALIFORNIA

W A DITTMER CONTROL CENTER
DESIGN CONCEPTS FOR COMPUTER SEISMIC LOADS

DRAWN D ZALAMBDA DATE 12/1 72
DESIGNED D CROSS
CHECKED D G C 9/25/73
APPROVED D J R 9/25/73

SHEET NO. D-2

AA JOB NO 7211-7
CONTRACT NO. 14-03-28GRN

(b) TIE-DOWN DETAILS

FIGURE 5-4. (CONTINUED)

the core memory cabinets, Group R1, consists of welded steel channels and plate members with a 4 in. maximum depth. This box-frame would be placed under the cabinets with existing cabinet feet modified to bolt directly to the frame edge members.

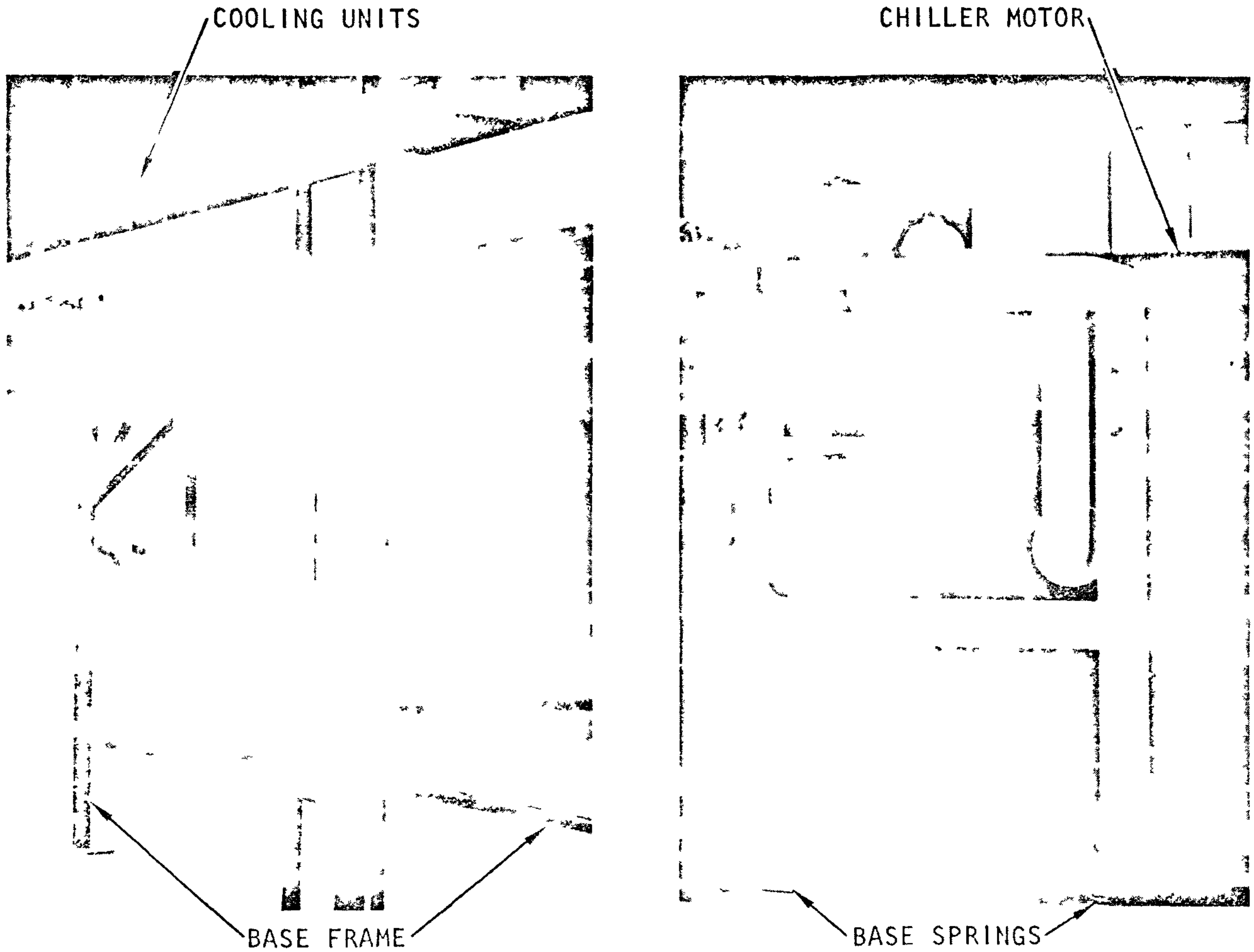
The concepts have been selected in a manner which minimizes floor modifications and system down-time. Costs will be correspondingly low with this criterion. Considerations of cabinet electrical ground and forced air cooling must be included when detailed drawings are prepared at a later date. No major difficulties are anticipated when insulators and cooling holes are incorporated into the box-frame design and existing electrical cables should not require modification.

HEAVY UNITS FOR ELECTRIC POWER AND AIR CONDITIONING

Classification for Earthquake Design Adequacy

The heavy units for building power, uninterruptible computer power, and air conditioning present a hazard to operating personnel during an earthquake. Classification of these units for seismic design adequacy was included within the project scope, together with design modifications for units classified in Category 2. However, because the necessary design data of weight and dimensional information could not be obtained in sufficient time, classification of individual units such as air conditioning blowers and boilers, and power transformers and control panels was not possible. For this reason, and because the power and air conditioning units are not Priority I items, only a general review for sliding and tip-over stability was provided.

Based upon information visually obtained during a field trip inspection, as shown in Figure 5-5 photographs, the power and air conditioning equipment is not adequately designed for lateral loading. Base attachment bolts are inadequate and in some cases deleted entirely. Flexible frames or vibration-isolator springs were also observed. For this reason the entire group of heavy equipment for uninterruptible power, building power, and air conditioning



(a) FLOOR AND CEILING MOUNTED ON FRAME SUPPORTS (COOLING UNITS)

(b) FLOOR MOUNTED ON VIBRATION ISOLATOR SPRINGS (A/C CHILLER)

FIGURE 5-5. PHOTOGRAPHS OF HEAVY UNITS FOR BUILDING POWER AND AIR CONDITIONING--SUBJECT TO LATERAL INSTABILITY was classified in Category 2. Without detailed structural data it was not possible to determine if specific units have sufficient lateral support to withstand seismic excitation.

Structural Modifications

In order to prevent sliding and tip-over of the heavy equipment, attention should be given to strengthening of the base attachment for each unit. The most common installation, as shown in Figure 5-6, consists of a set of individual bolts inserted through a base flange into predrilled concrete inserts. This method is adequate to carry seismic lateral loads if the local support structure is strong enough and the size and number of fasteners is sufficient to prevent excessive bolt tension loads. The bolt tension is dependent on the net overturning moment. The maximum bolt tension load, as determined by the net overturning moment, should result in a factor of safety of 3.0. Attention should also be given to flange or foot bending stresses due to the eccentricity in attachment loads. Local strengthening of the base structure may be necessary to prevent bending failures.

Recommendations

The recommended modifications for heavy units at Dittmer consists of installing additional high strength fasteners through existing base flanges. Accompanying high strength concrete inserts and flange stiffeners should also be incorporated to develop full fastener tensile strength, and provide a factor of safety of 3.0 for overturning.

STRUCTURAL RACKS FOR BATTERIES AND ELECTRONICS (MW AND UPS)

Classification for Earthquake Design Adequacy

The classification categories for the two battery rack designs (one for UPS power, one for MW/DATS communication) and the electronics rack design (for MW and LFC communication subsystems) utilized at Dittmer are presented in

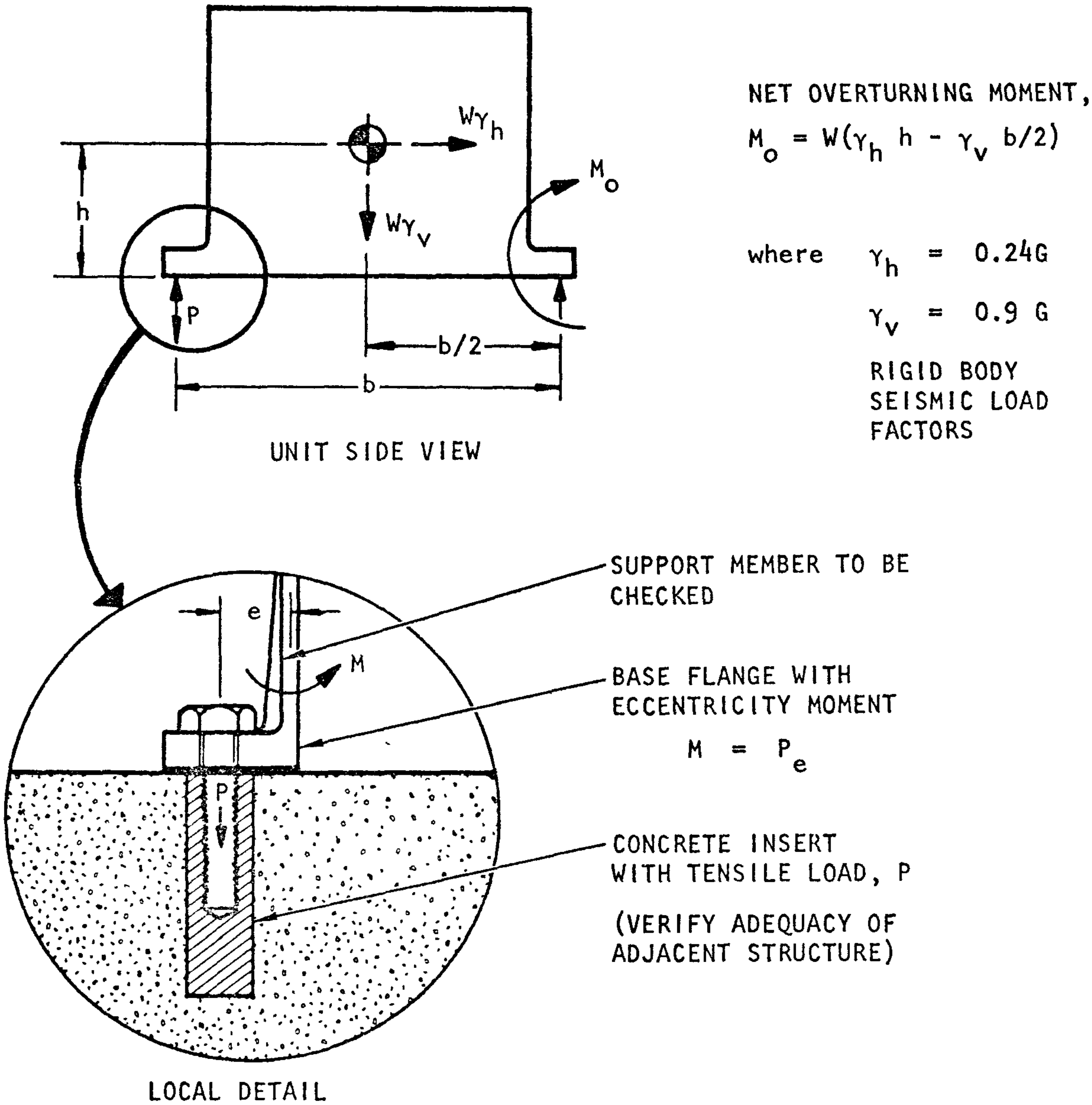


FIGURE 5-6. HEAVY UNIT BASE ATTACHMENT DESIGN

this section. The Uninterruptible Power Supply, UPS, subsystem for computer power is included in the Program Plan Exhibit 1 List. The Microwave, MW, and Load and Frequency Control, LFC, data acquisition subsystems are a vital part of the communication system and have been assigned to Priority I.

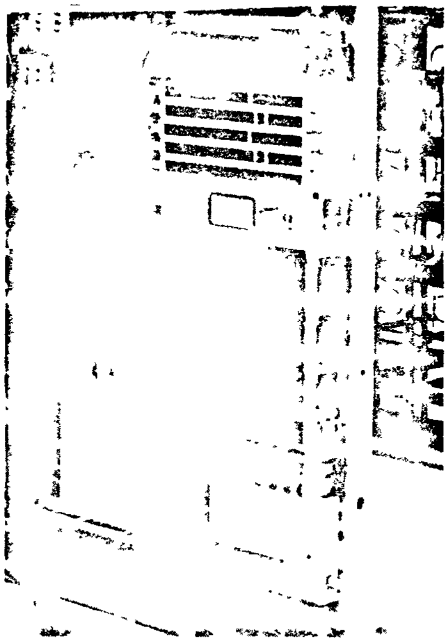
The UPS battery racks are structurally inadequate for the design earthquake condition, and have been classified in Category 2. The primary structural difficulty arises from the relatively long unbraced portions of the angle frames. Under lateral or longitudinal loading, large bending moments will be generated in the angle-section frame members, resulting in stresses above the material yield strength. Wall-mounting (as utilized for one of the three UPS battery racks) will substantially reduce the frame member bending stresses. However, member bending stresses will still be excessive, and additional frame bracing is required.

The MW (microwave subsystem) battery rack is designed with similar frame geometry to the UPS racks and produces equivalent member bending stresses. The rack has consequently been classified in Category 2.

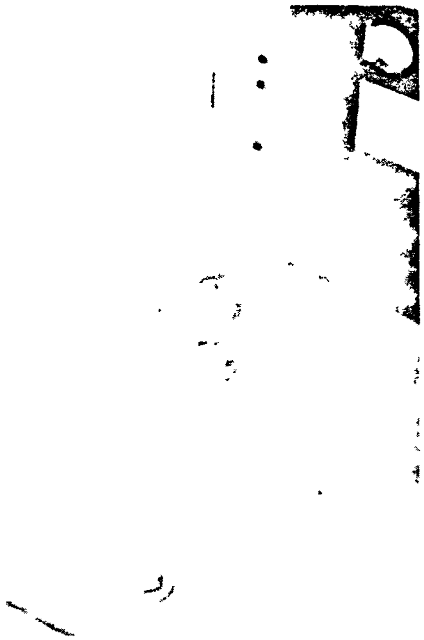
The racks supporting the electronics shelves for the LFC and MW communication subsystems, designated as "Radio Relay Racks," have been classified in Category 1 and require no redesign. The vertical members are relatively strong aluminum channels and are capable of withstanding the lateral earthquake response loading. Factors of safety are above 1.6 for bending stresses compared with material yield strength. Photographs of typical structural racks are shown in Figure 5-7.

Battery Rack Modifications

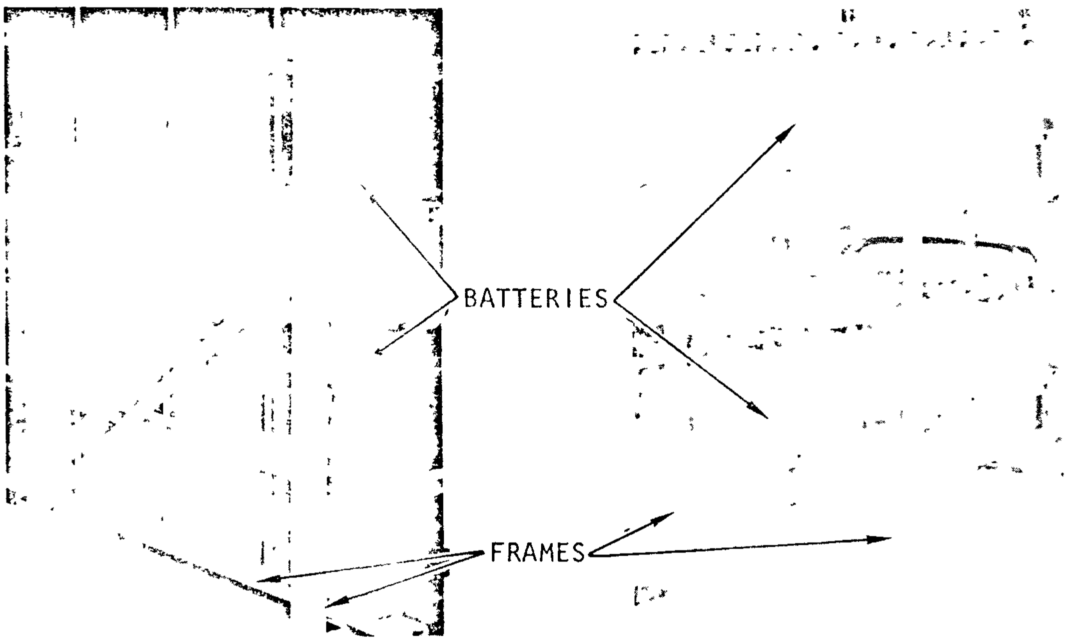
Recommended structural modifications are shown in Figure 5-8 for the UPS battery racks. Similar modifications will be required for the MW battery racks.



(a) MW ELECTRONICS RACK



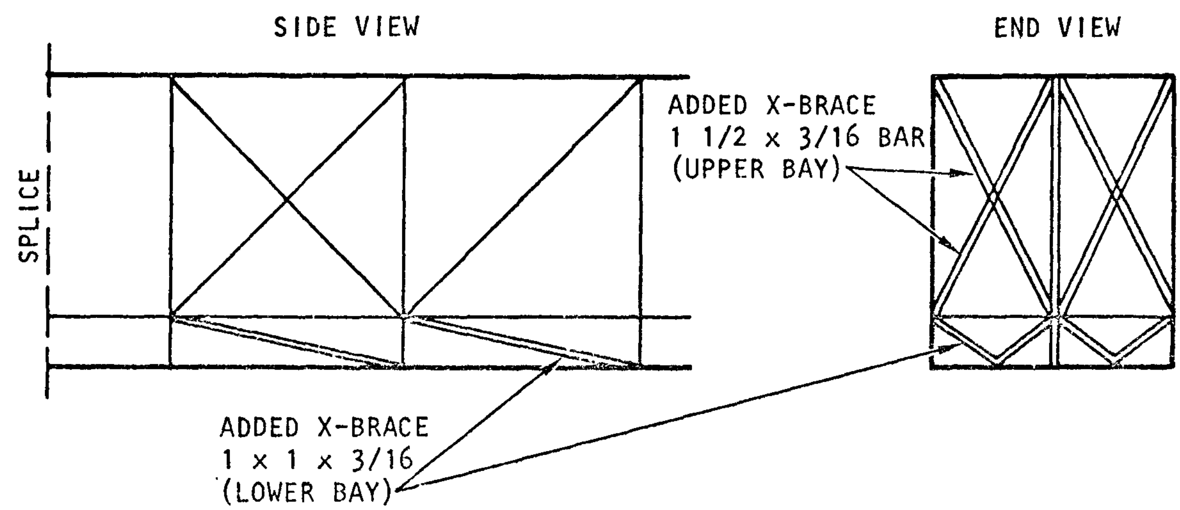
(b) BASE DETAIL OF RACK



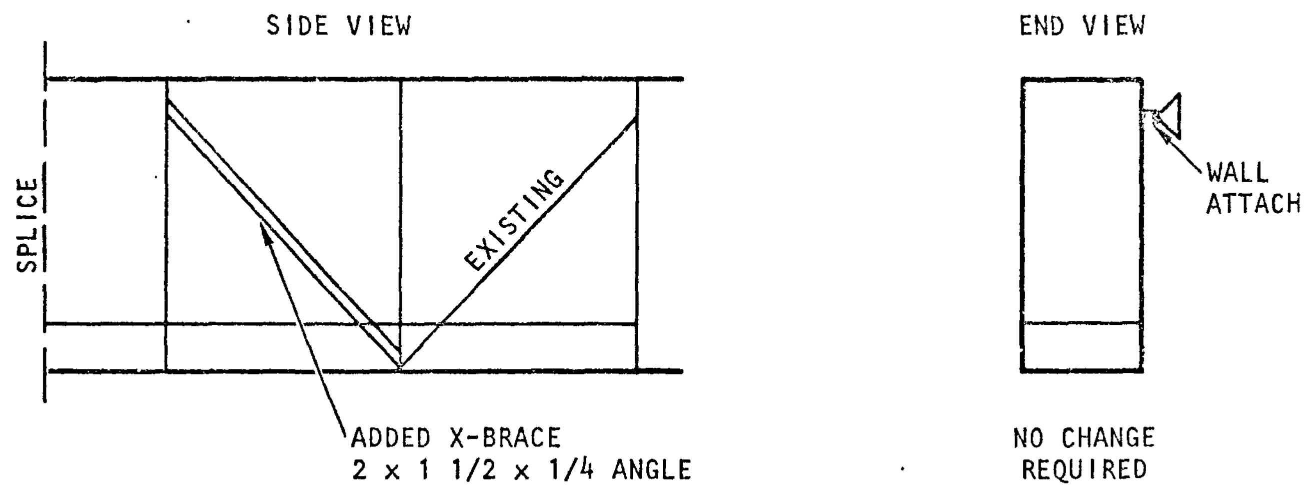
(c) WALL AND BASE-MOUNTED MW BATTERY RACK

(d) BASE-MOUNTED UPS BATTERY DOUBLE RACK

FIGURE 5-7. PHOTOGRAPHS OF STRUCTURAL RACKS FOR ELECTRONICS AND BATTERIES



(a) DOUBLE RACK, FREE STANDING



(b) SINGLE RACK, WALL MOUNTED

FIGURE 5-8. BATTERY RACK PROPOSED STRUCTURAL REDESIGNS

DISPLAY ROOM LIGHTING AND GROUP DISPLAY BOARD

The group display board and the overhead lighting in the operations display room are large flexible items which are included in Priority I and II, respectively, and could experience earthquake damage. However, it was not possible to completely evaluate these items within the time limit of the contract and these items have been placed in Category 3.

CONCRETE STRUCTURES--MICROWAVE TOWER, MAIN BUILDING, AND PRECAST PANELS

Classification for Earthquake Design Adequacy

The reinforced concrete structures at the Dittmer Control Center are susceptible to earthquake damage and corresponding risk to operating personnel. For this reason, these items were given a Priority 1 rating. Documentation of the classification of these structures follows:

Microwave Tower

The microwave tower is a reinforced concrete structure with a total height of 112 ft 4 in. The lower 49 ft 11 in. of the structure has 8 in. reinforced concrete shear walls on four sides and the upper 62 ft 5 in. of the structure has four 30 in. square columns spaced at 18 ft 6 in. on centers. There is no cross bracing between columns in the vertical plane. A platform and a massive hollow box girder weighing 245,000 lb are located at the top of the tower. There is a 72 in. I.D. precast concrete pipe at the center of the tower to house a spiral steel stair. The weight of the tower above the shear walls is 625,000 lb.

Estimated fundamental frequencies for the microwave tower are 1.0 cps lateral and 13 cps vertical. A damping factor of 5 percent of critical was used for estimating the response levels for concrete structures.

The tower was analyzed for the design earthquake response spectrum (0.24 g ground acceleration). A resulting lateral load of 0.4 g was applied with the tower dead load. The resulting moments in the tower columns were found to be considerably higher than their capacities. No further investigation was made of other components of the tower. The tower is susceptible to damage from a design earthquake and was, therefore, classified in Category 2. It should be structurally modified to meet the design level earthquake. Several feasible methods are available for the structural modifications. Further study of these methods is advisable prior to design.

Main Building Structure

This building is a 216 ft by 192 ft reinforced concrete building with two stories plus penthouse and basement. In the basement there are shear walls and typically 18 in. by 18 in. columns. At and above the first floor there are typically 18 in. by 18 in. interior columns, 18 in. by 48 in. exterior columns and some scattered shear walls. The estimated horizontal frequency of the building is between 5 and 10 cps. From the criteria spectrum for 5 percent critical damping, the horizontal acceleration would be above 0.5 g. The building was originally designed for the Uniform Building Code (UBC) Zone 2, i.e., horizontal acceleration of 0.04 g. A check of the second floor columns where there are essentially no shear walls, indicates the columns have inadequate bending resistance for the criteria earthquake motion. The building was therefore classified in Category 2. Modifications are required such as additional shear walls or lateral bracing and reinforcing of some structural elements.

Precast Concrete Panels

Precase concrete panels are located at stairs, over the lobbies, and as exterior walls. The panels are of various sizes, shapes, and weights and were originally designed for UBC Seismic Zone 2.

The estimated horizontal frequency of the building is between 5 and 10 cps. From the design spectra for 5 percent critical damping, the horizontal acceleration would be above 0.5 g. However, the UBC, 1970 edition, is more restrictive regarding connections of exterior panels. It requires that connections for exterior panels be designed for twice the horizontal acceleration predicted. Because of the difficulty in estimating the stress concentrations that develop in the connections, the higher accelerations are in our opinion, justified. Therefore, a value of 1.0 g was used for checking connections of exterior panels. On this basis the factors of safety is considerably less than required and the panels were placed in Category 2. It is recommended that the precast panel attachments be modified to withstand higher force levels with adequate safety.

Calculations for the concrete structures are presented in Reference 5-5.

REFERENCES

5-1. Agbabian Associates, *Program Plan for Increasing the Resistance of Electrical Facilities*, AA Report No. R-7211-2243, March 15, 1972.

5-2. Bonneville Power Administration, *Summary of Dittmer Subsystems*, June 29, 1972.

5-3. Los Angeles Department of Water and Power, *San Fernando Earthquake of February 9, 1971*, October 1971.

5-4. Agbabian Associates, "BPA/Dittmer Control Center Earthquake Design Analysis of Computer Cabinets and Display Consoles," AA Memorandum No. 7211-6-2525, October 17, 1972.

5-5. Agbabian Associates, "Reference Calculation Sheets for Dittmer Concrete Structure Seismic Analysis," AA Memorandum No. 7211-6-2583, November 29, 1972.

CHAPTER 6

IMPLEMENTATION PLAN FOR INCREASING EARTHQUAKE RESISTANCE LEVEL OF ELECTRICAL SUBSTATIONS AND SPECIAL INSTALLATIONS

INTRODUCTION

This chapter has two objectives. The first is to present a procedure for developing an effective implementation plan which establishes the sequence, or priority, of providing modifications to items of electrical equipment and support structures in the Bonneville Power Administration's (BPA) electrical substations. The modifications are those required to make the substations progressively more resistant to strong earthquake ground motion. The second objective is to provide a priority plan for implementing the modifications described in Chapters 4 and 5 to the special installations, the Celilo Converter Station, and the Dittmer Control Center, respectively. These installations require a high degree of operational reliability and represent large capital investments. The two objectives will be considered separately.

PROCEDURE FOR DEVELOPING AN IMPLEMENTATION PLAN FOR SUBSTATION MODIFICATIONS

The essential steps to be followed in developing the implementation plan for substation modifications are illustrated in the flow chart in Figure 6-1. The procedure requires the collection of three separate categories of information which are operational priorities, earthquake ground motion criteria and design modification identification and costs. The objective is to provide an economical justification of the modifications prior to establishing a schedule and budget. The items given the highest priority for modification should be those which have the highest operational priority, as well as a favorable benefit to cost ratio.

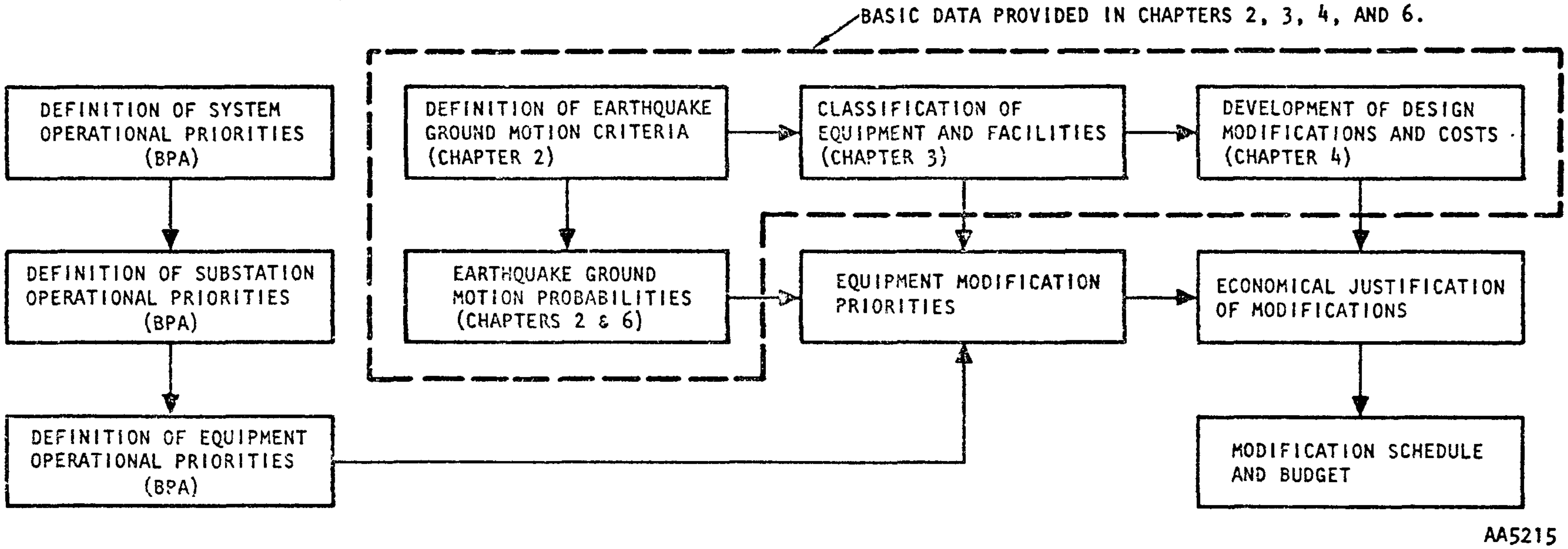


FIGURE 6-1. FLOW CHART OF IMPLEMENTATION PLAN FOR SCHEDULING MODIFICATIONS TO INCREASE EARTHQUAKE RESISTANCE LEVEL OF SUBSTATION EQUIPMENT AND SUPPORT STRUCTURES

RC7211C3-2859

Earthquake ground motion criteria for the development of the implementation plan is provided in Chapter 2. Criteria for identifying the items that require modification as well as modification concepts are provided in Chapters 3 and 4. Data on operational priorities and design modification costs are to be provided by BPA. However, in order to demonstrate the procedure, operational priorities and costs will be assumed in the discussion that follows.

OPERATIONAL PRIORITIES

The fact that substation equipment could suffer damage from strong earthquake ground motion in the BPA service area has been established by the procedures reported in Chapters 3 and 4. Also, specific types of equipment that would suffer damage and require modification are also identified. It is now necessary to establish the relative operational importance of the individual items of equipment in the various substations. This requires a careful systematic study of the overall system. The first step is to establish the relative operational importance of major elements, or segments, of the transmission system. The relative operational importance of the individual substations associated with each of the major segments of the system can then be identified and finally the relative operational importance of the various items of equipment in each substation can be determined.

The procedure can be best demonstrated with a hypothetical example based on assumed priorities. As an example, assume that after a careful study it is established that the 500 KV transmission network shown in Figure 6-2 is the most important major element of the BPA system. This is an assumption made to provide a simple example. Parts of the 345 KV and 230 KV system may be of more operational importance than certain segments of the 500 KV system and this should be considered in an actual study. Next the relative operational importance of different segments of this major element of the system must be defined. Assume that an analysis of the example problem leads to the priorities shown in Figure 6-2. The segments with the ten highest priorities are listed in Table 6-1:

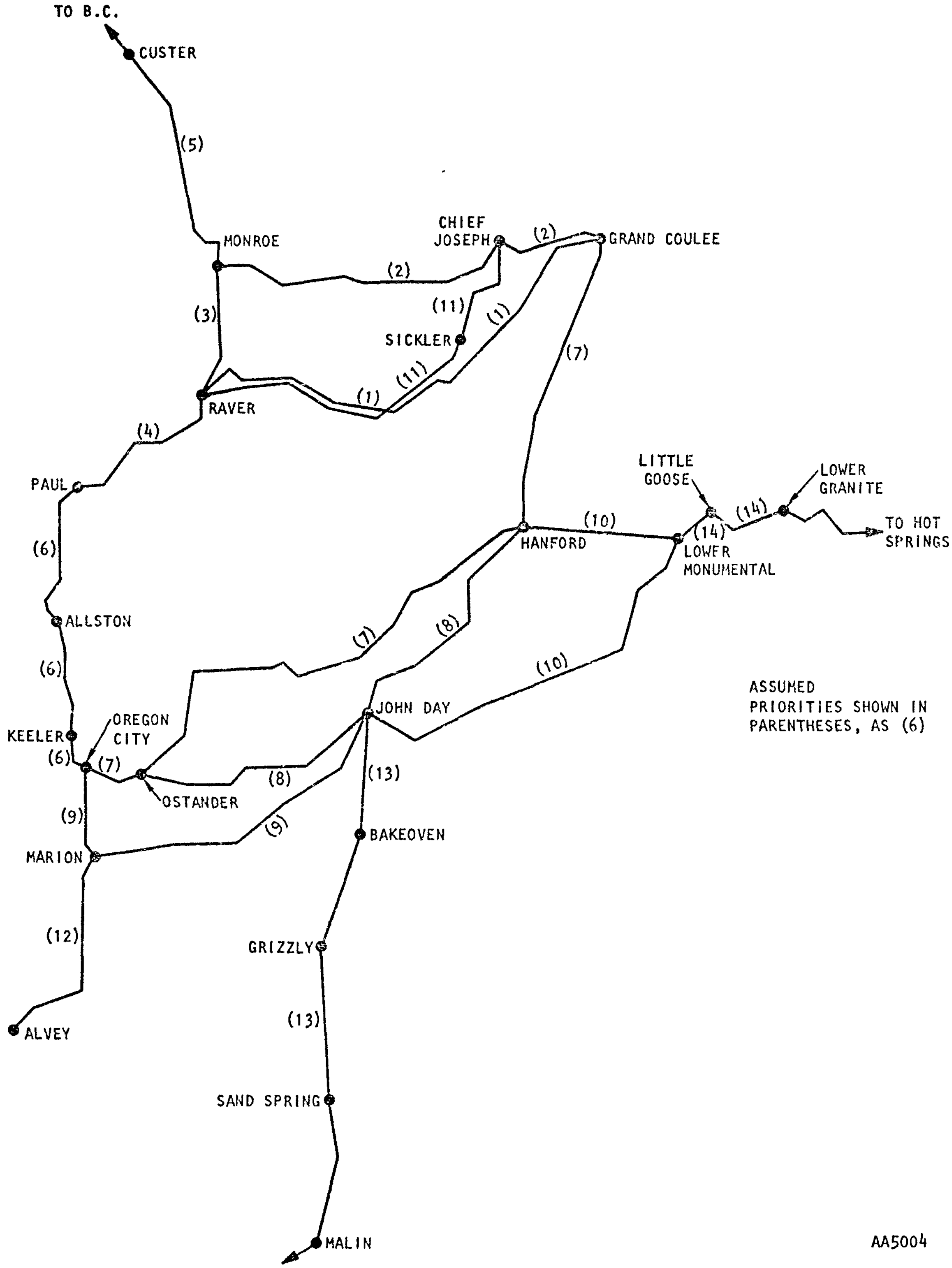


FIGURE 6-2. ASSUMED SYSTEM OPERATIONAL PRIORITIES FOR DIFFERENT SEGMENTS OF 500KV TRANSMISSION SYSTEM

TABLE 6-1. ASSUMED OPERATIONAL PRIORITIES OF DIFFERENT SEGMENTS OF 500 KV BPA TRANSMISSION SYSTEM

Priority	Segment of 500 KV System
1	Grand Coulee - Raver
2	Grand Coulee- Chief Joseph - Monroe
3	Raver - Monroe
4	Raver - Paul
5	Monroe - Custer
6	Paul - Oregon City
7	Grand Coulee - Hanford - Oregon City
8	Ostrander - Hanford
9	Oregon City - Marion - John Day
10	Hanford - Lower Monumental - John Day

Again this analysis should be as systematic and as quantitative as possible. Factors that might be considered are the total annual power transmitted on each segment, relative importance of different power loads, and the redundancy in transmission capability provided by different segments.

After the segmental priorities have been established, it is next necessary to assign operational priorities to each substation associated with the various segments of the system. The same factors noted above should be applicable to this analysis. Assume that the substation analysis leads to the list given in Table 6-2 for the substations having the highest fifteen priorities:

TABLE 6-2. ASSUMED OPERATIONAL PRIORITIES FOR 500 KV SUBSTATIONS

Priorities	Substation
1	Raver
2	Monroe
3	Grand Coulee
4	Chief Joseph
5	John Day
6	Hanford
7	Oregon City
8	Ostrander
9	Marion
10	Lower Monumental
11	Paul
12	Allston
13	Keeler
14	Custer
15	Sickler

The next step requires that operational priorities be assigned to the different items of equipment in each substation. Frequently, this will require considering the relative importance of different operational functions within the substation. For example, at the Monroe substation there is switching equipment for both 500 KV and 230 KV power transmission. In the hypothetical example only the 500 KV transmission system is being considered so the problem is relatively simple. It is important that this analysis be based strictly on operational considerations at this point. Air power circuit breakers (APCB's) are much more expensive than lightning arresters, but the relative cost should not influence the operational priority. Costs will be introduced later, as will the relative fragility levels of the various items of equipment. Safety hazards and potential damage resulting to other equipment would be factors that should be considered, however. Assume that this study leads to the equipment operational priorities for the 500 KV substations given in Table 6-3.

TABLE 6-3. ASSUMED EQUIPMENT OPERATIONAL PRIORITIES FOR 500 KV SUBSTATIONS

Priority	Equipment Item
1	APCB/CT
2	Lightning Arresters
3	Power Transformers
4	Capacitor Racks
5	Dead End Towers
6	Disconnect Switches
7	Bus Supports

At this point the essential steps in the implementation plan given in the left column of Figure 6-1 have been completed for the example problem.

CLASSIFICATION OF EQUIPMENT AND SUPPORT STRUCTURES

Referring to Figure 6-1, it will be noted that in addition to equipment operational priorities, equipment classification criteria and ground motion probability data are needed to establish the equipment modification priorities. Equipment classification criteria has been provided in Chapter 3 and earthquake ground motion for classification purposes is provided in Chapter 2. For the implementation plan, equipment classification studies are required of the equipment in the individual substations to ascertain which items require modification. Classification studies have been performed for specified items of equipment in three different 500 KV substations and the results are provided in Chapter 3. One is located in seismic Zone A (John Day), one is located in seismic Zone B (Allston), and one is located in Zone C (Custer).

While this is not an adequate coverage of all items of equipment in all seismic zones, the results of the classification studies for these substations will be used for the example problem on the assumption that the results are indicative of what would be obtained by a more comprehensive study of the substations listed in Table 6-2. The assumed results for the seven items of equipment listed in Table 6-3 are given below in Table 6-4.

TABLE 6-4. ASSUMED CLASSIFICATION OF SUBSTATION EQUIPMENT AND SUPPORT STRUCTURES ASSOCIATED WITH 500 KV SUBSTATIONS

Item	Classification* Seismic Risk Zone A	B	C
APCB/CT	2	2	2
Lightning Arresters	1	2	2
Power Transformers	2	2	2
Capacitor Racks	2	2	2
Dead End Towers	1	1	1
Disconnect Switches	1	2	2
Bus Supports	1	1	1

*Classification refers to Categories 1 and 2
1 = No modifications required
2 = Modifications required

Referring to Tables 6-3 and 6-4, it is apparent that Dead End Towers and Bus Supports in the 500 KV yards of the substations in all zones are adequate and will not be damaged by the expected strong earthquake ground motion in the BPA service area, (Note this is an assumption based on analyses at three substations.) Thus, for these items no modifications are needed and these items drop out and will not appear in the schedule and budget. The same statement applies to disconnect switches in Zone A, but not in Zones B and C. All other items would be damaged in all zones if not modified. Additional steps must, therefore, be taken before an equipment modification priority can be established for these items. These steps are a consideration of the different probabilities of strong earthquake ground motion occurring at the various substations, and a consideration of the different damage levels of the various items of equipment. Basic data for the former is provided in Chapter 2, and for the latter in Chapter 4. The application of the probability data, requires some additional assumptions which follow.

EARTHQUAKE GROUND MOTION PROBABILITIES

The probability of experiencing strong earthquake ground motion of different intensity during different recurrence intervals has been presented in Chapter 2 for the Puget Sound and Portland regions. Different sized areas have been considered in each case to provide some indication of areal effects. It is now necessary to extend this information to provide reasonable estimates of probabilities for the three seismic zones in which the substations of the example problem are distributed. As explained in Chapter 2, the basic assumptions upon which a probability analysis is based, are not completely satisfied by seismic phenomenology and considerable judgement and care must be exercised in using information of the type presented in Figures 2-26 through 2-29. Fortunately, great precision in probability prediction is not needed in developing the implementation plan. This will be evident from the discussion that follows.

The discussion in Chapter 2 indicates that the probability curves in Figure 2-26, which are based on an area of 21,000 square miles, are a better indication of the expected probabilities in the Puget Sound Zone C than the curves in Figure 2-27 which are based on a larger area. The curves in Figure 2-27 are likewise more applicable to Zone B than Zone C. Also, the curves in Figure 2-29 for the Portland region are more applicable to the Zone B area south of the 46° latitude than the curves in Figure 2-28. Therefore, for the example problem it will be assumed that the curves in Figure 2-26 are applicable to Zone C, and that a geometric average of the curves in Figure 2-27 and 2-29 are applicable to Zone B. This latter assumption is made because these final two sets of curves are quite similar. On this basis, the Zone B and C probability curves for a 40 year recurrence interval have been prepared in Figure 6-3. The 40 year recurrence interval has been selected as it is assumed that the useful life of the equipment to be studied is about 40 years. A curve crossing these two curves is also shown in Figure 6-3 which was taken from Figure 2-28. This curve signifies that the probability of strong motion in the Portland epicentral area is different than the average for Zone B. This is a realistic condition that should not be overlooked although it is not too significant in this example. The scope of the contract did not permit an analysis of sufficient depth to provide a reliable estimate of the probability of strong ground motion in Zone A. However, a curve based on judgement has been postulated for the example problem and is shown in Figure 6-3. These four curves will now be used to demonstrate the procedure of establishing equipment modification priorities for APCB's since these items have been indicated to have the highest operational priority in Table 6-3.

EQUIPMENT MODIFICATION PRIORITIES

Referring to Figure 6-1, it is evident that substation operational priorities, seismic zone earthquake ground motion probabilities and damage levels for the equipment are required to establish the modification priority. The substation priorities are provided in Table 6-2, the ground motion probabilities are given in Figure 6-3 and the damage level of the equipment can be developed from the information in Table 4-1 of Chapter 4. Table 4-1 indicates that the allowable response acceleration (expressed as a static load) for the 500 KV GE and Merlin Gerlin APCB's varies from 0.15 g to 0.20 g. For the damping inherent in this equipment, and the equipment natural frequencies,

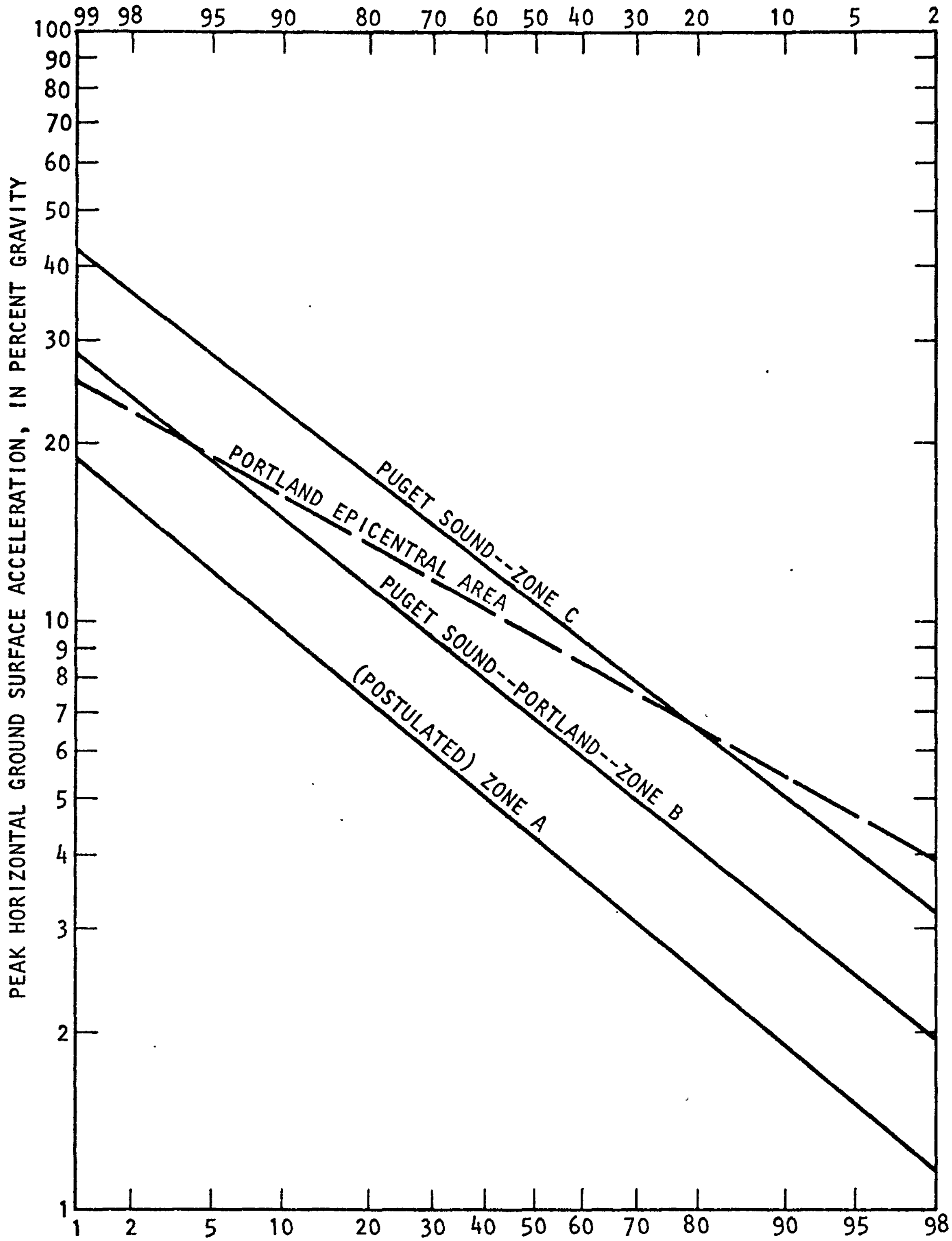


FIGURE 6-3. ASSUMED PERCENT PROBABILITIES THAT A GIVEN SUBSTATION IN A GIVEN SEISMIC ZONE WILL EXPERIENCE INDICATED PEAK ACCELERATION IN 40 YEAR RECURRENCE INTERVAL

this is equivalent to a peak horizontal ground surface acceleration of about 0,04 to 0,05 g. Since the allowable static load was based on a factor of safety of three, definite damage should result at peak horizontal ground surface accelerations of about 0,12 to 0,15 g with some damage possibly resulting for values as low as 0,07 g. Since the probability curves are nearly parallel for the three seismic zones, the difference in answer between 0,07 and 0,12 g is slight but the latter will be used for the example.

Entering Figure 6-3 with a damage level of 0,12 g, the probability of experiencing this value of peak horizontal ground surface acceleration in the different seismic zones are found to be the following: Zone A = 5 percent, Zone B = 18 percent, Zone C = 42 percent and in the Portland epicentral area the probability is 23 percent. These values can now be normalized by the probability value for Zone A. The following ratios are obtained: Zone A = 1,0; Zone B = 3,6; Zone C = 8,4; and Portland epicentral area = 4.6. This indicates that earthquake damage to 500 KV APCB's is 8.4 times more probable in Zone C than Zone A, 3,6 times more probable in Zone B than Zone A, and 4.6 times more probable in the Portland epicentral area than in Zone A. At this point it is obvious that the APCB modification priority will ordinarily first be determined by seismic zones and then by operational priority because of the very large probability ratios. On this basis the equipment substation modification priorities for 500 KV APCB's should be that indicated in Table 6-5.

TABLE 6-5. 500 KV APCB/CT MODIFICATION PRIORITY

Priority	Seismic Zone	Substation
1	C	Raver
2	C	Monroe
3	C	Paul
4	C	Custer
5	B-(Portland)	Allston
6	B-(Portland)	Keeler
7	B	Oregon City
8	B	Ostrander
9	C	Grand Coulee
10	C	Chief Joseph
11	C	John Day
12	C	Hanford
13	C	Marion
14	C	Lower Monumental
15	C	Sickler

The same procedure should now be followed for each of the remaining items of equipment listed in Table 6-3 except for the Dead End Towers and Bus Supports which do not require modification.

ECONOMICAL JUSTIFICATION OF MODIFICATION

At this point it is possible to determine whether it is economical to provide modifications to the 500 KV APCB's in the example problem, and to determine the benefit to cost ratio. The analysis requires an estimate of the probable cost of damage during the useful life of the equipment if modifications are not made, and the capitalized cost of the modifications for the same period. The cost of damage in this case is the benefit that would result from the modification investment. It is not an easy analysis and requires discussion.

The potential damage should include the cost of resulting equipment damage and replacement, and the cost to BPA of the lost revenue during the power outage as well as the cost of any liability that results from the power outage. Actually, the lost revenue and other power outage costs should be distributed to all of the equipment in the substation that requires modification. Since the costs of new equipment, lost revenues and other liabilities incurred will ultimately be determined by BPA, only assumed values will be used in the example problem.

The probability of the damaging ground motion occurring is a significant factor in the analysis. This can be most easily demonstrated by assuming that there are 100 substations in each seismic zone. Then based upon the probabilities listed above for a 0.12 g peak horizontal ground surface acceleration, 42 substations should be severely damaged in Zone C in a 40 year period, 18 in Zone B and 5 in Zone A. Since the relationship is linear and there are only four substations being considered in Zone C, in the example problem, the cost of damage to 4 x 0.42, or 1.68 substations, must be considered in the analysis of the cost of substation damage in Zone C. This cost must be divided by the capitalized cost of the modifications to <u>all four</u> substations. Considering the Raver substation, twelve, 500 KV APCB units are installed or planned for the near future. Assume that the replacement cost is $300,000 per unit. On this basis the APCB equipment damage at this station would be $3,600,000. Assume that the value of the lost revenue and the power outage liability is an additional $6,400,000, or the total damage averted by the modifications would be $10,000,000. The modification cost per unit has been estimated in Chapter 4 to be $15,000. This amount capitalized at 6 percent over a 40 year period gives an amount of $1,850,000. If we now assume, for simplicity, that the damage at Raver is typical of the damage occurring at all four stations in Zone C, the benefit to cost ratio for the 500 KV APCB's in Zone C is,

$$\text{Benefit to Cost Ratio} = \frac{10{,}000{,}000 \times 0.42 \times 4}{1{,}850{,}000 \times 4} = 2.28$$

In seismic Zone B the probability of damage in a forty year period is only 18 percent. If the damage to the four stations in Zone B are the same as above, the benefit to cost ratio would be roughly unity. On this basis, it would be marginally profitable to provide modifications in Zone B, and not profitable in Zone C. It should be noted that the above comparison is an example and actual damage costs at this time are unknown. Also, the actual number of stations in each zone must be considered and the estimated cost of damage and cost of modification to each station computed.

BUDGET AND SCHEDULE

After the benefit to cost analysis has been completed for all items of equipment in all seismic zones, a schedule based on the funds available annually for modifications can then be prepared. The schedule priorities can be established either on the basis of the most favorable benefit to cost ratio, or on the basis of operational priority of equipment and of substation for all items having a favorable ratio. The latter would normally be preferable.

SPECIFICATIONS FOR NEW EQUIPMENT

An important step in increasing the earthquake resistance level of the BPA substations and special installations would be to utilize a seismic specification for the procurement of all new equipment, and in the design of all new support structures which adequately provides for the seismic risk defined in Chapter 2. Since some items of equipment may be moved from one seismic zone to another, the specification for these items must provide for the strongest motion in the zones in which the equipment may be located. Other items, particularly structures, will be fixed in location and can logically and economically be designed for the seismic risk of that particular zone and site condition.

To assist in accomplishing this objective, a careful review was made of seismic specifications currently used by electrical utilities in California and a recommended specification prepared which can be keyed to the ground motion criteria defined in Chapter 2. This specification is provided in Appendix C-1. A brief review of the philosophy followed in preparing this specification is essential.

There are two identifiable philosophies in writing specifications, although many specifications confuse both philosophies. One philosophy is that the specification should be simple, brief and state only the requirements of the finished product. Under this philosophy the specification clearly describes the requirements of the finished product but does not tell the vendor how to perform his task. If the specification is restricted to statements of requirements, there can be little doubt as to who has not performed adequately if the final product does not meet the specification performance requirements. The problem is not really this simple. Usually, a simple specification is provided first, but a satisfactory product does not result. The buyer, or specification writer, then expands the specifications in an attempt to lead the vendor to the correct solution. This leads to the second philosophy, which is for the buyer to provide a very comprehensive specification that covers all details. There are two dangers in this type of specification. First, the specification can become quite rigid and inflexible, and an expensive product will result. Second, the buyer may find that he has assumed much of the responsibility for the performance of the finished product. For example, if a vendor is told how to analyze electrical equipment for seismic effects, and how to test it, and the instructions are followed but the equipment fails when a performance test is executed on the finished product by the buyer, is it not possible that these multiple requirements are not all consistent, or that something important has been overlooked?

In preparing the specification provided in Appendix C-1, an attempt has been made to follow the first philosophy and provide a simple performance, or requirement, type specification. Testing requirements have been reduced to a minimum and proof tests are not specified. If proof, or performance, tests are desired, it is recommended that these be provided under a separate contract funded jointly by several electrical utilities. It has been tacitly assumed in preparing this specification that the vendors will utilize qualified dynamists and experimentalists to support the analysis and design of the equipment, and that the buyer will use technically qualified personnel to prepare the specification and to review the analytical and experimental data submitted by the vendor. On this basis, detailed directions on how to make the analyses and perform the experimental tests to provide equipment to meet the requirements of the specifications are superfluous.

The seismic environment described in the specifications provided in Appendix C-1 is based on the envelope spectra for Zone C. This is the strongest seismic environment that would normally be specified for BPA equipment and facilities as currently there are no BPA substations or special installations in Zone D. Equipment and support structures designed in accordance with these specifications could be used throughout the present BPA service area. Fixed equipment and support structures that will be restricted to Zones A or B may economically and reliably utilize reduced response spectra and peak ground surface accelerations defined in Chapter 2 for the respective zones and site conditions.

EQUIPMENT MODIFICATION PRIORITIES FOR CELILO CONVERTER STATION AND JOHN DAY AND BIG EDDY SUBSTATIONS

The Celilo Converter Station and these two substations are special installations which represent a large capital investment and require a high degree of operational reliability. The design modifications required to raise the earthquake resistance of essential equipment and support structures at these stations to an acceptable level were determined by a special study which is reported in Chapters 3 and 4. Equipment modification priorities have been prepared for these installations and are summarized in Table 6-6. The priorities have been established independent of a system priority analysis and an economical justification. It is assumed that these types of analyses will be provided in developing an implementation plan when factual data on losses from power outages resulting from loss of station, and modification costs become available. Otherwise, modifications may be provided that are not economically justified.

TABLE 6-6. EQUIPMENT MODIFICATION PRIORITIES FOR CELILO CONVERTER STATION AND JOHN DAY AND BIG EDDY SUBSTATIONS

Priority	Equipment Item	Location	Damage Potential	Functional Importance	Safety Hazard	Proximity Damage Potential	Cost of Replacement	Cost of Modification	Total Rating
1	Current Dividers	Bridge Room--Celilo	4	5	4	5	5	3	26
	500 kv Air Power Circuit Breakers 500 kv Merlin-Gerlin 500 kv GE 500 kv Delle-Alsthom	John Day	5	5	2	4	5	5	26
	230 kv Air Power Circuit Breakers 230 kv GE 230 kv Allis-Chalmers	Big Eddy	5	5	2	3	5	5	25
2	Stone Work	Converter Station Building--Celilo	5	1	5	1	5	5	22
	AC Lightning Arresters 900 bil 1300 bil	Valve Camping Yard--Celilo	5	4	1	4	5	2	21
	DC Lightning Arresters 900 bil 1300 bil	Valve Damping Yard-Celilo	5	4	1	4	5	2	21
	DC SR Lightning Arrester	DC Yard-Celilo	5	4	1	3	5	2	20
	DC Line Arresters	DC Yard-Celilo	5	4	1	3	5	2	20
3	6th Harmonic Filter Capacitors	DC Yard-Celilo	5	4	1	2	3	2	17
	20th Harmonic Filter Capacitors	DC Yard-Celilo	5	4	1	2	3	2	17
4	5th and 7th Harmonic Filter Capacitors	Big Eddy	4	3	1	2	3	2	15
	11th and 13th Harmonic Filter Capacitors	AC Yard-Celilo	4	3	1	2	3	2	15
	HP Filter Capacitors	AC Yard-Celilo	4	3	1	2	3	2	15
	PF Capacitors	AC Yard-Celilo	4	3	1	2	3	2	15
5	Valve Damping Capacitors	Valve Damping Yard--Celilo	3	2	1	4	3	1	14
	HP Filter Resistors	AC Yard-Celilo	4	2	1	1	3	2	13
	Voltage Dividers	DC Yard-Celilo	4	2	1	1	3	1	12
6	5th and 7th Harmonic Filter Reactors	Big Eddy	3	2	1	1	2	2	11
	11th and 13th Harmonic Filter Reactors	AC Yard-Celilo	2	2	1	1	2	2	10
	HP Filter Reactors	AC Yard-Celilo	2	2	1	1	2	2	10
	Spare Equipment Storage 5th and 7th Filter reactors	Big Eddy	3	1	1	1	2	2	10
	Spare Equipment Storage HP Filter Reactors 11th and 13th Filter Reactors Degassing Transformers Degassing Reactors 1300 bil Transducers Voltage Dividers Isolation Transformers Lightning Arresters (parts) DC Reactors Converter Transformers Oil Tanks	Celilo	3	1	1	1	1	2	10
7	Converter Transformers	Valve Damping Yard-Celilo	1	2	1	1	3	1	9
	DC Smoothing Reactors	DC Yard-Celilo	1	2	1	1	3	1	9
	Power Circuit Breakers (230 kv)	Valve Damping Yard-Celilo	1	2	1	1	3	1	9
	Power Circuit Breakers (HSF6)	AC Yard-Celilo	1	2	1	1	3	1	9
	Oil Conservator	Degassing Transformer Yard--Celilo	1	1	1	2	1	1	7

AA5222

The modification priorities have been based upon a rating system which considered the following factors:

Damage Potential

Functional Importance

Safety Hazard

Proximity Damage Potential (damage to adjacent equipment)

Cost of Replacement

Cost of Modification

A subjective severity rating on a scale from 1 to 5 was assigned to each factor for each item of equipment to be modified. A rating of 1 implies a low severity rating while 5 implies a high severity rating. The items of equipment were then ordered according to the accumulated sum of the ratings of the categories for each equipment item. Seven modification priority groups of equipment were then established as indicated in Table 6-6.

EQUIPMENT AND STRUCTURE MODIFICATION PRIORITIES FOR DITTMER CONTROL CENTER

The Dittmer Control Center is also a special installation that represents a large capital investment and requires a high operational reliability. A special study of this installation was, therefore, provided and is reported in Chapter 5. Because of contract limitations in scope, only a preliminary seismic classification could be accomplished on some equipment and structural items considered essential to operations. Final classification of these items is an essential requirement before an implementation plan can be properly formulated for the modifications needed to increase the earthquake resistance level of this installation. Equipment modification priorities have been established on a subjective basis and are listed in Table 6-7. An economical justification of the modification is also required before the implementation plan can be developed.

TABLE 6-7. EQUIPMENT AND STRUCTURE MODIFICATION PRIORITIES FOR DITTMER CONTROL CENTER

Modification Priority	Item	Operational Priority	Additional Analysis Required	Modification Concepts Required
	Structures			
1	Precast Panels-Main Building	I		
2	Microwave Tower	I	X	X
3	Main Building	I	X	X
	Equipment			
4	Battery Racks, MW & UPS	I		
5	Group Display Boards	I	X	X
6	Computer Cabinets, Group A	II		
7	Display Room Lighting	II	X	X
8	Uninterruptible Power	II		X
9	Basement Air Conditioning	II	X	X
10	Basement Building Power	II		X
11	Emergency Generator	III		X
12	Penthouse Air Conditioning	III	X	X

APPENDIX A-1

CURRENT DIVIDER AND ANODE REACTOR SEISMIC ANALYSIS

APPENDIX A-1

CURRENT DIVIDER AND ANODE REACTOR SEISMIC ANALYSIS

by K. L. Merz

INTRODUCTION

The current dividers and anode reactors are suspended from the valve hall roof at the Celilo converter station with insulator hangers to form a three-dimensional pendular system which can experience large displacements during seismic excitation. During the 1971 San Fernando earthquake, the same type of suspension system failed on all 42 current dividers at the Sylmar converter station. The damage caused by the falling equipment resulted in the highest cost and longest downtime for repairs of all the equipment damaged by the earthquake. Assurance of the satisfactory performance of this suspension system is, therefore, a very important part of the design modification study and considerable effort has been expended on the analysis of this system. This memorandum summarizes the results of this study which consists of analyses using the three following mathematical models:

1. A modification and expansion of the linear model used in the initial Assessment Study.

2. A two-dimensional nonlinear model of the suspension system to test the nonlinearities of the system.

3. A three-dimensional nonlinear model of the suspension system used for the final analysis.

A brief review of each of the analyses is provided below.

MODIFIED LINEAR MODEL

An analysis was provided of the current divider and anode reactor support system in the initial Assessment Study which was based upon a linear elastic model. This required certain simplifying assumptions which were provided because of time limitations. Also, between the time of completion of this study and the initiation of the design modification study, experimental field data became available on the building and suspension system response which could be used to improve the reliability of this model. Therefore, as a first step, a review was made of this model for the purpose of modifying and expanding it. Before considering the modifications, a brief review of the initial model and analysis procedure is essential.

The initial analysis of the support system for the current dividers and anode reactors provided in the Assessment Study was based upon the assumption that the suspension insulator hangers had been replaced with post insulators which could resist both tensile and compressive loads. In the language of the analyst, this assumption permitted the development of a three-dimensional linear elastic mathematical model of the structure and support system which is described on pages 219-221 of the Assessment Study report. In this model, one entire bay of the building was modeled along with the current divider. Due to the size of the problem, simplifying assumptions were made in modeling the roof and hanger geometry. The results of this initial analysis was used in the Assessment Study to point out that both tensile and compressive forces could result from the earthquake motions, and since the hangers in reality could not resist compression but would go slack, the current dividers could impact on the hangers causing an increase in the force over that computed. By assuming an impact factor to be a value of 2, it was shown that the present hangers could be overstressed and modification was recommended.

As a result of the review of the model used in the Assessment Study, it was decided that this model should be separated into two models. These were the following:

1. A model of the building without the current divider.

2. A model which included a roof segment (1 bay) and a current divider and anode reactor.

This separation was possible due to the low frequency of the pendular system compared to the building frequency (i.e., the effective horizontal isolation of the current divider). The partition of the model into two separate models made it possible to expand the model of the roof segment, see Figure A-1, to provide a more precise definition of the hanger geometry. This was important since small changes in hanger geometry could cause large changes in computed force in the hangers.

Following the San Fernando earthquake shock on February 9, 1971, an accelerograph was installed on the roof of the valve hall at Sylmar and the actual fundamental frequency of the building was determined during an after-shock. The frequencies of the current divider and anode support system were also determined by tests conducted by the Los Angeles Department of Water and Power. The stiffness of the model of the building was modified from that initially used to provide the measured fundamental frequency of the building. The second model, the expanded model of the roof and current divider and anode support system, was also checked to make certain that the frequencies of the support system were in agreement with test results. The modified building model was then analyzed for a new earthquake ground motion consistent with the new response spectra developed for the modification studies (Ref. AA Memo 7211-2298 and AA Memo. 7211-2353). Three components of the Helena 1935 earthquake ground motion were used as input in contrast to the C-1 artificial record used in the Assessment Study analysis. The latter record, since all three components were in phase, probably gave a conservatively stronger response. The computed lateral response of the building roof was then used as input motion to the expanded roof/current divider model. The vertical input motion for the model was assumed to be the ground vertical motion (i.e., the building acted as a filter for the

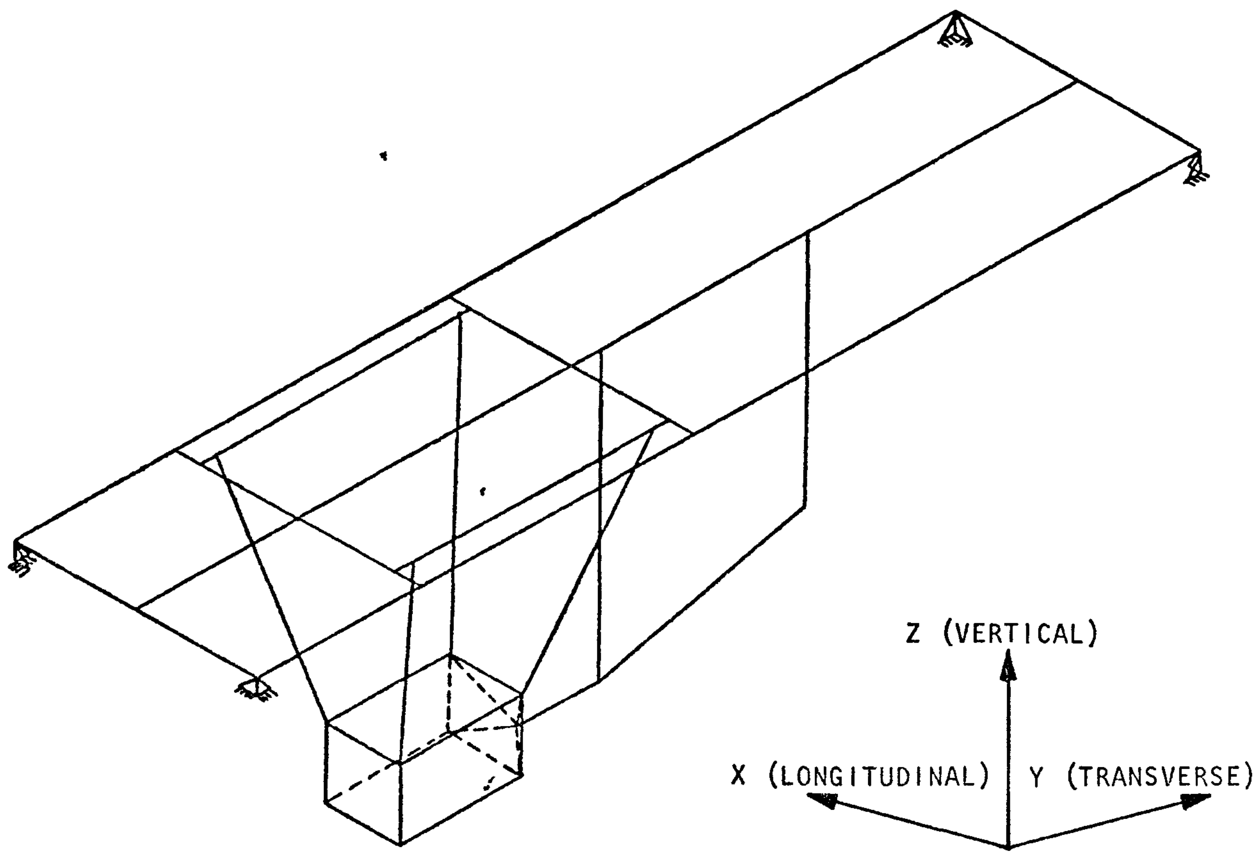


FIGURE A-1. EXPANDED CURRENT DIVIDER COMPUTER MODEL horizontal motion but not the vertical input motion). This analysis procedure is shown schematically in Figure A-2.

The results of the analysis is shown in Figure A-3 as the force time history in a post insulator hanger. The current divider response was such that the hangers did not quite go into compression, varying ±2000 lbs. Thus, the results of this analysis gave lower post insulator hanger stresses than the Assessment Study and indicated that a concept in which the suspension insulators are replaced with post insulators would perform well. Since this concept completely satisfies the model assumptions, we would have extremely high confidence in its performance but, replacing the suspension insulator hangers would be a very expensive modification. In addition, the nonlinear effect of vertical excitation on the pendular system was unknown. For this reason, additional analyses were provided which included the nonlinear behavior of the suspension insulator hangers.

TWO-DIMENSIONAL NONLINEAR MODEL

It was evident that the most economical modification that could be made to the present system would be merely to replace it with stronger suspension insulator hangers. However, in order to do this, it was necessary to show convincingly that the suspension insulator hangers would not be overstressed if impact occurred. Because it was recognized that a three-dimensional nonlinear model of the type required for this problem would be difficult, the two-dimensional nonlinear model was developed first to provide insight into the effects of the nonlinearities of the problem.

To begin the study of the nonlinear behavior of a pendular system, Reference A-1 and other related references were reviewed since pendular systems are used extensively for shock isolation in hardened military installations.

This review indicated that any statically stable pendular system can still be dynamically unstable so that oscillation amplitudes grow to unexpectedly large values as the result of a transient disturbance. Such a system is

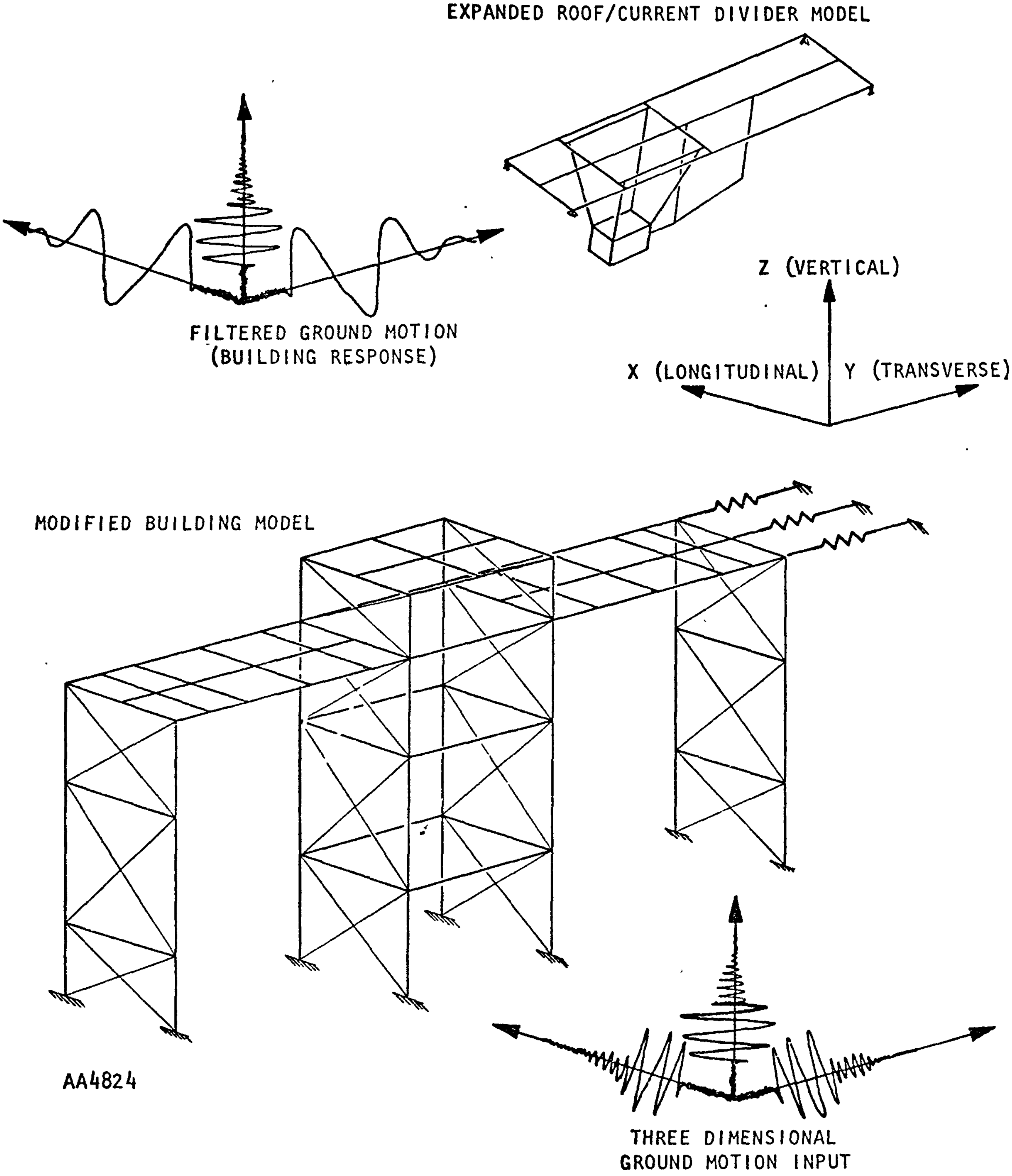


FIGURE A-2. ANALYSIS PROCEDURE

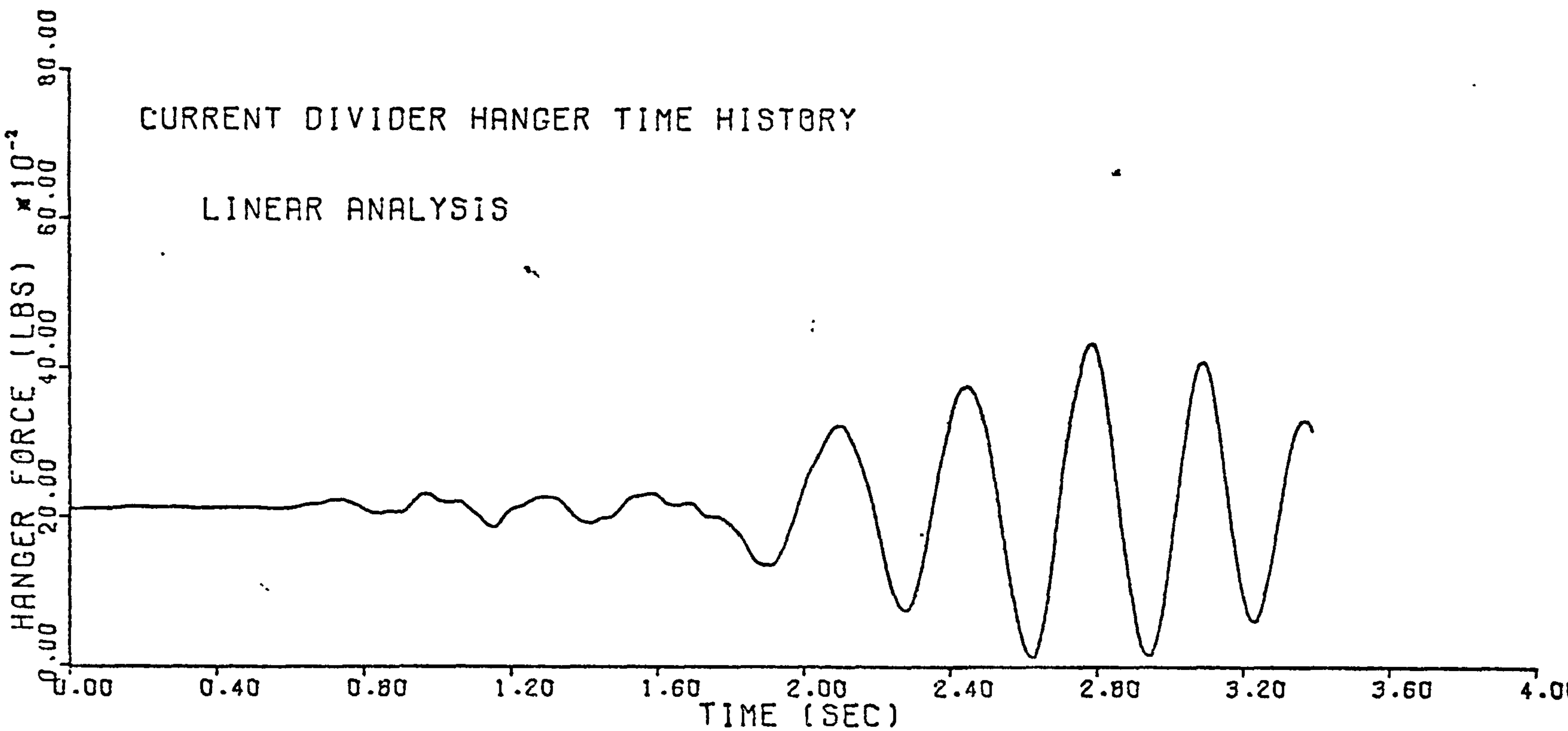


FIGURE A-3.

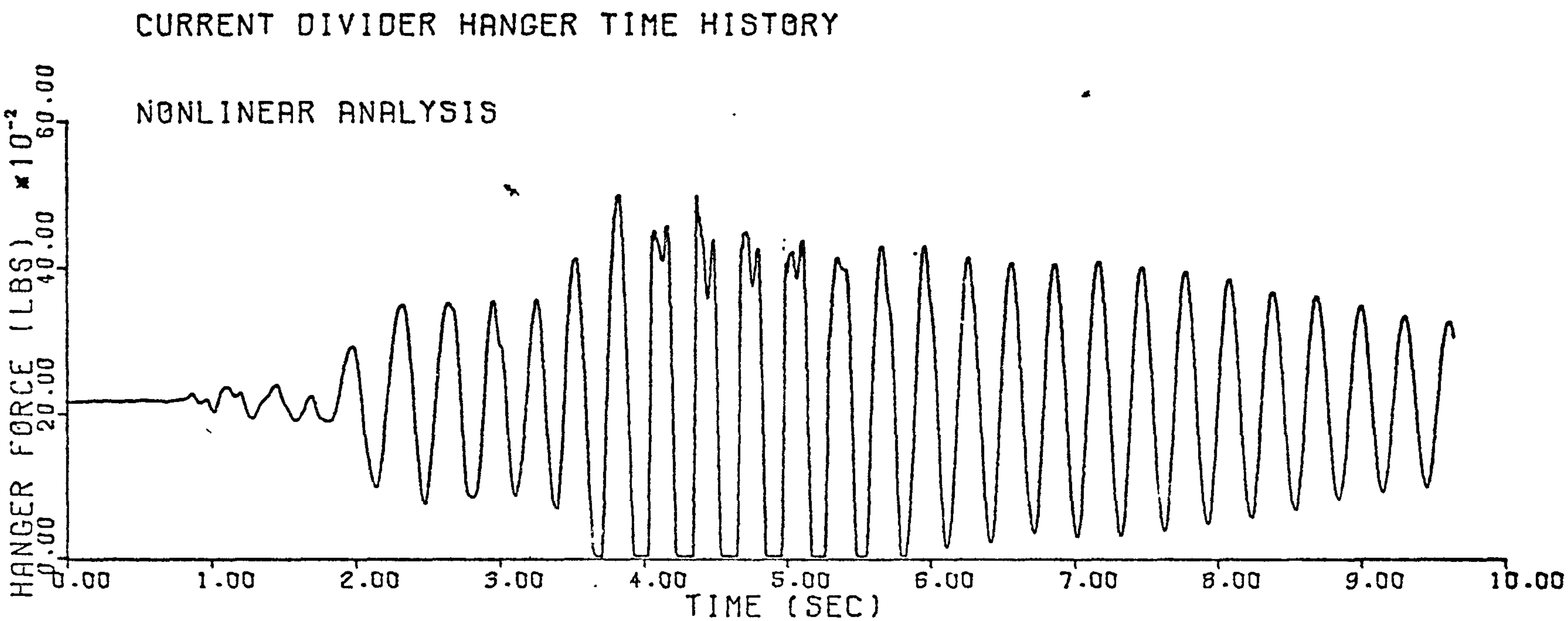
M-7211-4-2541

said to be parametrically excited, and thus the dynamic instability is a potential problem of any pendular system. This fact underscores the necessity of including the nonlinear effects of a pendular system in any model in addition to the suspension hanger nonlinearity. The AA GENSAP computer code is fully capable of including both nonlinear effects and a two-dimensional model with nonlinear capability was developed and executed with unexpected ease. The behavior when subjected to the filtered seismic input was almost completely linear with no unloading of the hangers. This is important because it indicates that the nonlinearities in this suspension system are due to the three-dimensional aspects of the suspension system.

The two-dimensional analysis, indicated that one potentially workable modification concept would be to use vertical suspension insulator hangers. However, this system would require damping, and the damping could contribute to the nonlinear behavior. This concept would also require that the hanger roof supports be modified, which should be avoided, if possible, because of the expense. Thus it was again demonstrated that an accurate prediction of the behavior of the present three-dimensional system was needed.

THREE-DIMENSIONAL NONLINEAR MODEL

The three-dimensional model with full nonlinear capability was developed and subjected to the filtered seismic input. One result of the analysis, the force time history in a suspension insulator hanger, is shown in Figure A-4. It should be noted that several periods of hanger unloading occur, resulting in a peak hanger force of about 5000 lbs. Impacting did not occur with the input considered. The displacement time history of a point on the current divider near a valve connection link is shown in Figures A-5, A-6, and A-7. The displacements shown are relative to the roof motion which is ± 0.5 in. relative to the ground. The valve platform can have motion which, at the valve tip, is ± 0.5 in. relative to the ground. Thus, approximately ± 1.0 in. should be added to the histories shown in Figure A-5 to give the estimated displacement of the current divider relative to the valve tip. An estimated upper bound of the extension required in a current divider valve link is ± 4.0 in.



CURRENT DIVIDER HANGER TIME HISTORY
NONLINEAR ANALYSIS
HANGER FORCE (LBS) ×10⁻²
0.00
20.00
40.00
60.00
0.00
1.00
2.00
3.00
4.00
5.00
6.00
7.00
8.00
9.00
10.00
TIME (SEC)

FIGURE A-4.

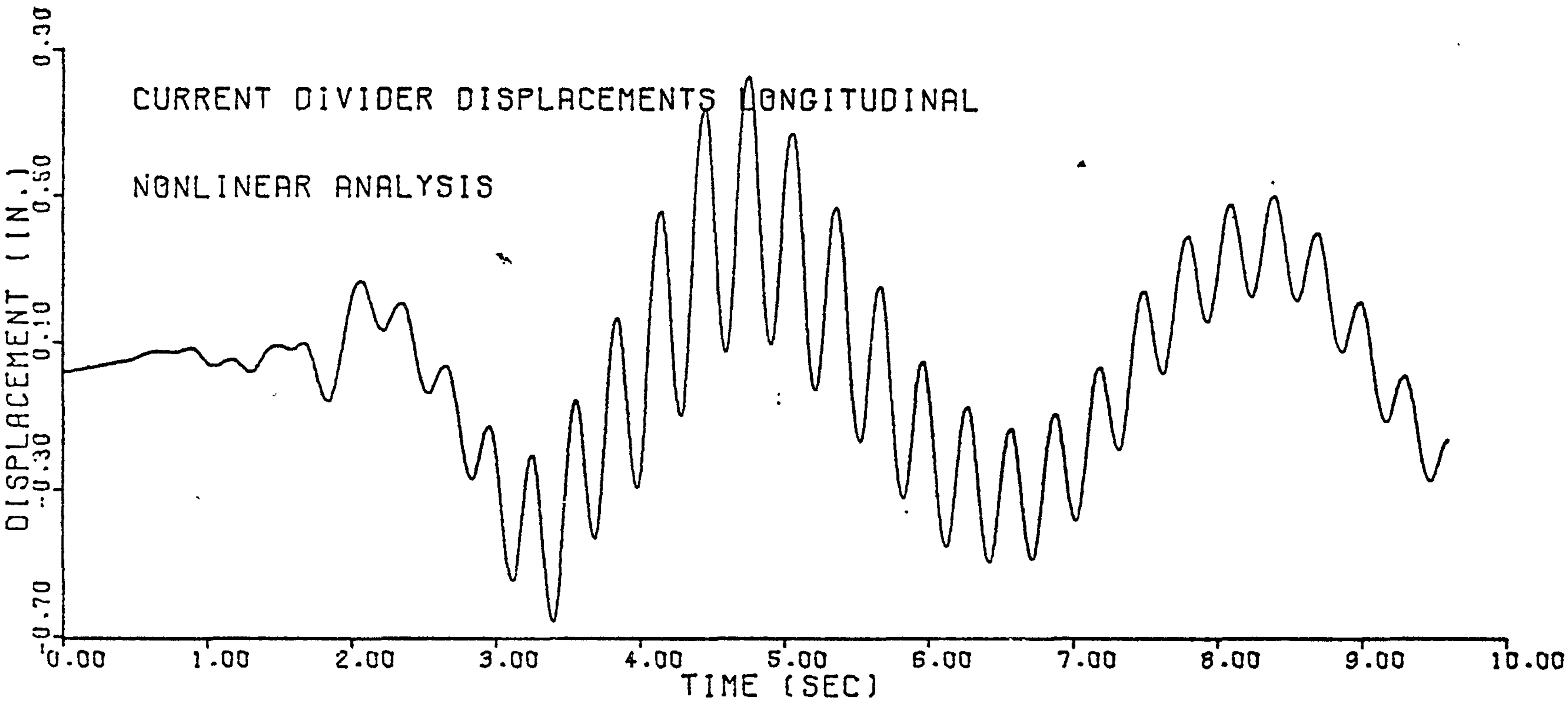


FIGURE A-5.

M-7211-4-2541

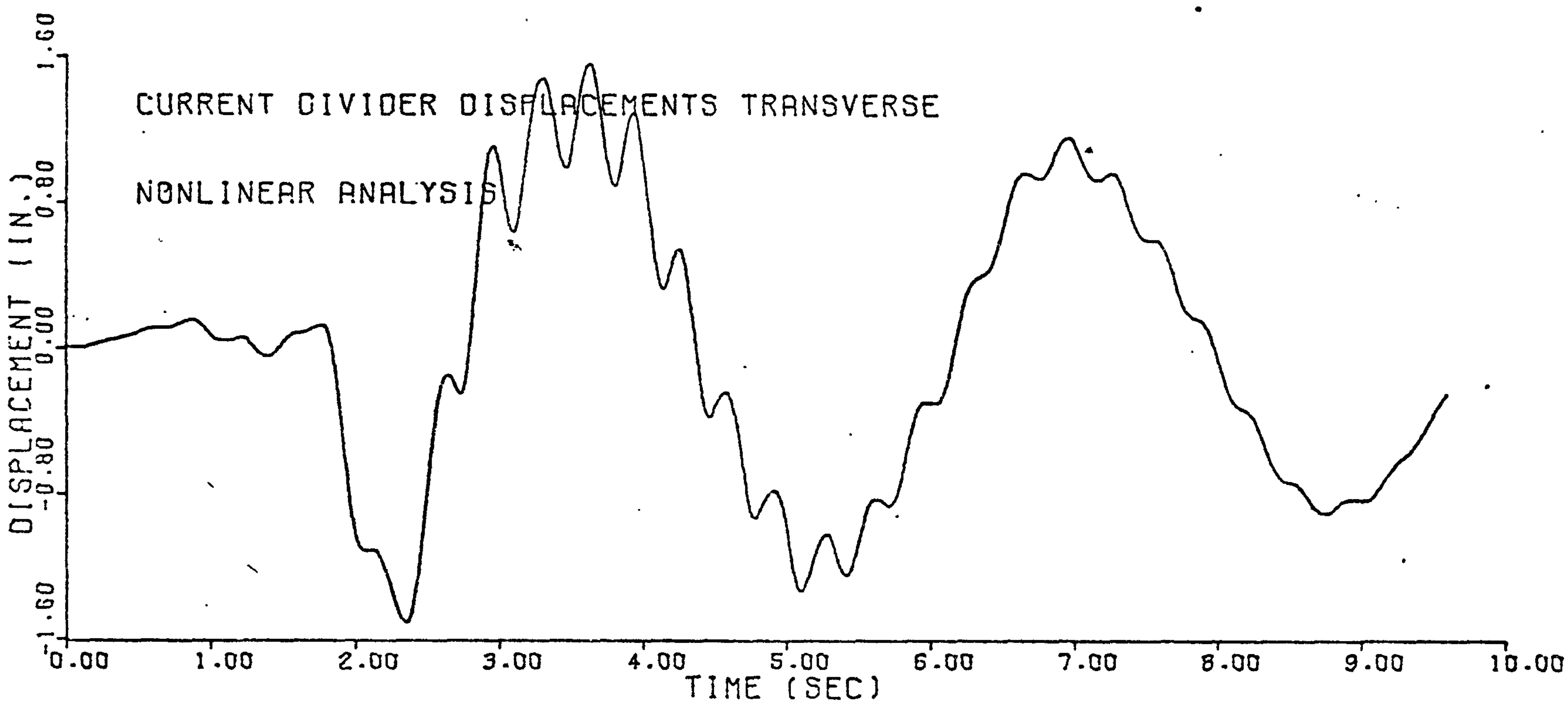


FIGURE A-6.

M-7211-4-2541

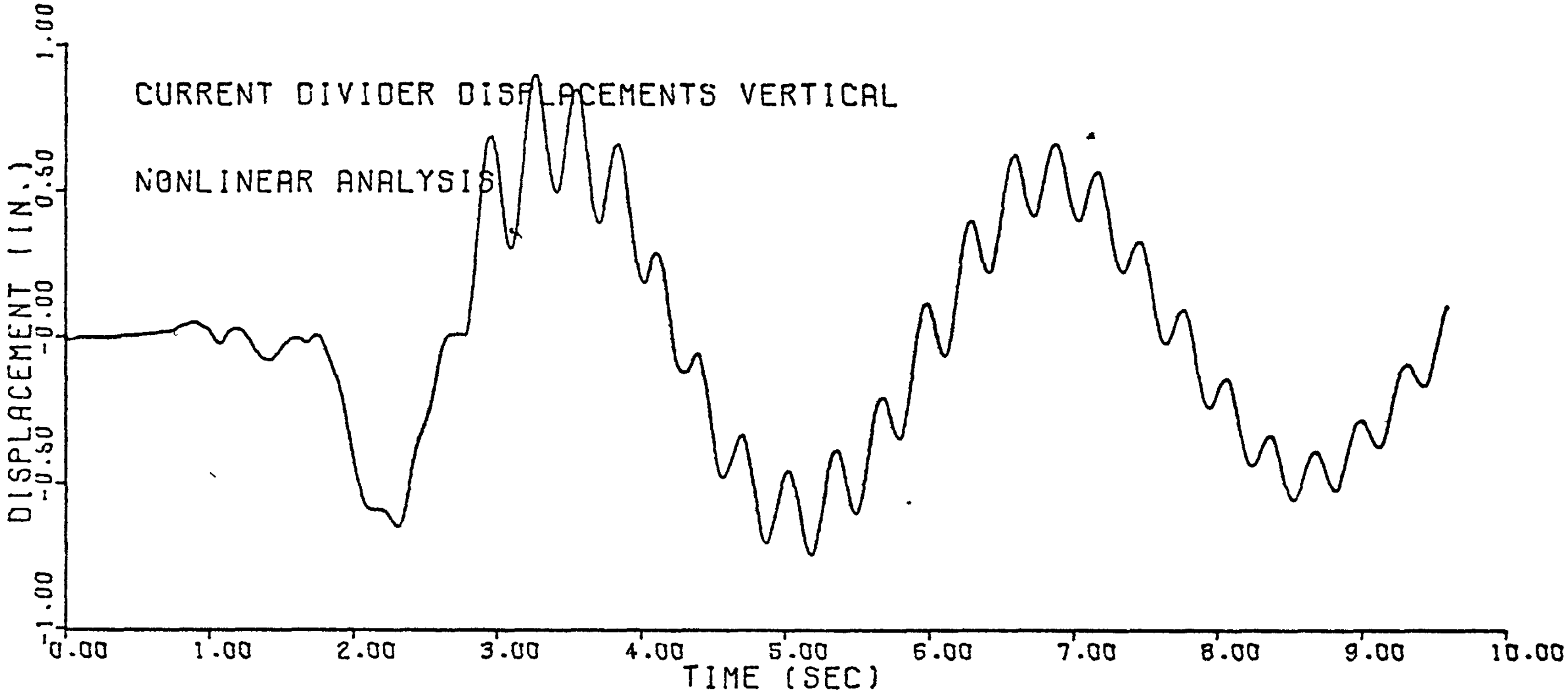

CURRENT DIVIDER DISPLACEMENTS VERTICAL
NONLINEAR ANALYSIS
DISPLACEMENT (IN.)
1.00
0.50
-0.00
-0.50
-1.00
0.00
1.00
2.00
3.00
4.00
5.00
6.00
7.00
8.00
9.00
10.00
TIME (SEC)

FIGURE A-7.

RECOMMENDATIONS

Since the present insulator hangers have a 15,000 lb rating, the factor of safety for the input considered is a value of 3.0. Thus, the existing system has a reasonable factor of safety against failure for the input considered and no revision of the support system is apparently required. However, the current divider valve connecting links should be modified to allow for about 5 in. of extension. This value is 25 percent greater than the upper extension bound computed and provides a factor of safety for displacement. Because of the nonlinearities of the system, the following factors should be noted.

1. Only one unique ground motion input has been considered in the analyses. Although the input used is typical of a hard site, such as Celilo, slightly different input motions could produce peak hanger forces which differ from those presented due to the nonlinearities of the problem. A different ground motion might result in a lower factor of safety. However, in our opinion the system should survive the earthquake ground motion predicted for Celilo with a reasonable factor of safety.

2. The factor of safety quoted above must be considered with the input motion used and must not be viewed as equivalent to a load factor. Because of the nonlinear behavior, it cannot be stated that the system would survive an input motion which is three times stronger than that considered.

Because of the importance of the equipment being considered, and because a nonlinear relationship exists between the input motion and the insulator hanger stress, BPA may wish to provide a higher factor of safety than 3.0 by installing higher strength insulator hangers. Additional analysis using other earthquake ground motions would also provide additional insight on the relationship between strength of earthquake ground motion and insulator hanger stress.

The computer analyses for the current divider suspension system was provided using the GENSAP Code. The GENSAP Code was developed by AA for the U. S. Army, Corps of Engineers, Huntsville Division under another contract for the analysis of Safeguard structures. This program is available through this agency.

REFERENCE

A-1. "Study of Shock Isolation for Hardened Structures," Agbabian-Jacobsen Associates for the U. S. Army Corps of Engineers, Contract No. DA-49-129-ENG-532, June 1966.

APPENDIX B-1

DESIGN MODIFICATION CONCEPTS FOR AIR POWER CIRCUIT BREAKERS

APPENDIX B-1

DESIGN MODIFICATION CONCEPTS FOR AIR POWER CIRCUIT BREAKERS

by K. L. Merz
and
S. F. Masri

INTRODUCTION

The purpose of this memorandum is to summarize the classification results for the air-power circuit breakers (APCB) which were examined at Big Eddy, John Day, Allston, and Custer Substations. The susceptibility of the APCB to earthquake damage is noted and modification concepts proposed to protect the APCB from loss of function resulting from an earthquake.

CLASSIFICATION OF APCB

The classification of the APCB is summarized in Table B-1. The range of natural frequencies for all APCB's reviewed was 1-3 cps and the factors of safety against collapse were less than 3.0 (required for porcelain). It should be noted that the APCB's were evaluated at particular sites with different peak ground acceleration values and response spectra. If all APCB's were evaluated for the Custer ground motion, the factor of safety against collapse would be less than 1.0 for all APCB's. The simple pedestal model shown in Figure B-1 was used to estimate the fundamental response of the APCB. In computing the response, damping was assumed to be 2 percent of the critical damping level.

MODIFICATION CONCEPTS

Due to the fragility and operational importance of the APCB, modification to insure operational survival in the specified seismic environment is required. It should be noted that APCB's have been designed to function primarily as electrical mechanisms and not as structures capable of resisting lateral inertia loading. In some designs, the porcelain support column must

TABLE B-1

CLASSIFICATION OF APCB

ABCB	Frequency Estimate, cps	Damping Estimate (% Critical Effective Damping)	Allowable Static Lateral Load	Factor of Safety Against Collapse			
				Big Eddy 0.12 g ⊛	John Day 0.12 g ⊛	Allston 0.24 g ⊛	Custer 0.36g ⊛
500 KV Merlin-Gerlin	2.9	2%	0.15 g	1.1			<0.4
500 KV Delle-Alsthom (Cogenel)							
4 support insulators per column	1.7	2%	0.30 g			1.1	
5 support insulators per column w/guys	2.6	2%	0.36 g	2.6			
500 KV GE (500-2Y)	1.9	2%	<0.20 g		<1.67 (<1.0*)		
220 KV GE	1.7	2%	0.20 g	1.6			
220 KV Allis-Chalmers	1.9	2%	0.21 g	1.6			
(220 KV Oerlikon** minimum oil PCB)	1.1	2%	0.62 g			1.8	

⊛ Peak ground acceleration
* Operational factor of safety
** Included due to configurational similarity to APCB

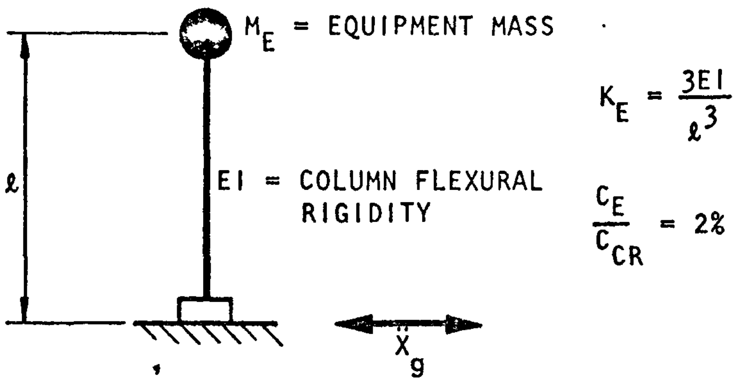


FIGURE B-1. BASIC PEDESTAL MODEL USED FOR CLASSIFICATION STUDIES

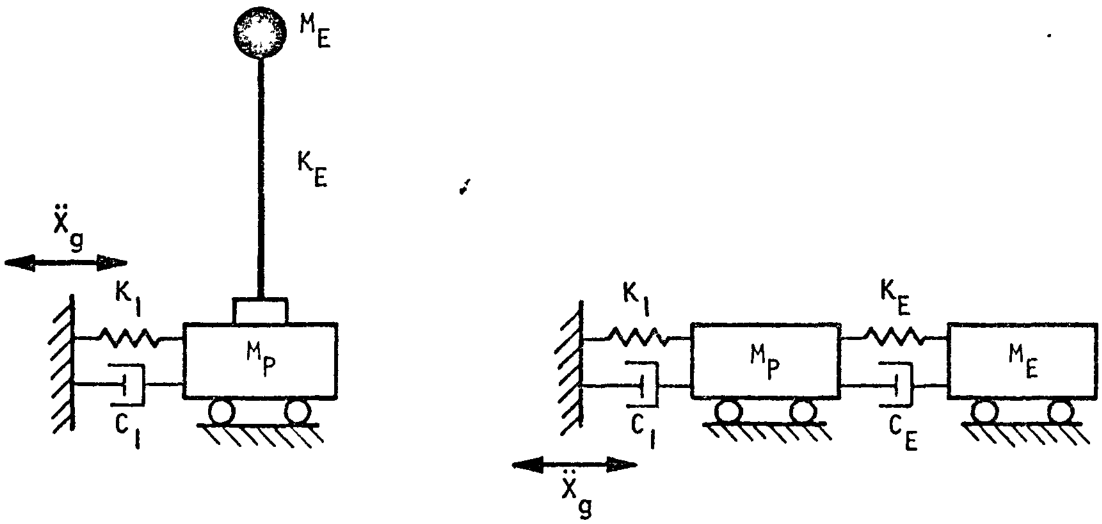


FIGURE B-2. BASIC EQUIPMENT ISOLATION MODEL FOR DESIGN STUDIES also function as a pressure vessel which causes initial stresses within the porcelain. Thus, the seismic fragility of the APCB should not be unexpected. The following modification concepts were considered:

1. Change of configuration
2. Strengthening of support columns
3. Incorporation of damping devices in the APCB
4. Isolation

Concept 1, which would change the configuration of the APCB (for example, introduce guys to brace the support columns), is desirable from a structural viewpoint but electrically undesirable due to the introduction of multiple leakage paths. In addition, a new configuration must be carefully analyzed to insure that dynamic amplication effects are reduced.

Concept 2, which provides for strengthening the support columns, is also a desirable structural modification. However, the support columns are usually specialized, costly porcelain elements which are not off-the-shelf replacement items.

Concepts 1 and 2 would both require electrical and dynamic testing for validation. In addition, the APCB would have to be dismantled for modification.

Concept 3, which would introduce damping devices in the APCB, has been successfully implemented and dynamically tested by European APCB manufacturers for South American utilities (References B-1 through B-5). This concept is very desirable to attenuate the dynamic response of the APCB, however, this concept can only be implemented by the manufacturer who has intimate knowledge of the mechanical design details. The incorporation of an auxiliary mass damper in the APCB design is an alternate means of controlling the equipment response which can also only be implemented by the manufacturer.

Concept 4, which would isolate the APCB from ground motion, appears to be the least-cost modification alternative for existing breaker installations. This concept would involve the placement of the APCB upon a support platform which would be isolated or heavily damped to attenuate the APCB response.

Both concepts 3 and 4 require dynamic testing for validation. For concept 4, no modifications to the APCB would be required. It is concluded that isolation, concept 4, is the least-costly and most feasible modification concept for existing in-plate equipment. This concept is, therefore, recommended. Discussion of two alternate isolation concepts follow.

SEISMIC ISOLATION CONCEPTS

The feasibility of two seismic isolation concepts were examined in detail for the APCB's listed in Table B-1. The basic two degree-of-freedom (DOF) model used in the design studies is shown in Figure B-2. The model allows freedom in choosing three support parameters, M_p (platform mass), K_I (isolation stiffness), C_I (effective isolation damping), to achieve attenuation of the equipment which is unmodified.

Previous studies (References B-6 and B-7) have indicated that the greatest reduction to APCB seismic response is given by a low frequency, damped support system. However, the displacement response of such a system can be several inches due to the required flexibility of the support. It should be noted that the large displacement response (stroke) is required in order for a damping device of realistic dimensions to achieve the required damping level. The simplest type of flexible isolation is a platform supported by rocker arms forming a pendular system. This type of system is shown in Figure B-3. Such a system has been proposed by a manufacturer in Reference B-8.

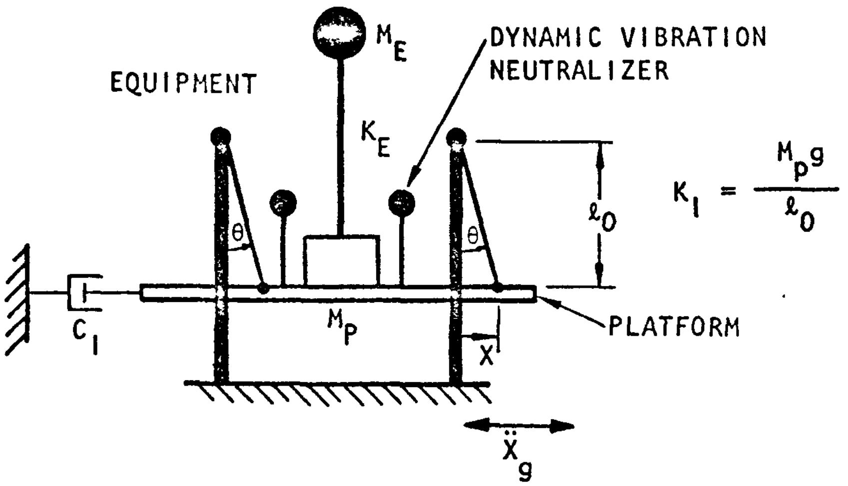


FIGURE B-3. PENDULAR ISOLATION CONCEPT

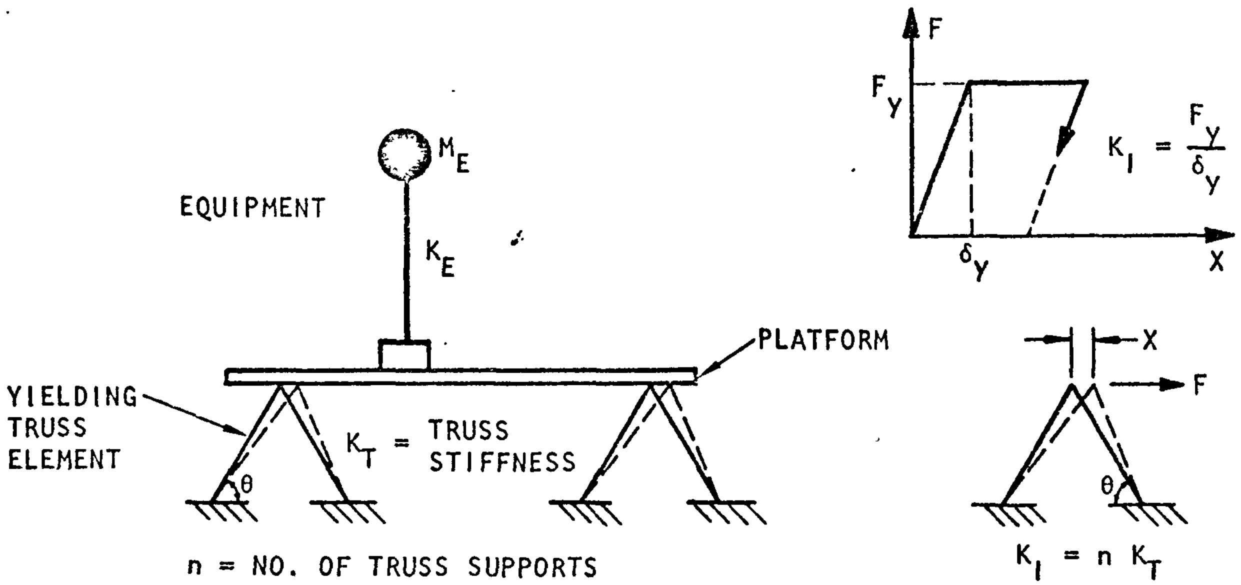


FIGURE B-4. YIELDING SUPPORT ISOLATION CONCEPT

An alternative support system is shown in Figure B-4. In this case, the supports are elasto-plastic elements which form a hysteretic two degree-of-freedom system. The seismic response of such systems is discussed in Reference B-9 and applied to the seismic modification of a D.C. reactor in Reference B-10.

The pendular platform system is applicable to all APCB configurations, since the entire APCB and structural supports would be placed upon the platform. The yielding platform system is applicable only to APCB configurations which do not have integral structural supports. The existing structural supports would be replaced by a yielding structural support. One APCB configuration listed in Table B-1, cannot utilize the yielding support system.

Attenuation of the vertical response of the APCB has not been indicated in either concept as it is probably not required. However, vertical attenuation can be provided with the pendular system. Since the isolation concepts proposed require a development program which will include model studies and verification tests of the prototype, the need for vertical isolation should be determined during the test phase of this program. More detailed discussion of the two systems follows.

PENDULAR PLATFORM ISOLATION

The concept of pendular isolation is shown in Figure B-5. An entire three-phase APCB, including CT, would be placed upon a steel frame platform and suspended from rocker arms. The arms would be hung from portal or T-frame ground supports. Heavy-duty truck U-joints would be utilized for the rocker pins. The detail design of the support frame is not shown, as this can be accomplished with normal structural design practice. The only requirement for the frame is that adequate vertical stiffness be maintained to eliminate vertical or platform amplification. It should be noted that additional rocker arms and supports can be added to the platform since the system response is a function of the rocker arm length. Thus, the platform can be supported at several points in order to shorten the platform spans. A typical 500 KV

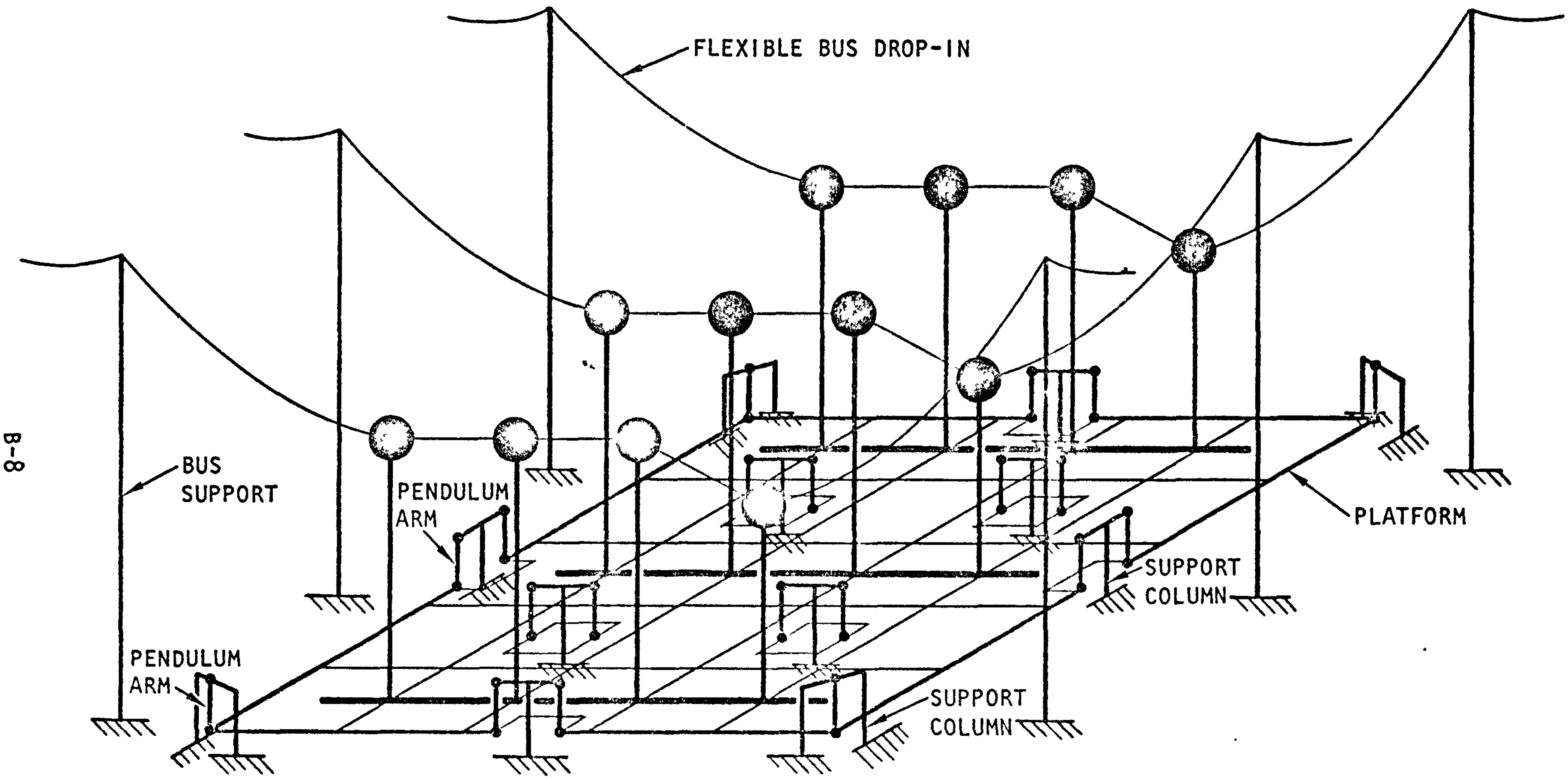


FIGURE B-5. TYPICAL THREE-PHASE APCB/CT ISOLATION PENDULAR PLATFORM CONCEPT

M-7211-7-2651

APCB/CT isolation platform would be 50 ft x 50 ft in plan and weigh 15,000 lb. The rocker arm would be 8 ft long and minimal platform ground clearance is required. A small friction damper would be added to prevent wind excitation. The fundamental frequency of a typical platform system would be 0.2 cps which would attenuate the APCB response to 0.10 g for the highest intensity ground motion in the BPA service area. However, the displacement response of the platform would be ± 10-20 in., requiring that the bus attachments accommodate the motion. A flexible bus drop-in as shown in Figure B-5 is recommended.

One way of reducing the displacement of the platform to earthquake excitation would be to increase the equivalent viscous damping in the pendulum arms through the introduction of damping devices. An effective damper device should have nonlinear characteristics. Since the excitation is transient, the response of the system becomes a complex process for which few analytical results are available. The amount of damping should be determined experimentally during the development program.

An alternative approach would be the use of the dynamic vibration neutralizer (DVN). The DVN consists of an auxiliary mass that is coupled to the vibrating platform by a linear spring. With proper tuning, the auxiliary mass can be made to exert a force on the platform that is equal in magnitude and opposite in direction to the exciting force, thus completely neutralizing the motion of the platform in the case of sinusoidal excitation, or reducing the response in the case of earthquake excitation. Several neutralizers can be attached to the platform as indicated in Figure B-3 thus distributing the concentrated mass and the corresponding neutralizing forces. A typical neutralizer would consist of a pipe that is welded to the platform at one end with a concentrated mass attached to its free end. The frequency of the neutralizer must be tuned to coincide with the frequency of the platform. The effect of the neutralizer is shown in Figures B-6 and B-7. By incorporating DVN in the platform design, the response spectra level over a wide frequency band can be attenuated by a factor of 25 percent. It can be seen from the Figure B-6 response spectra, with damping of $f = 0.02$, that the level of S_V is reduced from 2.7 to 2.1. An additional effect of the DVN is shown in Figure B-7 as a reduction in the distribution of peak displacement response. Thus, the major effect of the DVN is to smooth and reduce the response of the platform.

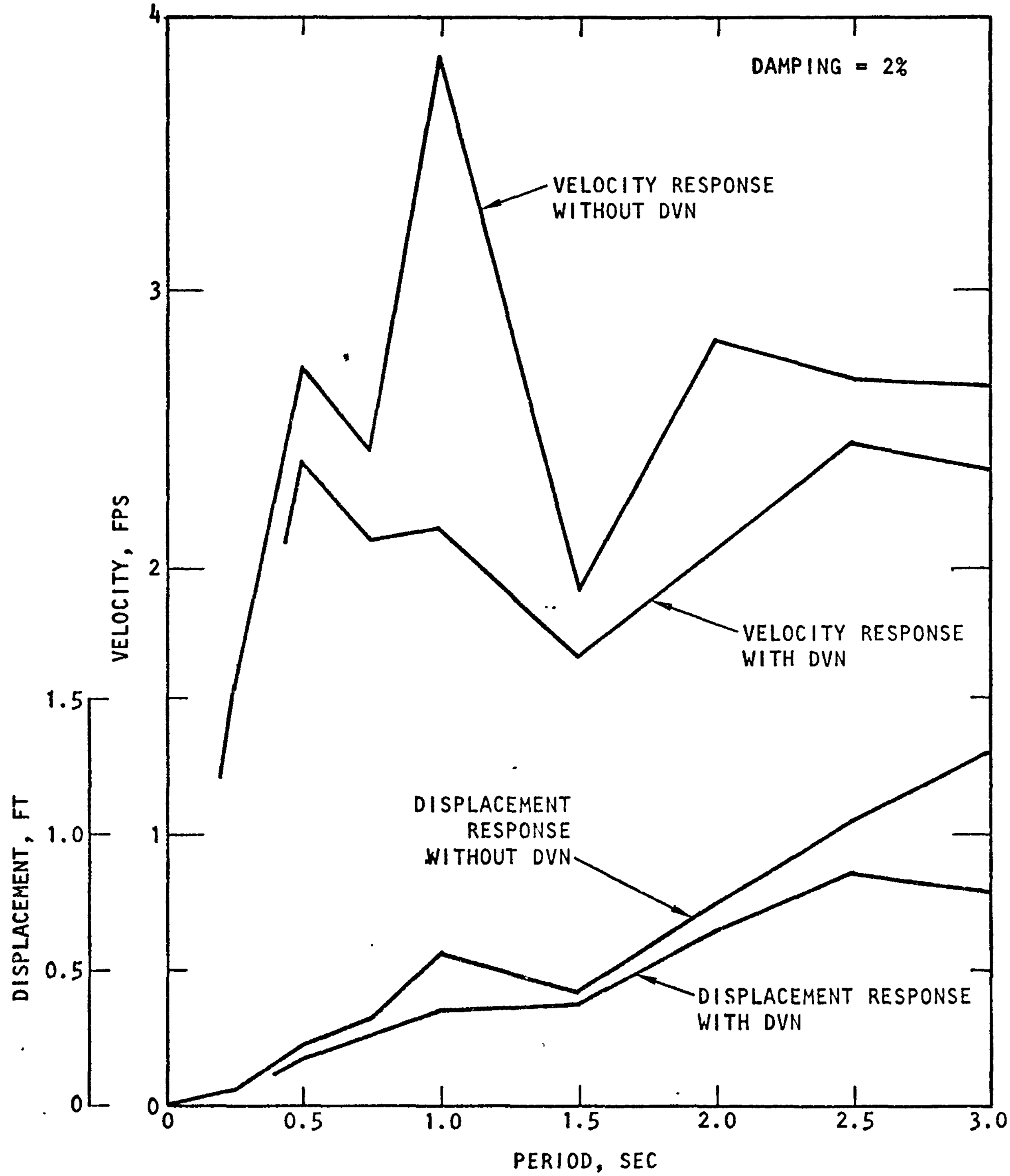


FIGURE B-6. RESPONSE SPECTRA FOR EL CENTRO (1940)

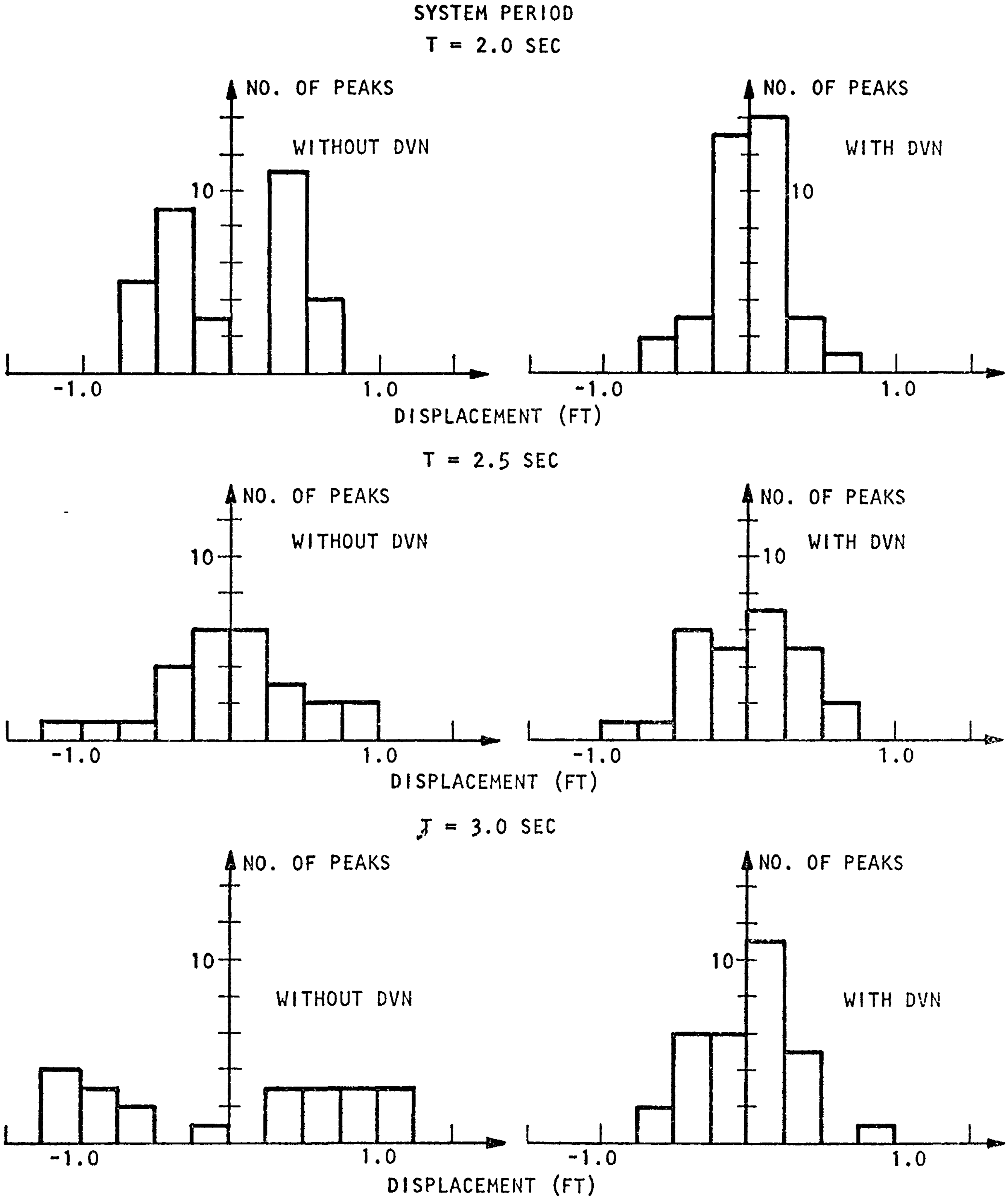


FIGURE B-7. DISTRIBUTION OF PEAK DISPLACEMENT RESPONSE TO EL CENTRO (1940)

YIELDING PLATFORM ISOLATION

The concept of a yielding support is shown in Figure B-8. An entire three-phase APCB, including CT, would be placed upon a steel frame platform supported on yielding tripod supports. Each truss element of the tripod would be an elasto-plastic device of the type presented in Reference B-11. Such devices are commercially available as a stock item from the supplier. The detail design of the support frame is not shown, as this can be accomplished in accordance with normal design practice. The platform should have adequate vertical stiffness to eliminate vertical amplification. The platform support points can be placed directly under the breaker columns, thus eliminating loaded beam spans. The tripod supports would replace the existing structural supports of APCB which are not integral units. The tripod-platform connection details would be designed to minimize bending or frame effects in the tripod. This concept does not introduce additional support flexibility but instead introduces hysteretic damping to attenuate the APCB response. The concept, in effect, isolates the platform and APCB from shear forces which exceed the yield level of the support tripods. Thus, this concept can only be utilized when the allowable static lateral load level of the APCB is greater than or equal to the peak ground acceleration. This concept was studied using a Delle-Alsthom APCB (4 support insulator column) as an example. The APCB and supports were modeled using the AA GENSAP computer code which is fully capable of including the nonlinear hysteretic effect. The three-dimensional aspects of the problem were maintained in the modeling.

A linear analysis without a yielding support, was performed in addition to a nonlinear analysis, with a yielding support. The results are compared in Figure B-9. Note that not only is the reduction of the APCB support column bending moment evident for the case of a yielding support, but the duration of a given moment level is reduced. The input used in the study was 8 sec of the Olympia (1949) record (3 components). This input is typical of the intensity level used to evaluate the Allston substation. The peak platform displacement response was ± 1 in. A certain amount of permanent deformation would result from any design level motion. Thus, the platform would have to be realigned with jacks after a seismic event.

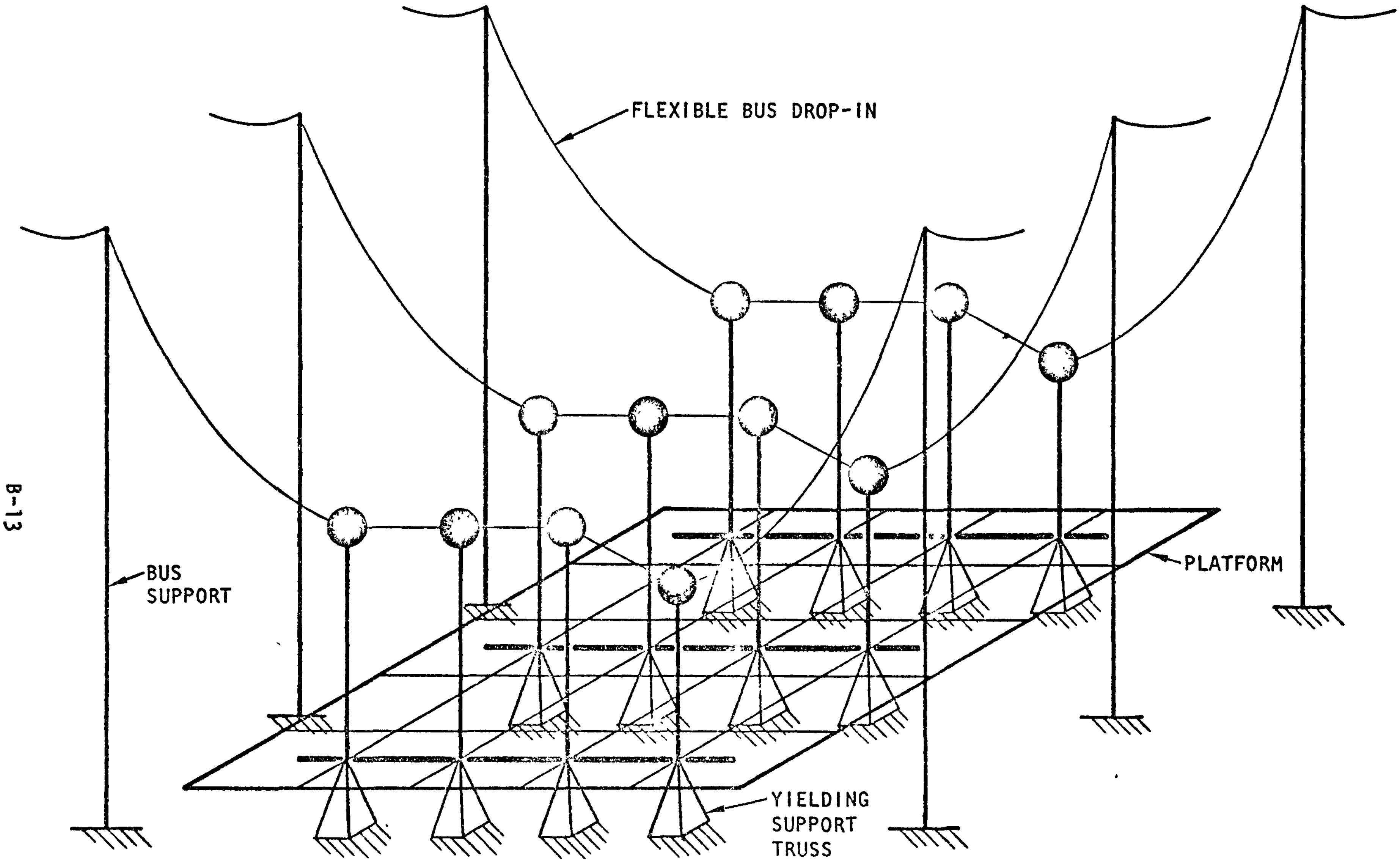


FIGURE B-8. TYPICAL THREE-PHASE APCB/CT ISOLATION YIELDING PLATFORM CONCEPT

M-7211-7-2651

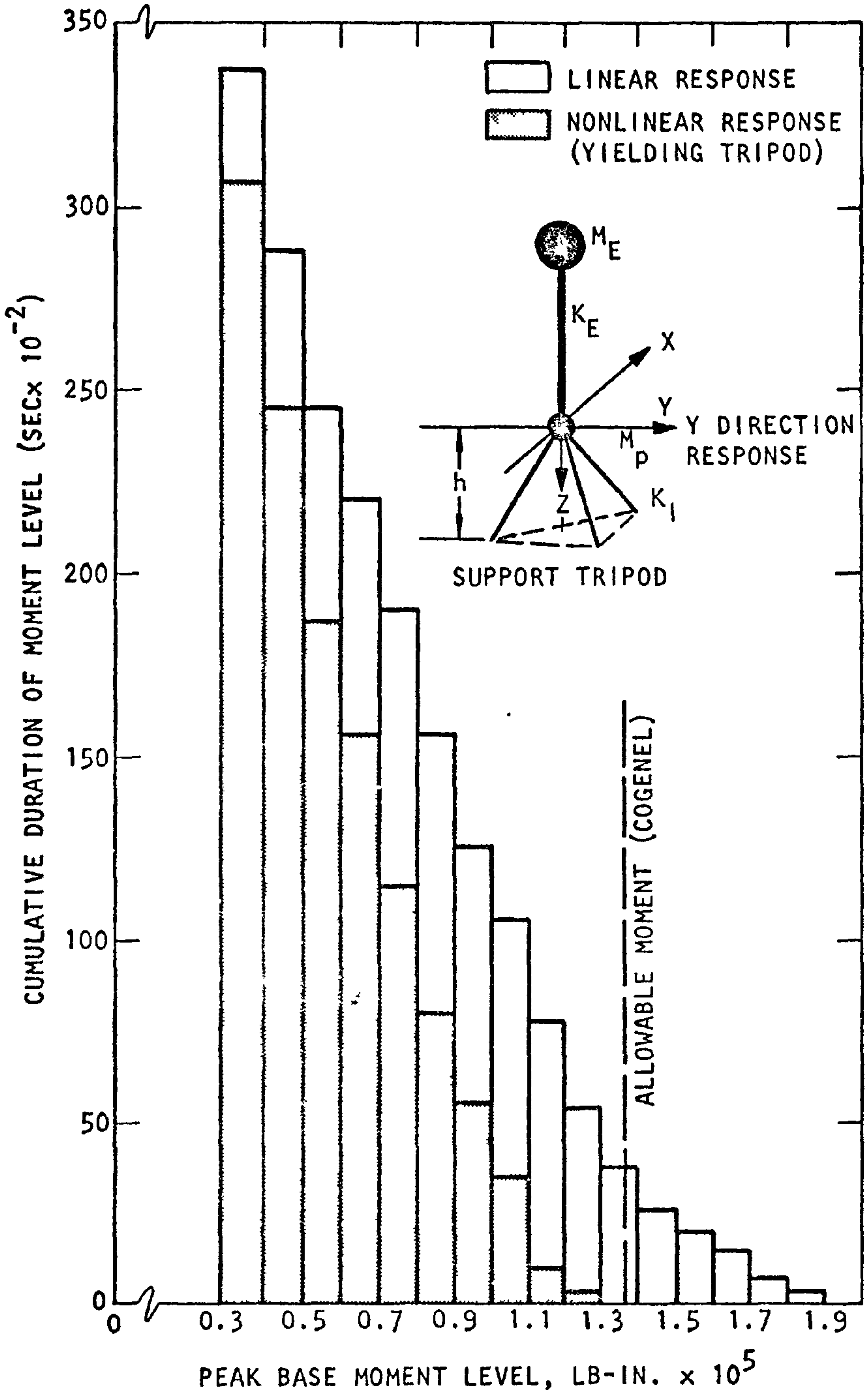


FIGURE B-9. RESPONSE OF APCB TO OLYMPIA (1949)

CONCLUSIONS AND RECOMMENDATIONS

1. All APCB reviewed require modification.

2. Since the manufacturers have in some cases developed modifications which will improve the earthquake resistance level of their APCB's, BPA should consult the manufacturers to determine if actual cost proposals from the manufacturers for equipment modifications are less expensive than the cost of the shock isolation concept proposed.

3. The estimated cost of fabrication and installation of the isolation concepts proposed above is about $15,000 for each 500 KV 3-phase APCB. This assumes that a large number of APCB's will be isolated and does not include the cost of development and testing of models and prototypes.

4. It is recommended that development studies be initiated on one, or both, isolation concepts. This should consist of a scaled model study to verify the parameters of each model, and to determine if vertical isolation is required. A prototype of the best concept should then be designed in detail, fabricated, and proof tested to validate the concept. Operational validation tests of the APCB should also be performed with the prototype.

REFERENCES

B-1. Ramseier, E. and R. Wehrli, "Operational Security of HV Circuit Breakers Subjected to Earthquakes", *Bulletin Oerlikon,* No. 391, 1969, pp. 9-13.

B-2. Cuk, N. and E. Ramseier, "High-Voltage Switchgear for Earthquake-Prone Areas", *Brown Boveri Review,* No. 12, 1970, pp. 548-553.

B-3. "DLF Airblast Circuit Breaker:--Protection--Against Earthquakes", *Electrical World,* July 15, 1972, pp. 30-31.

B-4. Beutler, P., "Seismic Considerations for High-Voltage Substation Facilities", *IEEE Paper,* No. 70 - TP579 - PWR, 1970.

B-5. Novoa, F., "Earthquakes and Substation Equipment Arrangement and Specifications", *CIGRE Paper 23.02,* 1970.

B-6. Winthrop, D. A. and H. C. Hitchcock, "Earthquake Design of Structures with Brittle Members and Heavy Artificial Damping by the Method of Direct Integration", *Bulletin of the New Zealand Society for Earthquake Engineering,* April 1971, pp. 294-300.

B-7. Hitchcock, H. C., "Electrical Equipment and Earthquakes", *New Zealand Engineering,* January 15, 1969, pp. 3-14.

B-8. Barton, R. S. and P. W. Dwyer, *General Electric ATB Aseismic Design,* Presented at the Doble Client Conference, Boston, Massachusetts, April 24-28, 1972.

B-9. Veletsos, A. S. and W. P. Vann, "Response of Ground-Excited Elastoplastic Systems", *Journal of the Structural Division, ASCE,* Vol. 97, No. ST4, Proc. Paper 8075, April 1971, pp. 1257-1281.

B-10. Gilmour, R. M. and H. C. Hitchcock, "Use of Yield Ratio Response Spectra to Design Yielding Members for Improving Earthquake Resistance of Brittle Structures", *Bulletin of the New Zealand Society for Earthquake Engineering,* April 1971, pp. 285-293.

B-11. Schrader, E. W., "Torus Absorbs Impact Energy", *Design News,* November 10, 1965.

APPENDIX C-1

RECOMMENDED SEISMIC SPECIFICATION

APPENDIX C-1

RECOMMENDED SEISMIC SPECIFICATION
ZONE C

SEISMIC REQUIREMENTS

OPERATIONAL REQUIREMENTS

The design shall be such that the apparatus and its supporting structures shall suffer no damage nor loss of function and shall remain operational during and following the seismic event described in Paragraph 1.2. The term "operational" implies that rotating equipment will not freeze, pressure vessels will not rupture, supports will not collapse, systems required to be leak-tight will remain leak tight, and components required to respond actively (such as control linkages, switch contacts, relays, motors, pumps, valves, etc.) will respond actively. In addition, equipment shall not be caused to change operational state due to the seismic event. For example, a circuit breaker in an open position shall remain open, or if closed shall remain closed.

SEISMIC ENVIRONMENT

The seismic environment for which the apparatus and supporting structures shall be designed is specified to be a vibratory ground motion having a maximum component of acceleration of 36 percent of gravity in any horizontal direction. The vibratory motion is further defined by the response spectra given in Figures C-1 and C-2 for single-degree-of-freedom oscillators.

DESIGN SPECIFICATIONS AND FACTORS OF SAFETY

All structural steel and aluminum elements shall be designed and fabricated in accordance with the specifications listed in References 1 and 2, respectively. The rated strength of porcelain elements shall not be less than three times the working stress. No increase in working stress for porcelain elements shall be permitted for seismic loads acting alone, or in combination, with the design dead and live loads. Factors of safety for all other materials must be submitted to the Buyer for approval.

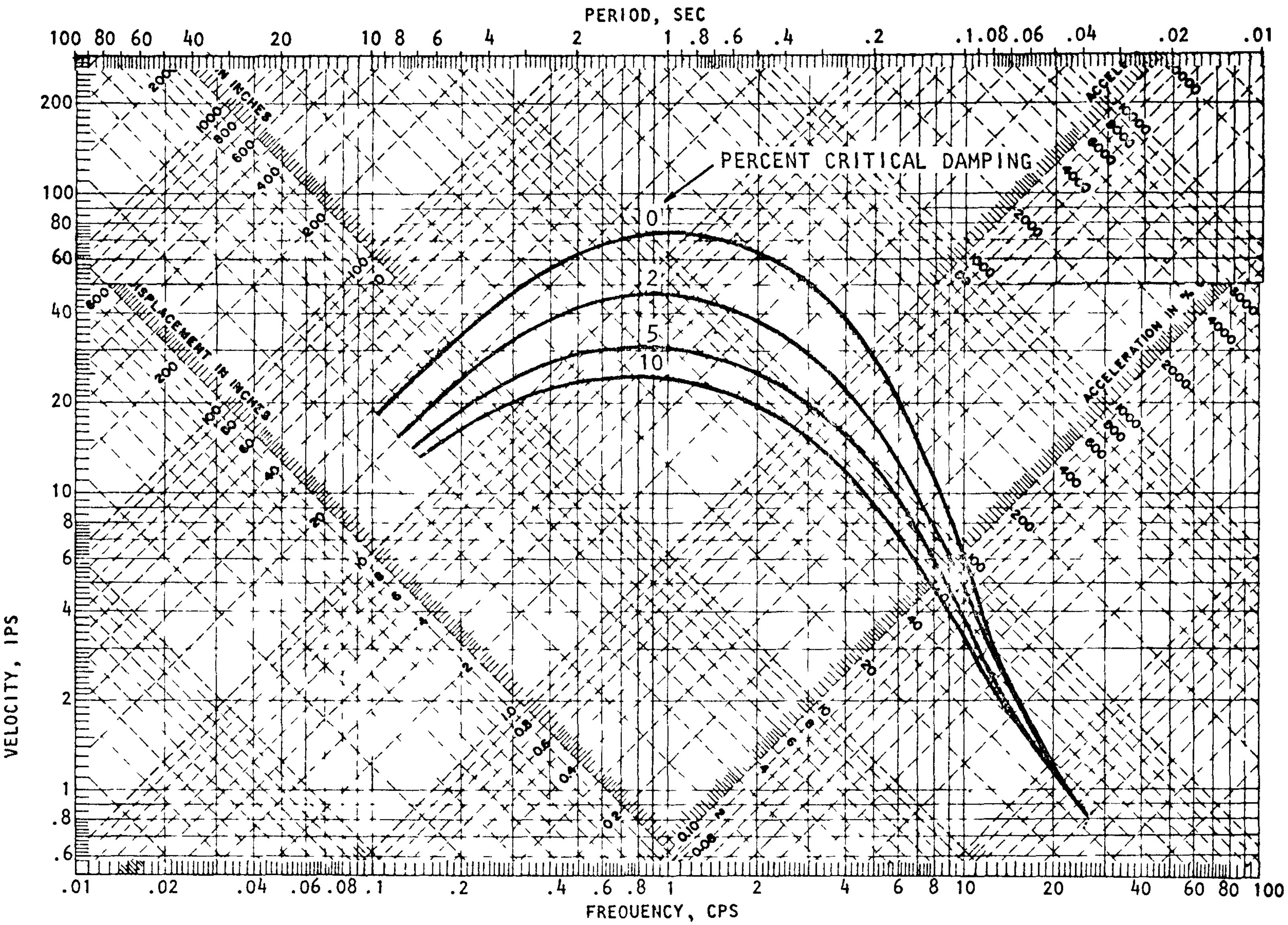


FIGURE C-1. RESPONSE SPECTRA FOR HORIZONTAL GROUND MOTION--ZONE C

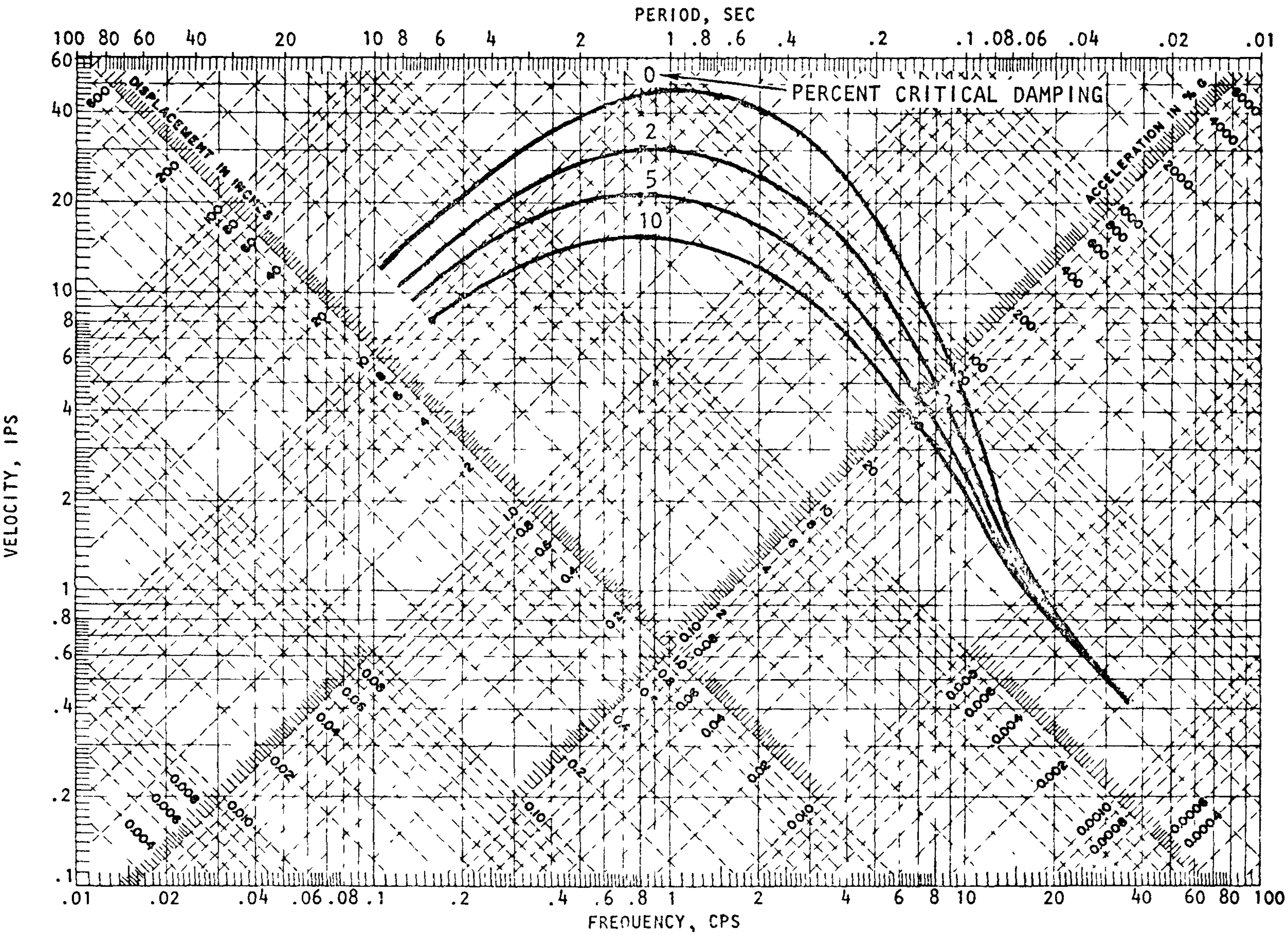


FIGURE C-2. RESPONSE SPECTRA FOR VERTICAL GROUND MOTION--ZONE C

R-7211-13-2859

DYNAMIC ANALYSIS

The manufacturer shall perform a dynamic analysis which considers the essential natural frequencies and damping of the apparatus and its supporting structures. A general outline of the procedures that will be followed must be submitted with the bid proposed. The natural frequencies and the damping used in the analysis shall be verified by experimental tests. The analysis shall consider all critical directions of response and shall be based upon the seismic environment defined in 1.2. If time motion records are used for the analysis, these records shall yield response spectra which are equal to or greater than the response spectra given in Figures C-1 and C-2 for comparable damping in the region defined by the essential natural frequencies of the equipment. The manufacturer shall submit to the buyer for review a clear and concise description of the method and the results of the dynamic analysis. This description shall indicate the natural frequencies and damping used, experimental test results, the safety factors computed for probable modes of failure, and the maximum deflections, shears, moments, and forces at all critical points in the apparatus and supporting structures.

FOUNDATIONS AND SUPPORTING STRUCTURES

If the foundations and/or suoporting structures are to be provided by the Buyer, the necessary structural information will be supplied to the manufacturer of the apparatus. The foundations and/or supporting structures shall be considered in the dynamic analysis provided by the manufacturer and described in 1.4.

RESPONSIBILITY OF THE MANUFACTURER

Review of the dynamic analysis of the apparatus and supporting structures by the buyer shall not relieve the manufacturer of the responsibility of providing apparatus that will withstand the seismic environment defined in 1.2 without damage or loss of function.

REFERENCES

C-1. *Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings*, American Institute of Steel Construction, as last revised.

C-2. "Specifications for Structures of Aluminum Alloy 6062-T6," *J. of the Structural Division, American Society of Civil Engineers*, Vol. 82, No. ST3, Proceedings Paper 3341, December 1962. Other specifications for alloys 6063-T5 and 6063-T6 are given in Paper 3342 of the same journal.

R-7211-13-2859

APPENDIX D-1

STRONG-MOTION DATA REQUIREMENTS FOR THE BONNEVILLE POWER ADMINISTRATION SERVICE AREA

A Report Submitted to Agbabian Associates
by

Stewart W. Smith
Seismological Consultant
Professor and Chairman
Graduate, Geophysics Program
University of Washington

October 9, 1972

APPENDIX D-1

STRONG-MOTION DATA REQUIREMENTS FOR THE BONNEVILLE POWER ADMINISTRATION SERVICE AREA

by Stewart W. Smith

STATEMENT OF THE PROBLEM

Damaging earthquakes have occurred in the northwest during historic times, and can be expected to continue in the future. Very little data on ground motion exists for this region because the rate of occurrence of earthquakes is small, and because in the past there have been few strong-motion seismographs. As a result, engineering decisions tend to be made using data from California earthquakes, for which much more data exists. This probably causes overly conservative estimates of ground motion. For example, in the Puget Sound region, the damaging earthquakes have been at depths of more than 60 km, and thus have produced accelerations substantially less than California earthquakes of the some magnitude but with much shallower depths. To make realistic assessments of future earthquake motion, it is necessary to have more strong-motion instruments in the Washington, Oregon, Idaho, western Montana region. These instruments should sample diverse geologic conditions in all the major seismic areas of this region.

PROPOSED SOLUTIONS

At the time this study was initially considered, there were 23 strong-motion instruments operating at 18 sites in the BPA service area. They are listed in Table D-1. At that time it appeared that the only way that adequate data on earthquake motion for BPA installations could be obtained would be for BPA to undertake a cooperative program with the Seismological Field Survey and the State Universities in this region. In such a program BPA would provide the instruments, and the other cooperating agencies provide the manpower for maintenance and analysis of data. This type of arrangement is quite easy to establish and has worked successfully in other parts of the country.

During the past eight months six new installations have been completed and at the time of this writing, crews from the Seismological Field Survey (NOAA) are installing twenty-five new instruments at Veterans Administration hospitals and U.S. Army Corps of Engineers' installations throughout the northwest region. An examination of BPA transmission facilities showed that the present distribution of instruments does not yet cover all the types of geological environments that are of concern to BPA; however, it does cover most of them. Discussions with Dr. Matthiesen, Director of the Seismological Field Survey, indicate that additional instruments for the northwest have a high priority in his agency's plans for the future. Federal funding for this type of activity is increasing rapidly, and based on the work done in the past year it is now reasonable to assume that during the next few years all of the geological environments of importance to BPA will be covered adequately with strong-motion seismographs. Specific problems concerning the response of electrical structures to earthquake motions can be investigated by BPA using this basic data.

CONCLUSIONS

As a result of this study, we conclude that it is not necessary for BPA to install strong-motion instruments at selected facilities. Rapid increases in the number of instruments installed and planned for the future make it clear that damaging earthquakes in the northwest region will be adequately recorded in the future. This data will make possible reliable seismic design of future facilities and equipment.

TABLE D-1

STRONG-MOTION SEISMOGRAPH LOCATIONS
IN
BONNEVILLE POWER ADMINISTRATION AREA

Location	Date of Installation	Agency
Bellevue, Wash. (Pac. State Life Ins. Co.)	2/11/72	Seismo. Field Survey
Renton, Wash. (Williams and Swanson Bldg.)	2/14/72	Seismo. Field Survey
Port of Tacoma, Wash.	2/14/72	Seismo. Field Survey
Tukwila (Tukwila Park)	2/14/72	Seismo. Field Survey
Everett (County Courthouse)	2/15/72	Seismo. Field Survey
Seattle (Johnson Hall, Univ. of Wash.)	2/15/72	Seismo. Field Survey
Seattle (Seatac C)	1/12/71	Seismo. Field Survey
Dworshak Dam, Idaho	9/ 9/71	Seismo. Field Survey
Seattle (West Seattle High)	6/24/70	Seismo. Field Survey
Seattle (Queen Ann High)	6/25/70	Seismo. Field Survey
Seattle (North Beach Elementary)	6/25/70	Seismo. Field Survey
Cascade Tunnel, Wash. (Stevens Pass)	1970	Univ. of Washington
Seattle (Seattle First - D)	6/18/69	Seismo. Field Survey
Seattle (Group Health)	6/18/69	Seismo. Field Survey
Portland State Building	6/13/69	Seismo. Field Survey
Green Peter Dam, Oregon (crest)	Prior to '68	Seismo. Field Survey
Green Peter Dam, Oregon (abutment)	Prior to '68	Seismo. Field Survey
Olympia, Wash. (Highway test. lab)	Prior to '68	Seismo. Field Survey
Ross Dam, Wash. (crest)	Prior to '68	Seismo. Field Survey
Ross Dam, Wash. (Right bank)	Prior to '68	Seismo. Field Survey
Seattle (Federal Office Building)	Prior to '68	Seismo. Field Survey
Seattle (Port of Seattle, Dry Commodity Bldg.)	Prior to '68	Seismo. Field Survey
Seattle, Wash. (First Nat'l. Bank, 4825 Rainier Avenue, S.)	Prior to '68	Seismo. Field Survey
Tacoma, Wash. (County City Bldg.)	Prior to '68	Seismo. Field Survey
Hungry Horse Dam, Montana	Prior to '68	Seismo. Field Survey
Butte, Montana (School of Mines)	Prior to '68	Seismo. Field Survey
Bozemann, Montana (State College)	Prior to '68	Seismo. Field Survey
Hanford, Wash. (accelerometer array)	Prior to '68	Battelle, N. W.
Seattle, Wash. (Univ. of Wash. Crew House)	1965	Univ. of Washington
Seattle, Wash. (Univ. of Wash. Geology)	1965	Univ. of Washington
Hanford, Wash. (N Reactor)		ARHCO (AEC)
Hanford, Wash. (6-station array, waste disposal site)		ARHCO (AEC)
V. A. Hospitals and Corps of Engineers Dams (25 stations currently being installed)	1972	Seismo. Field Survey

R-7211-13-2859

APPENDIX E-1

SEISMIC REGIONALIZATION STUDIES
BONNEVILLE POWER ADMINISTRATION SERVICE AREA

A Report Submitted to Agbabian Associates
by

Shannon & Wilson, Inc.
Soil Mechanics and Foundation Engineers
1632 S.W. Morrison Street
Portland, Oregon

October 1972

SEISMIC REGIONALIZATION STUDIES
BONNEVILLE POWER ADMINISTRATION SERVICE AREA
WASHINGTON, OREGON, IDAHO AND
WESTERN MONTANA

1. INTRODUCTION

Studies of the regional seismic conditions were made in the area of the electrical distribution system of the Bonneville Power Administration. The Service Area concerned includes the states of Washington, Oregon, Idaho and the western part of Montana. The objective of the study was to prepare regional seismic maps which could be used to evaluate the anticipated intensity of ground motions caused by earthquakes. This report presents the results of the studies together with our conclusions and recommendations.

1.1 SCOPE OF INVESTIGATION

The investigation consisted of compilation and analysis of geologic and seismic data. The geologic studies consisted primarily of compilation of available geologic and physiographic information from which regional maps were constructed. The seismic investigations consisted of a review of theoretical earthquake data, compilation of historical earthquake data, study of worldwide tectonics as it is related to the northwest states, construction of curves showing earthquake attenuation and recurrence - frequencies and development of seismic intensity maps. The results of the study include:

A. A physiographic map of the BPA Service Area. The map shows the major physiographic provinces of the region including the mountain ranges, lowlands, plateaus, inter-mountain basins and basin and range areas.

B. A geologic map of the BPA area. The map shows the distribution of the major petrologic units.

C. A map of the epicenters of the historical earthquakes within the BPA Service Area. The map indicates magnitudes in four intervals.

D. Estimates of the likely maximum magnitudes of earthquakes that can effect the BPA Service Area. Likely maximum earthquakes are postulated for the western Montana, Puget Sound, Gorda Basin, Portland and vicinity and Umatilla-Walla Walla areas.

E. Curves showing the attenuation of seismic energy for the BPA Service Area. Curves applicable to earthquake sources in Puget Sound, west of the Cascade Range, southwest Oregon, Basin and Range area and east of the Cascade Range are presented.

F. Estimates of the areas of occurrence of the likely maximum earthquakes.

G. Maps of the seismic intensities anticipated from earthquakes occurring in and about the BPA Service Area.

H. A composite map of seismic intensity in the BPA Service Area, showing the maximum anticipated intensities from all likely earthquakes occurring in and about the area.

The report is presented in three sections. The first is introductory. The second section is a summary of the physiography, master geologic units, structural history including Pleistocene and Holocene tectonics and the apparent relatioship of earthquake epicenters and geologic provinces. The third section deals with the present concepts of the origin of earthquakes, earthquake definitions, relations and estimations, historical earthquake data and theoretical plate tectonics. Also, this section includes a description of the techniques used for the determination of the maximum likely earthquakes, the development of curves for seismic energy attenuation and the preparation of seismic intensity maps. It includes a list of earthquakes that are suggested for consideration in the selection of design earthquakes for the BPA Service Area.

The introductory and second sections of the report were prepared by Robert J. Deacon of Shannon & Wilson, Inc., and the third section was prepared by Dr. Richard Couch, Oregon State University.

1.2 APPROACH TO SEISMIC ZONING

Historic earthquakes within the BPA Service Area indicate that seismic intensity levels range from high levels in the Puget Sound area and in the disturbed belt of western Montana and eastern Idaho to lower, but significant, levels at places in the region between these two areas of high levels. This study is regional in scope and is designed to provide an approach to seismic zoning. Within a background of the geology and tectonics, the report describes the seismicity of the region as it is indicated by the historic earthquakes and the maximum earthquakes likely to occur. Previous attempts at seismic zoning of the northwest states (Richter, 36, Algermissin, 2)* considered only the historic seismic record with no attempt to adjust seismic zone boundaries to geologic or physiographic boundaries.

In this regional investigation no attempt is made to evaluate ground response produced by earthquake intensities. At each particular place, the complexity of the geologic environment profoundly influences the effect of earthquake ground

*Numerals in parentheses refer to corresponding items in the list of references cited.

motion; consequently, the evaluation of each building site requires a detailed knowledge of subsurface conditions, distance from the source of energy release, extent of the source mechanism and character of the fault that produces the earthquake (length of fault, type of rupture - dip-slip, strike-slip, etc.).

The regional scope of this investigation limits it to general guidelines. It formulates seismic zones that are somewhat arbitrarily and conservatively drawn. The design basis for seismic risk at any specific location will continue to depend on detailed investigation of conditions of the site and on the local earthquake potential. The guidelines provided by regional studies will be subject to modifications based on detailed site investigations.

2. GEOLOGY

2.1 INTRODUCTION

As a background for seismic studies, geologic and physiographic maps were compiled for the BPA Service Area. The maps appear as Figure E1, Physiographic Province Map and Figure E2, Geologic Province Map. The physiographic map was constructed using information from Fenneman (18) and Dicken (14) with some modification of their province boundaries. The map shows the major physiographic compartments of the region including the mountain ranges, lowlands, plateaus, intermountain basins and Basin and Range areas.

Geologic information for the geologic map was obtained primarily from the Tectonic Map of North America (King, 28) and from state geologic maps included in the U.S. Geological Survey Mineral and Water Resource reports for Montana 1963 (47), Idaho 1964 (46), Washington 1966 (48) and Oregon 1969 (49). The geologic map shows the distribution of the twelve major rock units in the region. A correlation chart, Figure E3, summarizes the various physiographic provinces in which the major rock units occur.

The quality of the geologic mapping in the regions is highly variable; the accuracy ranges from detailed mapping of mineral deposits to large scale reconnaissance mapping. It is estimated that not more than 25 percent of the region is mapped on one inch to the mile topographic base maps, which is about the minimum scale of mapping to identify faulting. Reconnaissance mapping, however, is adequate to show the major rock units and the major structural systems. However, in many areas, stratigraphic detail is unknown and information on faulting is lacking.

Also, the quality of information on the geologic structure of the region is variable. Geologic mapping of the overthrust faulting in the disturbed belt of western

Montana is probably well done; in other areas, the quality of mapping at reconnaissance scales ranges from fair and poor in the central part of the region to poor in western Oregon and Washington. In large areas (principally Cascade Range and eastern Snake River Downwarp), faulting and geologic structure is masked by undeformed Quaternary volcanics; also, broad areas of alluvial deposits cover much of the structure and geology in the Puget Sound area, Willamette lowlands and intermountain basins. The density of the faulting indicated on a map may be a combination of actual faults observed and the training, judgment and philosophy of the geologist conducting the mapping at his prescribed scale. Hence, in some areas, faults are overmapped and conversely, in other areas, a few larger faults are indicated where numerous small faults may be present. Faults were not indicated on the geologic map, Figure E2, because of the complexity of evaluating some of the local fault systems.

2.2 SUMMARY OF GEOLOGIC AND PHYSIOGRAPHIC PROVINCES

The region contains major mountain ranges, plateaus, Basin and Range and lowland provinces in the northwestern United States. The physiographic provinces stand in sharp topographic contrast to each other and are underlain by various rock sequences. The mountain systems (from west to east) include the Coast Range and the Alpine-like Olympic Mountains; the Cascade Range (with high peak volcanoes and cones) of western Oregon and Washington; the mountains of central Oregon; the Okanogan Highlands of northern Washington; and the rugged middle and northern Rocky Mountains with the intermountain basins of central Idaho and western Montana and Wyoming. Between the Rocky Mountains and Cascade Range, plateaus cover nearly 100,000 square miles in central Oregon and Washington and southern Idaho. Within the plateaus are numerous structural basins. Major structural lows are present in the Willamette-

Puget lowland of western Oregon, in the Snake River Downwarp in southern Idaho and in the Wyoming basin, which is a broad undulating plateau in western Montana and Wyoming. Basin and Range topography, consisting of sharp fault block mountains separated by basinal lowlands are present in south central Oregon and Idaho. Figure E1 indicates the physiographic provinces of the region.

Rock sequences range in age from the Cambrian metamorphics to Holocene volcanics and glacial-alluvial deposits. The major groupings include Pre-Cambrian (Proterozoic) metamorphics and intrusives, Paleozoic and Mesozoic geosynclinal deposits, Mesozoic batholith (granite) implacements, Paleozoic and Mesozoic platform deposits. Tertiary rock sequences consist of marine and nonmarine deposits and Miocene and Pliocene flood basalt. Quaternary lavas, volcanoes and cones comprise much of the Snake River Downwarp and the high parts of the Cascade Range. The distribution of the major rock groupings is indicated on Figure E2.

Pre-Cambrian rocks consist of strongly metamorphosed middle Proterzoic sedimentary and volcanic rocks which include the Belt Series in eastern Washington, northern Idaho and western Montana. These rocks occur within the northern Rocky Mountain physiographic province. The rocks are intensely folded and faulted with lateral fault displacements of up to 12 miles. They were involved in major thrust sheets in the active Disturbed Belt of western Montana. Some of the faults of this Disturbed Belt show evidence of comparatively recent movement; the 1959 earthquake at Madison indicates continued structural activity to present times (U.S. Geological Survey, 46).

Paleozoic geosynclinal deposits include mainly Devonian to Permian marine sedimentary and volcanic rocks. In places Mesozoic geosynclinal deposits overlie the Paleozoics. The Paleozoic rocks are mostly metamorphosed in eastern Idaho, western Montana and northern Washington, in the Rocky Mountain and Okanogan physiographic provinces. These rocks have been deformed in Paleozoic, Mesozoic and Tertiary orogenies. Evidence of tectonic activity in this rock sequence since early Tertiary is generally lacking except within the Disturbed Belt.

Mesozoic geosynclinal deposits consist principally of partly metamorphosed Triassic and Jurassic marine and volcanic rocks. They occur in southwest and eastern Oregon and on Vancouver, Island, B.C., Canada. They were deformed mainly by the middle Jurassic Nevadian orogeny. Recent tectonic movements and earthquakes in the Blanco fault zone off the coast of southwest Oregon indicate offshore activity and one earthquake at Cape Blanco indicates present tectonic activity on or near shore.

Mesozoic granitic plutons include the batholic emplacements during the Mesozoic Nevadian and Laramide orogenies. The major granitic bodies include the Idaho batholith, the Boulder batholith in Montana and the Chelan, Similkameen, Colville, Loon Lake and Mt. Stuart batholiths in northern Washington. The granite bodies are present in the following physiographic provinces: Northern Rocky Mountains and Owhyee Upland in Idaho, the Okanogan Highlands and Northern Cascade Range in Washington and Klamath Mountains of southwest Oregon.

Platform deposits consist of Paleozoic and Mesozoic deposits on the Pre-Cambrian basement in the central plains (Wyoming Basin physiographic province) of western Montana and Wyoming. The deposits are near horizontal and gently tilted east of the Disturbed Belt; they were involved in complex overthrust faulting in the Disturbed Belt.

Nonmarine deposits include thick Cretaceous-Paleocene sequences of sedimentary rocks in the Bellingham (Chuckanut) and Swauk structural basins in northwest Washington. The deposits occur in the Northern Cascade Range and underlie the Puget lowland north and west of Bellingham. Narrow tight fold belts with parallel faulting are characteristic.

Marine deposits in western Oregon and Washington include eugeosynclinal deposits consisting of marine sediments interlayered with submarine lava and volcanic rocks. Nonmarine sediments (Puget Group and Chuckanut Formation) on the western flank of the Cascade Range in Washington are also included. The rock sequences are folded; they occur in the Coast Range and Northern Cascade Range physiographic provinces.

In western Oregon structural trends in Tertiary rocks are predominantly northeast-southwest and in northwest Oregon and western Washington, the trend swings to northwest-southeast. Faults are primarily normal and are present in numerous localities. Most of the faults have surface traces less than five miles long and no master fault system is known. The deformations were mainly of middle to late Tertiary age or accompanied the north-south warping of the Cascade Range and Coast Range in the early Quaternary. Uplift of Pleistocene terraces along the coast indicate vertical uplift of a hundred feet or more since late Pleistocene (12-15,000 years) and the present cycle of coastal erosion is one common to an emerging coast.

Volcanic rocks in the Cascade Range include early to late Tertiary andesitic and basaltic lava with volcaniclastic rocks. The earlier rocks occur mainly in the western Cascade Range physiographic province. These rocks are folded and faulted; fault traces are generally less than five miles. Deformation was mainly middle to late Tertiary but these rocks were also involved in the Cascade Range uplift in early Quaternary.

Plateau volcanics include middle to late Tertiary basaltic lava and volcaniclastic rocks in the Columbia Plateaus physiographic province of central Oregon and Washington, and western Idaho and the Basin and Range province in southern Oregon and Idaho. Deformation includes folding and faulting in central Washington, broad gentle folding with rare faulting in northern Oregon, numerous normal faults with small displacement in central Oregon and western Idaho and sharp Basin and Range fault block structures in southern Oregon and Idaho. Deformation was mainly late Tertiary (10.0 m.y.) to middle Pleistocene (0.5 m.y., million years before present).

Marine deposits in structural basins include early to late Tertiary sedimentary rocks in the Willamette structural basin in western Oregon and the Puget structural basin in northwest Washington. Deep well information indicates the thickness of sedimentary rocks in the Willamette basin is about 7500 feet and in the Puget basin from 12,000 to 25,000 feet (20,000 feet of Oligocene in the Clallam syncline alone). These rocks occur in the Willamette lowland and Puget lowland physiographic provinces and are folded and faulted. Faults are largely normal. Concealed faults with an indicated trace of more than 25 miles may be present in the Willamette basin near Portland and the Puget lowland near Seattle. Elevated surfaces of glacial deposits in the Puget Sound region (Couch, in press) indicate recent movement in this area.

Nonmarine sedimentary rocks in structural basins occur in central Washington, south central Oregon, southern Idaho and the intermountain basins of western Montana. The deposits are principally Pliocene and Pleistocene in Washington, Oregon and Idaho, and Oligocene and Miocene in the western Montana basins. They occur in the following physiographic provinces: Columbia Plateaus, Basin and Range, Snake River Downwarp and northern Rocky Mountains. Most of the basins are downdropped blocks bounded by normal faults, but some are broad structural lows without apparent faulting. Structural deformation of the basins of Oregon, Washington and Idaho occurred in late Pliocene to middle Pleistocene

(0.5 m.y.). Evidence of late Pleistocene movement is indicated on the fault system at the southwest side of the Pasco basin near Wallula and on faults in the Basin and Range province of southern Oregon and Idaho.

The youngest Quaternary volcanic rocks include lava and ash, parts of which were deposited throughout the Pleistocene; Holocene volcanic rocks less than 10,000 years old are locally present. The deposits occur in the following physiographic provinces: The High Cascade Range, Western Cascade Range, Columbia Plateaus (near Bend), southwest Idaho and the eastern part of the Snake River Downwarp in southeast Idaho and northwest Wyoming (Yellowstone National Park).

2.3 SUMMARY OF PLEISTOCENE AND HOLOCENE TECTONICS

The Disturbed Belt in western Montana is the only continental master fault system with evidence of Holocene movement in the BPA Service Area. Structural development of fault-controlled late Pliocene and Pleistocene basins in the Columbia Plateaus and the Basin and Range physiographic provinces indicate tectonic movements up to mid-Pleistocene (0.5 m.y.). Sharp topographic scarps and lineations together with hot springs indicate active tectonism on some faults in the Basin and Range province of southern Oregon and the Grande Ronde graben of northeast Oregon. Holocene volcanism in the High Cascade Range and in the eastern part of the Snake River Downwarp indicate some adjustments were still taking place within the last 2,000 years.

2.4 EARTHQUAKE EPICENTERS AND GEOLOGIC PROVINCES

All of the epicenters of historical earthquakes within the BPA Service Area are shown on Figure E4, Earthquake Epicenter Map. The map indicates earthquake magnitudes at four ranges:

1) All reported with magnitude ≤ 3.7, 2) magni-

tude $>$ 3.7 and $\leq$ 5.0, 3) magnitude $>$ 5.0 and $\leq$ 6.3 and 4) magnitude $>$ 6.3, as indicated.

From the clustering of most of the epicenters, it is apparent that meaningful comparisons can be made with geologic structure in some areas and in other areas a geologic source of the earthquake is obscure or unknown.

The concentration of epicenters in the Rocky Mountains of eastern Idaho and western Montana is obviously related to a north-south belt of structural disturbed rocks. The other main concentrations of epicenters are in the Puget Sound region of western Washington and Willamette lowland of northwest Oregon. Because of the alluvial cover in both of these western areas, earthquake producing structures are poorly defined. However, in both areas, structural basins are present and geologic information indicates some faults are present.

The one large historic earthquake near Port Orford on the Oregon coast is apparently related to the off-shore Cape Blanco fault system. A cluster of epicenters between Ashland and Medford indicate known fault sources, but other scattered onshore earthquakes of low intensity in the region cannot be related to known structural features. Epicenters scattered in the western and northern Cascade Range do not appear to be related to known faults but the geologic mapping in that area is mostly of reconnaissance quality and not adequate to define possible earthquake sources.

Clusters of epicenters in the Columbia Plateaus and Basin and Range physiographic provinces indicate local faults as the earthquake sources. Epicenters near Klamath Falls and in the Warner Valley both of the Basin and Range province in southern Oregon, indicate active tectonics; however, other large faults in that province, such as those along Albert Rim and Steens Mountain, have no historic earthquake activity. In the Columbia Plateau centering at Walla

Walla, epicenters along the eastern end of the Horse Heaven-Rattlesnake Hills structure indicate local activity on faults, but similar youthful faulting in the Grande Ronde Graben at La Grande, Oregon has little indicated earthquake activity.

Areas of relatively low seismic activity occur in central and southeastern Oregon, southwestern and north central Idaho and in north central Montana. However, part of the low count of earthquakes in these areas may be due to lack of reports from sparse rural populations.

3. EARTHQUAKE STUDIES

3.1 EARTHQUAKES (DEFINITIONS, RELATIONS)

Sudden releases of energy that occur to depths of 750 km in the earth cause earthquakes. These energy releases produce displacements at the source, presumably as offsets along one or more surfaces. The earthquake focus or hyop-center is the point of initial displacement at the source and the earthquake epicenter is the point on the earth's surface vertically above the focus. Earthquake focal depths are classified as shallow or normal when the depth is less than 70 km, intermediate for foci between 70 and 300 km, and deep when the focal depth is greater than 300 km. All reported earthquake focal depths for the Pacific Northwest are shallow. Most investigators consider earthquakes to result from a release of accumulated strain energy in the earth and that strain accumulation in shallow earthquakes is dependent essentially on rock strengths.

Intensity, magnitude and energy are used to characterize the severity of an earthquake. Of these quantities, seismic energy, the elastic wave energy that is radiated from the source, is the most significant, but it is difficult to determine. Empirical equations have been established (Richter, 35) to approximately relate intensity with magnitude and magnitude with energy.

Intensity describes the amount of shaking or damage at a specific location; it is generally highest in the epicentral region and decreases away from the region. Intensities are based on observed or felt effects of the earthquake. Intensities range on the Modified Mercalli Scale (1956 Edition), abbreviated M.M., from intensity I which is not felt except under very favorable circumstances to intensity XII in which damage is nearly total (Richter, 35).

Magnitude is a rating that is essentially independent of the point of observation and characterizes the amount of energy radiated from the source of an earthquake. Magnitudes are based on instrumental observations and range on a logarithmic scale from less than 1 for small shocks to over 8 3/4 for the largest earthquakes. In the United States three magnitude scales are in common use: M_L determined from records of local earthquakes, M_S determined from the amplitudes of surface waves for shallow teleseisms and M_B from the amplitude/period ratio of body waves for teleseims of shallow and deep-focus earthquakes (Gutenberg and Richter, 20, 21, and Richter, 35). The empirical relations between the magnitude scales are:

$$M_B = 2.5 + 0.63\ M_S$$

$$M_B = 1.7 + 0.8M_L - 0.01M_L^2$$

The National Oceanic and Atmospheric Administration presently reports magnitudes in the M_B scale. All magnitudes used in this report are in units of the M_B scale. The empirically determined equation $M_B = 1 + 2/3\ I_o$ (Gutenberg and Richter, 20) is used in this report to relate magnitude and intensity. The intensities observed during an earthquake of a given magnitude in addition to being dependent on the near surface earth materials, are strongly dependent on focal depth. The empirical relation of Richter (35) was obtained from observations of earthquakes in California where focal depths are believed to be approximately 16 km. Consequently, for a given magnitude, an earthquake which occurs at a depth less than 16 km may have relatively higher intensities and an earthquake occurring at a greater depth will generally exhibit lower intensities.

The severity of earthquakes in the Pacific Northwest was generally indicated by an assigned intensity prior to 1960 and an assigned magnitude after 1960. To permit more quantitative analyses the intensities of the historic earthquakes in this report are converted to magnitudes. The final results of the analyses, however, are converted to intensities for engineering evaluation.

The energy, E, of an earthquake may be calculated using the relation $\mathrm{Log}_{10}E = 5.8 + 2.4M_B$ (Richter, 35). The unit of energy E is ergs. An estimate of an earthquake's energy also may be obtained from the intensity by using the above empirical relationship between magnitude and intensity.

The time, place or amount of energy released from an earthquake cannot be predicted, although research on prediction is not being pursued in several countries. Statistically, earthquakes are found to occur along belts or zones and not commonly elsewhere. The earthquake activity for large regions over long periods of time, such as 100 years, can be estimated sensibly based on past history. However, it cannot be predicted that a locality, even in an active belt, will experience an earthquake of substantial magnitude within the next several years. Conversely, no region can be considered exempt from earthquakes at any time, even from those of the larger magnitudes.

It is not the purpose of this study to predict earthquakes but rather to provide only an estimate of the likely maximum seismic excitation levels to be anticipated in the Bonneville Power Administration Service Area. Because of the limitations in the historical data and the lack of a well defined technique of estimation the results of such a study tend to be conservative.

3.2 EARTHQUAKE DATA (HISTORICAL RECORDS, INTENSITY OBSERVATIONS)

Historical reports of earthquakes, including both those observed by the general populace and those instrumentality detected, form the major part of the data for this study. In addition, consideration was given to appropriate geological and tectonic studies of the region and applicable information on world earthquakes and related tectonism. This geologic and tectonic information was incorporated either directly or by analogy.

Earthquake summaries by Townley and Allen (44), Berg and Baker (8), Rasmussen (34), Wollard (53), Couch and Lowell (11), reports of the Department of Commerce (United States Earthquakes 1928 through 1970 ,45, and Earthquake Reports of the USC & GS)and hypocenter data from the National Geophysical Data Center of the National Oceanic and Atmospheric Administration (16) provide the historical earthquake data for this report. A meaningful earthquake history of the region extends back only to the late 1800's. This period is insufficient to establish directly either the largest magnitude of earthquakes to be expected or the frequency of occurrence of lesser shocks. The earliest reports extend from 1833 (Townley and Allen, 44) but are clearly dependent on the size and distribution of the populations.

Between 1833 and 1962 most earthquakes which occurred in the area were located by the felt effects reported by the people near the epicenter. Because the observed intensities are dependent on near surface geology and the location and distribution of the observers (people tended to settle in alluvium floored valleys), epicenters located in this way are uncertain and should not be assigned to a mapped fault without due consideration. From the late 1920's until 1962, some of the larger earthquakes of the region were located with seismographs of the University of California (which at that time included those at Corvallis) and the University of Washington. However, because of the distances to the epicenters, station limitations and uncertainties in travel-times, the instrumentally located epicenters of that period are probably as inaccurate as those estimated from the surface effects observed. However, the instrumental results during that period do suggest that no earthquakes greater than magnitude 5 passed unnoticed in the area.

Between 1960 and 1962 the USC&GS located a number of world wide seismograph stations in Washington, Oregon, Montana, Nevada and Utah. These stations together with a number of university-operated stations provided the ability to detect and reliably locate earthquakes in the region down to magnitude 3.5. Further, these stations provided data with which to determine the magnitude and focal depth of most observed earthquakes.

The accuracy with which epicenters can be located by triangulation involves some uncertainties in the arrival times of the seismic waves and in the velocities of the waves between the source and observing station. The larger earthquakes are observed at more stations and show the first waves more distinctly; hence, they are generally located with greater precision. Wave velocities, or transit times, between the source and observing station are obtained from travel-time curves. Jefferies and Bullen (27) and Gutenberg and Richter (20) have published global travel-time curves and Dehlinger, et al (13) have published local travel-time curves applicable to the Pacific Northwest.

An epicenter can be located by use of either the global travel-time curves or the local travel-time; for example, if an earthquake has been located in the Portland area by use of the global travel-time curves and arrival times at Blue Mountain and Tumwater, it would be approximately 5 to 6 km farther west-northwest by use of the local travel-time curves. This occurs because the local travel-time curves show an approximate 0.3 km/sec difference in wave velocity between the areas east and west of the Cascade Range, whereas, the global travel-time curves indicate a common velocity.

The difference in epicenter locations, computed with both the global and local travel-time curves, depends on the locations of the observing stations and on the location of the earthquake. A difference in epicenter location usually implies that there is also a difference in computed focal depth. The difference in the epicenter location cautions against assigning an observed earthquake to a particular fault; consequently, in this report we refer to fault or fracture zones or to seismically active areas.

Figure E4 is a map of the epicenters of earthquakes which have occurred in the Pacific Northwest. The earthquakes are indicated according to magnitude in four ranges: $M \leq 3.7$, > 3.7 and ≤ 5.0, > 5.0 and ≤ 6.3 and > 6.3. The sort interval of M = 1.3 was selected so the maps would include, in four increments, the range of magnitudes of historic earthquakes in the Pacific Northwest and would yield a four level intensity map of the BPA Service Area. It is likely that more earthquakes large enough to be felt occurred in the region but were not reported. However, it seems unlikely that earthquakes larger than M = 5 pass unreported within the past 100 years. Seismically active areas with earthquakes of up to magnitude M = 7.1 occur in the Puget Sound region of Washington and in northwestern Wyoming and southwestern Montana. Seismic areas of less areal extent with magnitudes up to M = 5+ occur in the vicinity of Portland, in the Basin and Range province of southern Oregon, along a general belt of minor seismicity extending southeast from Puget Sound to southern Idaho and in northeastern Washington and northwestern Montana. Relatively aseismic areas appear to occur in central and southeastern Oregon, in southwestern and north central Idaho, and in north central Montana. The aseismicity of these regions may in part be due to the lack of population-dependent observations.

The Earthquake Reports of the USC&GS provide considerable data on the observed intensities of a number of the larger earthquakes which have occurred in and about the BPA Service Area. Isoseismals drawn through hundreds of point observations of observed intensity clearly show both the level and pattern of the seismic excitation. The pattern depicted is dependent on the source mechanism, seismic energy attenuation within the earth and on the reaction of the near surface earth material to the seismic waves. The horizontal attenuation of the seismic excitation with distance is shown by the isoseismals.

Figure E5 shows the intensity IV isoseismals of the following earthquakes: Olympia, 1949; Hegben Lake, 1959; Crescent City, 1873; Kennedy, 1915, Portland, 1962; Milton-Freewater, 1936; and Grants Pass, 1906. The last two listed earthquakes and the Crescent City earthquake do not have published mapped isoseismals. The isoseismals drawn are estimated from the published reports of the observed effects of the earthquakes. Additional earthquakes in and near the study area yield isoseismals of intensity IV which essentially cover the area; this indicates that for the Bonneville Power Administration Service Area, an earthquake intensity M.M. IV is a minimum level to be anticipated.

3.3 TECTONICS (PLATE TECTONICS, TECTONISM IN THE NORTHWEST)

The present understanding of the cause of most of the world's earthquakes is contained within the concepts of plate tectonics (e.g., Morgan, 31; Isacks, Oliver and Sykes, 26; McKenzie, 30). The basic idea of plate tectonics is that the surface of the earth is divided into essentially rigid, undeformable plates that move relative to each other. The loci of interaction of these lithospheric plates are the main belts of earthquakes generation across the earth's surface. Deep sea trenches, such as the Aleutian and Peru-Chile trenches, and great thrust mountain ranges such as the Himalayas are interpreted as the result of plates colliding. Rift valleys, such as the East African rift and the Red Sea, and mid-ocean ridges are postulated loci of plate separation (divergence). Major strike-slip faults, such as the San Andreas fault in

California, the Fairweather in southeast Alaska and the large shear faults in New Zealand and the Philippine Islands, mark fracture zones where plates slide past one another.

Generally more strain energy is released per unit time per unit area in zones of thrusting or subduction than along fracture zones. The least energy released along plate perimeters occurs at diverging edges. Still less energy is released in plate interiors particularly in the ultrastable shield areas of continents. The reason for the difference in energy release along different interacting plate edges is unknown, but may be a function of the thickness of the lithosphere or the method of failure of lithospheric rocks during fracture; that is, whether in compression, shear or tension.

The kinematic theory of plate tectonics which describes the evolution of the earth's surface is well established. Although a number of processes have been proposed (for example, polar wobble, mantle convection currents or plumes and tidal friction), the driving mechanism of the lithospheric plates remains unknown. "The current absence of a detailed dynamic theory of plate motion in no way affects the use of the kinematic theory of plate tectonics" (McKenzie, 30).

Earthquakes in western North America are the result of, or caused by, the interacting motion of two large lithospheric plates, the North American and Pacific Plates. Off the Gulf of California, the locus of interaction of these two plates, as defined by earthquake epicenters, is associated with the East Pacific rise. North of the gulf, in southern California, the loci of earthquake epicenters appears to bifurcate (Barazangi and Dorman, 4). The western branch, associated with the San Andreas fault system in California, passes out to sea in the vicinity of Cape Mendocino. Between Cape Mendocino and the northern end of Vancouver Island, the locus of epicenters is associated with the ridge-rise-transform fault system which exists off the coasts of northern California, Oregon, Washington and Vancouver Island. From south to north the system is comprised of the eastern end of the Mendocino (or Gorda) escarpment, Gorda ridge, the Blanco fracture zone, Juan de Fuca ridge, Sovaco fracture zone, Explorer ridge and the Queen Charlotte-Fairweather fault system. The Fairweather fault system and an associated major fault, the Denali fault, extend into southern Alaska. The eastern branch of epicenters is less well defined, appearing as a broad zone or belt extending northward through Nevada, Utah, Wyoming and Montana. The eastern earthquake zone appears to either end in northern Montana or to be diffusely connected to the earthquake activity in the Puget Sound region. Oregon and parts of Washington and Idaho appear as a relatively quiet island between the two zones of plate interaction. The major strain release associated with the interaction of the two plates occurs off the coast of the Pacific Northwest, east of Idaho, south of Oregon and extends north-northwest from the Puget Sound region.

The tectonic situation of the region of relative seismic quiet within the active seismic belt is a moot question. Morgan (31) and Atwater (3) postulate that the small plate between Juan de Fuca ridge and the continental slope off Oregon, Washington and southern Vancouver Island is slowly underthrusting the continental margin, whereas Couch and MacFarlane (12) and Dehlinger, et al (13) postulate that the continental margin, and more generally the Pacific Northwest, is undergoing areal dilation. It is generally recognized, however, that the Pacific Northwest is not the site of major tectonic thrusting nor is it as inactive as the central area of a tectonic plate.

Focal depths of historic earthquakes in the region of study range from near surface to 57 km. Earthquakes occur as deep as 50 km both in the Puget Sound area and in Montana; however, most foci are between 10 and 25 km with the average depth of 272 earthquakes east of 114° W longitude being 13.5

km. West of 114^{o} and east of 121^{o}31' 16 focal depths average 17.8 km and west of 121^{o}31' 26 focal depths average 25.1 km. Because of the uncertainty in travel-time and consequent epicentral locations, these determinations of focal depths have large uncertainties. Attempts at better determination of focal depths generally yield shallower depths. No definite patterns are noted in the focal depths; however, most earthquakes occur in the crust.

Focal mechanism studies in the Pacific Northwest summarized by Dehlinger, et al (13) show that nearly all earthquakes occurring in the area of study are associated with either strike-slip or normal faulting or a combination of the two. In strike-slip faults the two alternative fault planes are usually right lateral oriented northwest-southeast and left lateral northeast-southwest. Couch and MacFarlane (12) and Dehlinger, et al (13) note that the minimum compressive stress vector remains oriented generally east-west and suggest the regional tectonic stress is tending to cause areal dilation of the Pacific Northwest. Several fault plane solutions in Puget Sound and in Idaho suggest that, at least locally, the stress pattern is more complex.

Considering the relatively low rate of release of strain energy in the area of study, the shallow foci and the focal mechanism solutions that indicate predominantly normal and strike-slip, the region appears more akin tectonically to a ridge-fracture zone area than either a major thrust zone area or the quiet of the internal plate. This suggests that the empirical relations between intensity and magnitude and the method of determining magnitude itself as worked out by Gutenberg and Richter (20, 21) and Richter (35) are, within their own limitations, applicable to the Bonneville Power Administration Service Area. Further, it suggests data for similar areas of the world may also be applicable to the area of study.

3.4 ESTIMATION OF "LIKELY MAXIMUM" EARTHQUAKE

A review of the earthquake reports of the USC&GS indicate the maximum intensities observed to date in the study area were caused by only a few large earthquakes in and about the area. Figure E5 shows the vast area that experienced intensity IV excitation from one or more of 7 larger earthquakes. Similar curves show that the intensity V isoseismals of the 1949 Olympia earthquake and the 1959 Hegben Lake earthquake alone enclose over one half the study area. This indicates the necessity of estimating a "likely maximum" earthquake or earthquakes for the study area and the immediate surrounding area and estimating their possible locations.

A review of the historical earthquakes provides a first estimate of the seismicity of the area. Figure E4 shows that the earthquakes of the area are not spaced uniformly or randomly over the region but occur mostly in local concentrations. For example, the Puget Sound region is the location of many epicenters, whereas epicenters in central Oregon are nearly absent. The groupings of epicenters indicate seismically more active areas, presumably areas in which deformation is more active. The areas of greater seismicity show a diminishing density of epicenters away from their centers. In addition, the larger earthquakes of the more seismically active areas are located near the centers, i.e., the earthquake magnitudes and consequently the release of strain energy tend to decrease away from the center of a seismic area until the regional background or ambient seismic level is reached. Consequently, the boundaries of the more active seismic areas are not well delineated.

A direct consideration of the historical data for a seismic area yields a first approximation on the energy release and the largest earthquake(s) to be anticipated. It should be noted that had an estimate been made in 1948 of the anticipated seismic activity in Puget Sound based strictly on the size of historical earthquakes, both the 1949 Olympia earthquake and the 1965 Tacoma earthquake would not have been expected to occur. Similar inadequacies indicate additional types of analysis of the historical data or different techniques are need to obtain a more realistic estimate of the magnitude of the likely maximum earthquake.

The equation: $\mathrm{Log}_{10} N = A - bM$ represents the general distribution of earthquakes for a given region over the observed range of magnitude (Richter, 35). Figures E6, E7, E8, E9 and E10 show the curve of the form $\mathrm{Log}_{10}N = A - bM$ fit to the available historical earthquakes of the seismic regions of western Montana, Puget Sound, the Gorda Basin, Portland and vicinity and the Umatilla-Walla Walla area, respectively. The interval of historical data on which the curves depend extends from 1861 through 1971. The horizontal bars on the data reflect the computer sort interval. Points without bars are exact reported magnitudes. On the vertical axis of the graph the number of earthquakes greater than a given magnitude is divided by the sample interval (e.g., 110 years) to yield a recurrence rate.

Examination of the curves shows that the historical recurrence rate of the earthquakes of smaller magnitude is less than predicted; the difference increases as the magnitude decreases. The decline in the historical recurrence rate at the low end of the magnitude scale is attributed to a lack of detection, reporting or recording of the lesser historical earthquakes, particularly in areas of low population. Divergence of the historical data from a straight line at low magnitudes indicates that observations of earthquakes which exhibited intensities of less than approximately V ($M_B = 4.5$) cannot be used to constrain the recurrence curve along the magnitude axis. At the high magnitude end of the scale $M_B=6$ the recurrence rate is low, consequently, the curve is not well constrained by the availalbe historical data at the high magnitude end. This indicates that the slope and position of the recurrence curve is constrained primarily by earthquakes with magnitudes between 4.5 and 6.

The value b in the equation $\text{Log}_{10} N = A - bM$ is the slope of the earthquake recurrence curve. Values of b determined for other areas of the world vary from 0.6 to 1.5 (Gutenberg and Richter, 19; Isacks and Oliver, 25; Sykes 41; Brazee and Stover, 10; and Lammlein, Sbar and Dorman, 29). However, when large numbers of earthquakes are used to determine a frequency curve the range of b values decreases; most occur between 0.8 and 1.1. Sykes (41) obtains a b value of 0.91 based on 267 earthquakes occurring in the Arctic. The region of his study is predominantly a ridge-transform fault area with focal mechanisms and focal depths similar to those occurring in the Pacific Northwest. Hence, the value b = 0.91 is adopted from the world data to additionally constrain the earthquake frequency curves applicable to the study area. In summary: A slope of 0.91 is adopted from world data to constrain the earthquake frequency curves in this report. The position of the curve on the magnitude axis is constrained by historical earthquakes of magnitude 4.5 or greater which occurred in the appropriate seismic area.

To arrive at an estimate of a likely maximum earthquake the projection of the earthquake frequency curve at the high magnitude end may be terminated with a vertical line or a horizontal line; that is, by magnitude or by time (recurrence interval). A magnitude limitation for shallow earthquakes is related primarily to the strength of the crustal or lithospheric rocks whereas the time limitation is dependent on strain accumulation rates. This suggests two different methods of arriving at a likely maximum earthquake. Clearly, however, the two methods are not independent and should arrive at the same likely maximum earthquake.

Studies of the parameters of earthquake sources, in terms of earthquake volume and strain (e.g., Bath and Duda, 6; and Duda, 5) and dislocation theory (e.g., Savage and Hastie, 39), have yielded estimates of source dimensions. Scholz, Wyss and Smith (40) explained their observations of motion on the San Andreas fault in terms of stick-slip and stable frictional sliding and provided crustal models to explain the variations in fault creep with space and time. Studies of source dimensions (Toucher, 43; Press, 33: Bonilla, 9) provide a relation between earthquake magnitude and fault length.

These and similar studies of dimensions and mechanisms of the source regions may provide a method of estimating the maximum earthquakes likely to occur along a fracture zone and a possible time of occurrence. However, it must be recognized, that the results obtained to date generally apply to a well instrumented and mapped area of California; in particular to the San Andreas fault, a right lateral strike-slip fault. This approach, although a promising method, is difficult to apply to the Bonneville Power Administration Service Area for the following reasons:

1. The depth to a plastic or low velocity zone is unknown in the Pacific Northwest.

2. Strain or strain rates are unknown.

3. Tectonic mechanisms are not well defined.

4. Faults are not adequately mapped.

5. The type of fault displacement (dip-slip, strike-slip, etc.) or age of movement on mapped faults is not well defined.

6. Focal mechanisms indicate that normal and possibly thrust faulting occur in addition to strike-slip faulting, for which source parameters are less well known.

7. In the Puget Sound region, in the area about Portland,

and in other broad alluvial areas, the postualted faults are covered with alluvium.

8. In the High Cascades, eastern Snake River plain and at Yellowstone, Quaternary volcanics cover the tectonic fabric and the probable faults.

Because of the above problems the direct applicability of such investigations to the BPA area without considerable additional research, it is elected to estimate the occurrence of the larger magnitude earthquakes based on recurrence rate or earthquake frequency.

The establishment of a recurrence frequency for large earthquakes implies a constant strength of the crustal or lithospheric rocks and a constant rate of strain accumulation. Application of a constantly increasing stress (or constant rate of strain) to a constant strength lithospheric element should yield maximum earthquakes at a well defined recurrence frequency. This could be considered a geo-seismic relaxation oscillator whose period is proportional to rock strength and strain rate.

Crustal and subcrustal rocks of a region, whether they yeild as brittle fracture or stick-slip can be considered, to a first approximation, to have a constant strength. Superimposed on the inherent rock strength are variations caused by rock heterogenities, changes in pore pressures, temperature and other factors of both local and distant origin.

Interpretation of oceanic magnetic anomalies (Vine, 50; Pitman and Heirtzler, 32) indicates that the relative movement of lithospheric plates is approximately constant over time spans of millions of years and, hence, tectonic stress or strain deformation is approximately constant in the zones of interaction between plates. Benioff (7) shows global strain release to be approximately constant over historical periods. Further, the frequency-magnitude curve (Richter, 35) implies that, for a given seismic area, a frequency of occurrence is predictable for all magnitude earthquakes up to and including the largest observed, i.e., the earthquake frequency curve itself implies a constancy of strain application assuming lithospheric elements of constant strength. Superimposed on the constant strain accumulation are fluctuations caused by tidal loading, denudation, polar wobble, world strain changes due to large distant shocks and other similar factors. However, lack of correlation between these events and earthquake occurrences suggest these strain effects are relatively small.

The above observations suggest that for a given seismic area there is a period of time over which application of constant rate of increase of strain will cause a lithospheric element of approximately constant strength to yield. Rock strength variations and fluctuations in strain cause this interval to vary somewhat (or to be statistical). It is proposed then, to determine a reasonable time interval for the study area during which the strain energy can accumulate to the maximum the rocks will sustain and thereby cause a "likely maximum earthquake".

Gutenberg and Richter (19), Richter (35), Isacks and Oliver (26), Sykes (41) and Lammlein, Sbar and Dorman (29) estimated recurrence rates or earthquake frequencies for selected seismic areas. Estimates of the time interval between the largest earthquakes for limited seismic areas range from 50 to 90 years for California to 250 years for southeastern Missouri. It is generally recognized that the Pacific Northwest is seismically less active (in number of earthquakes) than California but more active than Missouri, hence the recurrence interval for the largest earthquake is between these two estimates. The seismic history of the area which indicates one magnitude 7.1 in Puget Sound and one magnitude 7.1 in western Montana suggests the recurrence interval is longer than 100 years, but not much longer, for these earthquakes (which may be the largest attainable). Sykes (41) indicates that for an area of the Arctic a well defined recurrence curve indicates an interval of 16 years for M = 7 and 130 years for M = 8. He notes that shocks of M = 7 occur as predicted but no shocks of M = 8 are observed. This suggests that for his region of study the interval of 130 years is long enough to include the largest earthquakes which occur. The area of Sykes' study, although largely oceanic, is an area of ridge-transform fault tectonics, a region of dilation and an area of shallow focus earthquakes similar in seismicity to the Bonneville Power Administration Service Area. The area of seismicity within the area studied by Sykes is approximately the same as the BPA study area (2.2×10^6 km^2 vs. 1.6×10^6 km^2). Consequently, the 130 year interval is selected as a first approximation to the recurrence interval and is used in this report to estimate the magnitude of the largest earthquakes to occur in the seismic areas within the BPA Service Area.

It should be noted here that the 130-year interval is not intended to predict the recurrence of one large earthquake for a given area each 130 years, but this interval is presumed long enough that strain energy sufficient to produce the largest quake can accumulate and be released. In other words, the maximum size earthquake to occur in a given area can be smaller than the likely maximum, but it is not likely to be larger.

Figures E6, E7, E8, E9 and E10 show the historical data of the seismic areas of Montana, Puget Sound, Gorda Basin, Portland and vicinity and Umatilla-Walla Walla area, respectively. A line with a slope of b = .91 constrained by the historical data and extended to a 130 year recurrence interval yields the following likely maximum magnitudes: Montana M = 7.4, Puget Sound M = 7.4, Gorda Basin M = 8.0, Portland and vicinity M = 6.5, Umatilla-Walla Walla area M = 6.5. Extending the recurrence interval from 130 years to 150 years increases the magnitude of the likely maximum earthquake approximately 0.1 M. This reemphasizes the observation that the magnitude of the likely maximum earthquake is determined primarily by the historical earthquakes in the mid-magnitude range.

Earthquakes in eastern Idaho, western Montana and northwestern Wyoming (see Figure E4) show how much the frequency curve (Figure E6) is dependent on the population. Data obtained during the 10-year period following the installation of the WWSS Stations shows a very good fit to the plotted curve down to magnitude $M_B = 3.7$.

The earthquake frequency curve for Puget Sound (Figure E7) also shows how much the historical record is dependent on the population, especially at the low magnitude end; however, it is noted that data obtained during the period 1961 through 1971 also shows the same divergence from the frequency curve but to a lesser extent. It is possible that this is a real phenomenon related to the low incidence of after shocks associated with moderate earthquakes in the Puget Sound area.

Considerably more data is available to constrain the frequency curve of earthquakes in the Gorda Basin off southwest Oregon and northern California. The data points in Figure E8 are exact magnitudes; hence, no bars are drawn to indicate a sort interval. A best fit line drawn through the points is in very close agreement with the line having the adopted slope $b = 0.91$.

The data available for Portland and vicinity (Figure E9) are insufficient to show a convergence to the adopted slope; therefore, two curves are drawn through the data; a standard curve drawn through the highest magnitude data and a best fit curve. The standard curve indicates a magnitude $M_B = 6$ whereas the best fit curve yields a magnitude $M_B = 6.5$. As more data becomes available the difference between a best fit curve and a standard curve becomes smaller. As data become available a best fit curve will become asymtotic to the standard curve. Because not all earthquakes of small magnitude were reported and earthquakes of large magnitude are absent, the best fit curve will yield a larger likely maximum earthquake than the standard curve, i.e., it yields a "worst case" estimate. A conservative or "worst case" approach selects the magnitude M_B = 6.5 as the likely maximum for Portland and vicinity.

The data for the Umatilla-Walla Walla area (Figure E10) also shows a lack of convergence to the standard curve. The standard curve fit to the largest magnitude datum yields a magnitude M_B = 6.2, whereas a best fit curve yields a magnitude M_B = 6.5. The conservative approach selects the magnitude M_B = 6.5 as the likely maximum for the Umatilla-Walla Walla area.

Large earthquakes also occur south of the BPA Service Area. The magnitude M_B = 7.6 which occurred at Kennedy, Nevada, October 2, 1915, is selected as representing the likely maximum earthquake south of the study area. That earthquake is sufficiently close to give a basis for estimating the greatest intensities which might occur in the area.

Several comparisons can be made between the values of the likely maximum earthquakes, as selected just above, with those obtained by other methods previously mentioned. By taking the direction of preferred faulting (Algermissen and Harding, 1) in the larger quakes as an indication of the strike of the active faults in the Puget Sound area and postulating that a fault extends the width of Puget Sound beneath the alluvium (Hutting, et al, 24), a fault length of approximately 80 km can be obtained. Alternatively, a fault of approximately the same length is indicated on the Tectonic Map of North America (King, 28). Using 80 km as an assumed fault length and applying the empirical relation of Tocher (43) as modified by Press (3), M = 7.5 earthquake is indicated as a likely maximum to be anticipated in the Puget Sound region.

This value is in good agreement with the M = 7.4 indicated by the recurrence curve. Similarly, an assumed fault 30 km in length beneath the alluvium of Portland (Wells and Peck, 51) suggests a magnitude M = 6.8 which is in reasonable agreement with the M = 6.5 selected as the likely maximum earthquake in the Portland area.

Couch and Lowell (11), using the method of Benioff (7) and the assumption that the strain energy accumulation and release in the Portland area are in equilibrium over the interval of recorded earthquakes, estimated that the average rate of strain energy release was 2.6×10^{17} ergs/year for the period 1870 through 1970. They also note an approximate one magnitude increase in the rate of strain release after 1949. This strain energy, accumulated at the above given rate for 130 years, can when released generate an earthquake of magnitude 5.8 to 6.2. This suggests that the 130 year interval yields a conservative estimate compared to results from other methods based on seismic energy release. It must be recognized, however, that while these methods are in reasonable agreement in the above cases, this does not confirm the ability of any of the three methods to yield correct estimates of a maximum earthquake.

3.5 SEISMIC ENERGY ATTENUATION

Seismic energy is not uniformly radiated from the source; in addition, variations in attenuation characteristics along the ray paths of the propagating waves cause non-uniform attenuation, hence, the surface excitation pattern is not symmetric and in fact is very complex and difficult to predict. Figure 5 shows the asymmetry in the surface excitation pattern of several earthquakes in the BPA Service Area. Figure E11 illustrates the changes in earthquake intensity as a function of distance as observed (U.S. Earthquakes, 45) for the following earthquakes; Strait of Georgia, British Columbia, 1946;

Olympia, Washington, 1949; Tacoma, Washington, 1965; and Portland, Oregon, 1962. The ends of the vertical bars, marked by appropriate symbols, indicate the maximum and minimum distance at which a given intensity was observed. A similar symbol between the ends of the bars indicates a length of the radius of a circle which best fits the area of observed intensity. A smooth curve is drawn through the symbols which indicate the length of the radii to obtain the shape of the attenuation curves. The curves indicate that although attenuation varies greatly with azimuth the general shape of the curves is similar for these earthquakes all of which occurred west of the Cascade Range. Attenuation curves for the area west of the Cascade Range were constructed by drawing curves of the form indicated by the observed smooth curves through the maximums or worst case conditions. Figure E12 shows the attenuation curves applicable to the region west of the Cascade Range. The curves assume uniform radiation, consequently, they predict maximum surface excitation at all azimuths. Further analysis shows a significant difference in attenuation characteristics between sources within Puget Sound and sources outside Puget Sound, but west of the Cascade Range. Consequently a second set of attenuation curves was derived solely for the Puget Sound region; these curves are shown on Figure E13.

Similarly, attenuation curves were derived for southwest Oregon (Figure E14) for seismic waves originating off the coast primarily in the vicinity of the margin of the continental slope. Attenuation curves were also derived for the Basin and Range area (Figure E15) and for the area east of the Cascade Range (Figure E16).

3.6 SEISMIC INTENSITY MAPS OF THE BPA SERVICE AREA

In forming the seismic intensity maps of the BPA Service Area, the likely maximum earthquakes were positioned with reference to the historical earthquakes shown on Figure E4 and to the geologic and physiographic boundaries indicated on Figures E1 and E2. Also considered were faults and structural lineations indicated on the Tectonic Map of North America (King, 28) and on geologic maps of the states: Washington (Huntting, et al, 24), Oregon (Wells and Peck, 52; Walker and King, 51; Thiruvthukal, et al, 42), Idaho (Ross and Forester, 37) and Montana (Ross, Andrews and Witkind, 38). The likely maximum earthquake in a given seismic area (e.g., Puget Sound) was postulated as occurring anywhere along a fault or structural lineation as far as a distance (out from the center of seismicity) equal to the place where the observed seismicity would be decreased a magnitude interval of 2.6 (i.e., two sort intervals of Figure E4). For example, consider a seismic area whose likely maximum earthquake is postulated to be $M_B = 6.3$ and whose fault source extends from near the center of the area to outside the area. The $M_B = 6.3$ earthquake is postulated as occurring anywhere along the mapped fault as far as a distance out from the seismic center to equal to the place where the magnitude of the observed (historical) earthquakes would be decreased to $M_B = 3.7$.

After locating the likely maximum earthquake as described above, a contour was drawn around the limits of its occurrence. By use of the empirical relation between magnitude and intensity (p.E-15), an intensity was estimated for the likely maximum earthquake. This intensity is postulated to occur anywhere within the closed contour defined by the location of the likely maximum earthquake. The attenuation of the seismic excitation, or the decrease of intensity away from the epicenter of the likely maximum earthquake, was determined from the appropriate attenuation curve, Figures E12, E13, E14, E15, or E16, for the area. Contours are drawn at intensity levels of IX, VIII and VI. The regional background level is intensity IV and localized maximum levels of VIII-IX and

IX-X occur in the vicinity of the likely maximum earthquakes.

Figure E17 shows contours of the intensity generated by likely maximum earthquakes with magnitudes $M_B \geqq 7$. The earthquakes are postulated to occur off the coast of southern Oregon and northern California, in northern Nevada, Puget Sound, Georgia Strait, western Montana, northern Utah and northwestern Wyoming. The intensities shown in the center of the BPA area would be caused by the longer period seismic waves radiated by the postulated earthquakes.

Figure E18 shows seismic intensities that would be caused by postulated earthquakes of magnitude $M_B = 6.3 - 6.5$. The intensity patterns for the likely maximum earthquakes $M_B = 6.5$ postulated to occur in the Portland and Umatilla-Walla Walla areas are shown. It is expected that the intensities shown would be caused by waves of shorter periods than those of Figure E17.

Figure E19 shows intensity contours caused by postulated earthquakes of magnitude $M_B \doteq 5$. The localities of higher intensity would be caused by seismic waves of short period generated by earthquakes within those localities.

3.7 COMPOSITE SEISMIC INTENSITY MAP OF THE BPA SERVICE AREA

Figure E20 shows a composite seismic intensity map of the BPA Service Area formed by essentially stacking the maps shown in Figures E17, E18 and E19. Contours are drawn and smoothed to include the maximum intensity levels of the three separate seismic maps. The highest expected intensities occur in the Puget Sound region, along the southern Oregon coast and in western Montana and eastern Idaho. A small area that would generate slightly lower intensities is shown in the Umatilla-Walla Walla area. The composite map does not indicate anything of the spectral characteristics of the seismic waves which cause the mapped intensities; however, Figures E17, E18 and E19 provide a guide to those characteristics by indicating the level of the source of excitation. The composite map does not supply any information on the response of the earth's surface to the indicated levels of excitation in small areas. Such information requires detailed local studies.

It must be recognized that the maps of seismic intensity in this report are only an intermediate step between maps based simply on historical data and more definitive maps based on a tested prediction method or on, at least, a solid theoretical model. Consequently, these maps are intended to provide only a guide to the seismicity of the area. Design criteria for a particular site requires further detailed studies.

3.8 CONSIDERATIONS

It is recognized that the maps estimate higher intensities in the Cascade Range area of Washington, in southwest Oregon and in Idaho than historical earthquakes would suggest as likely. Part of this is due to the likely maximum earthquakes being larger than the historical earthquakes and part occurs because of the assumption of uniform radiation of seismic energy at all azimuths. It is recognized that the patterns of intensity are asymmetrical and there is a likelihood that additional study could modify the intensity curves so that a possible reduction of anticipated intensities would be shown in some areas.

An attempt was made in this report to treat the available data as analytically as possible; however, large uncertainties are associated with the intensities of historical earthquakes. This deficiency in the historical data, in addition to the uncertainty in the empirical relation between intensity and magnitude, causes large uncertainties in the magnitudes applied to historical data. Further, recently instrumentally determined magnitudes may be uncertain by ± 0.5 M. In some instances the magnitude scale used, whether M_L (local) or M_B (teleseismic), is unknown. The difference is small for moderate size earthquakes but large for small earthquakes. Where these problems were recognized the conservative approach generally was taken for the derivations.

Additional studies might lead to modification of the intensity contours. It is possible that the large earthquakes in the Puget Sound area occur at the greater depths, as indicated by the large earthquakes of 1949 and 1965, because of the inability of the upper crustal layers to sustain a large stress. Geologic studies are needed to provide additional information on post-Pliocene fault dimensions, focal mechanisms and intensity attenuation.

SHANNON & WILSON, INC.

by

RICHARD W. COUCH

by

ROBERT J. DEACON

AMERICAN INSTITUTE OF PROFESSIONAL GEOLOGISTS
CERTIFICATE NUMBER
471
AIPG
ROBERT J. DEACON
CERTIFIED PROFESSIONAL GEOLOGIST

REFERENCES CITED

1. Algermissen, S. T., and Harding, S. T., "The Puget Sound, Washington earthquake of April 29, 1965," Preliminary Seismol. Report, U. S. Dept. of Commerce Coast and Geodetic Survey, 1965, pp. 1-26.

2. Algermissen, S. T., "Seismic Risk Studies of the United States," Fourth World Conference on Earthquake Engineering, Vol.1, 1969.

3. Atwater, T., "Implications of plate tectonics for the Cenozoic tectonic evolution of western North America," Bull. Geol. Soc. Amer., Vol. 81, 1970, pp.3513-3536,18 figs.

4. Barazangi, M., and Dorman, J., "World seismicity maps"; compiled from ESSA Coast and Geodetic Survey Epicentral Data, Bull. Seismol. Soc. Amer., Vol. 59, 1969, 369 pp.

5. Båth, M., "The energies of seismic body waves and surface waves," H. Benioff; Ewing, M., Howell, B.F., Jr., F. Press (Editors), Contributions in Geophysics, Pergamon Press, New York, 1958, 244 pp.

6. Båth, M., and Duda, S.J., "Earthquake volume, fault plane area, seismic energy, strain, deformation and related quantities," Ann. Geofis. Rome, Vol. 17, 1964, pp. 353-368.

7. Benioff, H., "Earthquakes and rock creep," Part 1, Bull. Seismol. Soc. Amer., Vol. 41, 1951, pp. 31-62.

8. Berg, J.W., Jr., and Baker, C.D., "Oregon earthquakes, 1841 through 1958," Bull. Seismol. Soc. Amer., Vol. 53, 1963, pp. 95-108.

9. Bonilla, M.G., "Surface faulting and related effects," Earthquake Engineering, R.L. Wiegel (Coord. Editor), Prentice-Hall, Inc., New Jersey, 1970, pp. 47-74.

10. Brazee, R. J., and Stover, C.W., "The distribution of earthquakes with respect to magnitude (M_B)," Bull. Seismol. Soc. Amer., Vol. 59(2), 1969, pp. 1015-1017.

11. Couch, R.W., and Lowell, R.P., "Earthquakes and seismic energy release in Oregon," The OreBin, Vol. 33(4) 1971, pp. 61-84.

REFERENCES CITED Continued

12. Couch, R.W., and MacFarlane, W.T., "A fault plane solution of the October, 1969 Mt. Rainier earthquake and tectonic movements in the Pacific Northwest from fault plane and first motion studies," Trans. AGU, Vol. 52(2), 1971, 428 pp.

13. Dehlinger, P., Couch, R.W., McManus, D.A. and Gemperle, M., "Northeast Pacific structures," The Sea, A. Maxwell (Editor), John Wiley and Sons, New York, Vol. 4, 1971.

14. Dicken, S.N., Oregon Geography, Edwards Bros, Inc., Ann Arbor, Mich., 1st Edition, 1950, 127 pp.

15. Duda, S.J., "The earthquake volume," Bull. Seismol. Soc. Amer., Vol. 60(5), 1970,pp. 1479-1489.

16. Earthquake hyopcenter data for the period 1961 through 1970, Nat'l. Geophys. Data Center, Environmental Data Service, Nat'l. Oceanic and Atmospheric Admin., 1970

17. Ergin, K., "Observed intensity-epicentral distance relations in earthquakes," Bull. Seismol. Soc. Amer., Vol. 59(3), 1969, pp. 1227-1238.

18. Fenneman, N.M., Physiography of Western United States, McGraw-Hill Book Company, Inc., New York and London, 1931.

19. Gutenberg, B., and Richter, C.F., Seismicity of the Earth, 2nd Edition, Princeton Univ. Press, Princeton, N.J., 1954.

20. ____________, "Earthquake magnitude, intensity, energy and acceleration," Bull. Seismol. Soc. Amer., Vol. 46,1956(a), pp. 105-145.

21. ____________, "Magnitude and energy of earthquakes," Ann. Geofis. Rome, Vol. 9, 1956(b), pp. 1-15.

22. Howell, B.F., Jr., "Precision of different varieties of magnitude," Bull. Seismol. Soc. Amer., Vol. 62(3), 1972, pp.789-792.

23. Howell, B.F., Jr., Lundquist, G.M., and Yiu, S.K., "Integrated and frequency-band magnitude, two alternative measures of the size of an earthquake," Bull. Seismol. Soc. Amer., Vol. 60(3), 1970, pp. 917-937.

REFERENCES CITED Continued

24. Huntting, M.T., Bennet, W.A.G., Livingston, V.E., Jr., and Moen, W.S., "Geologic map of Washington," Division of Mines and Geol., Dept. of Natural Resources, State of Washington, 1961, 1:500,000.

25. Isacks, B.L., and Oliver, J., "Seismic waves with frequencies from 1 to 100 cycles per second recorded in a deep mine in northern New Jersey," Bull. Seismol. Soc. Amer., Vol. 54, 1964, pp. 1941-1979.

26. Isacks, B.L., Oliver, J., and Sykes, L.R., "Seismology and the new global tectonics," J.Geophys. Res., Vol. 73, 1968, 5855 pp.

27. Jefferies, H. and Bullen, K.E., Seismological Tables British Association, Gray-Milne Trust, 1940 and 1958, 50 pp.

28. King, P.B., "Tectonic map of North America," U.S. Geol. Survey, 1969, 1:5,000,000.

29. Lammlein, D.R., Sbar, M.L., and Dorman, J., "A microearthquake reconnaissance of southeastern Missouri and western Tennesee," Bull. Seismol. Soc. Amer., Vol 61(6), 1971, 1705-1716.

30. McKenzie, D.P., "Plate tectonics and sea-floor spreading," Amer. Sci., Vol. 60(4), 1972, pp. 425-435.

31. Morgan, W.J., "Rises, trenches, great faults and crustal blocks," J. Geophys. Res., Vol. 73, 1968, 1959 pp.

32. Pitman, W.C., and Heirtzler, J.R., "Magnetic anomalies over the Pacific Antartic ridge," Science, Vol. 154, 1966, 1164 pp.

33. Press, F., "Dimensions of the source region for small shallow earthquakes," Proc. of the VESIAC Conf. on the Current Status and Future Prognosis for Understanding the Source Mechanism of Shallow Seismic Events in the 3 to 5 Magnitude Range, 1967, pp. 155-164.

34. Rasmussen. N., "Washington state earthquakes 1840 through 1965," Bull. Seismol. Soc. Amer., Vol. 57(3), 1967, pp. 463-476.

35. Richter, C.F., Elementary Seismology, Freeman and Co., San Francisco, 1958, 768 pp.

REFERENCES CITED Continued

36. Richter, C.F., "Seismic Regionalization," Bull. Seismol. Soc. Amer., Vol.49(1), 1959, pp. 123-162.

37. Ross, C.P., and Forrester, J.D., "Geologic map of Idaho," Idaho Bureau of Mines and Geology, 1949, 1:500,000.

38. Ross, C.P., Andrews, D.A., and Witkind, I.J., "Geologic map of Montana", Montana Bureau of Mines and Geology, 1958, 1:500,000.

39. Savage, J.C., Hastie, L.M., "Surface deformation associated with dip-slip faulting," J. Geophys. Res., Vol. 71(20), 1966, pp. 4897-4904.

40. Scholz, C.H., Wyss, M., and Smith, S.W., "Seismic and aseismic slip on the San Andreas fault," J. Geophys. Res., Vol. 74(8), 1969, pp. 2049-2069.

41. Sykes, L.R., "The seismicity of the Arctic," Bull. Seismol. Soc. Amer., Vol. 55(2), 1965, pp. 519-536.

42. Thiruvathukal, J.V., Berg, J.W., Jr., and Heinrichs, D.F., "Regional gravity of Oregon," Geol. Soc. Amer., Vol. 81, 1970, pp. 725-738.

43. Tocher, D., "Earthquake energy and ground breakage," Bull. Seismol. Soc. Amer., Vol. 48, 1958, pp. 147-153.

44. Townley, S.D., and Allen, M.W., "Descriptive catalog of earthquakes of the Pacific coast of the United States 1769 to 1928," Bull. Seismol. Soc. Amer., Vol 29(1), 1939, pp. 1-297.

45. United States Earthquakes, U.S. Gov't. Print. Office, Washington, D.C., Annual Reports 1928 to 1970.

46. U.S. Geol. Survey in cooperation with Idaho Bureau of Mines and Geol., Idaho Dept. of Highways and Idaho Dept. of Reclamation, "Mineral and water resources of Idaho," Spec. Report No. 1, Nov. 1964.

47. U.S. Geol. Survey and Montana Bureau of Mines and Geology, "Mineral and water resources of Montana," Spec. Pub. No. 28, May 1963.

48. U.S. Geol. Survey in collaboration with the Washington State Dept. of Conservation, Div. of Mines and Geology and the U.S. Bureau of Reclamation and other Agencies, "Mineral and water resources of Washington," Reprint No. 9, 1966.

REFERENCES CITED Continued

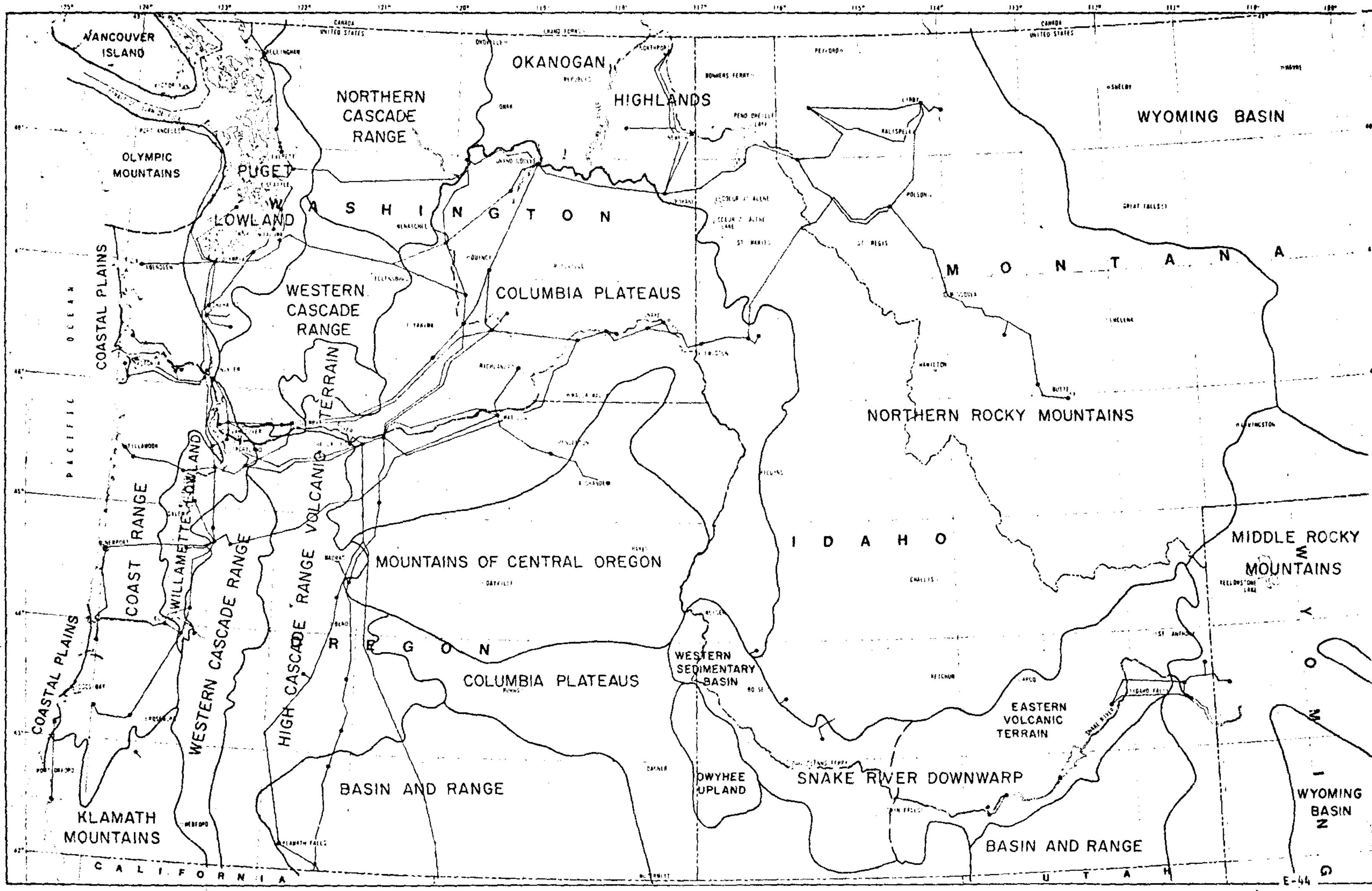
49. U.S. Geol. Survey and the State of Oregon Dept. of Geol. and Mineral Industries, "Mineral and water resources in Oregon," Bull. 64, 1969.

50. Vine, F.J., "Spreading of the ocean floor: new evidence," Science, Vol. 154, 1966, 1405 pp.

51. Walker, G.W., King, P.B., "Geologic map of Oregon," U.S. Geologic Survey Miscel. Invest. Map I-595, 1969, 1:2,000,000.

52. Wells, F.D., Peck, D.L., "Geologic map of Oregon west of the 121st meridian," State of Oregon, Dept. of Geol. and Mineral Industries, 1961, 1:500,000.

53. Wollard, G.P., "A catalogue of earthquakes in the United States prior to 1925," Hawaii Inst. of Geophy., Univ. of Hawaii, Data report No. 10 HlG-68-9, 1968.



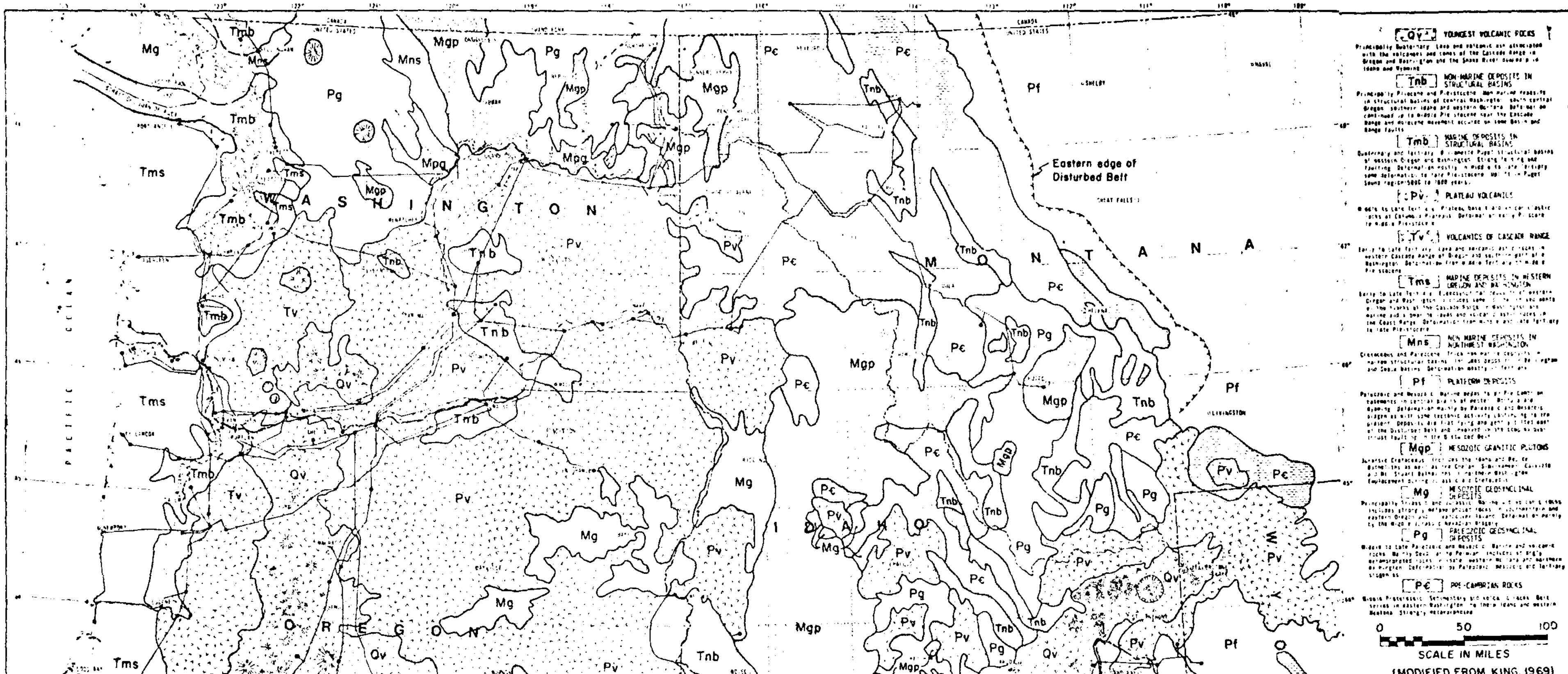
SEISMIC REGIONALIZATION STUDIES
AGBABIAN ASSOCIATES

PHYSIOGRAPHIC PROVINCE MAP
BPA SERVICE AREA
(WASHINGTON, OREGON, IDAHO AND WEST. MONTANA)

OCTOBER, 1972 0-653

SHANNON & WILSON, INC
PORTLAND, OREGON

FIG. E1



SEISMIC REGIONALIZATION STUDIES
AGBABIAN ASSOCIATES

GEOLOGIC PROVINCE MAP
BPA SERVICE AREA
(WASHINGTON, OREGON, IDAHO AND WEST MONTANA)

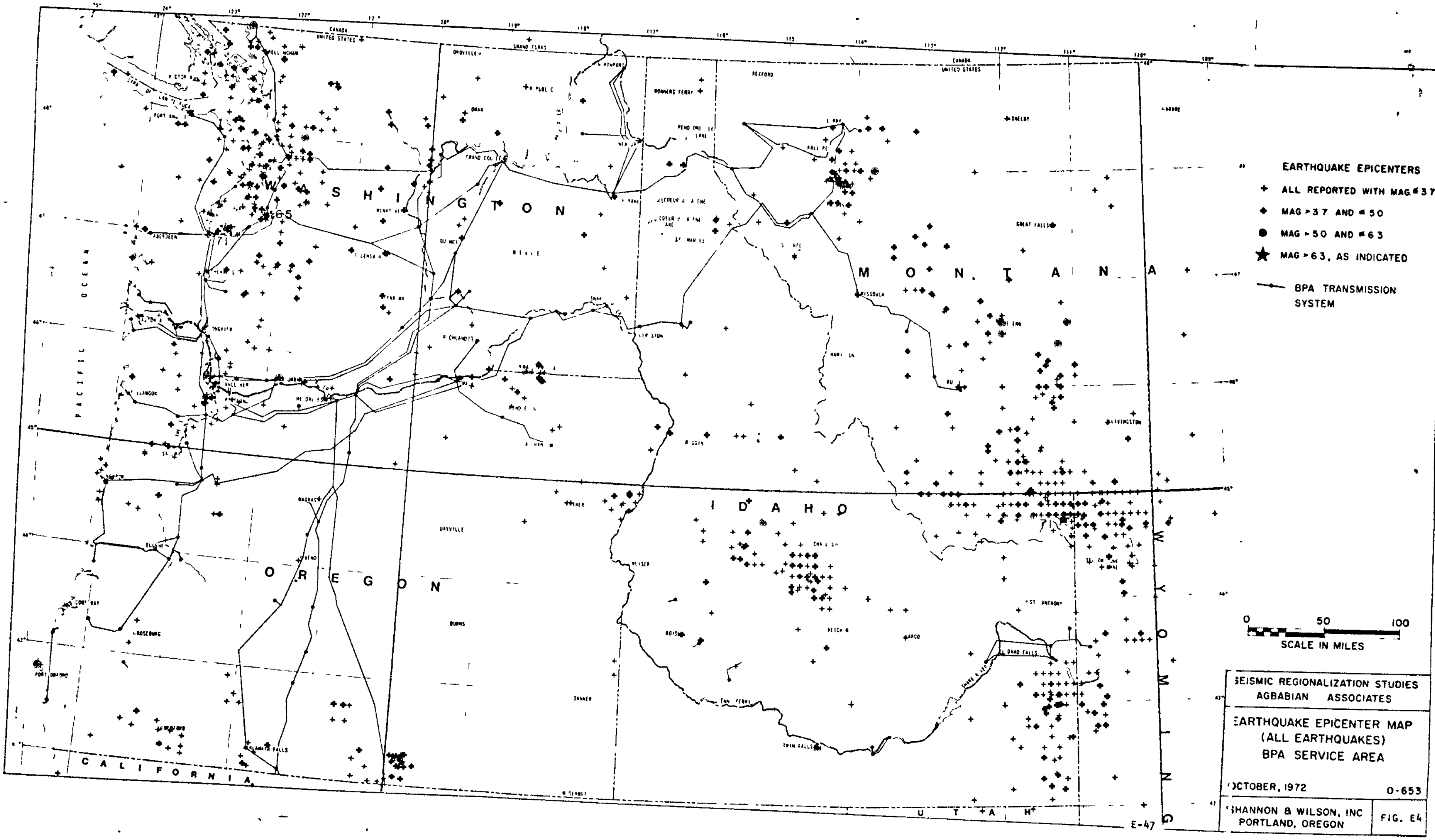
OCTOBER, 1972 | 0-653

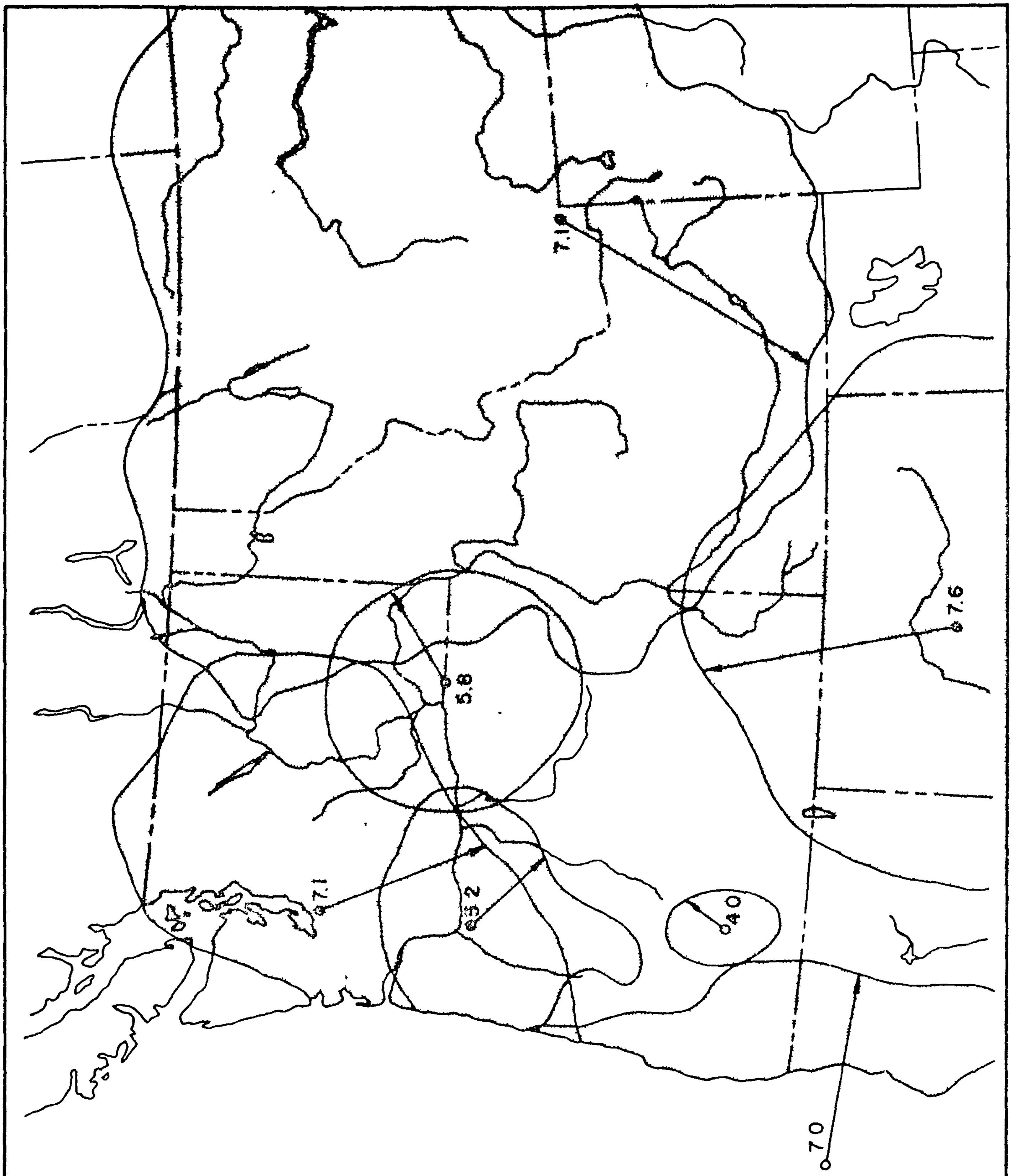
SHANNON & WILSON, INC.
PORTLAND, OREGON

FIG. E2

CORRELATION CHART

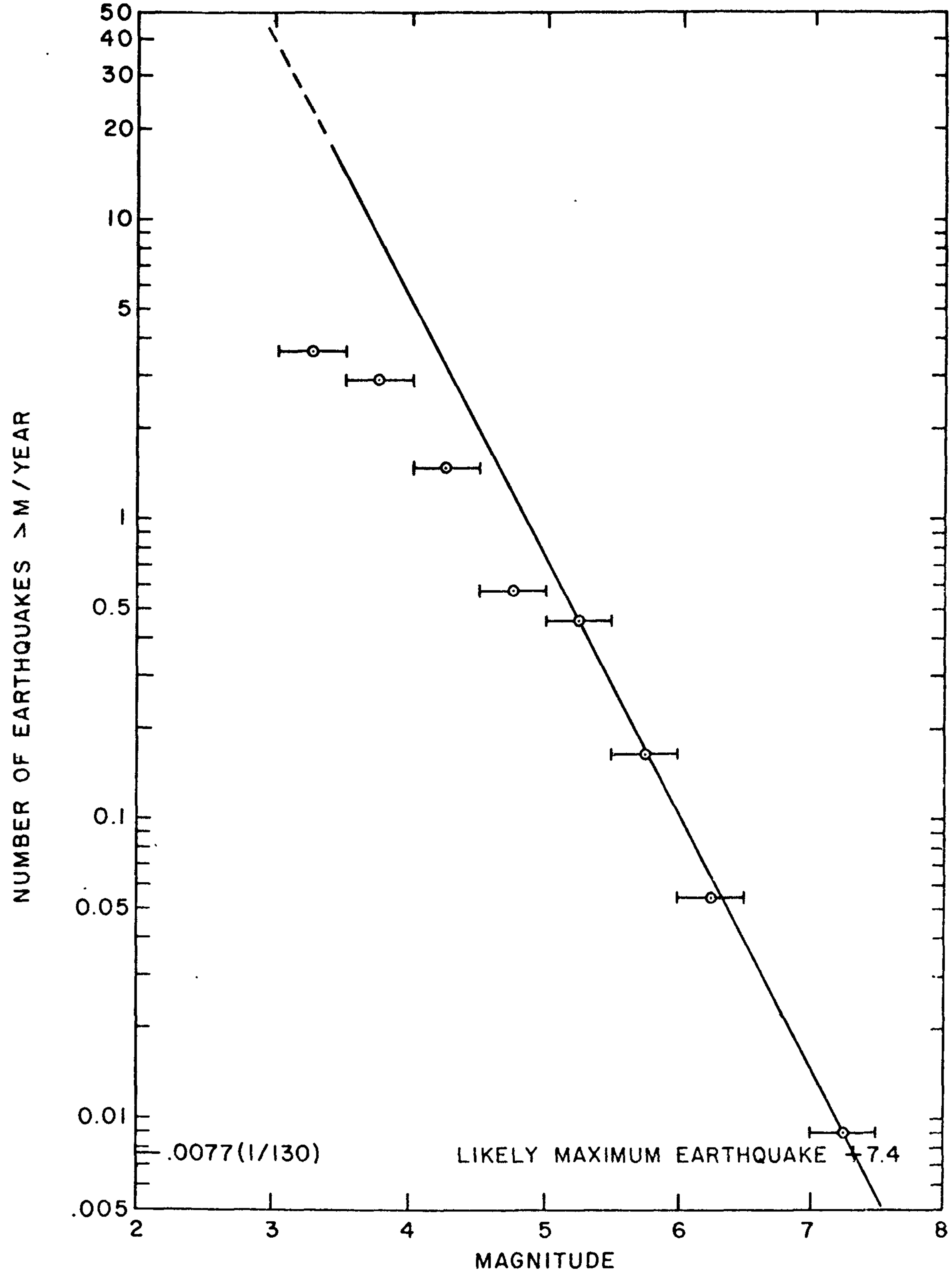
GEOLOGIC PROVINCE	PHYSIOGRAPHIC PROVINCE
YOUNGEST VOLCANIC ROCKS	High Cascade Range Volcanic Terrain Snake River Downwarp (Eastern Volcanic Terrain)
NON-MARINE DEPOSITS IN STRUCTURAL BASINS	Columbia Plateaus Basin and Range Snake River Downwarp (Western Sedimentary Basin) Northern Rocky Mountains (Intermountain Basins)
MARINE DEPOSITS IN STRUCTURAL BASINS	Willamette Lowland Puget Lowland
PLATEAU VOLCANICS	Columbia Plateau Basin and Range Northern Rocky Mountains (Southern Part) Middle Rocky Mountains
VOLCANICS OF THE CASCADE RANGE	Western Cascade Range
MARINE DEPOSITS IN WESTERN OREGON AND WASHINGTON	Coast Range Western Cascade Range
NON-MARINE DEPOSITS IN NORTHERN WASHINGTON	Northern Cascade Range
PLATFORM DEPOSITS IN CENTRAL PLAINS	Wyoming Basin
MESOZOIC GRANITIC PLUTONS	Northern Rocky Mountains Okanogan Highlands Northern Cascade Range Owyhee Upland Klamath Mountains
MESOZOIC GEOSYNCLINAL DEPOSITS	Klamath Mountains Mountains of Central Oregon Northern Rocky Mountains (Southern Part)
PALEOZOIC GEOSYNCLINAL DEPOSITS	Northern Rocky Mountains Middle Rocky Mountains Okanogan Highlands Northern Cascade Range
PRE-CAMBRIAN ROCKS	Northern Rocky Mountains Okanogan Highlands


EARTHQUAKE EPICENTERS
+ ALL REPORTED WITH MAG ≤ 3.7
◆ MAG > 3.7 AND ≤ 5.0
● MAG > 5.0 AND ≤ 6.3
★ MAG > 6.3, AS INDICATED
BPA TRANSMISSION SYSTEM
0 50 100
SCALE IN MILES
WASHINGTON
OREGON
IDAHO
MONTANA
WYOMING
CALIFORNIA
UTAH
PACIFIC OCEAN
CANADA
UNITED STATES
6.5
SEISMIC REGIONALIZATION STUDIES
AGBABIAN ASSOCIATES
EARTHQUAKE EPICENTER MAP
(ALL EARTHQUAKES)
BPA SERVICE AREA
OCTOBER, 1972
0-653
SHANNON & WILSON, INC
PORTLAND, OREGON
FIG. E4



INTENSITY IV ISOSEISMALS FROM PUBLISHED, MAPPED OR ESTIMATED OBSERVATIONS

SEISMIC REGIONALIZATION STUDIES AGBABIAN ASSOCIATES	
INTENSITY IV ISOSEISMALS EARTHQUAKES IN PACIFIC NORTHWEST	
OCT., 1972	0-653
SHANNON & WILSON, INC PORTLAND, OREGON	FIG. E5

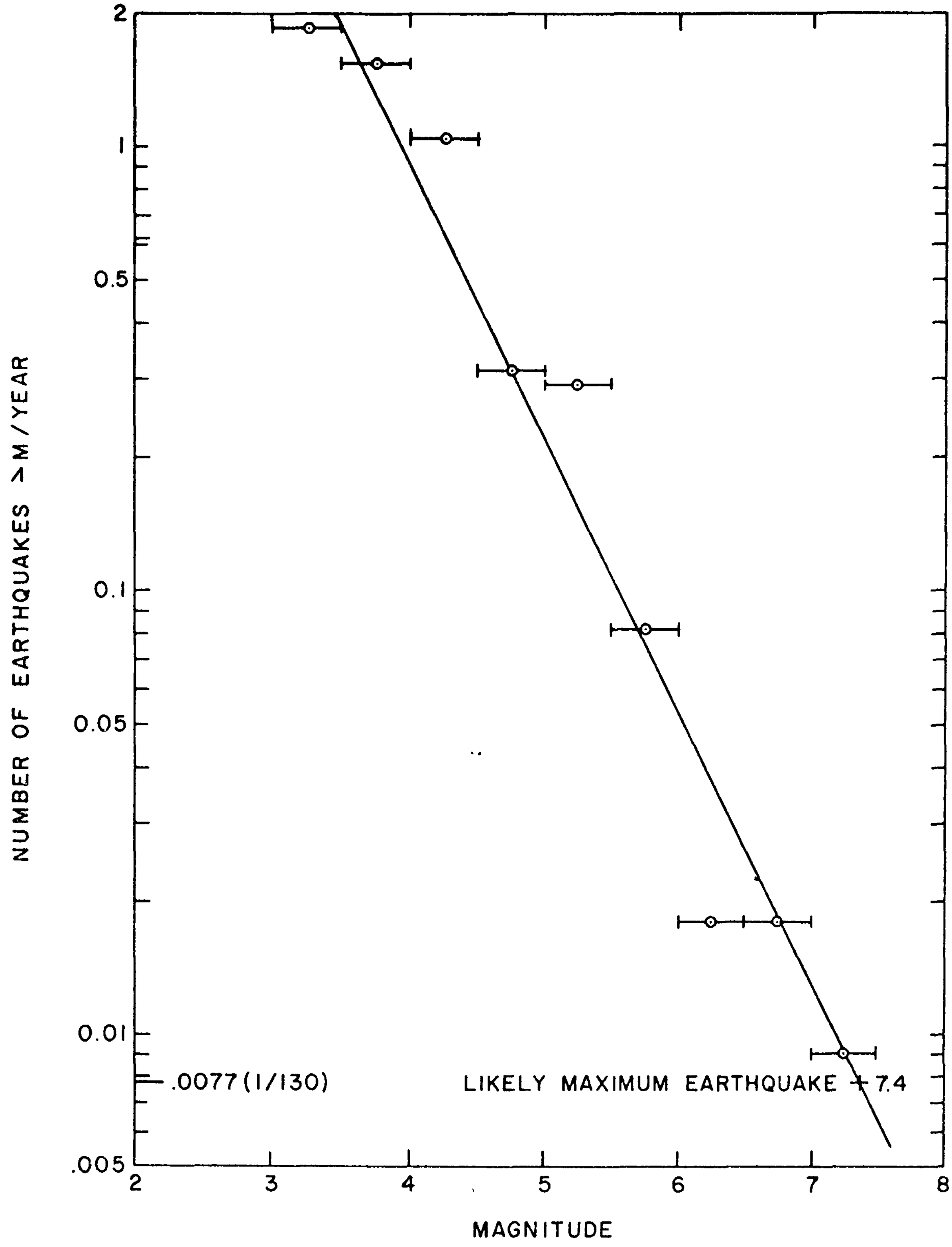


SEISMIC REGIONALIZATION STUDIES
AGBABIAN ASSOCIATES

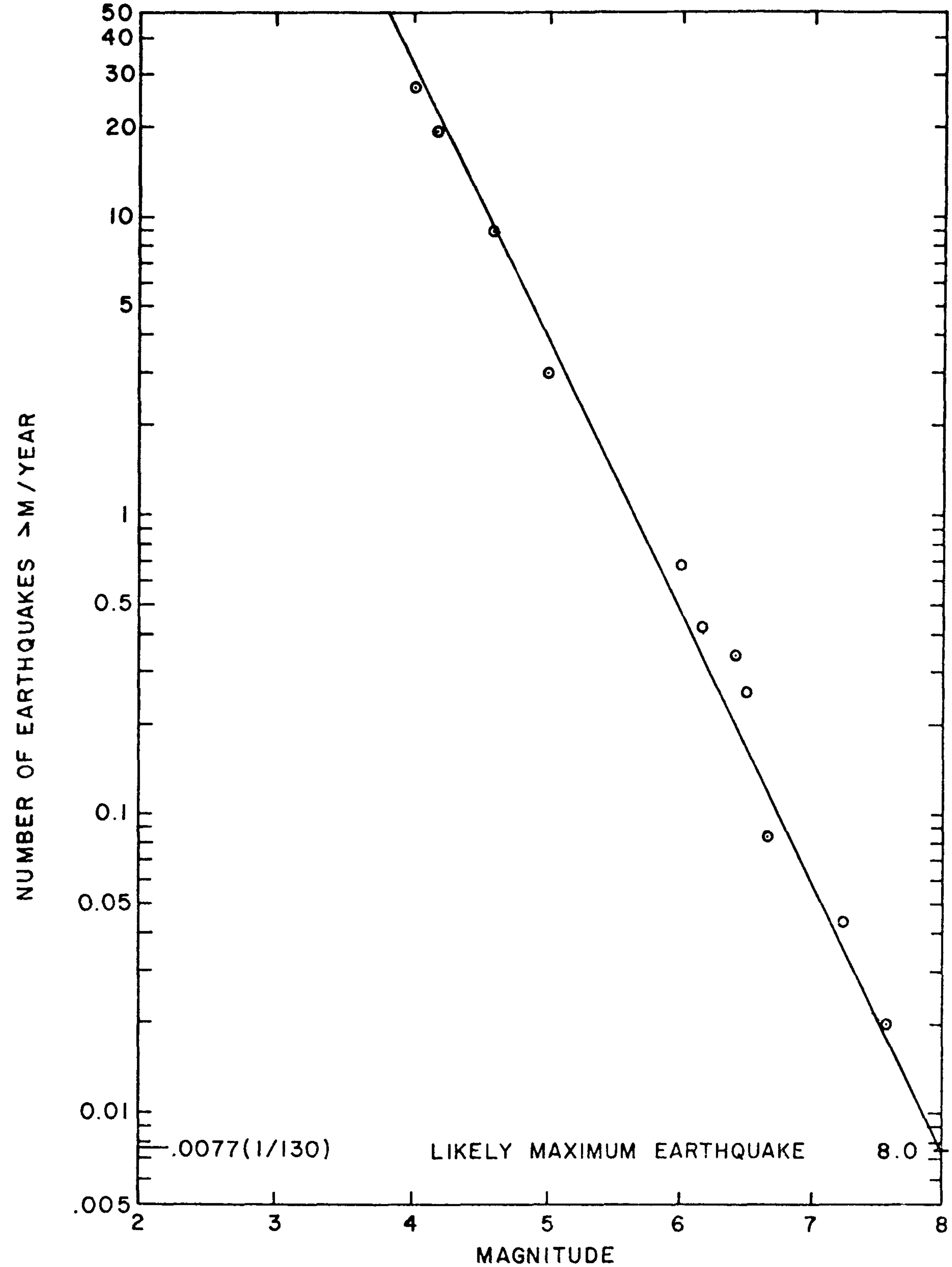
CUMULATIVE EQ CURVE
IDAHO, MONTANA AND WYOMING
(EAST OF 113.5° W)
OCT., 1972 0-653

SHANNON & WILSON, INC.
PORTLAND, OREGON

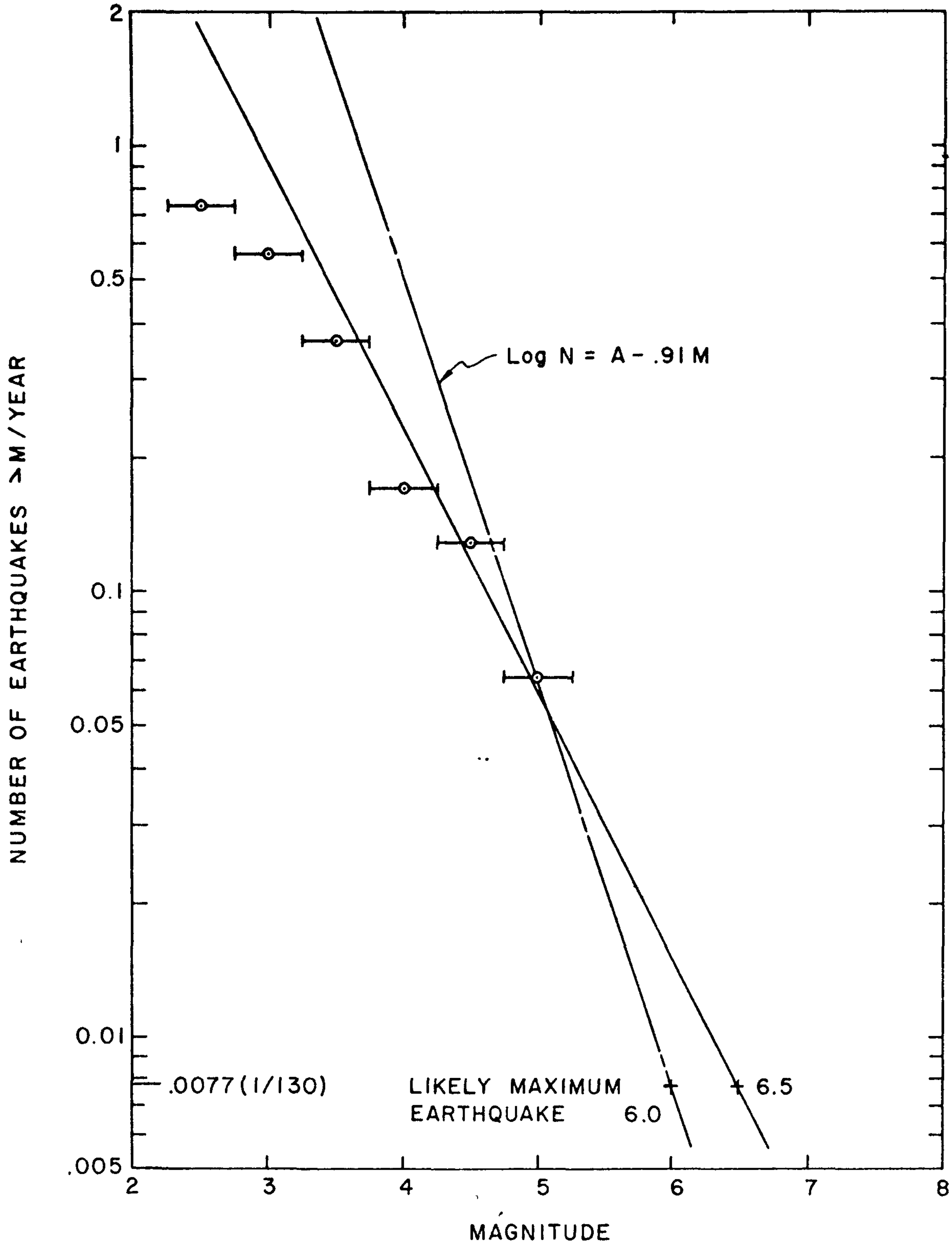
FIG. E6



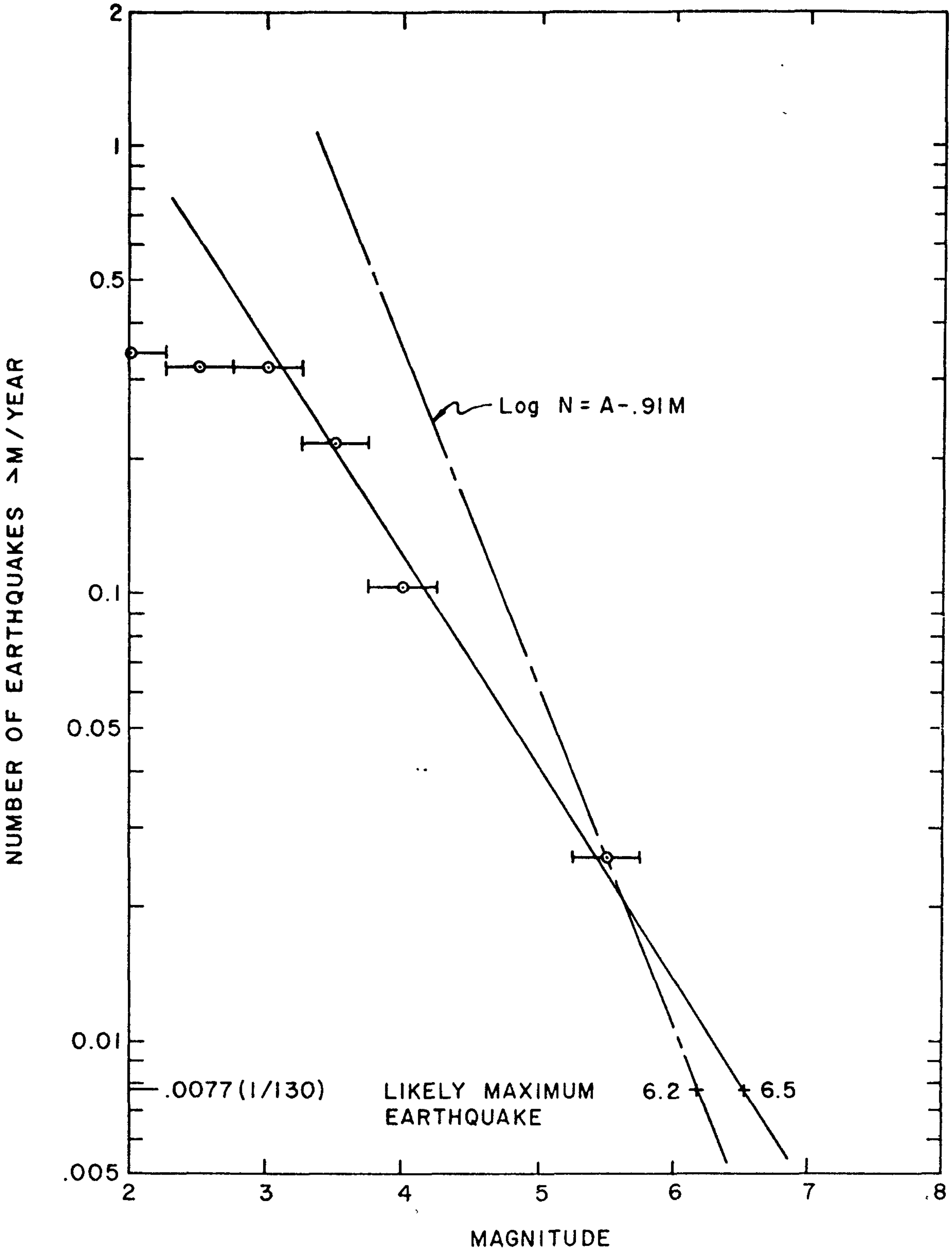
SEISMIC REGIONALIZATION STUDIES AGBABIAN ASSOCIATES	
CUMULATIVE EQ CURVE W. WASH. (PUGET SOUND)	
OCT., 1972	0-653
SHANNON & WILSON, INC. PORTLAND, OREGON	FIG. E7



SEISMIC REGIONALIZATION STUDIES AGBABIAN ASSOCIATES	
CUMULATIVE EQ CURVE GORDA BASIN (SOUTHWEST OREGON)	
OCT., 1972	0-653
SHANNON & WILSON, INC. PORTLAND, OREGON	FIG. E8

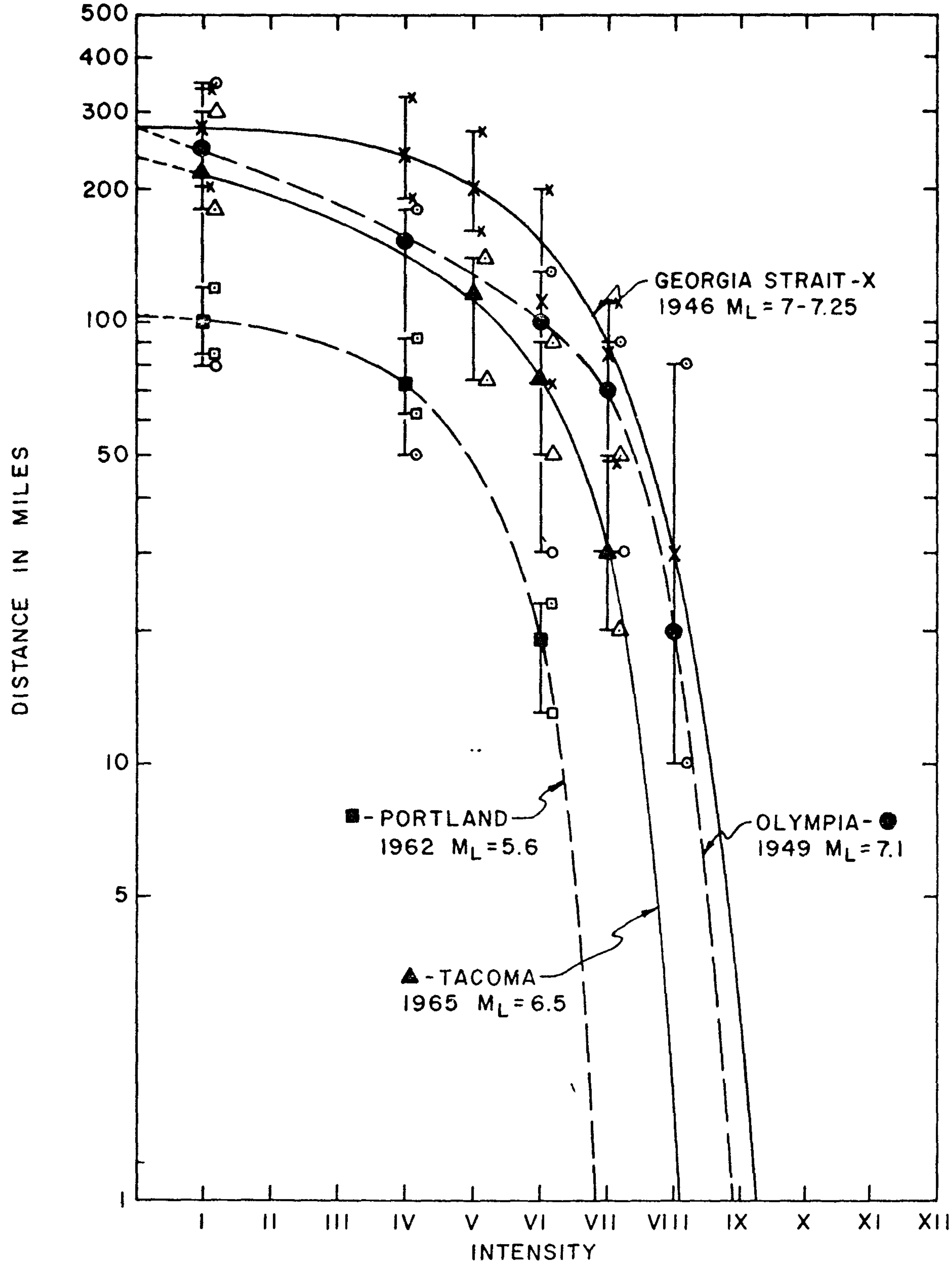


SEISMIC REGIONALIZATION STUDIES AGBABIAN ASSOCIATES	
CUMULATIVE EQ CURVE PORTLAND AND VICINITY	
OCT., 1972	0-653
SHANNON & WILSON, INC. PORTLAND, OREGON	FIG. E9



$\Sigma N = 27$ $\quad T = 75$ YEARS

SEISMIC REGIONALIZATION STUDIES AGBABIAN ASSOCIATES	
CUMULATIVE EQ CURVE UMATILLA-WALLA WALLA AREA	
OCT., 1972 0-653	
SHANNON & WILSON, INC. PORTLAND, OREGON	FIG. E10



EARTHQUAKE INTENSITY VS EPICENTRAL DISTANCE

CURVE INDICATES MEAN DISTANCE

BARS INDICATE APPROXIMATE MAXIMUM AND MINIMUM DISTANCES

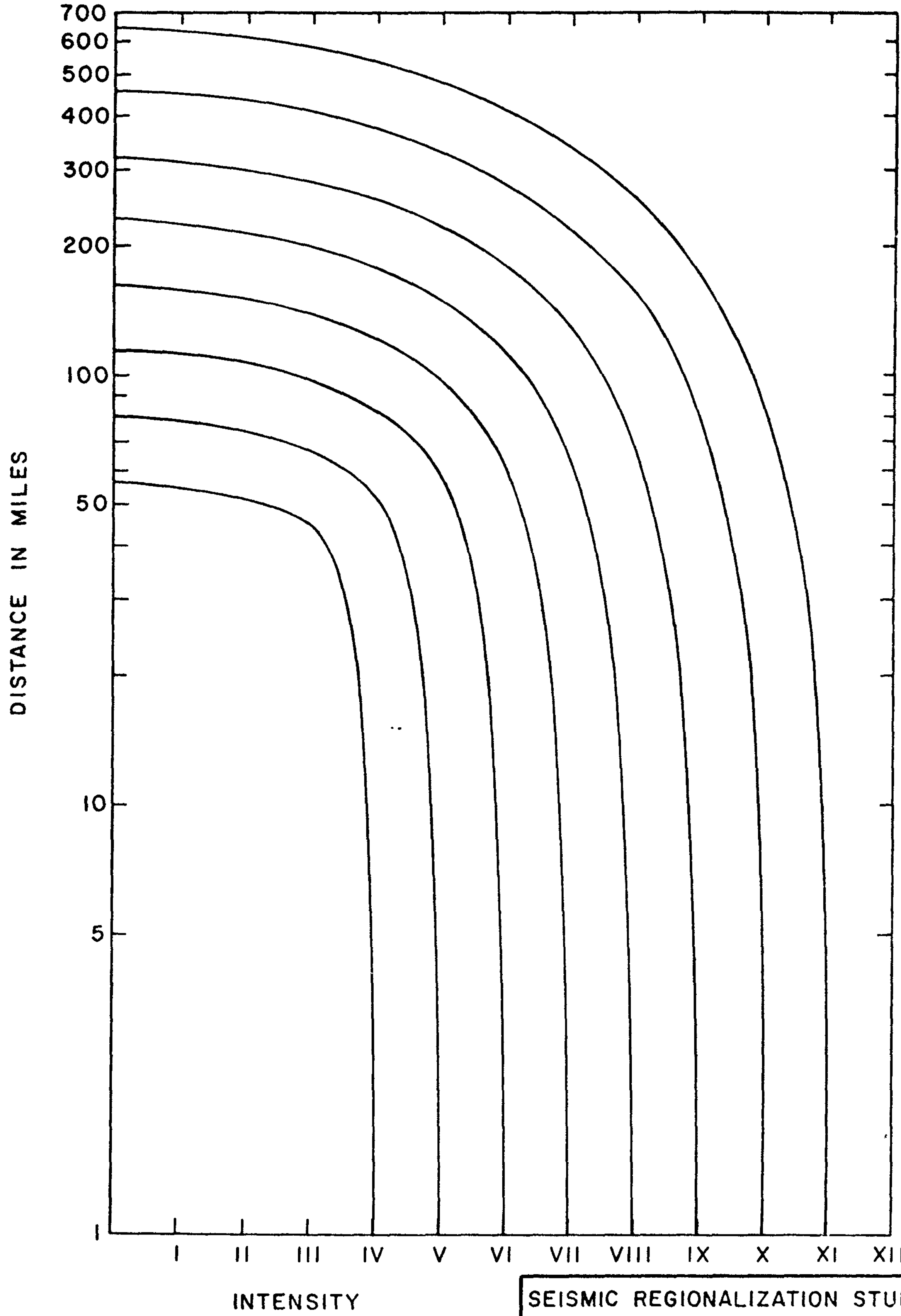
SEISMIC REGIONALIZATION STUDIES
AGBABIAN ASSOCIATES

OBSERVED AVERAGE CURVES
WEST OF CASCADE RANGE

OCT., 1972 0-653

SHANNON & WILSON, INC.
PORTLAND, OREGON

FIG. E11



EARTHQUAKE INTENSITY

VS

EPICENTRAL DISTANCE

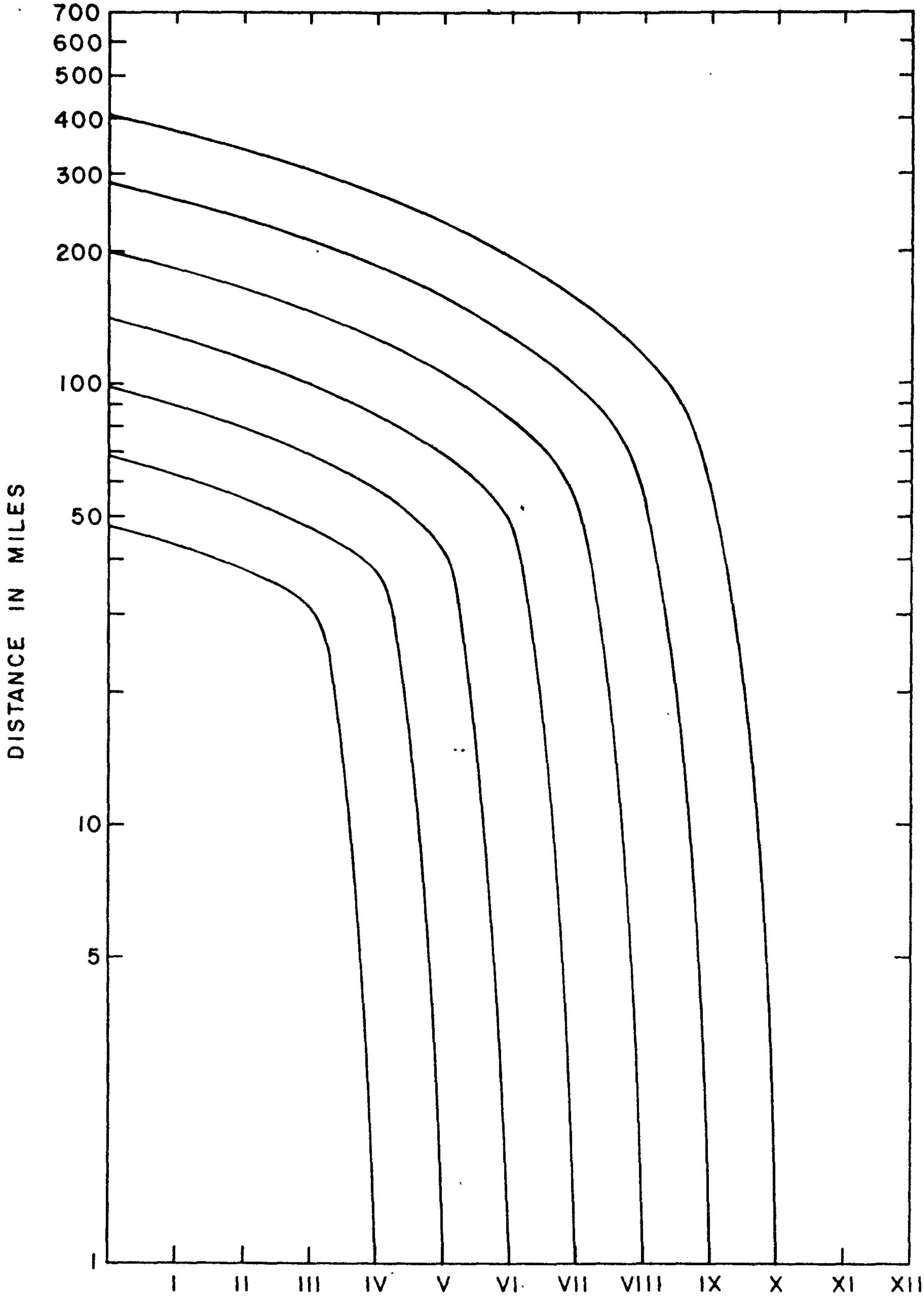
SEISMIC REGIONALIZATION STUDIES
AGBABIAN ASSOCIATES

ATTENUATION CURVES
WEST OF CASCADE RANGE

OCT., 1972 | 0-653

SHANNON & WILSON, INC.
PORTLAND, OREGON

FIG. E12



EARTHQUAKE INTENSITY
VS
EPICENTRAL DISTANCE

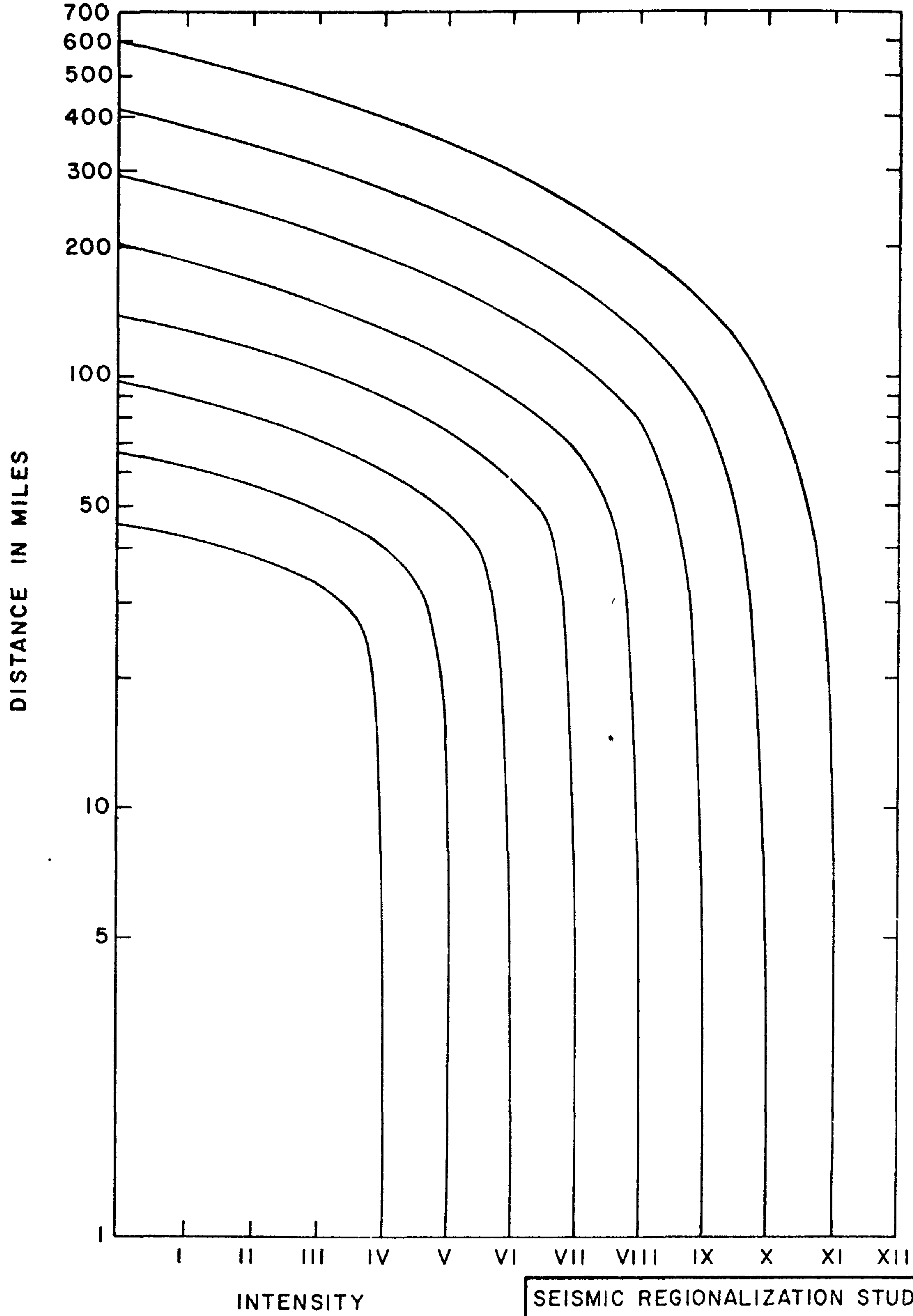
SEISMIC REGIONALIZATION STUDIES
AGBABIAN ASSOCIATES

ATTENUATION CURVES
WEST OF CASCADE RANGE
(PUGET SOUND)

OCT., 1972 0-653

SHANNON & WILSON, INC.
PORTLAND, OREGON

FIG. E13



EARTHQUAKE INTENSITY

VS

EPICENTRAL DISTANCE

SEISMIC REGIONALIZATION STUDIES
AGBABIAN ASSOCIATES

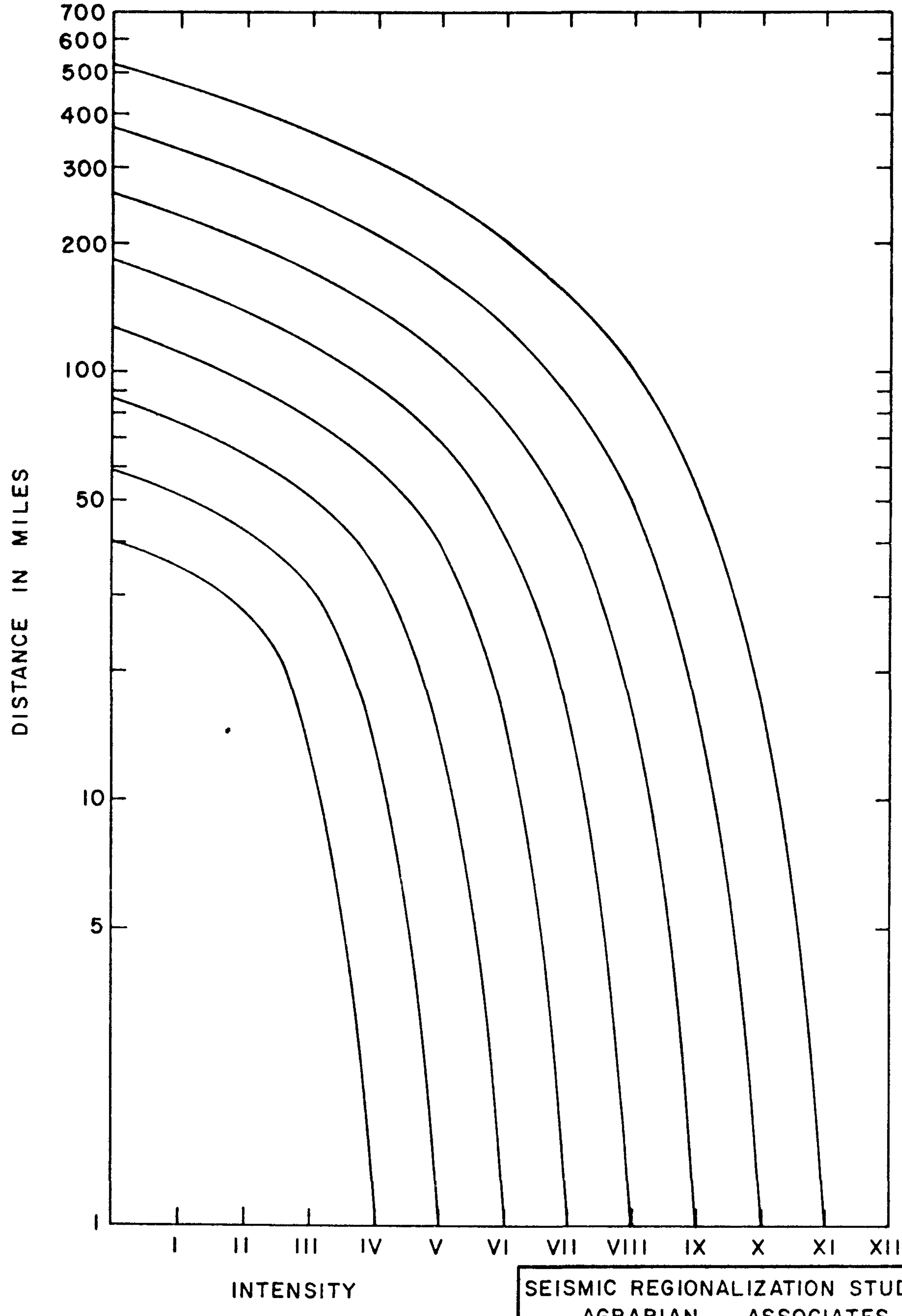
ATTENUATION CURVES

SOUTHWEST OREGON

OCT., 1972 0-653

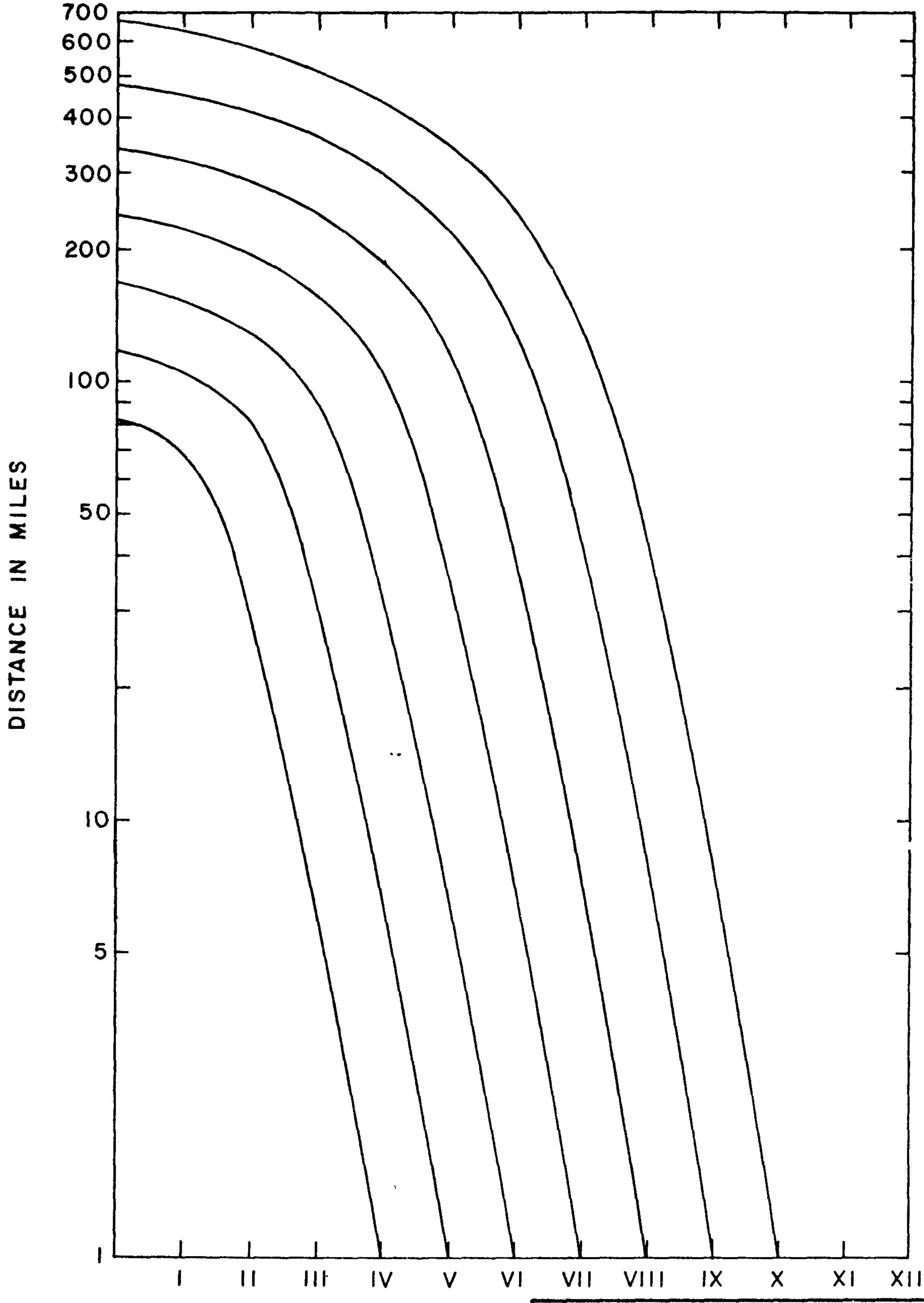
SHANNON & WILSON, INC.
PORTLAND, OREGON

FIG. E14



EARTHQUAKE INTENSITY
VS
EPICENTRAL DISTANCE

SEISMIC REGIONALIZATION STUDIES AGBABIAN ASSOCIATES	
ATTENUATION CURVES BASIN AND RANGE	
OCT., 1972	0-653
SHANNON & WILSON, INC PORTLAND, OREGON	FIG. E15



EARTHQUAKE INTENSITY
VS
EPICENTRAL DISTANCE

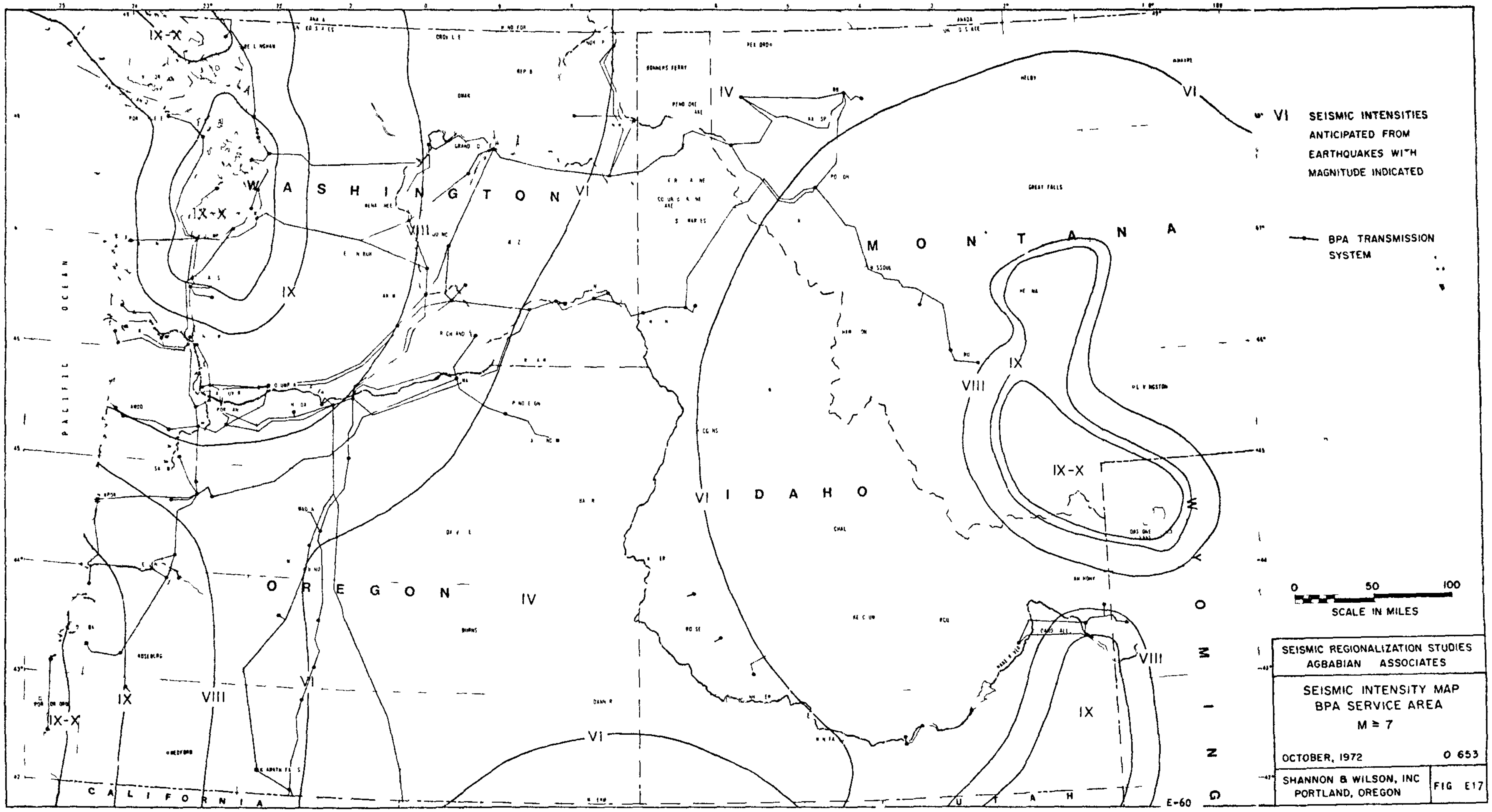
SEISMIC REGIONALIZATION STUDIES
AGBABIAN ASSOCIATES

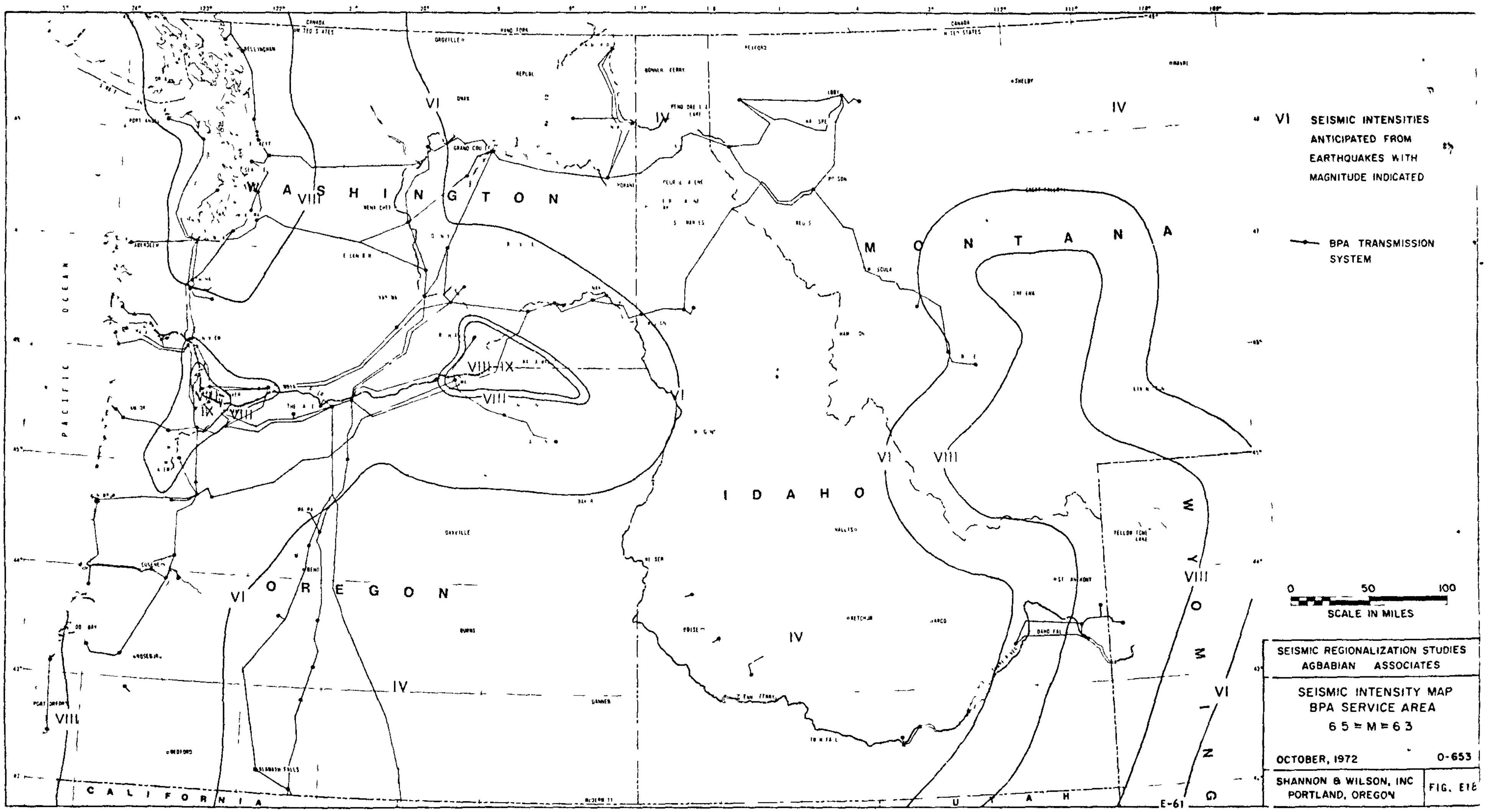
ATTENUATION CURVES
EAST OF CASCADE RANGE

OCT., 1972 0-653

SHANNON & WILSON, INC.
PORTLAND, OREGON

FIG. E16


VI SEISMIC INTENSITIES ANTICIPATED FROM EARTHQUAKES WITH MAGNITUDE INDICATED
BPA TRANSMISSION SYSTEM
0 50 100
SCALE IN MILES
SEISMIC REGIONALIZATION STUDIES
AGBABIAN ASSOCIATES
SEISMIC INTENSITY MAP
BPA SERVICE AREA
M ≥ 7
OCTOBER, 1972
0 653
SHANNON & WILSON, INC
PORTLAND, OREGON
FIG E17
WASHINGTON
MONTANA
IDAHO
OREGON
WYOMING
UTAH
CALIFORNIA
PACIFIC OCEAN
IX-X
IX
VIII
VI
IV
E-60

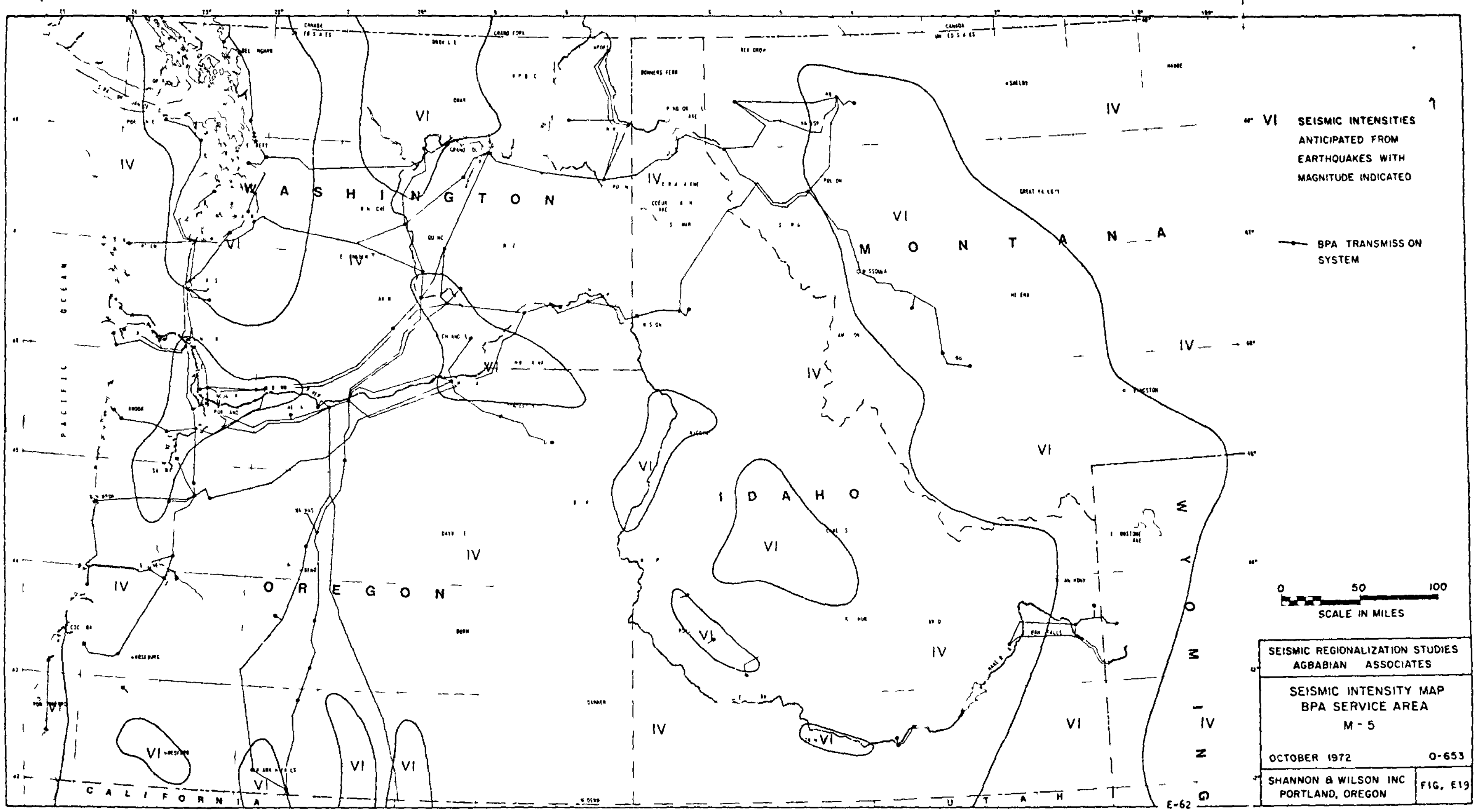


SEISMIC REGIONALIZATION STUDIES
AGBABIAN ASSOCIATES

SEISMIC INTENSITY MAP
BPA SERVICE AREA
6.5 ≥ M ≥ 6.3

OCTOBER, 1972 | 0-653

SHANNON & WILSON, INC
PORTLAND, OREGON | FIG. E18



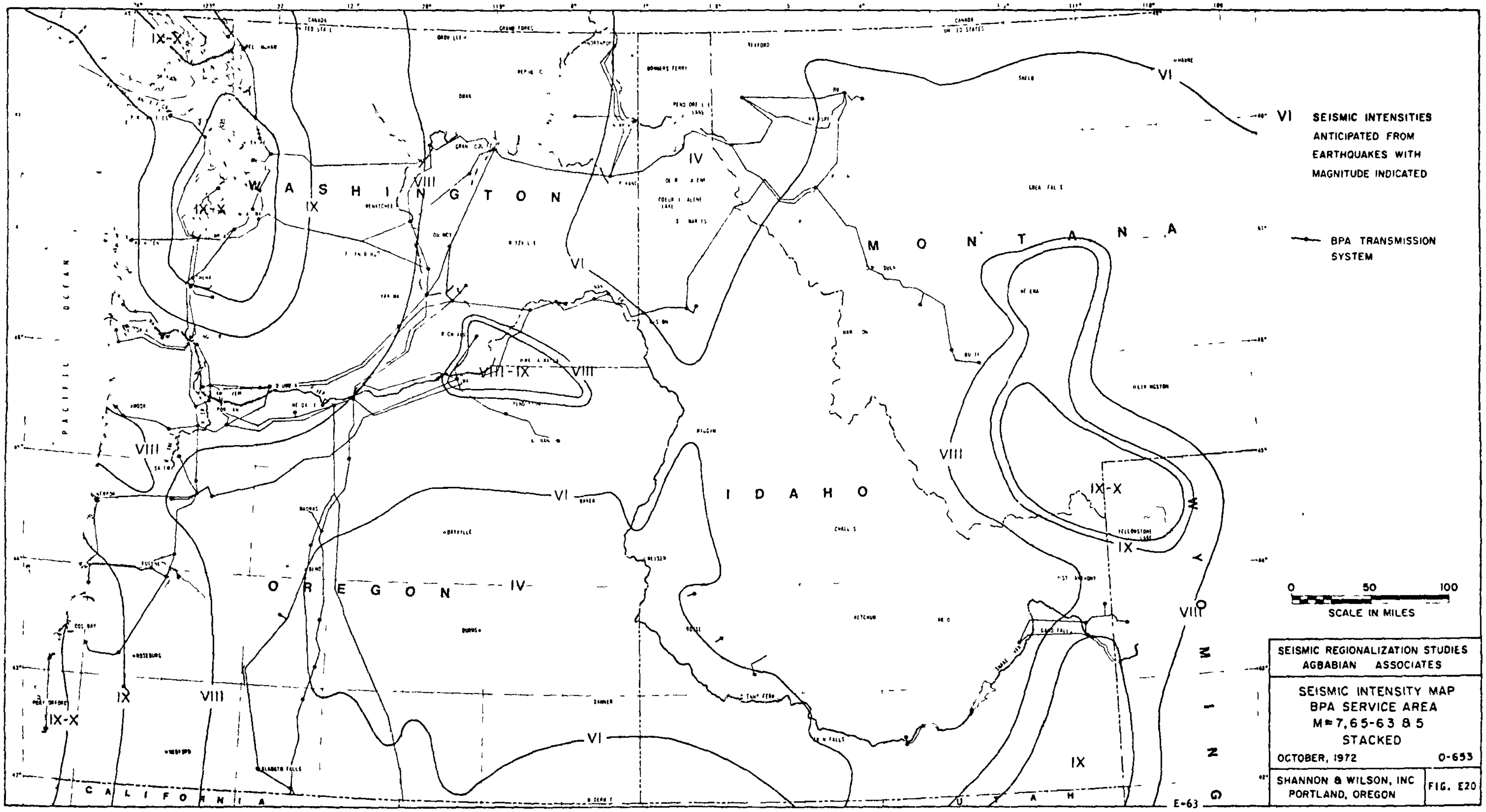
SEISMIC REGIONALIZATION STUDIES
AGBABIAN ASSOCIATES

SEISMIC INTENSITY MAP
BPA SERVICE AREA
M - 5

OCTOBER 1972 | O-653

SHANNON & WILSON INC
PORTLAND, OREGON

FIG. E19



SEISMIC REGIONALIZATION STUDIES
AGBABIAN ASSOCIATES

SEISMIC INTENSITY MAP
BPA SERVICE AREA
M≈7,6.5-6.3 & 5
STACKED

OCTOBER, 1972 0-653

SHANNON & WILSON, INC
PORTLAND, OREGON

FIG. E20