

**REPORTS OF VISITING RESEARCHERS
AT
GROUND VIBRATION DIVISION OF
PUBLIC WORKS RESEARCH INSTITUTE**

- In-Situ Testing for Liquefaction —**
- Large Scale Landslide Induced by Earthquake —**
- Attenuation of Peak Ground Motions
from Mexico Earthquakes of 1985 —**

January, 1988

Ground Vibration Division
Earthquake Disaster Prevention Department
Public Works Research Institute
Ministry of Construction
Asahi 1-banchi, Tsukuba-shi, Ibaraki-ken, 305 Japan

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- In-Situ Testing for Liquefaction —
— Large Scale Landslide Induced by Earthquake —
— Attenuation of Peak Ground Motions
from Mexico Earthquakes of 1985 —

edited by

YASUSHI SASAKI, DR. ENG.

Synopsis :

This report presents the works done by three foreign researchers at the Ground Vibration Division, Earthquake Disaster Prevention Department of Public Works Research Institute during the fiscal year of 1986. Following three subjects are included:

- 1) Study of In-Situ Testing for Evaluation of Liquefaction Resistance and Occurrence
- 2) Research on the Effects of Topographic Amplification of Earthquake Shaking on Slope Stability
- 3) Attenuation of Peak Ground Motions from Mexico Earthquakes of September 19 and 21, 1985

Key Words : In-Situ Testing, Liquefaction, Slope Stability, Attenuation of Ground Motion, Mexico Earthquake of 1985, Earthquake Engineering

Preface

This report presents the works done by three foreign researchers stayed at the Ground Vibration Division, Earthquake Disaster Prevention Department of Public Works Research Institute during the fiscal year of 1986.

Mr. Jeffrey A. Farrar, who stayed during the period of October 4 through November 17, 1986, is a geotechnical engineer dispatched from the Soil and Rock Mechanics Section, Division of Research and Laboratory Services, Engineering and Research Center, Bureau of Reclamation, U.S. Department of Interior. Mr. Farrar is engaged in the research work concerning the seismic stability of fill type dams dealt by the Bureau of Reclamation.

During his stay at the Ground Vibration Division, Mr. Farrar studied the in-situ testing for assessing the liquefaction potential during earthquakes and in his study, he introduced the concept of steady state to analyze the cone penetration test results. His study was supported by the Japanese Government Research Awards for Foreign Specialists through Science and Technology Agency.

Dr. Randall W. Jibson, who stayed from January 12 through March 31, 1987, is a researcher of Branch of Geologic Risk Assessment, Office of Earthquakes, Volcanoes and Engineering, Geologic Division, Geological Survey, U.S. Department of Interior, where he is engaged in the study of land slides induced by earthquakes.

During his stay, Dr. Jibson challenged to clarify the main cause of large land slides induced by earthquakes. He studied the efficiency of the Newmark's deformation for assessing the slope stability during earthquakes, and as well he joined the shaking table test on the ridge effect to the ground motion amplification using the silicon rubber model which were just in course of progress by the Ground Vibration Division. His study was supported by the Important and Fundamental Research Program of Science and Technology Agency.

Mr. Alonso Echavarria Luna, who stayed during January 9 through February 22, 1987, visited from the Geotechnical Department of Capture and Handling of Water, Ministry of Water Resources, Mexican Federal Government.

During his stay, Mr. Echavarria studied the strong motion records obtained in Mexico during the Mexico Earthquake of September 19 and 21, 1985. He analyzed the attenuation characteristics of maximum acceleration with epicentral distance using the records by the Guerrero Array which had been kindly sent to PWRI from Professor J. G. Anderson of University of California, San Diego after the earthquake. His visit was a part of his stay in Japan under the 15th. Personal Exchange Program between Japan and Mexico supported by the Japan International Cooperation Agency.

Main projects in progress at the Ground Vibration Division, PWRI, are reduction of hazards caused by ground liquefaction and the rationalization of input ground motion for earthquake resistance design of civil engineering structures. During their stay, aforementioned three engineers joined these projects from each approach and have made important contribution, and completed their respectable reports.

Their sincere efforts and passion to complete their work during the stay at PWRI should be greatly noted.

Almost one year has passed since their leave, and the limitation of budget delayed the publishing of their report, however, this report would advance the concerned research field and the friendship between Japan and U.S., and Japan and Mexico.

Finally all who were concerned to exchange these three engineers are greatly acknowledged.

January 19, 1988

Yasushi SASAKI, Editor
Head, Ground Vibration Division,
Earthquake Disaster Prevention Department,
Public Works Research Institute

ま え が き
(Preface in Japanese)

この報告は昭和61年度に土木研究所地震防災部振動研究室に滞在した3人の外国人招へい研究員が滞在中に実施した研究成果を収録したものである。

昭和61年10月4日から11月17日まで1箇月余滞在したジェフリー・ファラー氏は、米合衆国内務省開拓局工学研究センターから派遣された土質工学分野の研究者である。工学研究センターにおいてファラー氏は開拓局が関与するフィルタイプダムの地震時安定性に関する検討を担当している。

ファラー氏の招へい期間中の研究題目は「地震時の地盤液状化判定のための原位置試験法に関する研究」と題するもので、コーン貫入試験結果の評価に定常状態パラメーターの概念を導入しようとする試みを実施した。氏の招へいは科学技術庁外国人研究者招へい制度によるものである。

昭和62年1月12日から3月31日まで滞在したランドール・ジブソン氏は、米合衆国内務省地質調査所において地震時の斜面安定問題に関する研究を担当する研究者である。ジブソン氏の招へい期間中の研究は地震によって発生する大規模斜面崩壊の発生原因を検討することを目的とするもので、研究題目は「地震時の大規模斜面安定に及ぼす地震動の影響」と題するものである。このため大規模斜面の地震時安定性を評価する手法としてのニューマーク変位の有用性について検討するとともに、振動研究室で実施しているシリコンゴムを用いた模型振動実験結果の検討に参画した。氏の招へいは科学技術庁重点基礎研究制度によるものであった。

昭和62年1月9日から2月22日まで滞在したアロンソ・エチャバリア・ルナ氏はメキシコ合衆国農業水資源省導水管関連施設担当課長である。エチャバリア氏が滞在中に実施した研究は、1985年9月19日および21日に発生したメキシコ地震による強震動特性の検討である。メキシコ国内に設置されていたゲレロアレイで取得された強震記録を用いて、最大加速度の距離減衰特性に関する検討を実施した。氏の滞在は、海外協力事業団による第15回日墨交流計画に基く研修員としてのものであった。

振動研究室の実施している研究の主要な柱は地盤の液状化被害の軽減、ならびに耐震設計用地震動の合理化である。上記の3氏は、それぞれ異なった立場からこれらの研究に参画され、貴重な貢献をなされ、それぞれ立派な報告書を完成された。

不自由な異国の地で限られた滞在期間に仕上げられた報告書と、研究実施に際して示された熱意に深甚の敬意を表したい。

既に3氏の滞在期間から約1年が経過しており、報告書としての刊行が予算の制約のため遅れてしまったが、本書の刊行が関連分野の研究の進歩と日米、日墨間の友好の助けとなることを期待している。

昭和63年1月19日

編集者

建設省土木研究所地震防災部振動研究室
室長 佐々木 康

Contents

(目次)

Preface

Preface in Japanese (まえがき)

1. Abstracts 1
2. Abstracts in Japanese (要旨) 9
3. Study of In-Situ Testing for
Evaluation of Liquefaction
Resistance and Occurrence
(地震時の地盤液状化判定のための原位置試験法に関する研究) . . . 15
4. Research on the Effects of
Topographic Amplification of
Earthquake Shaking on Slope
Stability
(地震時の大規模斜面安定に及ぼす地震動の影響) 53
5. Attenuation of Peak Ground
Motions from Mexico Earthquakes
of September 19 and 21, 1985
(1985年メキシコ地震による最大地震動の距離減衰特性) 173

Abstracts

1. Study of In-Situ Testing for Evaluation of Liquefaction Resistance and Occurrence

Jeffrey A. Farrar

1.1 Preface

As a recipient of the Japanese Government Research for Foreign Specialists, I stayed at the Public Works Research Institute, Ministry of Construction from October 4, 1986 to November 17, 1986.

I would like to thank all of the people who made it possible for this assignment in Japan. Everyone involved helped me in such a kind and gracious manner to make my stay productive and enjoyable. My thanks are extended to the Science and Technology Agency for financial support and promotion of such international exchanges which surely have great benefit to the countries involved. The efforts of the Director of International Affairs, Miyabayashi of the Science and Technology agency are very much appreciated. My stay was also made possible through the interests of exchange of the U.S.-Japan Panel on Wind and Seismic Effects, UJNR and such interests are highly acknowledged.

I would like to express my gratitude towards the host agency, Public Works Research Institute, Ministry of Construction for accepting my stay and providing such kindness and excellent working conditions. The mission and facilities of the Public Works Research Institute provide most important research work in the field of civil engineering with superior capabilities for undertaking such studies. My sincere gratitude is extended to Director General-Dr. Tominaga, Deputy Director-Narita, and Assistant Director-Fukui. The efforts and interests of the Earth Disaster Prevention Department and Ground Vibration Division are appreciated. I would like to express my gratitude towards Dr. Iwasaki and Dr. Sasaki and all of the staff in these departments for such friendly reception and excellent working conditions.

1.2 Research Objectives and Activities

1.2.1 Objective

The objective of this study was to evaluate testing methods of ground soils for their ability to predict soil liquefaction during earthquakes. Liquefaction is a process where soil loses its strength and undergoes large distortion causing significant damage to structures and facilities. In this study, testing that is performed in the field using standard testing procedures was evaluated.

Specifically, in-situ field tests consisting of Standard Penetration (SPT), Cone Penetration (CPT), Flat Plate Dilatometer (DMT), and Vibrocone (VCPT) testing were evaluated. Results of a comprehensive U.S.-Japan field study of soil liquefaction sites performed in 1983 were evaluated. The study sites were located in Akita and Niigata. A field testing program using the DMT was

performed during my stay. The evaluation of the DMT testing was added to the original objectives of my study submitted in application to the Science and Technology Agency.

1.2.2 Activities

The activities during my stay are summarized below in chronological order:

October 4	Arrive in Japan
October 5-12	Working at Public Works Research Institute
October 13-14	Observing Vibrocone testing in Nagoya
October 15-17	Working at Public Works Research Institute
October 17	Field trip to Haneda Airport to observe ground improvement of reclaimed land
October 28-31	Field testing of DMT at Akita, Hachiro Gata Lagoon
November 1-13	Working at Public Works Research Institute and preparing report
November 14	Presentation of findings at Public Works Research Institute
November 17	Return to U.S.

1.3 Presentation of Findings

A draft report was presented to the Public Works Research Institute summarizing the evaluations of the testing methods. The report is attached to this report. Conclusions of the report are given below:

Conclusions

Comparisons of Liquefaction resistance evaluation techniques using the SPT, CPT, DMT, and Vibrocone have been performed with the data obtained from the 1983 U.S.-Japan cooperative field study sites. Pertinent conclusions regarding the methods considered along with recommendations for further study are as follows:

1) SPT data was reanalyzed to incorporate energy and sampler corrections using the N160 method proposed by Seed (1984) and compared to the Iwasaki et al. (1980) method. The methods agree very well with only slight differences at low of safety. At large factor of safety the methods diverge due to the limiting shear strain concept of the N160 method.

2) CPT data was analyzed using the qcl method proposed by Seed (1986). Results of an SPT dependent analysis and independent qcl factor of safety comparison show the method was very reliable and suitable for use in liquefaction analysis. Additional field performance comparisons in silty sands subjected to higher levels of cyclic loading are required.

3) CPT data was analyzed for a qualitative comparison to the method proposed by Olsen (1984). The method in concept is fundamentally correct but modifications would be required the relationship to the mechanical cone.

4) CPT data was analyzed and a method developed to evaluate liquefaction resistance using the state parameter proposed by Been and Jefferies (1985). The method should have comparable

accuracy as the qcl method and has additional benefit of estimating static stress-strain behavior in shear. Additional study will be required to evaluate prestress effects and extending the correlation to silty sands.

5) A series of six DMT soundings were performed by the Public Works Research Institute at the Hachiro Gata Lagoon area during October and November of 1986. Comparisons of static and dynamic Kd values did not indicate a trend correlated to liquefaction resistance. Static Kd values were compared to the method proposed by Robertson (1984) with favorable results. Additional study of silty sands subjected to higher levels of cyclic loading are required.

6) Of all static-dynamic penetration techniques the vibrocone test has the best potential for accurate prediction of liquefaction resistance. Equipment or calculation method need to be improved to overcome rod friction effects. the correlation of penetration decrease to cyclic strength requires further study.

2. Research on the Effects of Topographic Amplification of Earthquake Shaking on Slope Stability

Randall W. Jibson

2.1 Preface

As a recipient of the Japanese Government Research for Foreign Specialists grant, I visited the Public Works Research Institute, Ministry of Construction, from January 12 through March 31, 1987.

I am grateful for the opportunity to work at the Public Works Research Institute. During my visit, everyone showed the greatest Kindness and worked to make my stay comfortable and productive. Special thanks to the headquarters office of the Ministry of Construction for supporting my research and for making opportunities for exchange of ideas between scientists from different countries possible.

Many individuals at the hosting agency, the Public Works Research Institute, Ministry of Construction, worked to make my visit possible and to insure that it was productive. I extend my thanks to Director General Kamijo and to Deputy Director Narita and Assistant Director Fukui. I am particularly grateful for the efforts of Dr. Iwasaki, Director of the Earthquake Disaster Prevention Department, and Dr. Sasaki, Head of the Ground Vibration division, who extended the invitation for this trip and who provided excellent working conditions and a stimulating research project. The entire staff of the Ground Vibration Division was most helpful and made my stay very enjoyable.

2.2 Research Objectives and Activities

2.2.1 Objective

The objective of this research was to determine the effects

of topographic amplification of earthquake shaking on the dynamic stability of slopes, particularly in the case of large, deep-seated landslides. The massive debris avalanche from the south flank of Mt. Ontake triggered by the 1984 Naganoken-Seibu earthquake was investigated in detail as a possible case of slope instability caused by topographic amplification.

The research was executed as a series of discreet studies related to this objective:

1) A method was developed to estimate the relative dynamic performance of slopes having different stabilities in different levels of ground shaking. This required relating an index of dynamic slope performance, Newmark displacement, to seismic slope stability and earthquake magnitude and proximity.

2) Data from a previous field study of topographic amplification of earthquake shaking, conducted by the United States Geological Survey after the 1985 Central Chile earthquake, were analyzed for comparison with the Mt. Ontake landslide and with results from model tests described below. Also, data from the Matsuzaki dense seismic observation array, operated by the Public Works Research Institute, were analyzed for comparison.

3) Shaking-table tests on silicon ridge models were designed and conducted by the Public Works Research Institute. Results from these tests were analyzed and compared to results from the Chile earthquake experiment, the Matsuzaki dense seismic array, and stability studies of the Mt. Ontake landslide.

4) The dynamic performance of the ridge that failed as the Mt. Ontake landslide was analyzed, and the performances of several nearby intact ridges on Mt. Ontake were also analyzed using the method described above in #1. Results of this analysis were used to determine the likelihood that topographic amplification of earthquake shaking caused the Mt. Ontake landslide.

2.2.2 Activities

Activities during my visit to the Public Works Research Institute (PWRI) are summarized below:

January	12	Arrive in Japan
January	13-31	Research at PWRI on dynamic slope performance
February	1-4	Research at PWRI on seismic shaking intensity
February	5-8	Consultation with Dr. Ishihara at Tokyo University, Drs. Kobayashi and Sassa at Kyoto University, and Dr. Harp of the U.S. Geological Survey
February	9-18	Research at PWRI on Chile earthquake experiment
February	19-21	Visit dense seismic observation arrays at Matsuzaki, Numazu, and Shizuoka
February	22-	
March	16	Research at PWRI on results of shaking-table tests
March	17-19	Visit Nagano Prefecture to observe landslides triggered by the 1984 Naganoken-Seibu earthquake

March	20-26	Research at PWRI on stability of Mt. Ontake ridges
March	27	Tour of National Center for Research on Disaster Prevention
March	28-30	Preparation of final report at PWRI
March	31	Departure from PWRI

2.3 Presentation of Findings

A draft report summarizing my research activities and findings was presented to the Public Works Research Institute. The conclusions of the report are summarized below:

1) A method was developed to estimate relative dynamic slope performance of slopes having different stabilities subjected to different levels of ground shaking. The following equation can be applied in a variety of ways to the problem of dynamic slope performance:

$$\log(D_n) = 1.34(M) - 1.85\log(R) - 6.36(A_c) - 5.14$$

where D_n is the Newmark displacement in centimeters, M is the moment magnitude of the earthquake, R is the earthquake source distance in kilometers, and A_c is the critical acceleration of the landslide block in g 's. The Newmark displacement is an index of relative dynamic slope performance.

2) Results from the seismic site-response experiment conducted after the 1985 Central Chile earthquake illustrated the differences in ground shaking on ridges and intervening valleys. Significant amplifications (ratios greater than 2) occur for all ridge stations in a broad peak around 2-4 Hz and in a narrower peak around 8 Hz. These resonant frequency ranges may relate to some elements of the ridge geometry, such as height or width, that are about the same lengths as the incident seismic waves.

3) Results from analysis of the data from the Matsuzaki dense seismic array show that amplification of about a factor of 3 occurs at station over the thickest parts of the ridge and near the midpoint of the crestline at the top of the steepest part of the ridge as compared to stations at the base of the ridge.

4) Shaking table tests of silicon slope models showed that the resonant frequency, about 14-16 Hz for all models, is independent of the overall height, length, width, crestline angle, and side-slope angle. The aspects of the model geometries held constant for the models included slope length and maximum ridge thickness and the 45 degree angle of the rigid model base; therefore, the resonant frequency must be controlled by one or more of these constant factors.

The spatial distribution of acceleration ratios along the model crestlines and vertically along the maximum cross sections shows the inertial effects of amplification is greatest near the midpoints of the crestlines; for higher frequency responses (peaks 2 and 3), amplification maxima are near the one-third points along crestlines. Maximum amplification ratios are about 6 for peak 1; in higher frequency modes the ratios are less. In the maximum vertical cross section, amplifications range from 1 (no amplification) at the base of the ridge to more than 6 at the

ridge surface; amplifications in the fundamental mode are greater than those of higher frequency modes.

The first derivative of the acceleration distribution curves is an index of the shear stress (Shear-stress Index) induced by differential amplification within a ridge. The Shear-stress Index along the ridge crestlines, and maximum values are randomly scattered along the central 80 percent of the crestlines. The Shear-stress Index is much less within the ridge than along the crestline.

5) The estimated peak ground acceleration at the Ontake slide with no amplification effects is 0.39g, sufficient to cause catastrophic failure in all but the most favorable ground-water conditions. The maximum amplification required to cause failure in the most favorable ground-water condition is only 1.1. Therefore, it appears unlikely that amplification was required for failure.

Comparison of the dynamic performance of 38 ridges and profiles on Mt. Ontake revealed that only two had Newmark displacements greater than the ridge that failed. Thus, it appears that the ridge that failed did so because it was the most susceptible to failure in the conditions present during the 1984 earthquake, and no extraordinary effects such as topographic amplification would have been necessary to cause failure.

6) All of the evidence presented here indicates that (1) inertial effects and shear stresses induced by topographic amplification are greatest along ridge crests and are much less significant deep within and near the bases of ridges, and (2) no amplification of ground shaking was necessary to cause failure of the Mt. Ontake debris avalanche. Thus, topographic amplification may have little effect on deep-seated landslides. This is consistent with my observations and with published reports that ridges experiencing in past earthquakes displayed primarily surficial effects--churned earth and shallow, disrupted landsliding--located along their crests. Much information regarding the Mt. Ontake landslide is unavailable, however, and new data could indicate some effect of amplification.

3. Attenuation of Peak Ground Motion from Mexico Earthquakes of September 19 and 21, 1985

Alonso Echavarria Luna

3.1 Preface

As a trainee of the Personal Exchange Program between Japan and Mexico, I stayed at the Public Works Research Institute from January 9 to February 22, 1987. I would like to thank all of the people of Japan International Cooperation agency who made it possible for me to visit Japan. People of the hosting agency, the Public Works Research Institute are also acknowledged.

3.2 Research Objectives and Activities

3.2.1 Objective

For determining appreciate seismic effects to be considered in the design of civil engineering structures, it is essential to assess intensities and frequency characteristics of ground motions. According to results of strong motion observation, it is known that the characteristics of ground motion depend on the factors of earthquake magnitude, epicentral distance, ground condition of the site and so on. "Attenuation Equation" is the equation which assess the intensities of ground motion from the aforementioned factors. There have been many studies on attenuation equations and various equations have been proposed by the Public Works Research Institute and so forth in Japan.

This work presents the result of the regression analysis on the attenuation characteristics of ground motion records that were obtained at the Guerrero Array during the Mexico Earthquakes of September 19 and 21, 1985. Main objectives of this work are the followings:

- 1) Analyze the attenuation characteristics of the maximum acceleration, velocity and displacement.
- 2) Analyze the attenuation characteristics of acceleration response spectra.
- 3) Compare the attenuation characteristics of ground motion records of the Mexico Earthquake with the attenuation equation proposed by Public Works Research Institute.

3.2.2 Activities

January	9	Arrive at Public Works Research Institute
January	12-16	Review of the studies on the attenuation of ground motions
January	19-23	Editing records of the Mexico Earthquake
January	26-	
February	6	Regression Analysis
February	9-18	Preparation for the report of analysis
February	19-21	Visit dense instrument array at Matsuzaki, Shizuoka

3.3 Conclusions

The conclusions of this work are summarized below:

1) As for records obtained by the Mexico Earthquake of 1985, the attenuations of maximum acceleration, velocity and displacement with epicentral distance is smaller in comparison with the attenuation equations of the Public Works Research Institute. The attenuation characteristic of records of the Mexico Earthquake is relatively close to the attenuation equation for rock and diluvial site.

2) The attenuation of acceleration response spectrum of the Mexico Earthquake is close to the attenuation equation of the Public Works Research Institute in the short natural period range and is a little smaller than the equation in the long natural period range.

要 旨
(Abstracts in Japanese)

1. 地震時の地盤液状化判定のための原位置試験法に関する研究

米合衆国内務省開拓局工学研究センター

Jeffrey A. Farrar

(ジェフリー・ファラー)

1. 1 はじめに

筆者は、日本政府による招へい研究者として昭和61年10月4日より11月17日まで建設省土木研究所に滞在した。筆者の来日の機会を与えて頂いた科学技術庁、受入れ機関である建設省土木研究所ならびにUJNR（日米天然会議）耐風耐震構造専門部会の関係各位に感謝の意を表します。

1. 2 研究目的と研究活動

1. 2. 1 研究目的

地震時の地盤液状化判定のための原位置試験法に関する研究を実施した。

昭和58年5月に発生した日本海中部地震の後にUJNR耐風耐震構造専門部会の活動の一環として液状化判定のための原位置試験法の日米の実施方法の比較を行うための日米共同現地調査が秋田、新潟他で実施されている。

その成果のうち標準貫入試験法の日米の試験方法の違いによる差についてはUJNR耐風耐震構造専門部会第16回合同部会（昭和59年5月）でも報告されている。

しかし、標準貫入試験以外のコーン貫入試験や振動式貫入試験については突っ込んだ解析が十分なされておらず、上記現地調査で得られたこれら試験のデータを基に液状化判定法の詳細な解析を行うこととした。

さらにまた、この解析に際して必要となった地盤データを補足するため、新しい原位置試験法として最近注目されているフラットプレートダイレトメーターを米国から持参して建設省土木研究所と共同で現地調査を補足実施した。

これらの検討の目的は以下の通りである。

- 1) コーン貫入試験結果を用いた液状化判定法の妥当性解析
- 2) 振動式貫入試験による液状化判定法の妥当性解析
- 3) フラットプレートダイレトメーターによる液状化判定法の妥当性解析

1. 2. 2 研究活動

1986年 10月 4日（土） 来日

10月 6日（月）

土木研究所にてデータ解析

[振動式貫入試験現場視察（名古屋，11/13～14）]

[軟弱地盤改良工事現場視察（羽田，11/17）]

10月27日（月）

10月28日（火）

秋田県八郎潟で現地調査

[御所ダム強震計他視察]

11月 1日（土）

11月 4日(月)

土木研究所にてデータ解析, 報告書作成
[研究成果報告講演会(土研, 11/
14)]

11月17日(月)

11月17日(月) 離日

1. 3 研究成果

研究成果の詳細は別添報告書(英文レポート)の通りであるが、その主要な結論は以下の通りである。

- 1) 標準貫入試験結果に基いた液状化判定はSeedらのN₁₆₀法, 岩崎・龍岡の方法のいずれによってもほぼ同じ結果を得ることができる。
- 2) コーン貫入試験結果に基いた液状化判定は他の方法によるものと同一の結果を得ることができる。Seedらの提案したq_{c1}法は満足すべき結果であるが、Olsenの方法はメカニカルコーンにより得られたデータを用いる時はその判定表を修正すべきである。
- 3) フラットプレートダイレトメーター試験によるK₀値を用いてRobertsonによる判定を行った結果、判定した範囲内ではほぼ満足すべき結果を得たが、密な砂、シルト質砂については判定表を修正すべきかも知れない。
- 4) 全ての原位置試験法の中で信頼できる原位置試験法として最も有望なものは振動式貫入試験法である。しかしその判定法を満足すべきものに修正していくためにさらに幾つかの検討が望まれる。

2. 地震時の大規模斜面安定に及ぼす地震動の影響

米合衆国内務省地質調査所
R a n d a l l W. J i b s o n
(ランドール・ジブソン)

2. 1 はじめに

筆者は、日本政府による招へい研究者として昭和62年1月12日より3月31日まで建設省土木研究所に滞在した。筆者の来日の機会を与えて頂いた建設本省、土木研究所の関係各位に感謝の意を表します。

2. 2 研究目的と研究活動

2. 2. 1 研究目的

本研究の目的は、地震時大規模斜面安定性に及ぼす地震動の地形による増幅性を検討することである。1984年長野県西部地震により発生した長野県御岳山南麓の大規模斜面崩壊が、地形による地震動増幅により発生したと考えられた。研究を進めるに当っては、この問題に関連した以下に示す4つの小課題を設定し、各小課題に対して検討を行った。

- 1) 異なった地震動による種々の大規模斜面の動的安定性を評価するため、斜面の動特性を表わすニューマーク変位を、地震規模、震央距離により求める手法を検討する。
- 2) 1985年チリ地震の際、米国地質調査所が地形による地震動増幅に着目して行った現地観測データを、御岳崩れ、土木研究所が実施した斜面模型

振動実験及び土木研究所の松崎地区高密度強震観測記録と比較検討する。

- 3) 土木研究所が実施したシリコン斜面模型振動実験を、上記チリ地震観測データ、松崎高密度強震観測記録及び御岳崩れと比較検討する。
- 4) 御岳崩れの発生した大規模斜面及び御岳山腹の他の斜面に、1)で開発した手法を適用する。この結果より、御岳崩れの発生に及ぼした地震動の地形による増幅特性の影響、程度について検討する。

2. 2. 2 研究活動

1987年 1月12日

来日

1月13日

大規模斜面の動的特性に関する検討
(前項1)

1月30日

2月 1日

地震動の地形による増幅特性に関する検討
(前項2)

2月21日

2月 5日～ 8日 東京大学 石原教授
京都大学 小林教授
佐々教授

2月19日～21日 土木研究所松崎地区
高密度強震観測所現地調査

2月22日

土木研究所の斜面模型振動実験結果
検討 (前項3)

3月16日

3月17日

木曾御岳山斜面の地震時安定性
(前項4)

3月27日

3月17日～19日 長野県木曾御岳山現地調査

3月27日 科学技術庁国立防災センター訪問

3月28日

最終報告書作成 研究発表会

3月30日

3月31日

離日

2. 3 研究成果

研究成果の詳細は別添報告書(英文レポート)のとおりであるが、その主要な結論は以下の通りである。

- 1) 作用地震動の特性により異なった安定性を有する斜面について、斜面の相対的な動的特性推定法を開発した。次式は、様々な斜面の動的安定問題に適用可能である。

$$\log D_n = 1.34M - 1.85 \log \Delta - 6.36A_c - 5.14$$

ここで、

D_n : ニューマーク変位 (cm)
 M : モーメントマグニチュード
 R : 震源距離 (km)
 A_c : 限界加速度 (g)

- 2) 1985年チリ地震の余震を斜面上で観測した結果より、2-4 Hzの振動数領域及び8 Hz付近において地震動増幅が2倍以上になった。この共振振動数は、地震動波長と同程度の斜面高さ、幅等の地形条件に関連すると思われる。
- 3) 松崎地区高密度強震観測所で得られた記録より、最大加速度の増幅が3倍以上となる箇所は、斜面幅が最も広い箇所及び斜面中央部付近である。
- 4) 土木研究所で実施したシリコン斜面模型振動実験結果より、斜面の1次共振振動数付近では、斜面中央部付近で、また2次、3次共振振動数付近では、斜面の1/3地点付近に加速度応答が最大となる点があった。また、加速度応答の差から、せん断ひずみの相対的分布を求めることにより、斜面方向のせん断ひずみが、斜面最大断面内のせん断ひずみに比べて大きいことが明かとなった。
- 5) 御岳山周辺の38斜面について1)で求めた式を適用した結果、御岳崩れ発生地点の斜面より崩壊の発生しやすい斜面は、2斜面であった。また、地形による地震動の増幅特性がなくとも、御岳崩れが発生するのに十分な地震動が作用した可能性を認めた。
- 6) 地形による地震動の増幅は、斜面の尾根部の表層近くで大きくなり、ここでは加速度及びせん断応力が大きい。斜面内部や斜面深部ではそれほど増幅されない。このため、1984年長野県西部地震による御岳崩れは、地形による地震動の増幅特性がなくとも発生したと考えられる。

3. 1985年メキシコ地震による最大地震動の距離減衰特性

メキシコ合衆国農業水資源省

Alonso Echavarría Luna

(アロンソ・エチャバリャ・ルナ)

3. 1 はじめに

筆者は、日墨交流計画による研修員として昭和62年1月9日から2月22日まで建設省土木研究所地震防災部振動研究室に滞在した。筆者の来日の機会を与えて頂いた海外協力事業団、受入れ機関である建設省土木研究所の関係各位に感謝の意を表します。

3. 2 研究目的と研究活動

3. 2. 1 研究目的

公共土木施設を合理的に耐震設計するとともに各種の地震防災上の措置を講じるためには、建設地点等の当該着目地点で期待される地震動の強度を正しく

予想することが必要とされる。既往の強震観測の成果によれば、地震動の特性は、地震のマグニチュード、震央距離および当該地点の地盤条件等の要因により変化することが知られている。これらの要因と地震動強度との関係を求めたものが距離減衰式である。距離減衰式に関しては各種の検討が行われており、日本でも建設省土木研究所をはじめとして種々の距離減衰式が提案されている。

本研究は、1985年9月19日に発生したメキシコ地震およびその最大余震（同年9月21日発生）の際にメキシコ合衆国太平洋側沿岸に設置されていたゲレロアレで得られた記録をもとに、重回帰分析により地震動強度の距離減衰特性について検討したものである。本研究の主たる目的は、次のとおりである。

- 1) 最大地震動（最大加速度、最大速度、最大変位）の距離減衰式を求める。
- 2) 加速度応答スペクトルの距離減衰式を求める。
- 3) メキシコ地震による距離減衰式と建設省土木研究所から提案されている距離減衰式を比較する。

3. 2. 2 研究活動

1987年	1月 9日	土木研究所来所
	1月12日	
	1月16日	地震動の距離減衰特性に関する文献調査
	1月19日	
	1月23日	メキシコ地震記録の編集
	1月26日	
	2月 6日	重回帰分析
	2月 9日	
	2月18日	研究成果のとりまとめ、報告書作成
	2月19日	
	2月21日	土木研究所松崎地区高密度強震観測場視察

3. 3 研究成果

研究成果の詳細は別添報告書（英文レポート）のとおりであるが、その主要な結論は以下のとおりである。

- 1) 1985年メキシコ地震による最大地震動の距離減衰は、建設省土木研究所の距離減衰式と比較して震央距離による減衰が小さい。また、メキシコ地震による地震動の距離減衰は、日本の1種もしくは2種地盤（岩盤、洪積地盤）上の距離減衰式に比較的近い値を示す。
- 2) メキシコ地震による加速度応答スペクトルの距離減衰は、短周期領域では建設省土木研究所の距離減衰式とほぼ同程度の減衰特性を有し、長周期領域では若干減衰が小さくなっている。

STUDY OF IN-SITU TESTING FOR EVALUATION OF
LIQUEFACTION RESISTANCE AND OCCURRENCE

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STUDY OF IN SITU TESTING FOR EVALUATION OF LIQUEFACTION RESISTANCE AND OCCURRENCE

Jeffrey A. Farrar¹

ABSTRACT

Analysis of penetration testing for liquefaction resistance evaluations performed at field liquefaction sites in Japan is described. The study reviews results of standard penetration, cone penetration, and vibrocone tests performed in October and November of 1983 at six field study sites which exhibited evidence of liquefaction in Akita and Niigata, Japan. Standard penetration test data with energy measurements and corrections to account for sampler dimensions are analyzed using N1 and Japanese simplified methods, and factors of safety against liquefaction are compared. Mechanical cone penetration test data are analyzed according to methods proposed by Seed and Olsen. A method to evaluate liquefaction resistance from cone penetration testing by using the state parameter is developed and evaluated. Flat plate dilatometer tests were performed at three liquefaction sites in the Hachiro-Gata Lagoon area of Japan. Dilatometer tests were performed using quasi-static and dynamic advancement techniques, and ability to predict liquefaction occurrence is evaluated. Vibrocone testing and its ability to quantify cyclic strength is evaluated. Findings of the report indicate additional case study data from silty sand sites and with differing loading conditions are required to further proof test relationships.

KEYWORDS: Cone Penetration; In Situ Testing; Standard Penetration; Vibrocone

1. INTRODUCTION

The purpose of this report is to investigate various in situ testing techniques for evaluation of liquefaction occurrence in Japan. The current report is an extension of U.S.-Japan cooperative studies initiated in 1983 with fieldwork. This report focuses on the cone penetration (CPT) and flat plate dilatometer (DMT) tests but also addresses the use of standard penetration (SPT) and vibrocone penetration (VCPT) testing. Data obtained in a comprehensive field study performed in 1983 at six study sites are reviewed, and a field study with the DMT was performed. Results of these studies have developed valuable case history information on various methods of geotechnical testing and will improve our ability to accurately predict occurrence of earthquake liquefaction.

From the results of the 1983 field study, several important papers have been published to date. However, there is yet no evaluation of the CPT test from the data collected. Several researchers have recently published papers on the use of CPT for liquefaction resistance evaluation, but lack case history validation. Results of the U.S.-Japan 1983 field data can be

used to check these methods. Another new in situ test, the DMT, has recently become very popular among geotechnical groups. Laboratory chamber testing in sands has indicated that the instrument may have good potential for evaluation of liquefaction. To date, only one field liquefaction study site in the U.S. has been investigated. In this study of the DMT, three locations in the Hachiro-Gata Lagoon were investigated by quasistatic and dynamic advancement techniques.

1.1 Background Information

A comprehensive field study was performed to investigate in situ testing techniques in October and November 1983 in Akita and Niigata prefectures. Three sites from each prefecture were selected for study, with two sites showing evidence of liquefaction and one site showing none. As shown on Fig. 1, the Niigata sites were affected by the 1964 Niigata earthquake of Richter magnitude 7.5 and Akita sites were affected by the 1983 Nihonkai-Chubu earthquake of Richter magnitude 7.7. According to strong motion seismographs in areas nearby study sites, maximum ground accelerations of 0.17 and 0.16 g were used in analyses of data for Akita and Niigata sites, respectively.

Study sites in Akita were labeled sites A, B, and C. The sites are all located in the Hachiro Gata Lagoon area as shown on Fig. 2. The Hachiro-Gata area is reclaimed land surrounded by protective dikes. Site A is located on the approach to Gomyoko bridge at dike station WC 15+553. This location experienced serious lateral spreading of the approach fill. Site B is located at dike station FD 6+900 where there was a slope failure of the dike and numerous sand boils. Site C is located at dike station WC 0+580 where no damage was experienced.

Study sites in the Niigata area were labeled D, E, and F (Fig. 3). Site D is the Kawagishi-Cho apartment building site which is famous for the rotational movement of apartment buildings. Ishihara and Koga (1) have documented soil conditions at the site through SPT, CPT, and undisturbed sampling. Site E is on the left bank of the Showa Ohashi bridge where extreme damage to the bridge and surrounding ground mass was experienced. Site F is on the right bank of the Showa-Ohashi bridge site where no damage occurred. Analysis of the Showa-Ohashi bridge damage was performed by Iwasaki, et al. (2).

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The field study at these study sites was probably one of the most intensive of its kind to date. A listing of the testing performed at each study site is given in Table 1. Some of the tests performed included SPT with energy measurements, CPT, Swedish dynamic cone, Swedish weight sounding, piezocone, vibrocone, and other dynamic cone tests. Borings and soundings were generally spaced at distances of 2 m from one another. Soil types present at the sites are summarized on figure 4. The sites are almost completely comprised of clean fine to medium sands.

Following the field investigations, OYO Corporation compiled the sounding and laboratory data obtained from the study in a report dated March 1984. Presentation of SPT energy measurements and vibrocone results were made by Sasaki, et al. (3), at the meeting of the U.S.-Japan Panel on Wind and Seismic Effects (UJNR) in May 1984. Results of SPT measurements were documented by Kovacs, et al. (4), in National Bureau of Standards report dated August 1984. With additional data on SPT energy made available, Seed reevaluated the SPT data base to refine the N1 method (5). At the UJNR meetings of May of 1985 and 1986, Sasaki, et al. (6, 7), presented papers refining the vibrocone test. On August 17-20, 1985, the UJNR group sponsored a workshop on in situ testing with results soon to be published. Ohya and Iwasaki (8) presented a paper which concentrated on the dynamic penetration testing and its correlation with liquefaction resistance. In this paper, relationships were proposed for a ratio of static penetration from the CPT to dynamic penetration from the Swedish dynamic cone or automatic ram sounding test to predict liquefaction.

Several approaches to liquefaction resistance evaluations from CPT testing have evolved over the past 10 years. Many initial proposals were not very refined and sometimes depended on extensive site-specific correlation with the SPT test. In 1985, both Robertson (9) and Ishihara (10) proposed relationships with normalized cone tip resistance and cyclic strength. Both methods were felt to only be applicable to clean, low compressibility sands as an increase in fines or more compressible minerals such as micas would greatly reduce cone resistance. At the ASCE specialty conference on in situ testing, Seed (11) proposed a similar relationship based on refined SPT-CPT correlation. Only two case history sets of information were available to validate the proposed method. The method included correlations to account for fines content.

Some researchers have argued that the cone appeared to be too insensitive to stress-fabric effects which are beneficial to liquefaction resistance. Olsen (12) presented a method which incorporates normalized tip and sleeve resistance to evaluate liquefaction strength. This method was developed for the electric cone

with a sleeve directly behind the tip. The difficulty with this method is that sleeve data must be accurately determined. In cohesionless soils, sleeve resistance is typically one-tenth of the tip resistance; and additional accuracy of less rugged, independent load-cell electric cones is required for the method.

Researchers working on the Beaufort Sea drilling islands developed a correlation of state parameter to normalized cone resistance to study failures. The concept of the state parameter was presented by Been and Jeffries (13) in 1985. The state parameter measures the distance or position of a sand in given void ratio stress condition to a reference steady-state line. The steady-state line is the same as defined by Castro and Poulos and originally proposed in the critical void ratio concept of Casagrande. The advantage to this approach is that many significant engineering parameters such as static stress - strain behavior can be correlated to the state parameter. In addition, positive values of the state parameter are indicative of collapse potential of sand structure under existing stress conditions resulting in very large strain development in loading events such as earthquakes. Recently, Been, et al. (14), have determined a correlation of normalized cone tip resistance with state parameter by determining steady-state lines for sands used in cone chamber tests.

In this report, each of the concepts for CPT evaluation of liquefaction will be studied. Depending on the results, more in-depth studies will be made. Conclusions regarding the general applicability of the methods will be made.

An additional study was performed with the flat plate dilatometer (DMT) testing device to gain valuable case history experience with the instrument. The Public Works Research Institute arranged for a field study with this device which took place in the Hachiro Gata Lagoon area during October 29 through November 1, 1986. The DMT test has been discussed by Marchetti (15) who developed the test in the mid-1970's. In 1982, Marchetti (16) discussed the possible use of the test for liquefaction evaluations. Robertson (17) proposed a correlation of the horizontal stress index to cyclic stress ratio. Based on discussions at the UJNR workshop on in situ testing and with users and manufacturers of the DMT, the differences between dynamic and quasi-static insertion were also evaluated to see if sensitive sands could be identified.

2. DISCUSSION

2.1 SPT Simplified Methods

Two methods of evaluation of liquefaction resistance using the SPT find widespread usage. In Japan, Iwasaki and Tatsuoka, et al. (18), have developed a method whose basis is in a large quantity of undisturbed sample testing to

develop a correlation between cyclic strength for 20 cycles of equivalent loading (R_{120}), SPT N value normalized for overburden pressure effects, and particle size expressed in terms of mean particle diameter (D_{50}).

In the U.S. Seed (5) and coworkers developed a method which separates liquefaction and nonliquefaction from observed field performance data on a plot of cyclic stress ratio and SPT N_1 value normalized to a stress level of 1 ton/ft². The method also distinguishes for fines content in sands. Neither method is applicable for low-plasticity silts or gravelly soils.

When studying previous liquefaction sites, the most difficult task is to determine the actual zones of liquefaction. Since SPT testing is the most widely accepted method of evaluating liquefaction, it will form the basis for testing cone penetration and other methods for their ability to predict liquefaction. As a result, the SPT analysis must be carefully performed to evaluate zones of liquefaction.

Results of SPT analysis using both SPT techniques by Ohya and Iwasaki (8) are shown on Fig. 5. On this figure, liquefied and critical zones where factor of safety is close to 1 are shown. Inspection of these plots indicates there is good agreement among the methods in their predictions. One will also notice, however, that the two methods diverge at high and low factors of safety. This trend has been identified in the past by others. The divergence at large factors of safety could possibly be explained by loosening of undisturbed samples of dense sands during the sampling process, thereby reducing the R_{120} strength.

The analysis of factor of safety by the Seed N_{160} method was reperformed to incorporate energy corrections and corrections for sampler barrels in accordance with the 1984 report. Calculations for this analysis are summarized on Table 2. The boring logs used for this analysis were reported by OYO Corporation in March 1984. To estimate the induced cyclic stress ratio at the depth of interest, the simplified equation proposed by Seed was used. To simplify calculations, a single unit weight of 120 lb/ft³ was used and the reduction factor r_d was the equation proposed by Iwasaki (18). SPT N values obtained with the ASTM sampler were increased to account for larger inside barrel diameter by using the recommendation by Kovacs (4). The factor of 20 percent is probably too large for low blow count soils, and a factor of 10 percent is probably more appropriate. After conversion for sampler size, the N value was then adjusted to an energy level of 60 percent using the equation proposed by Seed and Kovacs (4, 5). At each field study site, only several energy data points were available at depths of greater than 30 ft. Where energy data were available, they were directly applied to the N value obtained; and an average value was used for the remainder

of the boring. At site E, energy data were not available; but the same operator and drill rig previously used at site D were again used. At site F, no data could be assumed; therefore, Japan average energy was used (4). Corrections for rate and borehole size, as recommended by Seed (5), were not used as it was felt that they were purely speculative and probably overemphasized.

After conversion to N_{60} , the value was then normalized to an overburden pressure of 1 ton/ft² using the following equation for the factor C_N ;

$$C_N = (\sigma'_v)^n = (\sigma'_v)^{-0.5} \quad \text{Equation 1}$$

with σ'_v (vertical effective stress) in ton/ft²

The graphic C_N values proposed by Seed (1984) were not used as it requires prejudging the relative density of the deposit. The values of C_N using the above equation are median points of those from the two curves recommended by Seed.

After obtaining values of N_{160} along with fines content information, cyclic strength was determined by visual examination of the N_{160} chart. If fines were present, increased strength was interpolated from the curves for various ranges of fines contents. In some cases, the cyclic strength was estimated as infinite following concepts of limiting shear strain potential.

The resulting change in factor of safety from both SPT methods is shown on figures 6 and 7. Factors of safety from the Japanese method were not recomputed but scaled graphically from the report by Ohya and Iwasaki. It can be seen that, at lower factors of safety, the agreement is now better between the two methods in most cases. This can be attributed to the higher energy level (68 percent) delivered in Japan. Correction to the N_{60} value results in an increase of the N value. Remaining differences at a low factor of safety can be attributed to slightly higher cyclic strength observed in the Japan undisturbed sample test data due to consolidation of loose sands.

2.2 Analysis of 1983 Field Data Using the QC1 Method

The method presented by Seed (11) is refined correlation of cyclic stress ratio to normalized cone penetration resistance (Q_{c1}). The effects of particle size on the correlation were inferred by refining the Q_c/N versus D_{50} data base to incorporate energy consideration on the SPT value and then using the SPT data base to develop the relationship. In this approach, the N value should be normalized to the 60-percent energy level to check the Q_c/N_{60} versus D_{50} relationship. Using Q_c/N_{60} and D_{50} , the Q_{c1} method could be extended to silty sands by inferring fines content from D_{50} ; and dividing lines separating occurrence of liquefaction were developed. The chart for clean sands is very

similar to the charts proposed by Robertson and Ishihara (9,10). Seed compared this approach to case history data at the Niigata Kawagishi-Cho apartment site [Ishihara and Koga (1)] and the Imperial Valley study sites in the U.S. [Youd (19)]. Additional field performance data from this study will assist in validating this method.

In the 1985 UJNR workshop on in situ testing, Ohya and Iwasaki presented evidence that the ratio of Q_c to dynamic blow count N_d from the Swedish dynamic penetration test could be used to predict occurrence of liquefaction. Larger ratios of Q_c/N_d had indicated increasing possibility of liquefaction. In this study, the Q_c/N_{60} ratio was also studied for this effect. A comparison of Q_c/N_{60} versus D_{50} particle size is shown on figure 8. On the figure, points which did or did not liquefy according to SPT N_{60} analysis are appropriately marked. From this plot, there is no consistent trend in liquefaction occurrence; and there is wide scatter among the data.

Begemann friction mantle cone (ISSMFE M2 penetrometer) tests were performed at each of the study sites. Tip resistance data, Q_c , were normalized for overburden pressure effects using the same correction factor C_N as used in the N_{60} analysis. It should be noted that this normalization proposed by Seed (11) is not in accordance with findings presented by Jamiolkowski (20), who recommended the n exponent of equation 1 to be -0.72 with σ'_v in kg/cm^2 . As with the SPT analysis, a single unit weight of $120 \text{ lb}/\text{ft}^3$ is used with all stress computations.

A dependent analysis of occurrence of liquefaction was performed to evaluate the Q_{c1} method. At each SPT depth interval, Q_{c1} values were averaged. Plots of Q_{c1} and cyclic stress ratio were made on the proposed chart, and points which did or did not liquefy from SPT prediction could be marked. A summary of dependent data is given in Table 3, which also shows the average friction ratio in the SPT interval. All of the values are less than 2.5 percent, which is the common borderline dividing sands and sand-silt-clay mixtures using the Begemann friction mantle cone. For electrical cones, this value will be different. The Q_{c1} method should not be used for liquefaction assessment using the Begemann cone if higher friction ratios are obtained.

A summary chart, Fig. 9, shows that liquefaction occurrence is accurately predicted at the sites. From the individual charts for each site the cyclic strength was determined by visual inspection, taking into account the percentage of fines and D_{50} particle size. The cyclic strength and factor of safety for the Q_{c1} data are summarized in Table 4 and factor of safety is plotted on Figs. 6 and 7 for comparison to SPT methods. The Q_{c1} factor of safety compares very well with SPT methods. It is interesting

to note that, at site A, liquefaction is predicted to greater depth using the Q_{c1} method. This may possibly reflect on the greater uncertainty of overburden pressure corrections for the SPT at shallow depths.

Results of the study of the Q_{c1} method indicate that the method is very suitable for evaluation of liquefaction resistance for clean sands. Since no sample is obtained in the test, the friction cone should always be used to check friction ratio. Further field evidence is required to evaluate silty sands and sand deposits which have been subject to larger cyclic stress ratios. Additional study is also required for sands of varying compressibilities due to varying mineralogies.

2.3 Evaluation of Olsen's CPT Method of Liquefaction Assessment

A method of liquefaction resistance evaluation using normalized cone and friction sleeve data from the CPT test was proposed by Olsen (1984). By incorporating normalized sleeve resistance, sensitive soils can be detected. The CPT tip resistance is relatively insensitive to stress history effects, such as overconsolidation, and some aging effects, such as slight cementation. The sleeve is more sensitive to effects, such as overconsolidation.

Olsen developed a soil characterization chart that identifies sensitive, normally consolidated, and overconsolidated behavior. The advantage of this method is that it may also identify sensitive silts and silt-sand mixtures which are potentially liquefiable. Normalization of the tip resistance Q_c uses the same form as previous sections of this report; but the exponent n in equation 1 varies with soil type, and an iterative process is used. Sleeve friction is normalized by dividing by the effective overburden pressure, since it is a measure of remolded strength. Using SPT-CPT and laboratory cyclic stress ratios, stress ratio contours of liquefaction resistance strength were drawn on the characterization chart.

The relationship developed by Olsen was developed for electric cone testing with friction sleeves located immediately behind the tip. The sleeve friction measured by the Begemann friction mantle cone is located further back behind the cone tip. The sleeve has a shoulder or bearing surface at its base and, therefore, measures higher levels of sleeve friction. DeRuiter (21) states that the electric friction is, on the average, only one-half of Begemann mechanical friction values. In electric cone testing with subtraction-type cones, the sleeve friction can be in error as much as $\pm 0.3 \text{ ton}/\text{ft}^2$, making it difficult to obtain accurate friction readings.

For this study, the Begemann cone data were normalized for a qualitative comparison with

Olsen's chart to evaluate if use of such a chart was feasible for mechanical cones. The exponent n to obtain the correction factor to normalize Q_c was taken as -0.72 , since practically all of the soils were clean sands. Friction sleeve data were not reduced to be more comparable to electric cone friction values. Normalized tip and sleeve resistance data points are summarized in Table 3. A summary plot with data points for all six sites is shown on the soil characterization chart on Fig. 10. On Fig. 11, data from site B are plotted on the characterization chart, which contains the contours of cyclic strength. Beside the data points on this figure, the SPT N_{160} cyclic strength values are shown.

From review of the figures, it is evident that the Begemann friction values are higher than the electrical data. It is possible that the cyclic stress ratio contours and characterization zones could be shifted to the right to make Olsen's method applicable to the Begemann cone. The general trend of liquefiable data points lying on the sensitive side of the normally consolidated zone confirm the concepts of this method. Dense dilative sands fall in the overconsolidated zone.

Overall, conclusions regarding this approach are that the method in concept is fundamentally correct. Possibilities exist to modify this relationship for mechanical cones. The benefit of this approach is that it may be able to identify sensitive silts and sand-silt mixtures and eventually apply cyclic strength estimates. For now, if sensitive sands with fines or silts are identified by the method, laboratory testing may be required. Disadvantages to the method are its log-log relationship, which makes resolution difficult. Also, sleeve friction measurement is more sensitive and subject to error.

Since accurate conversions of mechanical to electrical cone data cannot be made, electrical cone data are required from case history sites in clean sands, sands with fines, and silts to validate the method.

2.4 Evaluation of the State Parameter for Liquefaction Assessment

The state parameter (ψ) proposed by Been and Jefferies (13) is simply a convenient method to develop a common referencing system which can quantify static stress - strain behavior of sands and eventually sand-silt mixtures. The concept of the state parameter is shown on Fig. 12. The unique reference line is the steady-state line which is fairly easily measured on remolded contractive specimens. The state of the soil is expressed as a function of mean normal stress and void ratio. Soils, whose position is below the steady-state line with a negative value of state parameter, will exhibit dilatant behavior in shear. Soils, whose state lies above the steady-state line with a

positive value of state parameter, are contractive in shear. The contractive condition indicates potential collapse of structure with extremely large deformations. Contractive soils are rare in natural deposits; but if identified, they can exhibit extensive deformation upon earthquake or additional static loading. Been and Jefferies (13) have related the state parameter to angle of phase transformation, drained peak angle of shearing resistance, dilation rate, and decrease in peak to steady-state friction angle.

In a currently unpublished paper, Been, et al. (14), have determined the steady-state lines for all of the sands used in CPT chamber testing and have found a unique relationship to Q_c normalized to mean effective confining pressure and the state parameter in clean sands. The initial correlation developed is shown on Fig. 13. Additional study is needed to determine the relationship for silty sands.

In this study, it was desired to calculate the state parameter for qualitative comparison to liquefaction resistance. The equation proposed by Been, et al. (14), to determine the state parameter from Q_c is as follows:

$$\psi = -.092 \ln \left[\left(\frac{Q_c - I_1}{I_1'} + 17 \right) / 31 \right] \quad \text{Equation 2}$$

where I_1 and I_1' are the first total and effective stress invariants (mean normal stress). In equation 2, stress is expressed in kPa. Calculation of state parameter requires knowledge of the value of K_0 (ratio of in situ vertical to horizontal stress). Based on results of DMT tests to be presented later in this paper, a K_0 value of 0.6 was assumed for liquefied sites and 1.0 for nonliquefied sites. The state parameter was calculated for all Q_c values.

Average state parameter values in SPT depth intervals are summarized in Table 4. Inspection of these data reveals extremely low state parameter values in a low cone bearing zone from 5.0 to 6.5 m in depth at site B. Liquefaction sites D and E had more moderate values of the state parameter.

The state parameter for the zone present in site B is very close to being a contractive value; but if the equation for state parameter is examined, it is found that extremely low values of tip resistance would be required to indicate a contractive condition. In fact, the tip resistance may approach zero since the sand structure collapses away from the cone.

Results of this analysis indicate that moderately dilative sands are subject to level ground liquefaction and resulting excessive deformations. In order to develop a correlation in terms of the state parameter, the Q_{c1} versus

cyclic strength relationship was used. Q_{c1} values for clean sands from the chart proposed by Seed were converted to state parameter values by assuming $K_0 = 0.5$ with typical sand unit weight and ground-water location at the ground surface. Then, state parameter points calculated in SPT intervals were plotted with liquefaction occurrence as predicted by the SPT N_{160} method noted. The results are shown on Fig. 14.

The results indicate that the majority of the liquefied data points fall in the proper position, with exception of two near-surface data points at sites B and C and a point at site D at 7.3 m with high cone bearing. At site C, there was no evidence of liquefaction. As discussed earlier, shallow blow counts from the SPT are more difficult to evaluate due to uncertainty of pressure normalization. Also, erratic energy transmission is a problem with the SPT at shallow depths. It is highly probable that the cone can better estimate near-surface liquefaction occurrence.

The effect of deriving a state parameter versus cyclic strength correlation based on $K_0 = 0.5$ conditions will have a similar effect as the N_1 method, which disregards varying K_0 in pressure correction of the N value. In equation 2, the state parameter is most sensitive to the Q_c value and less sensitive to mean normal stress, since the tip resistance is typically two orders of magnitude larger than mean normal stress levels. At higher K_0 values, the dominant increase in Q_c will cause a more negative value of state parameter to be estimated if $K_0 = 0.5$ conditions are assumed. This will result in predicted increase in cyclic strength, which is appropriate for liquefaction studies. However, the $K_0 = 0.5$ assumption would overestimate the dilational tendency for static stress - strain behavior predictions.

In conclusion, there seems to be a relationship between the state parameter and occurrence of liquefaction. Further study should be performed to evaluate the effects of higher K_0 values on the state parameter and its ability to predict liquefaction. Since it is difficult to determine field K_0 values, a $K_0 = 0.5$ assumption is appropriate for young alluvial soils which are typical subjects of liquefaction studies. The shortcoming of the method is that relationships still need to be developed for silty sands and silts. Additional studies are required to test sand in a contractive condition and sands which contain fines. The benefit of this approach for clean sand at this time is that static stress - strain behavior of the soil can also be estimated. As with the Q_{c1} method, there is no sample obtained, so the method should only be applied with friction ratios of less than 2.5 percent and gradation of the sand should be checked to be sure the fines content is less than 10 percent.

2.5 Analysis of Flat Plate Dilatometer Test Results

The flat plate dilatometer test (DMT) was introduced by Marchetti in the mid 1970's. Marchetti (15) presented a summary of the equipment and data reduction procedures in the ASCE Geotechnical Journal. The DMT consists of a flat blade-shaped penetrometer with a 60-mm circular expandable membrane located on the face of the blade. The blade is usually quasi-statically advanced at 20-mm/s at 20-cm intervals, while thrust pressure at the surface is recorded. At each 20-cm increment, pressure is released from the rods and the membrane is expanded. Two readings are obtained at each interval:

- First, an internal pressure to lift the membrane into an approximately planar condition
- Second, an internal pressure to extend the membrane approximately 1.1 mm into the soil

The two readings are obtained in about 30 seconds. After subtraction of inertial pressures required to expand the membrane freely in air, values of P_0 and P_1 are obtained from the first and second readings, respectively. Based on the two readings, three index parameters are determined as follows:

$$\text{Material or Deposit Index, } I_d = \frac{P_1 - P_0}{P_1 - U_0}$$

$$\text{Lateral Stress Index, } K_d = \frac{P_0 - U_0}{\sigma_{v0}}$$

$$\text{Dilatometer Modulus, } E_d = 34.7 (P_1 - P_0)$$

The material index is used to determine the soil type. It represents the normalized difference between 1-mm expansion pressure, P_1 , and contact pressure, P_0 , which is the measure of soil rigidity. Since soil classification is based only on rigidity, some errors in estimated soil type may exist. The horizontal stress index is related to lateral earth pressure at rest. The dilatometer modulus is a numerical solution for $(E/1 - \mu^2)$ when expansion is modeled as loading of a circular area in an elastic half space. The modulus obtained must be empirically related to soil modulus since drainage conditions and disturbance vary.

Dilatometer parameters have been related to K_0 , δ' , S_u , P'_c , E_{25} (sand), and E (clays). The soil friction angle in sands is now based primarily on thrust data and bearing capacity analysis [Schmertmann (22)]. Based on initial

chamber tests in normally consolidated and prestressed sands, Marchetti (16) felt that the instrument was more sensitive to prestress than the CPT and proposed it as a viable method for evaluation of liquefaction potential. Marchetti proposed that K_d values of less than 1.5 would indicate sands at relative densities of less than 60 percent and that resistance to liquefaction in terms of cyclic strength may be roughly proportional to $K_d/10$. A most recent paper showing additional chamber test results in sands was presented by Baldi, et al. (23). Robertson (17), after consideration of chamber test data and a relationship between relative density and liquefaction resistance proposed by Christian and Swiger (24), proposed a correlation between the K_d and liquefaction resistance. After attending the 1985 UJNR workshop on in situ testing, DMT users felt there may be some relationship of liquefaction to the change in K_d with static and dynamic advancement. In this study, both the effects of static and dynamic driving and Robertson's method will be analyzed.

On October 29 through November 1, 1986, the Public Works Research Institute, Ministry of Construction, Japan, performed a series of nine soundings using DMT and CPT testing in the Hachiro Gata Lagoon area. DMT equipment was supplied by the Bureau of Reclamation for this study. Six soundings were located in sites B and C of the 1983 field study. An additional site A' was located near the Gomyoko Bridge at dike station WC 13+620 where the dike had settled up to 1 m, and approximately 6 m of banking soil were available for testing. CPT and SPT data were available for site A' from previous investigations. At each site, a cone penetration test using a Dutch mantle cone (ISSMFE penetrometer M1) was performed to evaluate any change in soil conditions since the dikes were rehabilitated after 1983. The rebuilding did not include subsurface densification.

At each site, dynamic and static DMT tests were performed to evaluate effects on the K_d index. Static DMT tests were advanced with a portable cone penetration machine which used anchors to develop additional reaction. Thrust pressure to quasi-statically advance the DMT was measured at the surface with CPT hydraulic pressure gauges. At site B, the pressure gauges malfunctioned and a second sounding with the DMT blade was performed. The Swedish automatic ram sounding equipment was used to dynamically advance the DMT. This device uses a chain cam mechanism to lift and drop the hammer so that consistent energy is applied. The 63.5-kg hammer is dropped from 50 cm. The DMT blade was driven with CPT rods for 20 cm, and the number of blows were recorded for each increment. On the two dynamic soundings at sites B and C, the hammering action loosened the wire contact at the base of the sensing disk at about 7 and 5 m, respectively; and the soundings were terminated. No other specific equipment problems occurred.

The completed sounding numbers, test site, test type, and depths are listed below:

Sounding No.	Site	Test type	Depth
86-1-2	B	Quasi-static	15 m
86-1-3	B	Dynamic	7 m
86-2-2	A'	Quasi-static	8 m
86-2-3	A'	Dynamic	7 m
86-3-2	C	Quasi-static	5 m
86-3-3	C	Dynamic	4 m

DMT data were reduced using IBM PC Basic program DILLY4 distributed by GPE, Inc., the U.S. distributor of the DMT. The program provides tabular output of all parameters and a graphic output of strength, stress, and modulus values. Computer data reduction used methods recommended by Schmertmann (22) for plane strain friction angle and K_0 . For the dynamic soundings, the thrust data from the adjacent static tests were input instead of trying to convert the number of blows to thrust.

Summary graphs which contain pertinent data from both quasi-static and dynamic tests on one graph for each site are shown on Figs. 15, 16, and 17. Cone penetration data from the 1983 field studies and the current study are plotted in terms of Q_{c1} for sites B and C are shown on Figs. 18 and 19. From Fig. 18, it can be seen that Q_{c1} has increased to a depth of 5 m since 1983. Below 5 m, the same zone of low cone bearing sands exists with little increase in Q_{c1} . At site C, it can be seen that Q_{c1} has increased over a depth of 2.5 to 5 m.

At site A', the DMT summary graph (Fig. 16) indicated an interbedded deposit of silty sands and sands with highly erratic data. Cone tip resistance data also indicated soils with fines. Data from site A' were not included in the analysis of liquefaction resistance. It will be included when additional gradation and cone data from the previous borings can be analyzed.

The effects of dynamic driving of the DMT in loose and dense sands can easily be distinguished from the summary graphs. Dynamic driving causes an increase in material index and friction angle and a decrease in horizontal stress index and modulus. Of interest for liquefaction potential is the decrease in K_d . At site B, dynamic driving resulted in reducing K_d to below 1 in critical liquefied zones. At site C, the reduction in K_d is also apparent; but values remained above 2. In an attempt to quantify this reduction, the ratio of dynamic to static K_d were analyzed. At site C in dense sand, a 60 percent reduction of K_d occurred at

3 to 4 m in depth. At site B, an 80-percent reduction in K_d occurred at a depth of 5 to 6 m in loose sand. However, a 90-percent reduction in K_d occurred at 4 to 5 m at site B where normalized cone bearing was significantly higher. Additional checks of dynamic and static K_d ratios resulted in no correlation to liquefaction resistance. The only conclusion that can be drawn is that, if dynamic K_d is less than 1, liquefaction seems highly likely.

The quasi-static K_d values were then compared to Robertson's method. K_d values were averaged in the 1983 SPT intervals. Using the SPT factor of safety as a guide, K_d values in zones which had liquefied at site B could be plotted on the proposed relationship by Robertson on Fig. 19. These data confirm a limiting K_d value of 4.2 at cyclic stress ratio of 0.18. At site C, the K_d value varied considerably in the SPT interval; and a bar was used to show the nonliquefied zone. Since Q_{c1} had increased in 1986, the Q_{c1} cyclic strength was used to show increased strength of the deposit. This approach indicates the K_d versus cyclic strength line may increase more sharply than proposed by Robertson. Additional field performance data will be needed to confirm the K_d relationship at higher cyclic stress ratios.

In conclusion, the field tests at Hachiro Gata have resulted in a better understanding of the DMT in sands. Major changes can occur between static and dynamic DMT tests to the point where dynamic driving should be discouraged in loose and dense sands alike. It is doubtful that a useful correlation between dynamic and static K_d values and liquefaction resistance can be developed. Robertson's proposed correlation works well, but additional information will be needed at higher levels of cyclic loading and in dense sands at low levels of cyclic loading.

Since material index is not necessarily a good indicator of soil type, there may also be questions regarding fines contents in the soils tested.

2.6 Comments on the Static and Dynamic Penetration Resistance

So far on this project, there have been four types of static dynamic comparisons of penetration resistance attempted. The ratio of cone resistance to SPT resistance was shown on Fig. 10 to be highly erratic. Likewise, K_d ratios obtained in this recent study do not show consistent trends. Ohya and Iwasaki (1985) showed some success in correlating CPT Q_c to the Swedish dynamic penetration N_d value. However, there is also considerable scatter among the data; and the relationship could not be correlated to quantify cyclic strength. Sasaki and others at the Public Works Research Institute have shown the vibrocone to have good success, at least in a qualitative sense. If any method of a static to dynamic comparison methods stands

a chance of success, it seems to be the vibrocone.

Recent efforts to quantify the decrease in cone penetration resistance, D , from the VCPT with cyclic strength are summarized by Sasaki, et al. (7). In Table 4, the most recent version of the equation expressing cyclic strength (R_{120}) versus decrease in cone resistance and effective overburden pressure was used to calculate strength. The D value was obtained for the study sites from plots shown in Sasaki, et al. (3). A review of Table 4 indicates that liquefaction would not have been predicted at any of the study site SPT intervals.

Review of the decrease in resistance diagrams indicates a general trend of reduction of D with depth. In current field use, the force on the cone tip is being measured at the ground surface. As a result, friction buildup on the rods is influencing the penetration resistances. The recent modification to incorporate effective overburden pressure into the strength equation may compensate for this effect; but it is possible that friction buildup occurs at different rates for the static and dynamic soundings which are performed independently. If this is the case, the static and dynamic components of D may have to be adjusted separately. Static Q_c can be compared to CPT Q_c data and friction can be evaluated; but the only solution to evaluating the effect on vibratory Q_c is to locate a load cell within the tip or just above the vibratory section.

A second problem with the correlation is the use of cyclic strength data from undisturbed samples obtained the field. The sampling process has a tendency to densify loose sand. Trimming and reapplication of confining stresses may further densify the sand. The converse effect can occur with dense sands. As a result, there is a general flattening of the cyclic strength versus D curve. On Fig. 21, the two equations proposed for the vibrocone are plotted on a cyclic strength versus D diagram. Also plotted on the diagram are the SPT N_{160} cyclic strength data and D value summarized in Table 4. In the area of 100- to 60-percent reduction ($1 - D = 0.0$ to 0.4), SPT strength data for a given D value seem reasonable. At smaller values of D or as $1 - D$ approaches 1, there is considerable scatter. The point of lower cyclic strength in this area originate from deeper zones at liquefied sites where friction has reduced the magnitude of decrease. If the friction problem is solved, it may be possible to develop the anticipated trend line shown on the figure.

To summarize, two recommendations can be made to improve VCPT performance. First, the effects of rod friction must be omitted or determined accurately. Second, either of the three approaches which follow could be taken to develop the cyclic strength correlation; laboratory chamber studies of sands of known cyclic

strength, field cyclic strength determinations on sands which are frozen prior to sampling; or correlations to accepted in situ testing techniques such as Iwasaki - Tatsuoka, N_{160} , or Q_{c1} approaches. In order to extend the prediction into sands containing fines, it may be helpful to add a friction sleeve to the static penetrometer system to assist in identifying these materials.

3. CONCLUSIONS

Comparisons of liquefaction resistance evaluation techniques using the SPT, CPT, DMT, and vibrocone have been performed with the data obtained from the 1983 U.S.-Japan cooperative field study sites and a field study performed in 1986. Pertinent conclusions regarding the methods considered along with recommendations for further study are as follows:

a. SPT data were reanalyzed to incorporate energy and sampler corrections using the N_{160} method proposed by Seed (5) and compared to the Iwasaki, et al. (18), method. The methods agree very well with only slight differences at low factor of safety. At a large factor of safety, the methods diverge due to the limiting shear strain concept of the N_{160} method.

b. CPT data were analyzed using the Q_{c1} method proposed by Seed (11). Results of an SPT dependent analysis and independent Q_{c1} factor of safety comparisons show the method was very reliable and suitable for use in liquefaction analysis. Additional field performance comparisons in silty sands and sands subjected to higher levels of cyclic loading are required.

c. CPT data were analyzed for a qualitative comparison to the method proposed by Olsen (12). The method in concept is fundamentally correct; but modifications would be required to develop the relationship for mechanical cones.

d. CPT data were analyzed and a method developed to evaluate liquefaction resistance using the state parameter proposed by Been and Jeffries (1985). The method should have comparable accuracy as the Q_{c1} method and has the additional benefit of estimating static stress - strain behavior in shear. Additional study will be required to evaluate prestress effects and extend the correlation to silty sands.

e. A series of six DMT soundings were performed by the Public Works Research Institute at the Hachiro Gata Lagoon area during October and November of 1986. Comparisons of static and dynamic K_d values did not indicate a trend correlated to liquefaction resistance. Static K_d values were compared to the method proposed by Robertson (1984)

with favorable results, additional studies of silty sands and sands subjected to higher levels of cyclic loading are required.

f. Of all static-dynamic penetration techniques, the vibrocone test has the best potential for accurate prediction of liquefaction resistance. Equipment or calculation methods need to be improved to overcome rod friction effects. The correlation of penetration decrease to cyclic strength requires further study.

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Table 1 Items of field investigation

Region		Akita (Lagoon Macitro)			Niigata		
Site No.		A	B	C	D	E	F
Location		Gomyoko bridge	Front levee	West levee	Kawagishi-cho	Showa Ohashi bridge left side right side	
Occurrence of Liquefaction		yes	yes	no	yes	yes	no
Boring	No.1 (m)	0~20.0	0~20.0	0~20.0	0~20.0	0~20.0	0~20.0
	No.2 (m)	0~20.0	0~20.0	0~20.0	0~20.0	-	-
S.P.T. (JIS sampler)	No.1 (points)	10	10	10	10	10	10
	No.2 (points)	8	4	9	4	-	-
S.P.T. (ASTM sampler)	No.1 (points)	10	10	10	10	10	10
	No.2 (points)	8	4	8	4	-	-
Sand Sampling (points)		5	5	2	7	-	-
Piezocone (m)		0~9.7	0~20.0	0~12.7	0~20.0	0~18.0	0~10.5
Automatic Ram Sounding (m)		0~14.4	0~20.0	0~20.0	0~20.0	0~20.0	0~20.0
Dutch Cone (m)		0~6.2	0~11.8	0~6.2	0~10.4	0~12.0	0~6.2
Vibratory Cone (m)		0~6.8	0~20.0	0~7.4	0~16.8	0~15.2	0~7.2
Swedish Sounding (m)		0~12.0	0~15.0	0~15.0	0~20.0	0~15.5	0~1.0
Dynamic Cone Penetration Test (m)		0~19.0	0~20.0	0~15.5	0~19.3	0~17.2	0~16.3
PS Logging (m)		0~21.0	0~20.0	0~20.0	0~20.0	-	0~20.0
Magnetic Susceptibility Logging (m)		-	8.0~15.0	-	1.5~1.8	3.4~21.2	3.5~20.0
Density Logging (m)		0~20.0	3.0~15.8	1.0~20.5	0.5~20.5	0~21.4	3.0~20.1
Caliper Logging (m)		0~20.8	-	0~20.5	0~20.6	0~20.9	0~20.8
Electric Logging (m)		3.5~20.0	8.5~14.5	4.5~19.8	1.2~20.5	3.4~20.5	3.5~19.0
Classification Test (points)		25	25	22	26	-	-
Cyclic Triaxial Test (series)		3	3	1	4	-	-
Resonant Column Test and Cyclic Torsional Test (series)		2	2	-	2	-	-

Table 2(1) Summary of SPT N160 calculations - Site A -
Hachiro Gata - WC 15+553

DEPTH m.	D50 mm.	MINUS #200 SIEVE %	N ASTM BPF	N LINER BPF	N JIS BPF	ROD ENERGY %	N60 BPF	VERT. STRESS TSF	CN	N160 BPF	FIELD CYCLIC STRESS RATIO	CYCLIC STRENGTH N160 METHOD
1.3	0.22	6	6	6	2	62	1.6	0.21	2.2	4	0.13	0.04
2.3	0.20	21			12	62	4.6	0.30	1.8	9	0.17	0.09
3.3	0.26	10	13	16.6		62	12.4	0.39	1.6	20	0.18	0.32
4.3	0.20	13			18	62	16.1	0.49	1.4	23	0.18	0.33
5.3	0.22	13	14	16.8		62	18.6	0.68	1.3	24	0.19	0.36
6.3	0.26	5			36	62	17.4	0.68	1.2	22	0.19	0.28
7.3	0.16	16	22	26.4		62	36.2	0.77	1.1	40	0.19	>.5
8.3	0.18	10			49	62	27.3	0.86	1.1	31	0.18	>.5
9.3	0.18	9	51	61.2		62	50.6	0.96	1	61	0.18	>.5
10.3	0.18	10			59	62	63.2	1.06	0.97	61	0.18	>.5
11.3	0.17	14	29	34.8		60	59	1.15	0.93	56	0.18	>.5
12.3	C				6	66	36.9	1.24	0.9	32	0.18	>.5
13.3	C		6	6		60	6	1.33			0.17	
14.3	C				6	62	6.2	1.43			0.17	
15.3	C		7	8.4		62	6.2	1.52			0.17	
16.3	C				6	62	8.7	1.62			0.17	
17.3	C				6	62	5.2	1.71			0.16	
18.3	C		6	7.2		62	7.4	1.80			0.16	
19.3	C				7	62	7.2	1.90			0.16	
20.3	C		10	12		62	12.4	1.99			0.16	

Table 2(2) Summary of SPT N160 calculations - Site B -
Hachiro Gata - FD 6+900

DEPTH m.	D50 mm.	MINUS #200 SIEVE %	N ASTM BPF	N LINER BPF	N JIS BPF	ROD ENERGY %	N60 BPF	VERT. STRESS TSF	CN	N160 BPF	FIELD CYCLIC STRESS RATIO	CYCLIC STRENGTH N160 METHOD
1.3	0.28	12			12	76	11.4	0.23	2.10	24	0.12	0.32
2.3	0.28	11	3	3.6		76	3.6	0.32	1.80	6	0.15	0.10
3.3	0.40	3			10	76	12.7	0.42	1.60	20	0.16	0.22
4.3	0.20	6	5	6		76	7.6	0.61	1.40	11	0.17	0.12
5.3	0.24	3			7	76	8.9	0.60	1.30	12	0.18	0.13
6.3			6	7.2		76	9.1	0.70	1.20	11	0.18	0.12
7.3					12	76	15.2	0.79	1.10	17	0.18	0.18
8.3	0.22	6	10	12		76	15.2	0.89	1.10	17	0.18	0.18
9.3	0.24	5			12	76	15.2	0.98	1.00	15	0.18	0.17
10.3	0.23	7	9	10.8		76	13.7	1.07	0.96	13	0.18	0.16
11.3	0.28	6			10	70	11.7	1.17	0.92	11	0.17	0.12
12.3	0.30	1	10	12		78	16.6	1.26	0.89	14	0.17	0.16
13.3	0.28	6			10	73	12.2	1.36	0.86	11	0.17	0.12
14.3	C		5	6		84	8.4					
15.3	C				4	76	6.1					
16.3	C		3	3.6		76	4.6					
17.3	C				1	76	1.3					
18.3	C		2	2.4		76	3					
19.3	C				6	76	7.6					
20.3	C		3	3.6		76	4.6					

Table 2(3) Summary of SPT N160 calculations - Site C -
Hachiro Gata - WC 0+580

DEPTH m.	D50 mm.	MINUS #200 SIEVE %	N ASTM BPF	N LINER BPF	N JIS BPF	ROD ENERGY %	N60 BPF	VERT. STRESS TSF	CN	N160 BPF	FIELD CYCLIC STRESS RATIO	CYCLIC STRENGTH N160 METHOD
1.3	0.36	1			2	72	1.8	0.28	1.98	4	0.11	0.06
2.3	0.28	2	4	4.8		72	4.5	0.44	1.5	7	0.11	0.08
3.3	0.28	3			4	72	4.8	0.56	1.34	6	0.12	0.06
4.3	0.28	3	16	19.2		72	23.6	0.66	1.24	29	0.13	>.6
5.3	0.28	2			16	72	18	0.76	1.16	21	0.14	0.23
6.3	0.23	6	14	16.8		72	20.2	0.84	1.09	22	0.15	0.26
7.3	0.22	4			22	72	26.4	0.93	1.03	27	0.15	0.32
8.3	0.19	4	17	20.4		72	24.5	1.03	0.99	24	0.15	0.28
9.3	0.16	4			28	72	33.8	1.12	0.94	32	0.15	>.6
10.3	0.16	6	39	46.8		72	56.2	1.22	0.91	51	0.16	>.6
11.3	0.16	8			25	70	29.2	1.31	0.87	25	0.16	0.28
12.3	0.17	6	33	39.6		69	45.6	1.40	0.84	38	0.16	>.6
13.3	0.16	4			30	72	36	1.50	0.82	29	0.16	>.6
14.3	0.18	8	20	24		72	28.8	1.59	0.79	23	0.16	0.27
15.3	0.13	18			36	74	43.2	1.69	0.77	33	0.16	>.6
16.3	0.18	4	26	31.2		69	35.9	1.78	0.76	27	0.15	0.36
17.3	0.11	17			17	71	20.1	1.87	0.73	14	0.16	0.20
18.3	0.11	22	22	26.4		70	30.8	1.87	0.71	22	0.15	>.6
19.3	0.11	12			20	77	26.7	2.06	0.7	18	0.14	0.24
20.3	0.11	16	23	27.6		75	34.5	2.16	0.68	24	0.14	>.6

Table 2(4) Summary of SPT N160 calculations - Site D -
Kawagishi-cho - Niigata

DEPTH m.	D50 mm.	MINUS #200 SIEVE %	N ASTM BPF	N LINER BPF	N JIS BPF	ROD ENERGY %	N60 BPF	VERT. STRESS TSF	CN	N160 BPF	FIELD CYCLIC STRESS RATIO	CYCLIC STRENGTH N160 METHOD
1.3	0.32	3			11	64	8.8	0.24	2	18	0.11	0.27
2.3	0.36	4	8	9.6		64	7.7	0.33	1.7	13	0.14	0.21
3.3	0.36	2			6	64	6.4	0.42	1.5	10	0.15	0.11
4.3	0.40	2	4	4.8		64	5.1	0.52	1.4	7	0.16	0.07
5.3	0.36	4			6	64	6.4	0.61	1.3	8	0.16	0.08
6.3	0.42	3	7	8.4		64	8.96	0.71	1.2	11	0.17	0.12
7.3	0.34	3			10	64	10.7	0.80	1.1	12	0.17	0.13
8.3	0.18	12	7	8.4		64	8.96	0.89	1.06	10	0.17	0.13
9.3	0.34	3			8	64	8.5	0.99	1	9	0.16	0.10
10.3	0.41	1	9	10.8		64	11.6	1.08	0.96	11	0.16	0.13
11.3	0.27	1			8	59	7.9	1.18	0.92	7	0.16	0.07
12.3	0.28	3	8	9.6		63	10.2	1.27	0.89	9	0.16	0.10
13.3	0.43	2			8	64	8.5	1.36	0.86	7	0.16	0.07
14.3	0.27	2	9	10.8		64	11.5	1.46	0.83	10	0.16	0.13
15.3	0.27	4			17	66	18.7	1.55	0.8	15	0.16	0.17
16.3	0.26	4	16	18.6		63	19.2	1.66	0.78	15	0.15	0.17
17.3	0.26	4			25	62	25.8	1.74	0.76	20	0.15	0.22
18.3	0.25	13	21	25.2		64	26.7	1.83	0.74	20	0.15	0.28
19.3	0.22	6			35	68	39.7	1.93	0.72	29	0.15	>.6
20.3	0.26	6	29	34.8		66	37.1	2.02	0.7	26	0.14	0.39

Table 2(5) Summary of SPT N160 calculations - Site E -
Showa Bridge - Niigata

DEPTH m.	D50 mm.	MINUS #200 SIEVE %	N ASTM BPF	N LINER BPF	N JIS BPF	ROD ENERGY %	N60 BPF	VERT. STRESS TSF	CN	N160 BPF	FIELD CYCLIC STRESS RATIO	CYCLIC STRENGTH N160 METHOD
1.3	0.28	30			15	64	12	0.26	1.98	24	0.1	>.5
2.3	0.22	9	5	6		64	4.8	0.36	1.67	8	0.13	0.14
3.3	0.10	39			2	64	2.1	0.45	1.48	3	0.14	
4.3	0.25	5	6	7.2		64	7.7	0.55	1.35	10	0.15	0.12
5.3	0.29	2			9	64	9.6	0.64	1.25	12	0.16	0.13
6.3	0.52	3	11	13.2		64	14.1	0.74	1.16	16	0.16	0.17
7.3	0.40	1			12	64	12.8	0.83	1.09	14	0.16	0.15
8.3	0.42	3	11	13.2		64	14.1	0.93	1.04	15	0.16	0.16
9.3	0.40	3			10	64	10.7	1.02	0.99	11	0.16	0.16
10.3	0.38	5	12	14.4		64	15.4	1.11	0.95	16	0.16	0.16
11.3	0.41	4			10	64	10.7	1.21	0.91	10	0.16	0.11
12.3	0.41	6	8	9.6		64	10.2	1.30	0.87	9	0.16	0.10
13.3	0.30	3			39	64	41.6	1.40	0.84	35	0.16	>.5
14.3	0.30	4	24	28.8		64	30.7	1.49	0.82	25	0.15	0.32
15.3	0.26	4			27	64	28.8	1.59	0.79	23	0.15	0.27
16.3	0.26	6	27	32.4		64	34.6	1.68	0.77	27	0.15	0.36
17.3	0.30	3			36	64	38.4	1.77	0.75	29	0.15	0.40
18.3	0.28	5	19	22.8		64	24.3	1.87	0.73	18	0.15	0.20
19.3	0.22	4			25	64	26.6	1.95	0.71	19	0.14	0.21
20.3	0.20	8	31	37.4		64	39.7	2.06	0.69	27	0.14	0.40

Table 2(6) Summary of SPT N160 calculations - Site F -
Showa Bridge - Niigata

DEPTH m.	D50 mm.	MINUS #200 SIEVE %	N ASTM BPF	N LINER BPF	N JIS BPF	ROD ENERGY %	N60 BPF	VERT. STRESS TSF	CN	N160 BPF	FIELD CYCLIC STRESS RATIO	CYCLIC STRENGTH N160 METHOD
1.3	0.70	11			33	68	28	0.26	1.97	55	0.1	>.5
2.3	0.40	2	4	4.8		68	4.1	0.39	1.59	7	0.12	0.08
3.3	0.04	64			3	68	3.4	0.49	1.43	5	0.13	
4.3	0.01	94	0	0		68	0	0.58	1.31	0	0.14	
5.3	0.26	6			25	68	28.3	0.68	1.20	34	0.15	>.5
6.3	0.24	4	16	19		68	21.5	0.77	1.10	24	0.15	0.32
7.3	0.24	5			32	68	36.3	0.87	1.07	39	0.15	>.5
8.3	0.24	3	27	32.4		68	36.7	0.96	1.02	37	0.15	>.5
9.3	0.24	5			36	68	40.8	1.05	0.97	40	0.16	>.5
10.3	0.27	2	27	32.4		68	36.7	1.15	0.93	34	0.16	>.5
11.3	0.26	6			64	68	61.2	1.24	0.89	55	0.15	>.5
12.3	0.26	5	24	28.8		68	32.6	1.34	0.86	28	0.15	0.34
13.3	0.26	4			28	68	31.7	1.43	0.83	26	0.15	0.28
14.3	0.27	6	25	30		68	34	1.53	0.81	28	0.15	0.37
15.3	0.26	7			17	68	19.3	1.62	0.78	15	0.15	0.17
16.3	0.24	7	22	26.4		68	29.9	1.71	0.76	23	0.15	0.28
17.3	0.21	7			29	68	32.9	1.81	0.74	24	0.15	0.28
18.3	0.24	5	33	39.6		68	44.9	1.90	0.72	33	0.14	>.5
19.3	0.16	8			24	68	27.2	2.00	0.71	19	0.14	0.21
20.3	0.22	5	34	40.8		68	46.2	2.09	0.69	32	0.14	>.5

Table 3 Summary of cone penetration calculations - all sites

SITE	DEPTH	FIELD CYCLIC STRESS RATIO	SPT N160 STRENGTH	FRICTION RATIO %	MINUS NO.200 SIEVE %	D50 mm.	QC/N60 RATIO	QC1 AVG. n=0.5 TSF	CYCLIC STRENGTH QC1 METHOD	QC1 AVG. n=0.72 TSF	FSM SLBBVS FRICTION TSF	STATE PARAMETER
A	1.3	0.13	0.04	1.9			10	45	0.10	64	1.44	
A	2.3	0.14	0.09	1.2	6	0.22	4	44	0.11	58	0.70	-0.11
A	3.3	0.18	0.32	4.8	21	0.20	2	27	0.13	33	1.35	
A	4.3	0.18	0.33	1.0	10	0.26	3	60	0.17	71	1.70	-0.13
A	5.3	0.19	0.35	0.5	13	0.20	5	90	0.29	95	3.20	-0.18
B	1.3	0.12	0.32	1.5	12	0.28	5	121	>.5	169	1.57	
B	2.3	0.15	0.10	1.2	11	0.28	12	108	0.35	139		-0.20
B	3.3	0.16	0.22	1.1	3	0.40	5	90	0.20	109	2.15	-0.16
B	4.3	0.17	0.12	1.1	5	0.20	2	20	0.06	24	0.47	-0.06
B	5.3	0.17	0.13	0.6	3	0.24	2	27	0.06	30	0.20	-0.05
B	6.3	0.18	0.12	2.3			2	19	0.05	20	0.34	-0.04
B	7.3	0.18	0.18	1.5			3	44	0.11	48	0.82	-0.09
B	8.3	0.18	0.18	1.5	6	0.22	4	65	0.16	67	0.58	-0.11
B	9.3	0.18	0.17	1.6	5	0.24	2	28	0.08	29	0.42	-0.06
B	10.3	0.18	0.16	1.0	7	0.23	3	44	0.11	43	0.38	-0.08
B	11.3	0.18	0.12	1.0	5	0.28	5	54	0.14	52	0.63	-0.08
C	3.3	0.12	0.06	1.3	3	0.28	10	59	0.13	69	0.44	-0.11
C	4.3	0.13	>.5	1.7	3	0.28	3	97	0.21	109	2.18	-0.14
C	5.3	0.14	0.23	1.9	2	0.28	4	90	0.22	98	2.36	-0.11
C	6.3	0.15	0.26		5	0.23	5	120	0.33	127		-0.14
D	1.3	0.11	0.27	0.9	3	0.32	7	120	0.30	170	2.70	
D	2.3	0.14	0.21	0.3	4	0.36	10	129	0.33	169	1.64	-0.20
D	3.3	0.15	0.11	1.7	2	0.36	5	54	0.12	67	1.02	-0.12
D	4.3	0.16	0.07	1.4	2	0.40	6	45	0.10	53	0.58	-0.11
D	5.3	0.16	0.08	0.8	4	0.36	7	56	0.12	64	0.72	-0.12
D	6.3	0.16	0.12	2.2	3	0.42	5	51	0.11	56	1.69	-0.13
D	7.3	0.17	0.13	0.9	3	0.34	6	74	0.17	79	0.99	-0.13
D	8.3	0.17	0.13	1.9	12	0.18	5	46	0.16	48	0.56	-0.10
D	9.3	0.17	0.10	1.2	3	0.34	6	50	0.11	51	0.89	-0.09
D	10.3	0.16	0.13	0.6	1	0.41	6	63	0.13	63	0.31	-0.10
E	2.3	0.1	0.11	1.0	9	0.22	8	61	0.17	79	1.06	-0.13
E	3.3	0.14	NA	1.4	39	0.10	13	40	0.23	56	0.87	
E	4.3	0.15	0.12	1.0	5	0.25	4	41	0.12	47	0.48	-0.10
E	5.3	0.15	0.13	1.3	2	0.29	5	55	0.13	62	0.79	-0.12
E	6.3	0.16	0.17	1.4	3	0.52	4	59	0.11	64	1.06	-0.11
E	7.3	0.16	0.15	1.4	1	0.40	3	45	0.10	52	0.79	-0.09
E	8.3	0.16	0.16	1.3	3	0.42	4	64	0.12	67	0.79	-0.12
E	9.3	0.16	0.12	1.6	3	0.40	5	50	0.11	52	0.82	-0.10
E	10.3	0.16	0.16	2.1	5	0.38	4	50	0.11	49	0.55	-0.08
E	11.3	0.16	0.11	1.6	4	0.41	4	43	0.10	41	0.53	-0.08
F	2.3	0.12	0.08	2.2	2	0.40	15	102	0.23	129	3.35	
F	5.3	0.15	>.5	1.3	6	0.26	3	95	0.30	105	1.50	-0.15
F	6.3	0.15	0.28	1.3	4	0.24	6	141	>.5	150	1.70	

Table 4(1) Summary of factor of safety and strength data

SITE	DEPTH m.	FIELD CYCLIC STRESS RATIO	N160 BPF	CYCLIC STRENGTH N160 METHOD	FS N160	CYCLIC STRENGTH 1&T METHOD	FS 1&T	QC1 TSP	CYCLIC STRENGTH QC1 METHOD	FS QC1	VIBRO CONE D X	STRENGTH BL20 VIBRO CONE	STATS PARAMETER
A	2.3	0.17	9	0.09	0.5	0.18	1.1	44	0.11	0.6	0.70	0.32	-0.11
A	3.3	0.18	20	0.32	1.8	0.23	1.3	27	0.13	0.7	0.30	0.34	
A	4.3	0.18	23	0.33	1.8	0.23	1.3	60	0.17	0.9	0.20	0.35	-0.13
A	5.3	0.19	24	0.35	1.8	0.27	1.4	90	0.29	1.5	0.20	0.34	-0.18
A	6.3	0.19	22	0.28	1.5	0.23	1.2						
A	7.3	0.19	40	>.5		0.31	1.6						
A	8.3	0.18	31	>.5		0.28	1.6						
A	9.3	0.18	51	>.5		0.35	1.9						
A	10.3	0.18	61	>.5		0.36	2.0						
A	11.3	0.18	55	>.5		0.36	2.0						
A	12.3	0.18	32	>.5		0.29	1.6						
B	2.3	0.15	6	0.10	0.7	0.12	0.8	108	0.35	2.3	0.90	0.31	-0.2
B	3.3	0.16	20	0.22	1.4	0.17	1.1	90	0.20	1.3	0.50	0.32	-0.16
B	4.3	0.17	11	0.12	0.7	0.16	0.9	20	0.08	0.4	0.85	0.28	-0.06
B	5.3	0.18	12	0.13	0.7	0.16	0.9	27	0.08	0.3	0.80	0.27	-0.05
B	6.3	0.18	11	0.12	0.7	0.15	0.8	19	0.05	0.3	0.80	0.25	-0.04
B	7.3	0.18	17	0.18	1.0	0.19	1.1	44	0.11	0.6	0.40	0.29	-0.09
B	8.3	0.18	17	0.18	1.0	0.19	1.1	65	0.16	0.9	0.80	0.22	-0.11
B	9.3	0.18	15	0.17	0.9	0.18	1.0	28	0.08	0.4	0.70	0.22	-0.06
B	10.3	0.18	13	0.16	0.9	0.17	0.9	44	0.11	0.6	0.75	0.20	-0.08
B	11.3	0.17	11	0.12	0.7	0.15	0.9	54	0.14	0.8	0.75	0.20	-0.08
B	12.3	0.17	14	0.16	0.9	0.15	0.9				0.75	0.19	
B	13.3	0.17	11	0.12	0.7	0.14	0.8				0.75	0.18	
C	3.3	0.12	6	0.06	0.5	0.12	1.0	59	0.13	1.1	0.30	0.32	-0.11
C	4.3	0.13	29	>.5		0.22	1.7	97	0.21	1.6	-0.40	0.50	-0.14
C	5.3	0.14	21	0.23	1.6	0.2	1.4	90	0.22	1.6	0.10	0.35	-0.11
C	6.3	0.15	22	0.26	1.7	0.22	1.5	120	0.33	2.2	0.05	0.42	-0.14
C	7.3	0.15	27	0.32	2.1	0.24	1.6						
C	8.3	0.15	24	0.28	1.9	0.23	1.5						
C	9.3	0.15	32	>.5		0.27	1.8						
C	10.3	0.16	51	>.5		0.34	2.1						
C	11.3	0.16	25	0.28	1.8	0.26	1.6						
C	12.3	0.16	38	>.5		0.3	1.9						
C	13.3	0.16	29	>.5		0.26	1.6						
C	14.3	0.15	23	0.27	1.8	0.22	1.5						
C	15.3	0.15	33	>.5		0.28	1.9						
C	16.3	0.15	27	0.35	2.3	0.24	1.6						
C	17.3	0.15	14	0.20	1.3	0.22	1.5						
C	18.3	0.15	22	>.5		0.25	1.7						
C	19.3	0.14	18	0.24	1.7	0.21	1.5						
C	20.3	0.14	24	>.5			0.0						

Table 4(2) Summary of factor of safety and strength data

SITE	DEPTH	FIELD CYCLIC STRESS RATIO	N160 BPF	CYCLIC STRENGTH N160 METHOD	FS N160	CYCLIC STRENGTH IAT METHOD	FS IAT	QC1 TSF	CYCLIC STRENGTH QC1 METHOD	FS QC1	VIBRO CONE D %	STRENGTH BL20 VIBRO CONE	STATE PARAMETER
D	2.3	0.14	13	0.21	1.5	0.17	1.2	129	0.33	2.4	1.00	0.31	-0.20
D	3.3	0.15	10	0.11	0.7	0.13	0.9	54	0.12	0.8	1.00	0.30	-0.12
D	4.3	0.16	7	0.07	0.4	0.11	0.7	45	0.10	0.6	0.90	0.28	-0.11
D	5.3	0.16	8	0.08	0.5	0.12	0.8	56	0.12	0.8	0.40	0.30	-0.12
D	6.3	0.16	11	0.12	0.8	0.13	0.8	51	0.11	0.7	0.80	0.25	-0.13
D	7.3	0.17	12	0.13	0.8	0.16	0.9	74	0.17	1.0	0.40	0.28	-0.13
D	8.3	0.17	10	0.13	0.8	0.17	1.0	46	0.16	0.9	0.30	0.30	-0.10
D	9.3	0.17	9	0.10	0.6	0.13	0.8	50	0.11	0.6	0.30	0.30	-0.09
D	10.3	0.16	11	0.13	0.8	0.12	0.8	63	0.13	0.8	0.10	0.38	-0.10
D	11.3	0.16	7	0.07	0.4	0.13	0.8				0.40	0.26	
D	12.3	0.16	9	0.10	0.6	0.14	0.9				0.20	0.33	
D	13.3	0.16	7	0.07	0.4	0.11	0.7				0.40	0.26	
D	14.3	0.16	10	0.13	0.8	0.15	0.9				0.10	0.39	
D	15.3	0.16	15	0.17	1.1	0.18	1.1				0.00	0.45	
D	16.3	0.15	15	0.17	1.1	0.17	1.1				0.10	0.40	
D	17.3	0.15	20	0.22	1.5	0.20	1.3						
D	18.3	0.15	20	0.28	1.9	0.20	1.3						
D	19.3	0.15	29	>.5		0.25	1.7						
D	20.3	0.14	26	0.39	2.8	0.22	1.6						
E	2.3	0.13	8	0.11	0.8	0.18	1.4	61	0.17	1.3	1.00	0.31	-0.13
E	4.3	0.15	10	0.12	0.8	0.16	1.1	41	0.12	0.8	1.00	0.27	-0.10
E	5.3	0.16	12	0.13	0.9	0.16	1.0	55	0.13	0.8	0.95	0.25	-0.12
E	6.3	0.16	16	0.17	1.1	0.14	0.9	59	0.11	0.7	0.90	0.24	-0.11
E	7.3	0.16	14	0.15	0.9	0.15	0.9	45	0.10	0.6	0.90	0.22	-0.09
E	8.3	0.16	15	0.16	1.0	0.15	0.9	64	0.12	0.8	0.90	0.19	-0.12
E	9.3	0.16	11	0.12	0.8	0.13	0.8	50	0.11	0.7	0.80	0.21	-0.10
E	10.3	0.16	15	0.16	1.0	0.15	0.9	50	0.11	0.7	0.75	0.20	-0.08
E	11.3	0.16	10	0.11	0.7	0.13	0.8	43	0.10	0.6	0.70	0.20	-0.08
E	12.3	0.16	9	0.10	0.6	0.12	0.8				0.60	0.21	
E	13.3	0.16	35	>.5		0.26	1.6				0.70	0.28	
E	14.3	0.15	25	0.32	2.1	0.17	1.1				0.40	0.26	
E	15.3	0.15	23	0.27	1.8	0.22	1.5						
E	16.3	0.15	27	0.36	2.4	0.23	1.5						
E	17.3	0.15	29	0.40	2.7	0.24	1.6						
E	18.3	0.15	18	0.20	1.3	0.18	1.2						
E	19.3	0.14	19	0.21	1.5	0.21	1.5						
E	20.3	0.14	27	0.40	2.9	0.25	1.8						

Table 4(3) Summary of factor of safety and strength data

SITE	DEPTH	FIELD CYCLIC STRESS RATIO	N160 BPF	CYCLIC STRENGTH N160 METHOD	FS N160	CYCLIC STRENGTH I&T METHOD	FS I&T	QCI TSF	CYCLIC STRENGTH QCI METHOD	FS QCI	VIBRO CONE D X	STRENGTH RL20 VIBRO CONE	STATE PARAMETER
P	2.3	0.12	7	0.08	0.7	0.11	0.9	102	0.23	1.9	0.10	0.37	
P	5.3	0.15	34	>.5		0.26	1.7	95	0.30	2.0	0.25	0.32	-0.15
P	6.3	0.15	24	0.32	2.1	0.23	1.5	141	>.5				
P	7.3	0.15	39	>.5		0.28	1.9						
P	8.3	0.15	37	>.5		0.26	1.7						
P	9.3	0.16	40	>.5		0.29	1.8						
P	10.3	0.16	34	>.5		0.25	1.6						
P	11.3	0.15	55	>.5		0.31	2.1						
P	12.3	0.15	28	0.34	2.3	0.23	1.5						
P	13.3	0.15	26	0.28	1.9	0.22	1.5						
P	14.3	0.15	28	0.37	2.5	0.22	1.5						
P	15.3	0.15	15	0.17	1.1	0.18	1.2						
P	16.3	0.15	23	0.28	1.9	0.21	1.4						
P	17.3	0.15	24	0.28	1.9	0.23	1.5						
P	18.3	0.14	33	>.5		0.24	1.7						
P	19.3	0.14	19	0.21	1.5	0.22	1.6						
P	20.3	0.14	32	>.5		0.25	1.8						

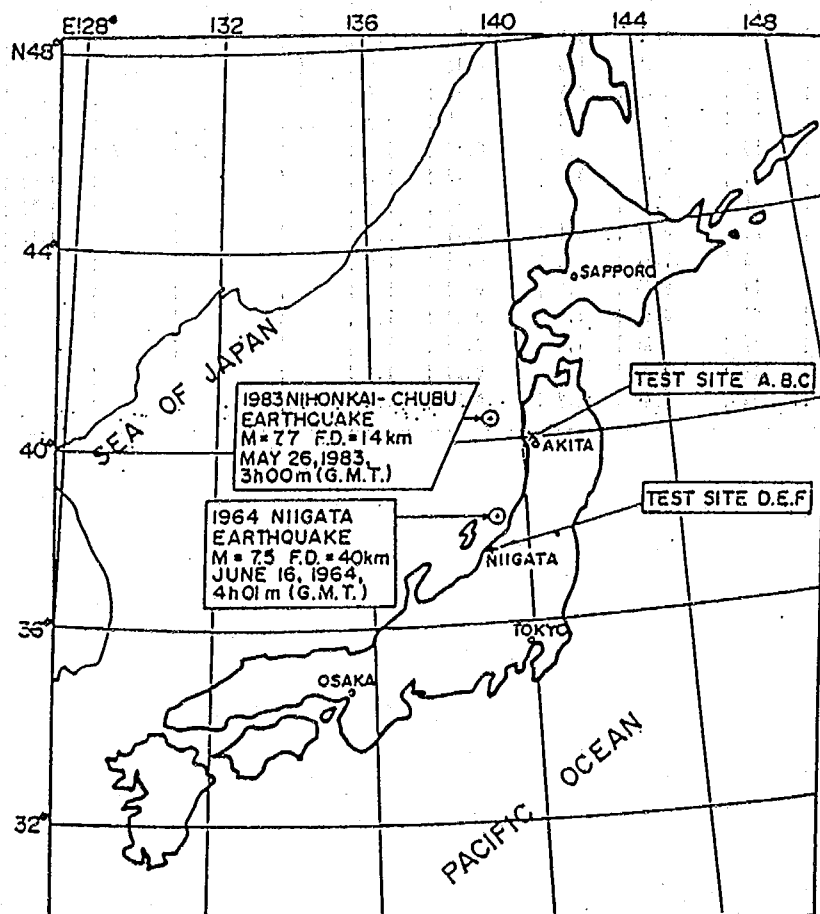


Figure 1 Location of test sites and the earthquake epicenters

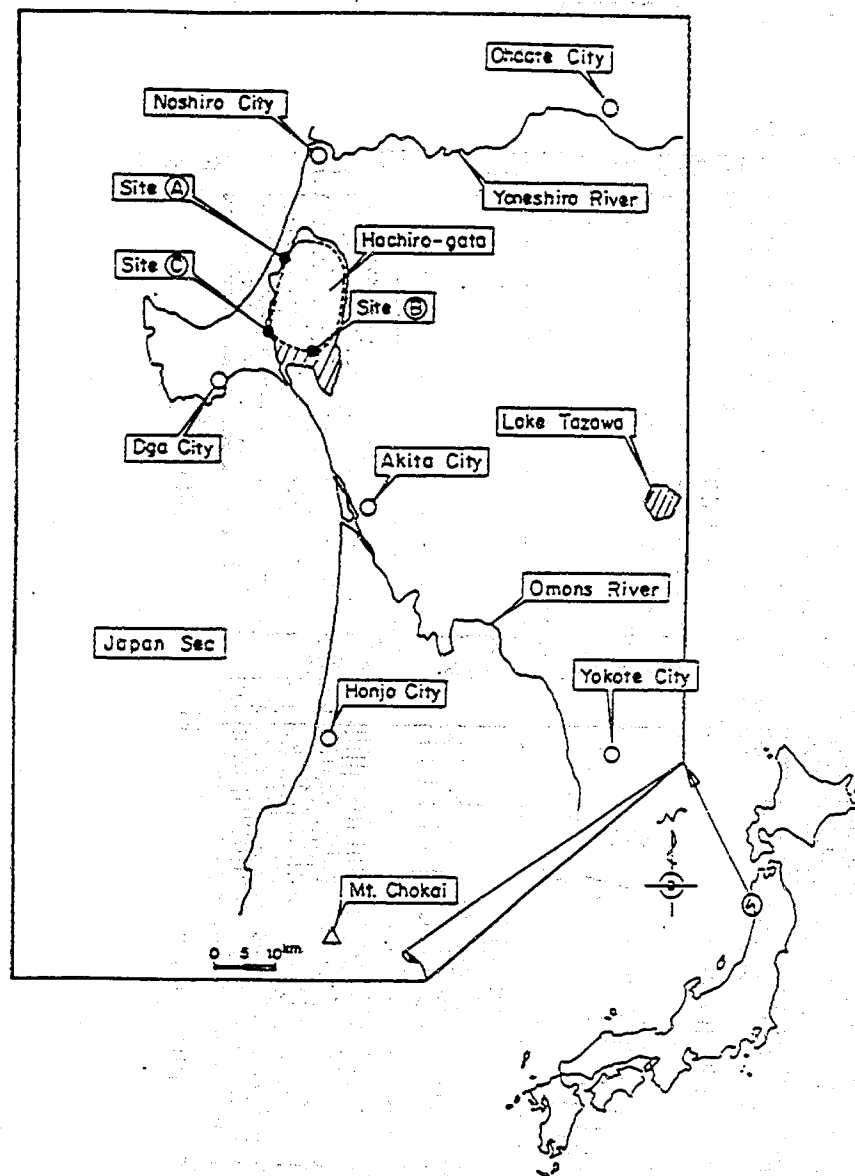


Figure 2 Guide map of Akita prefecture

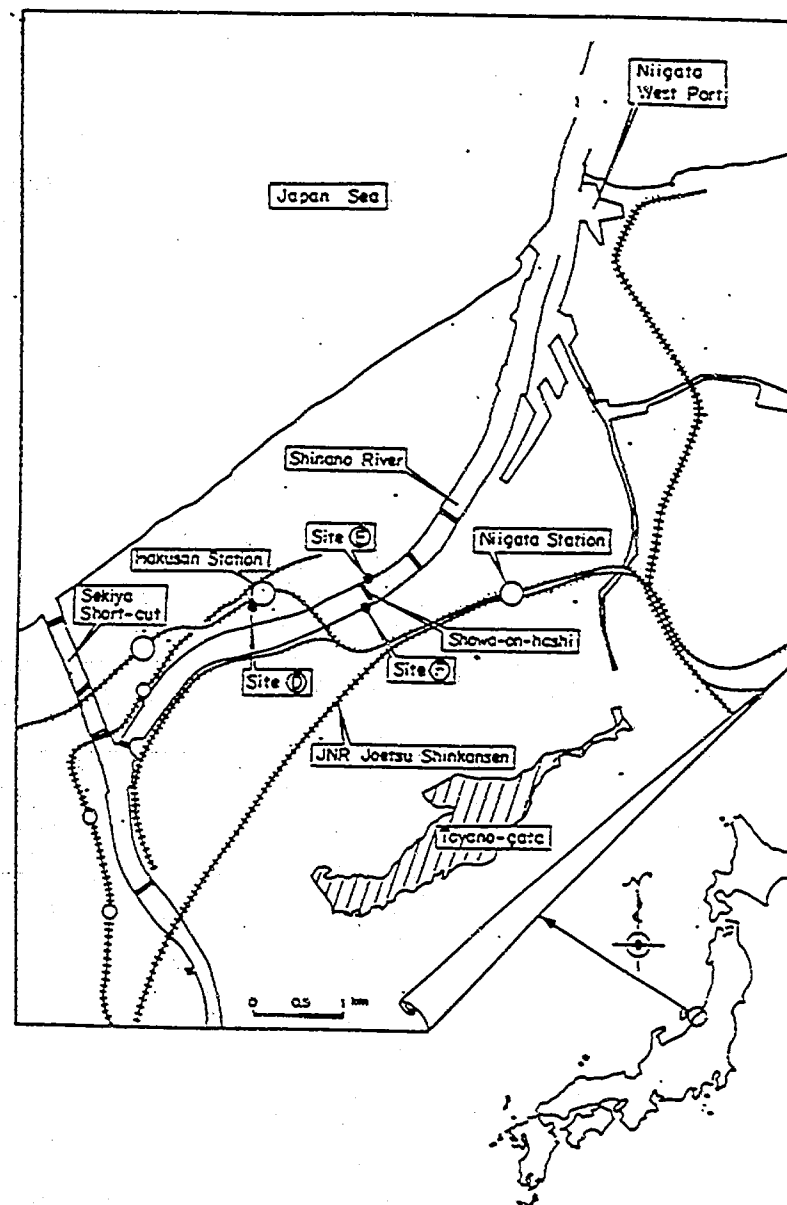


Figure 3 Guide map of the Niigata prefecture

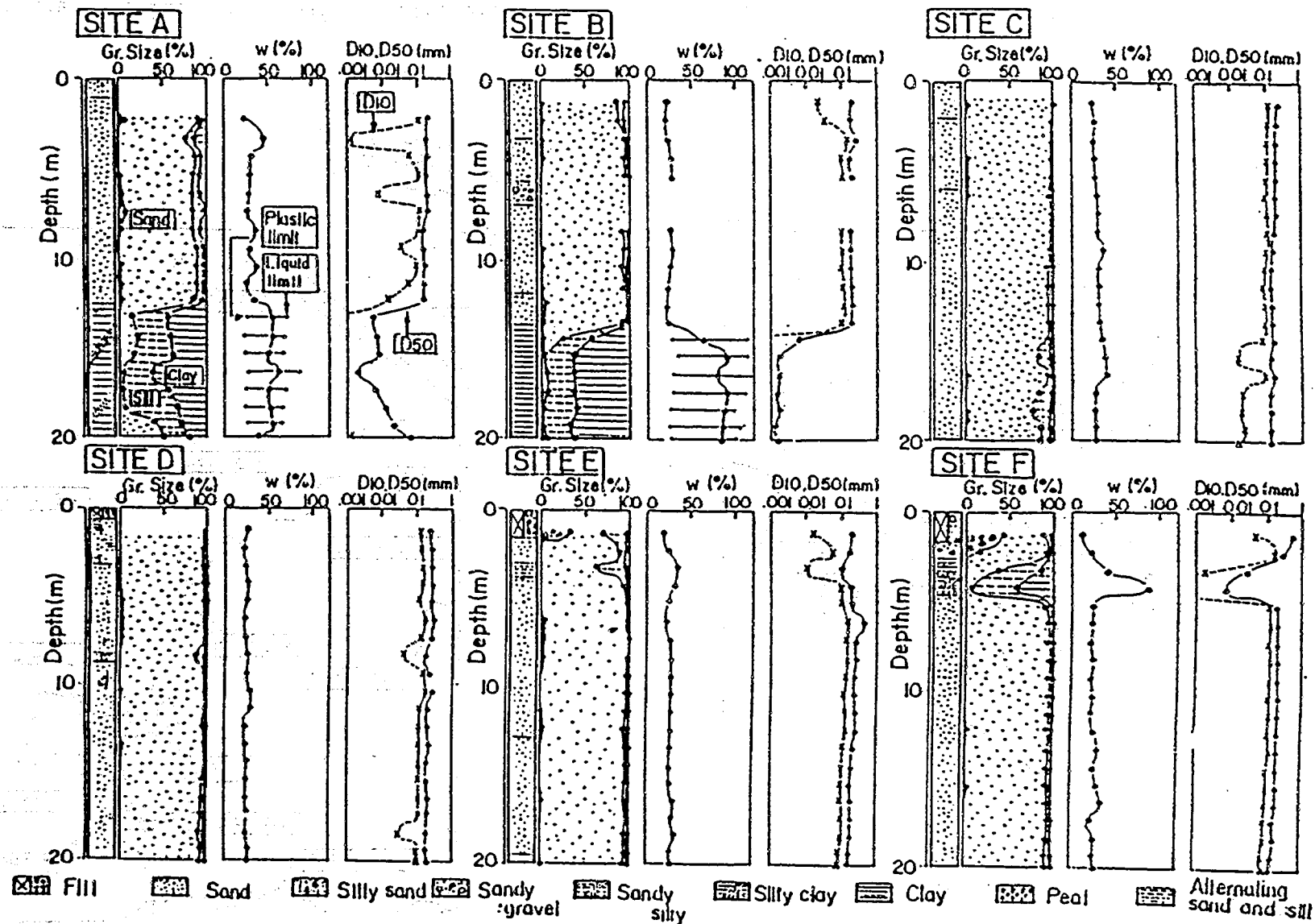


Figure 4 Soil properties and soil logs at test sites

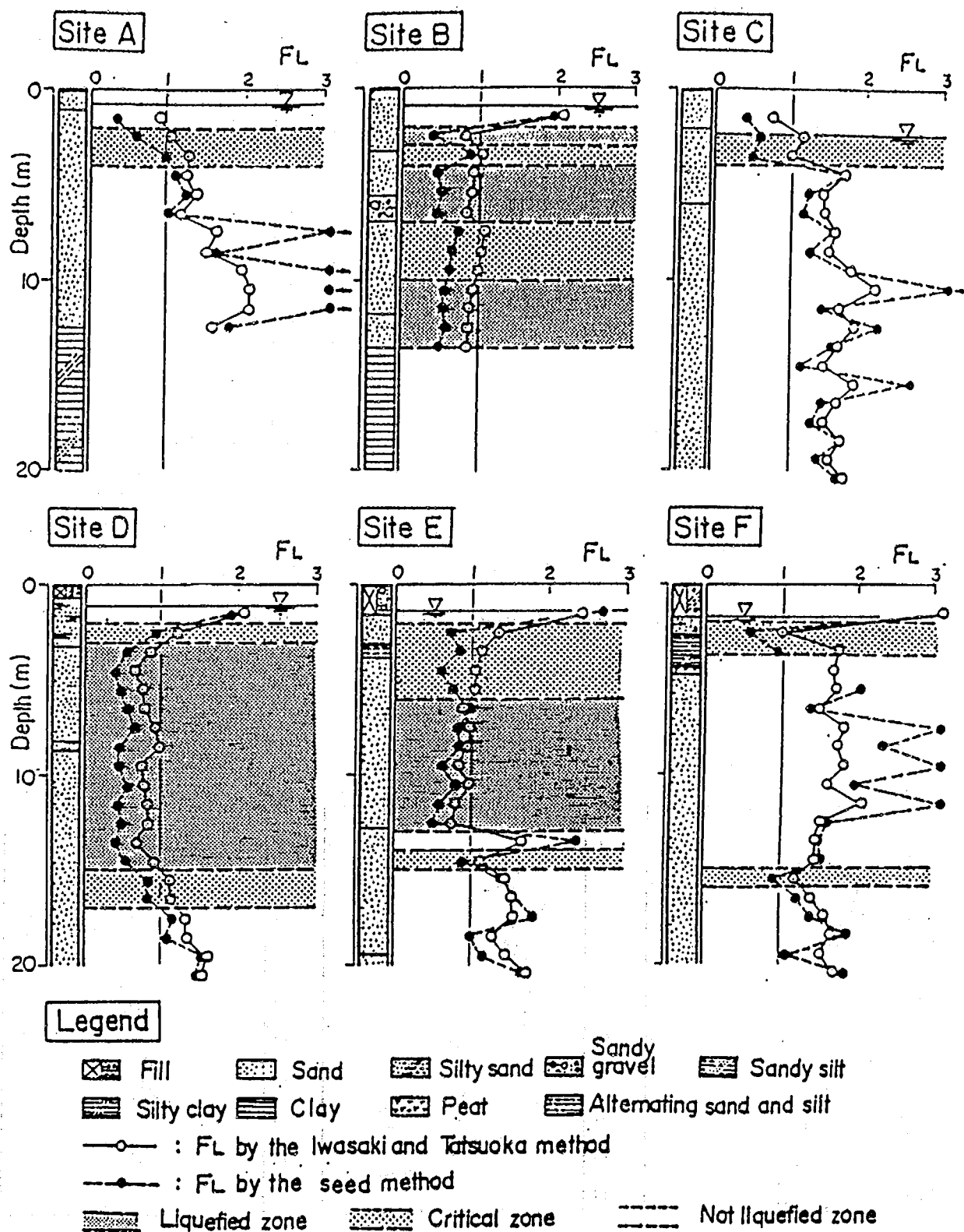


Figure 5 Comparison of FL distribution by the Iwasaki and Tatsuoka's method and by the Seeds' method

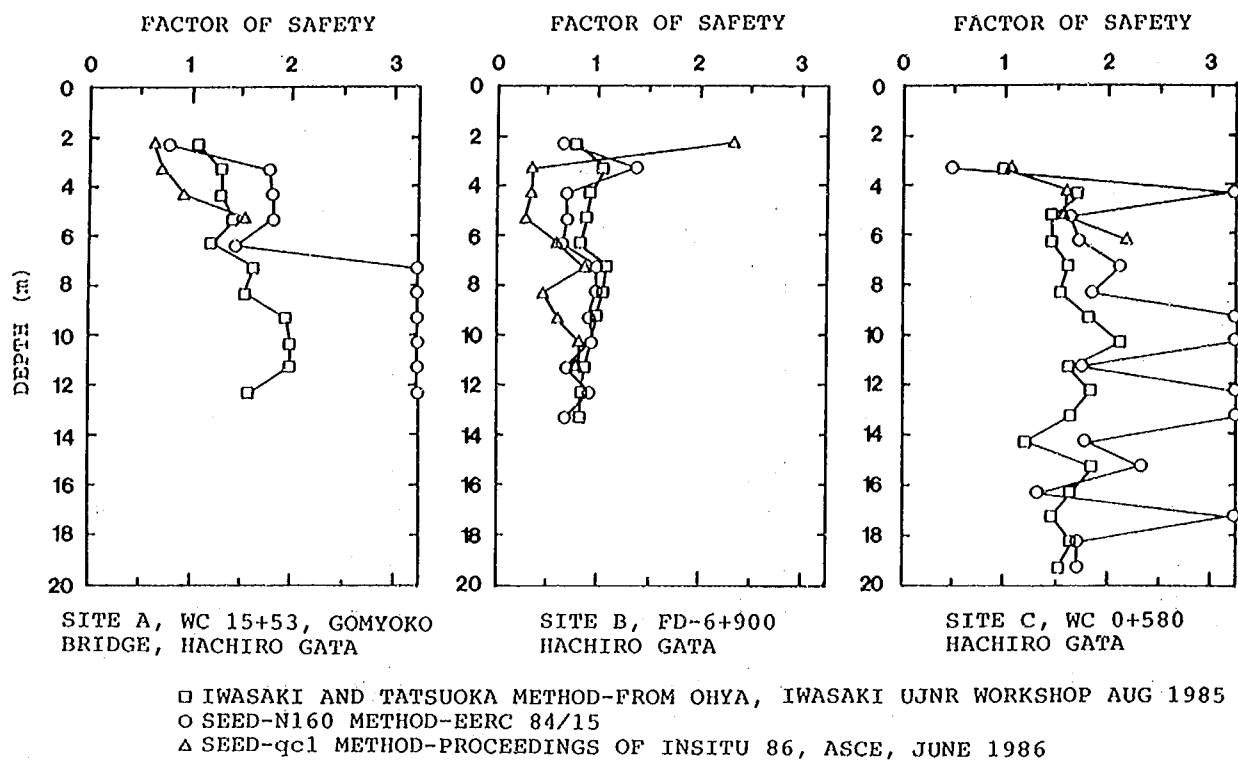


Figure 6 Summary of predicted factor of safety

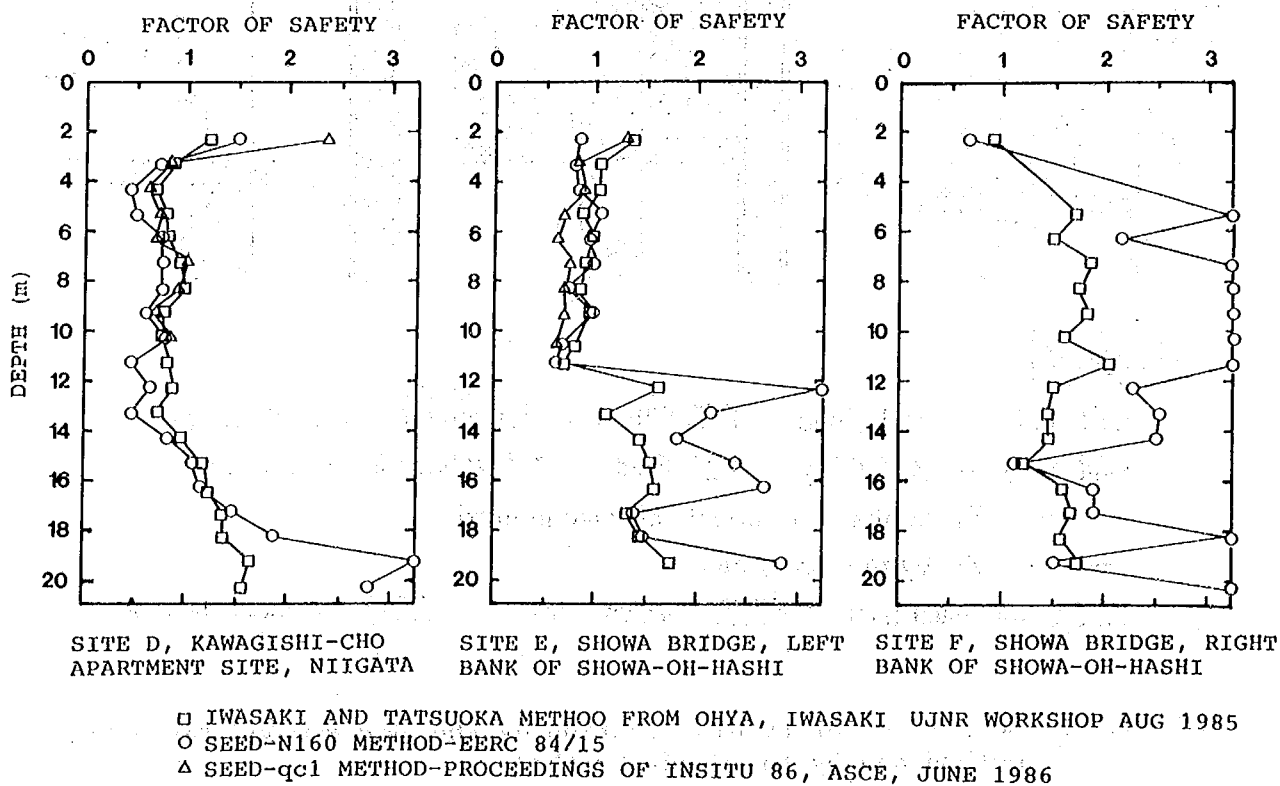


Figure 7 Summary of predicted factor of safety

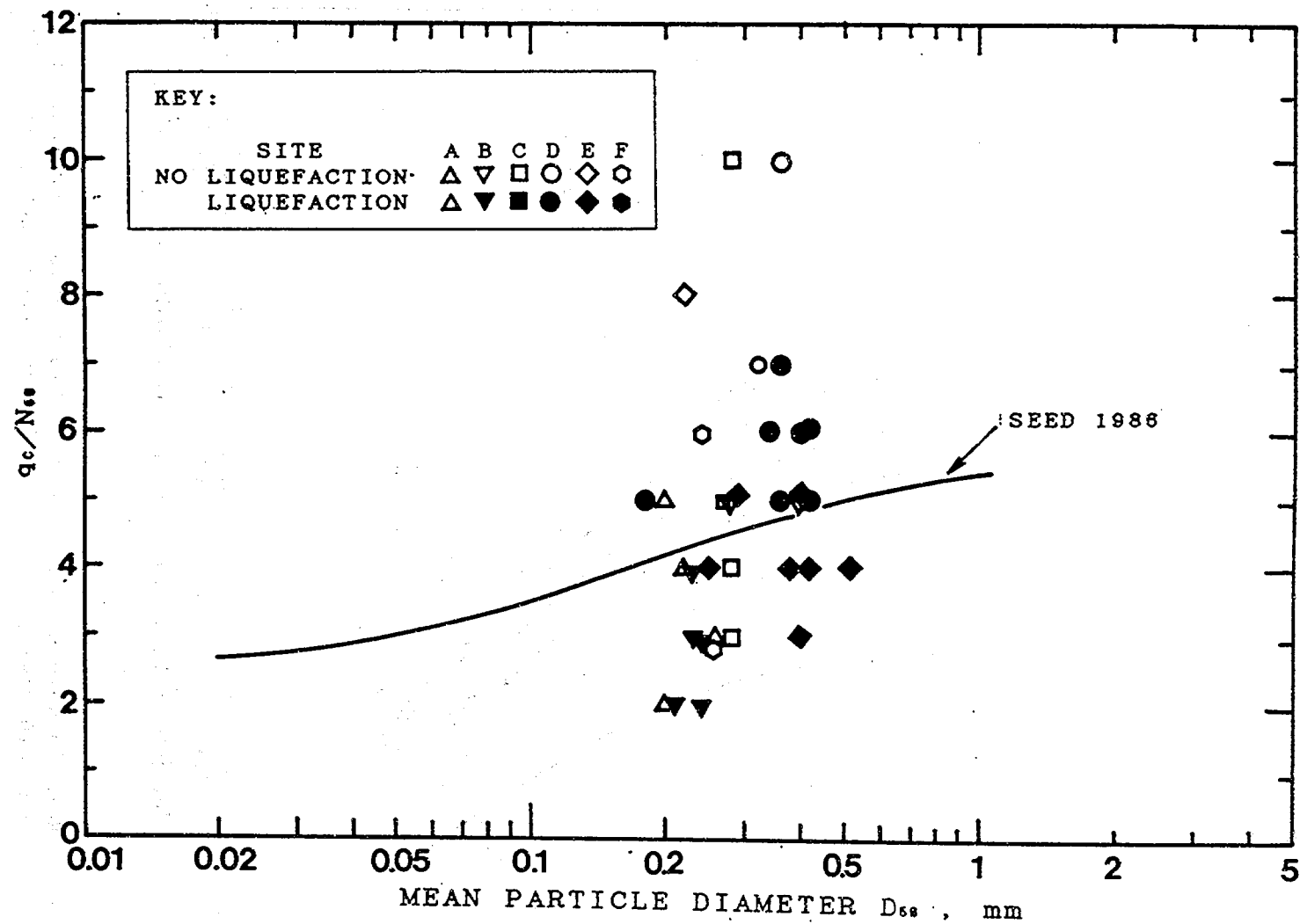


Figure 8 Ratio of cone resistance and SPT N_{60} value vs. mean particle diameter

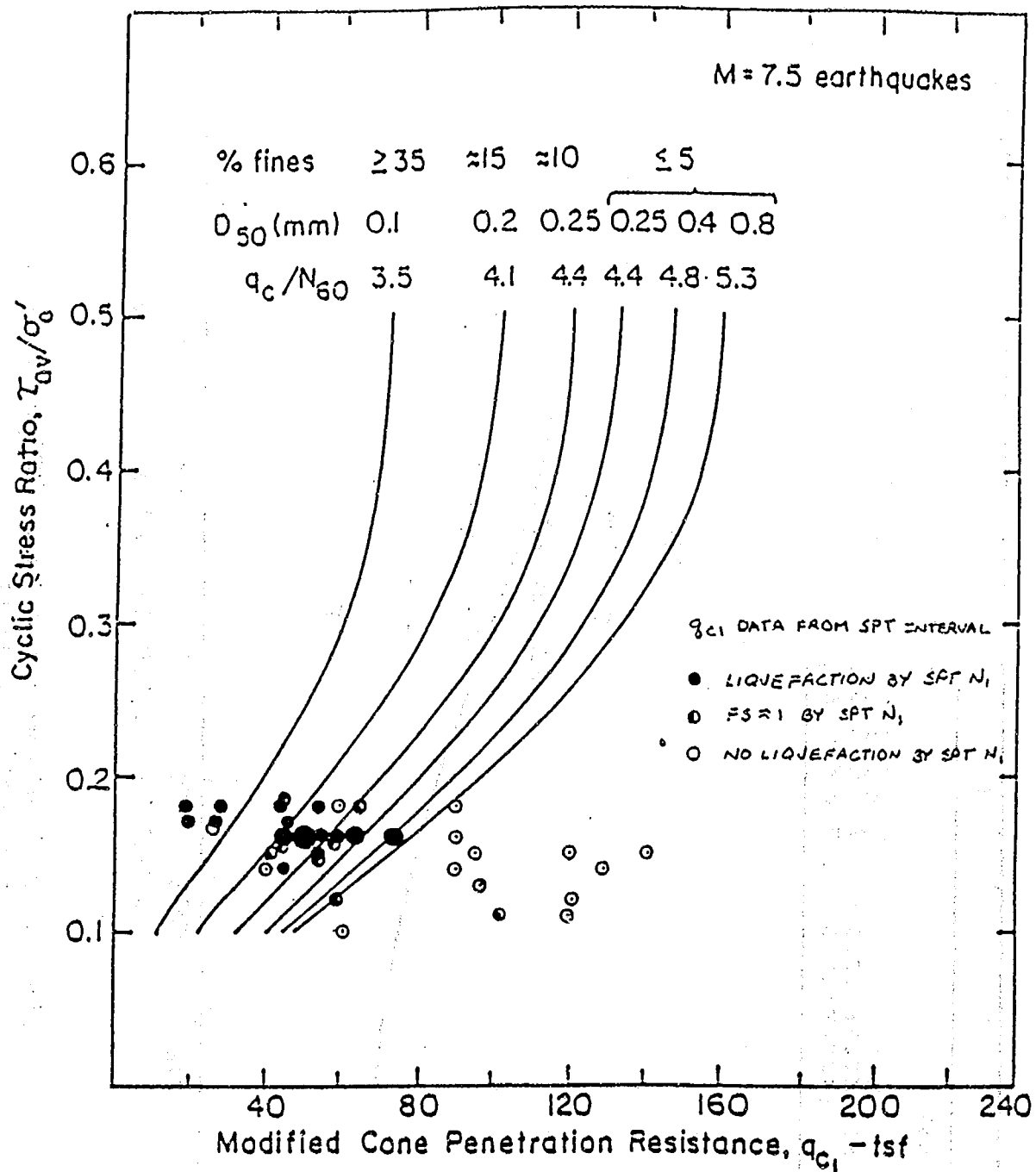


Figure 9 Relationship between stress ratio causing liquefaction and cone tip resistance for sands and silty sands - (seed 1986)

Summary of dependent Q_{c1} data points from the 1983 US - Japan joint study

CPT SOIL CHARACTERIZATION CHART

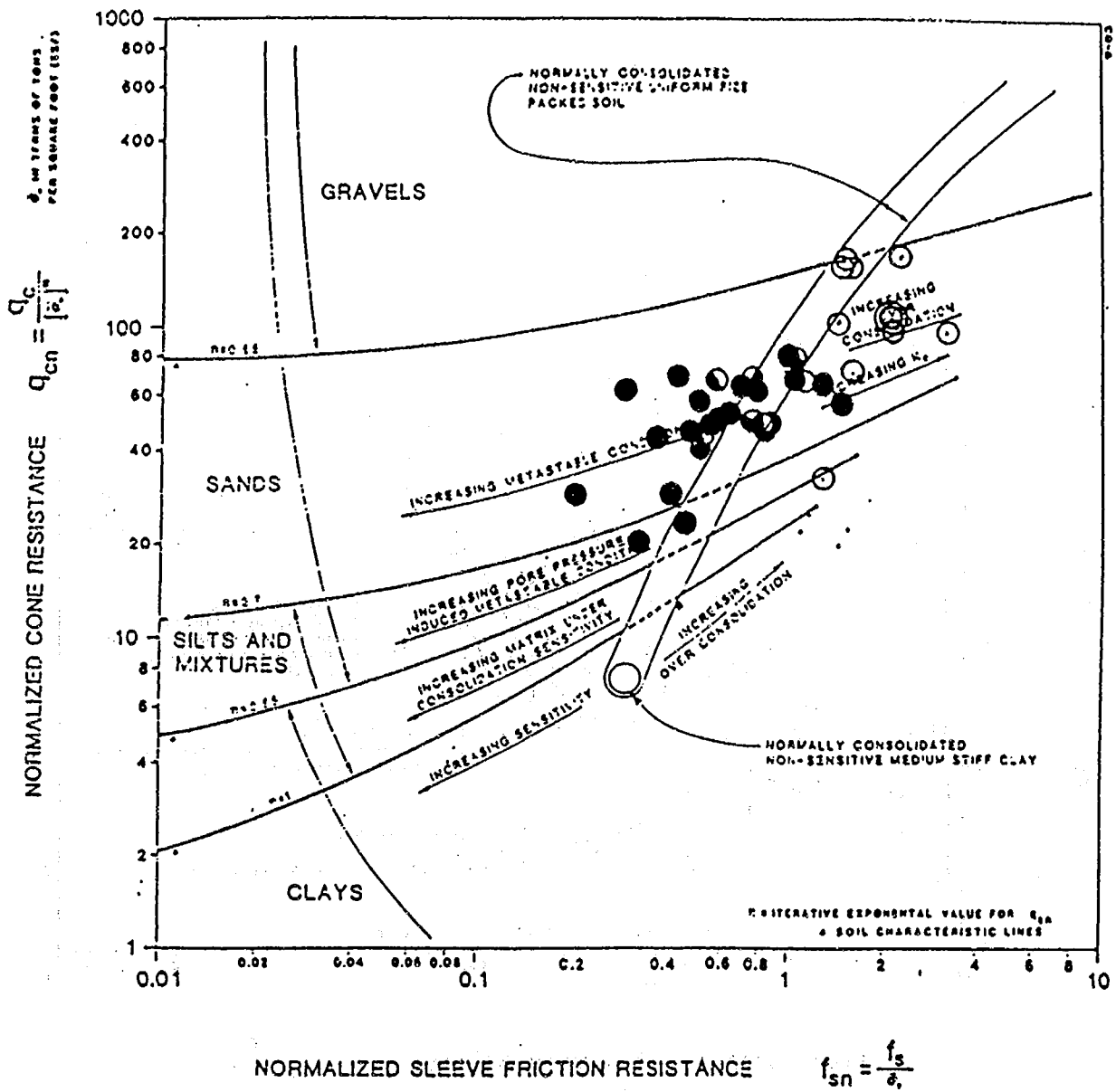
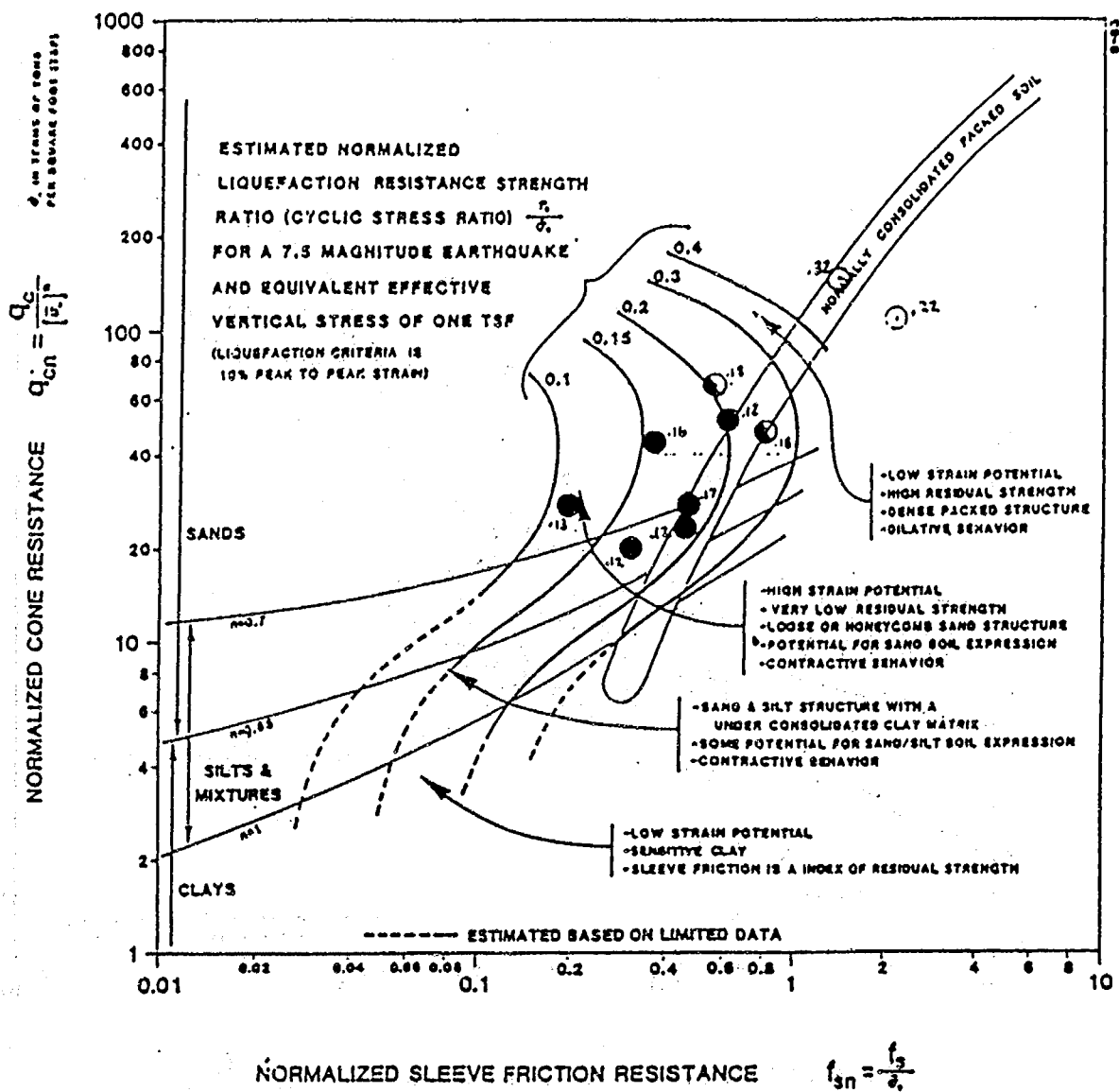


Figure 10 Summary of normalized tip and sleeve friction CPT Data - 1983 US - Japan joint study - all sites



NO LIQUEFACTION
CRITICAL FS= 1.0
LIQUEFACTION

LIQUEFACTION OCCURRENCE FROM
SPT N160 METHOD - NUMERAL
BESIDE POINT IS ESTIMATED N160
STRENGTH

Figure 11: Summary of normalized tip and sleeve friction CPT
Data - Site B - FD 6+900 - Hachiro Gata

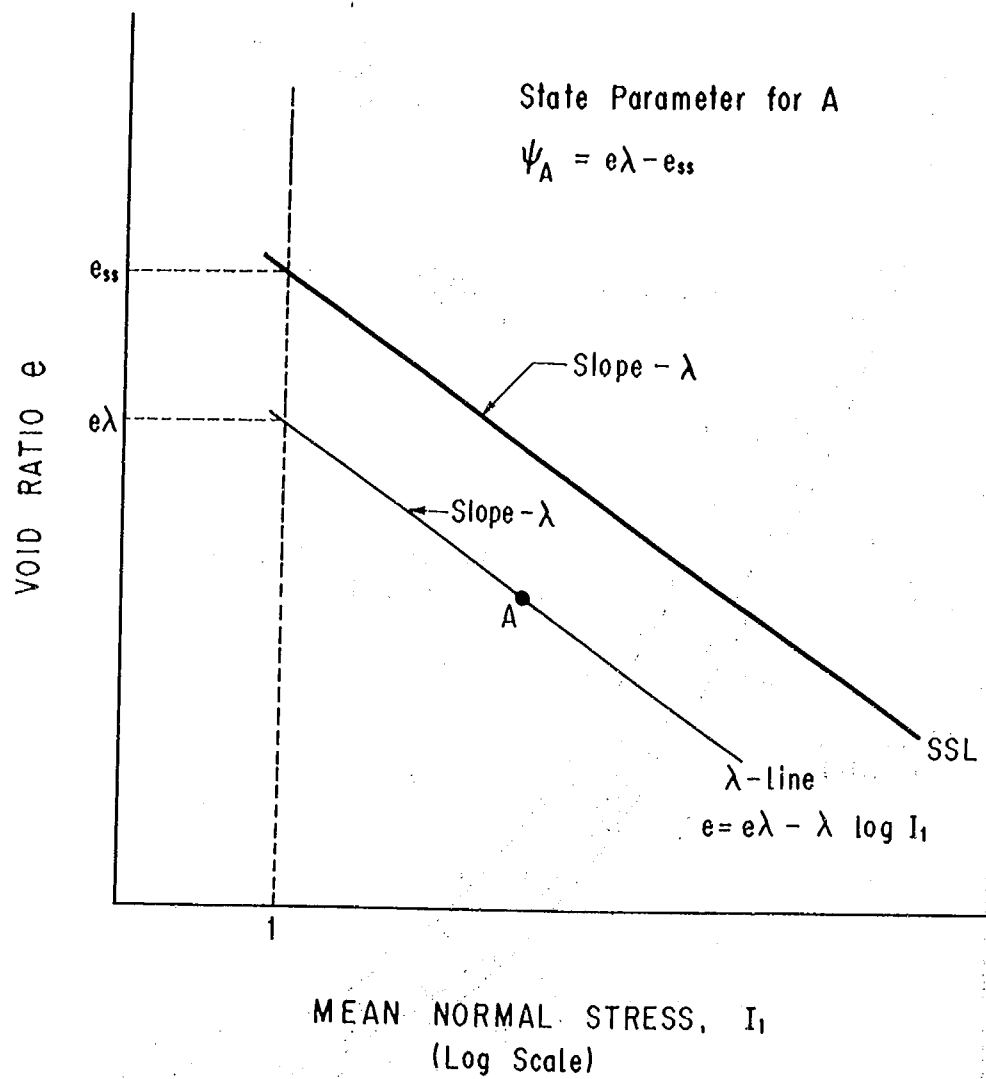


Figure 12 Definition of state parameter ψ

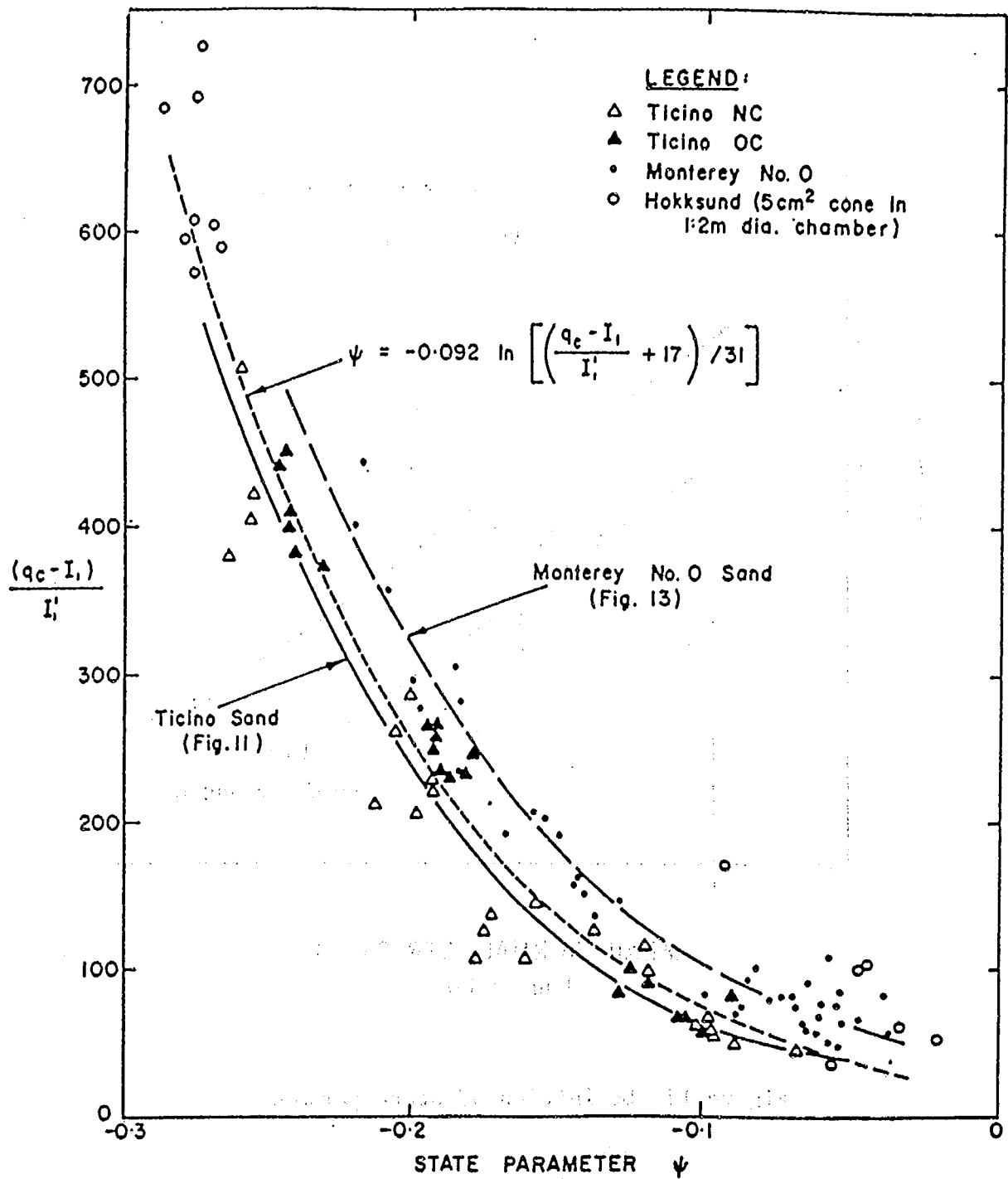


Figure 13 Comparison of normalized cone resistance state parameter correlations for Hokksund Ticino and Monterey #0 sands

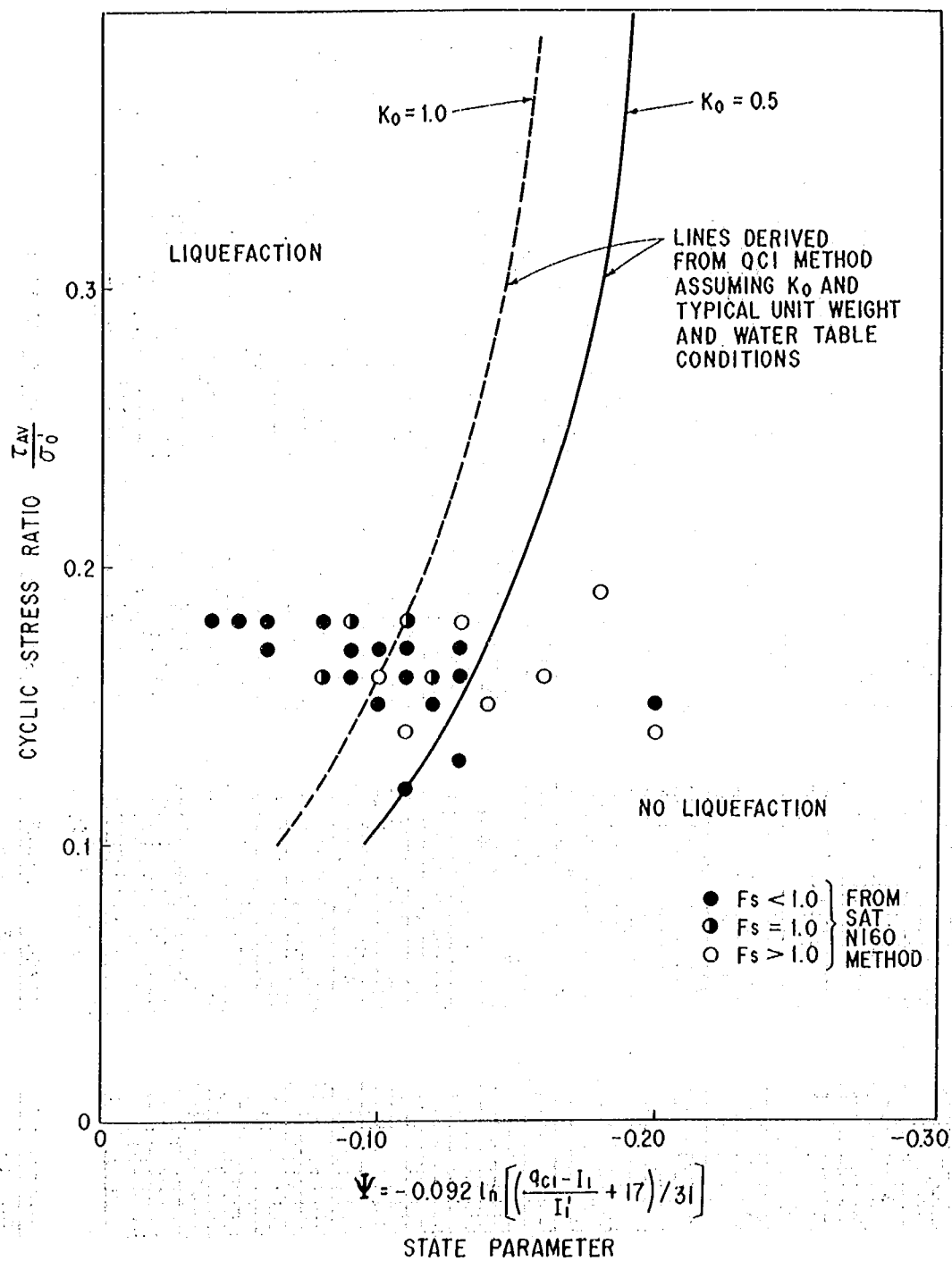


Figure 14 Proposed Relationship between Stress Ratio causing Liquefaction and State Parameter for Clean Sands

FIGURE 15

GRAPHS OF RESULTS FROM DILATOMETER SOUNDING No.

Sounding location: SITE B, ED 6+920 2'S.B. HACHIRO SATO LA3001
 Graphs plotted by: WU - JAF Date: 1/22/76

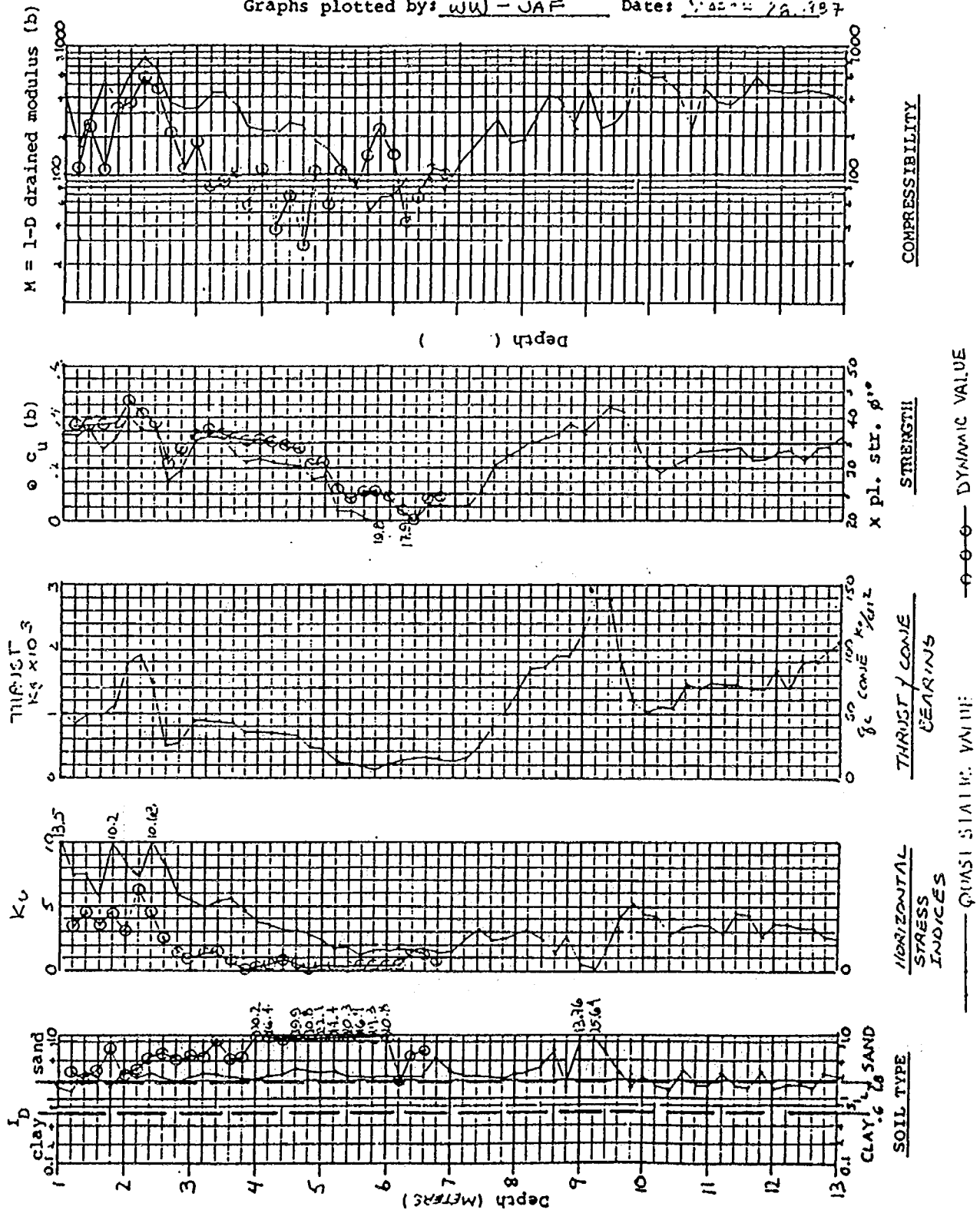


Figure 15 Graphs of results from dilatometer sounding

FIGURE 16

GRAPHS OF RESULTS FROM DILATOMETER SOUNDING No.

Sounding location: STATION W^a 13 + 620 24m D. E. UACHIRO 3A7A 12500N
 Graphs plotted by: WW - JAF Date: MAR 26, 1967

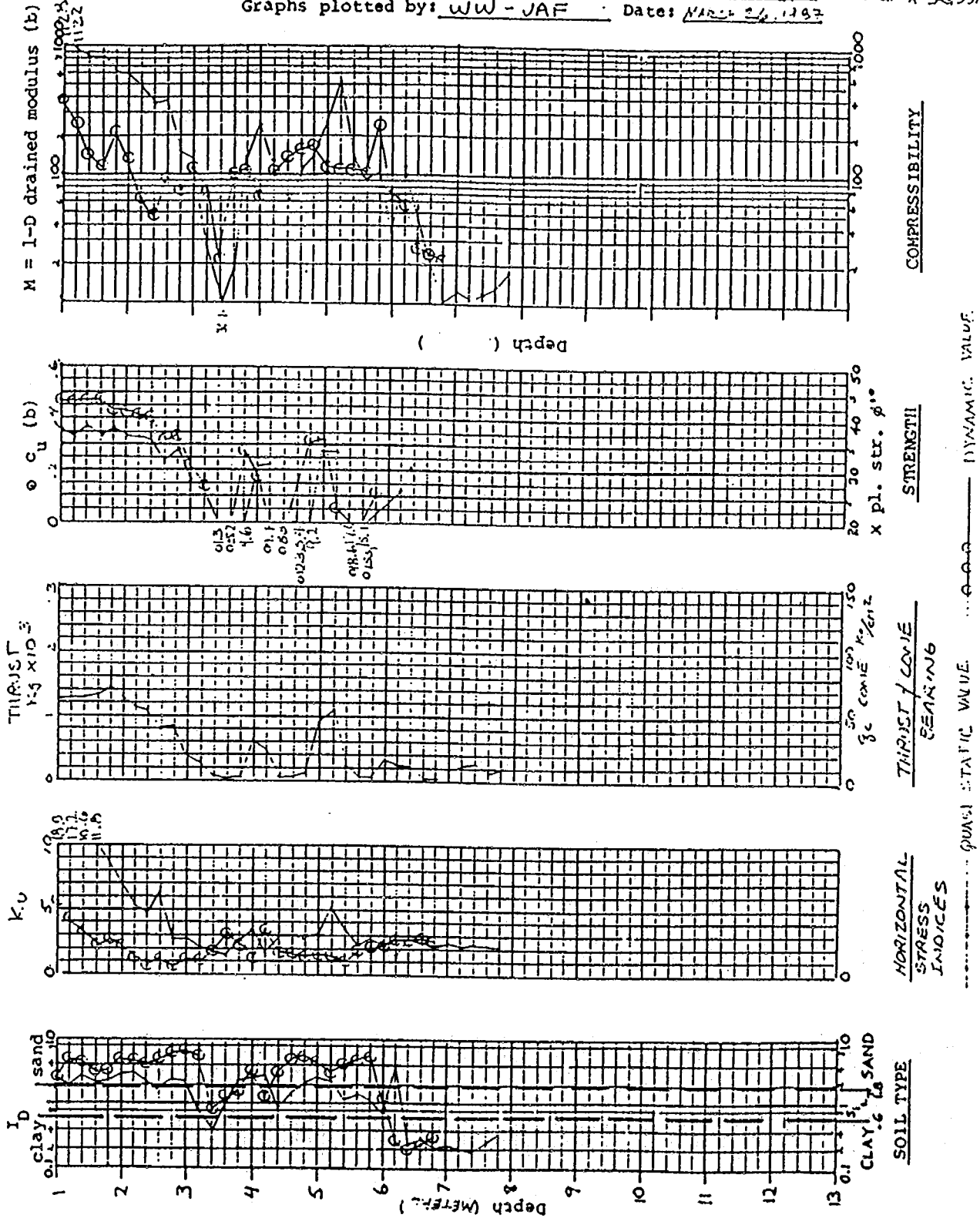


Figure 16 Graphs of results from dilatometer sounding

FIGURE 17

GRAPHS OF RESULTS FROM DILATOMETER SOUNDING No.

Sounding location: SITE C, STA W 0+530 HACHING DATA LA 5001
 Graphs plotted by: WW - JAF Date: MARCH 5, 1977

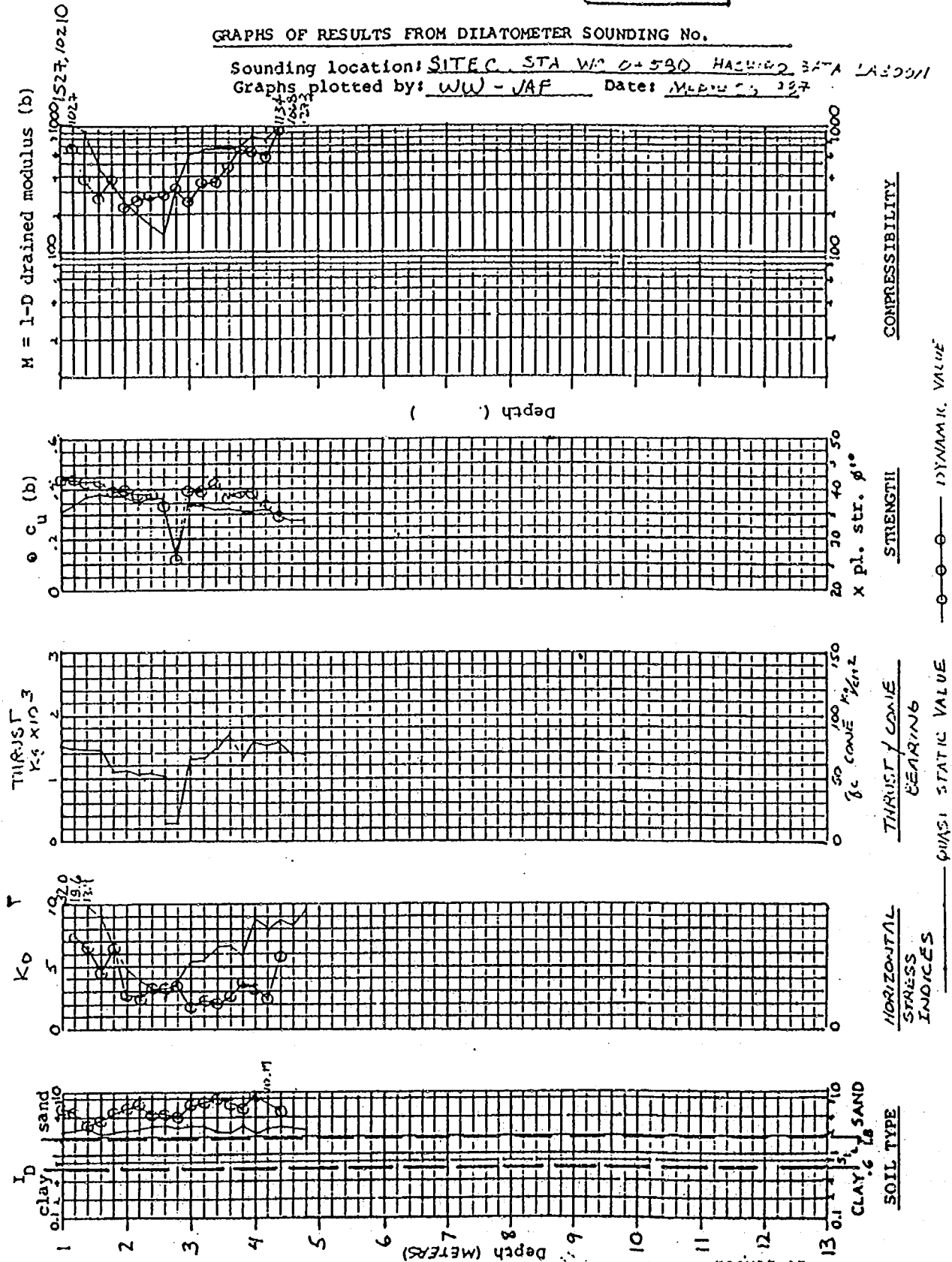


Figure 17 Graphs of results from dilatometer sounding

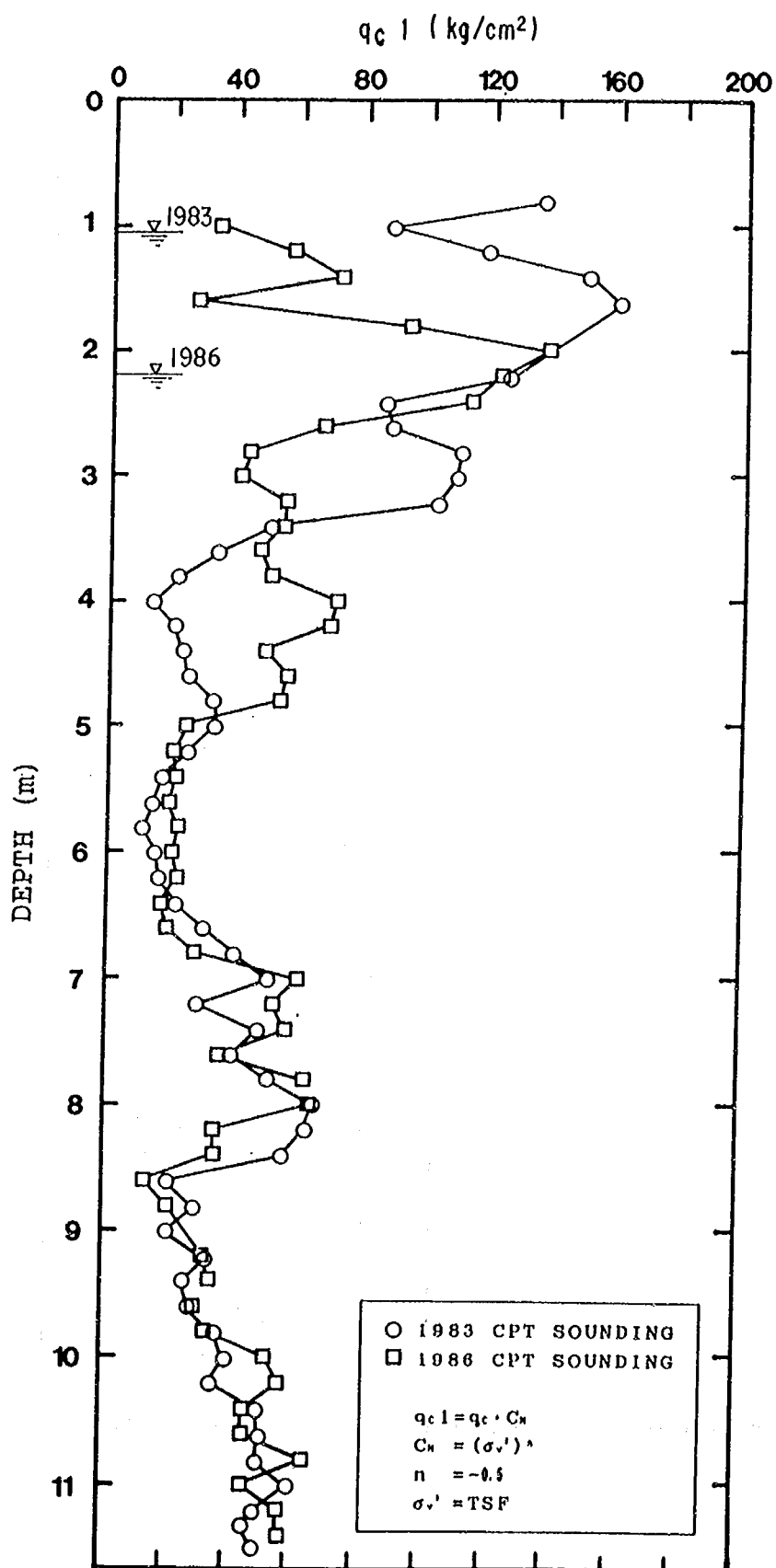


Figure 18 Comparison of 1983 and 1986 QC1 values - Site B - FD 6+900

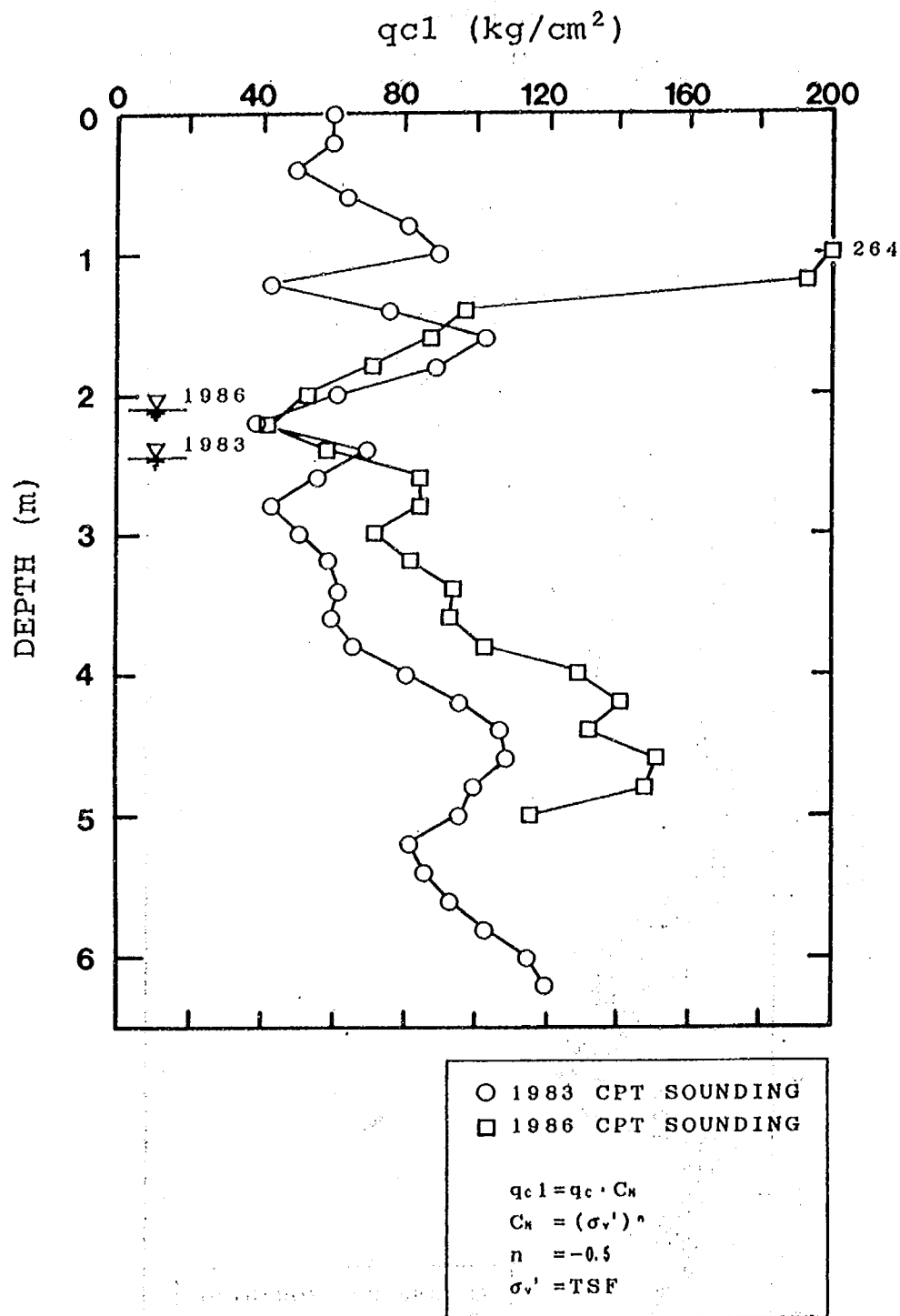
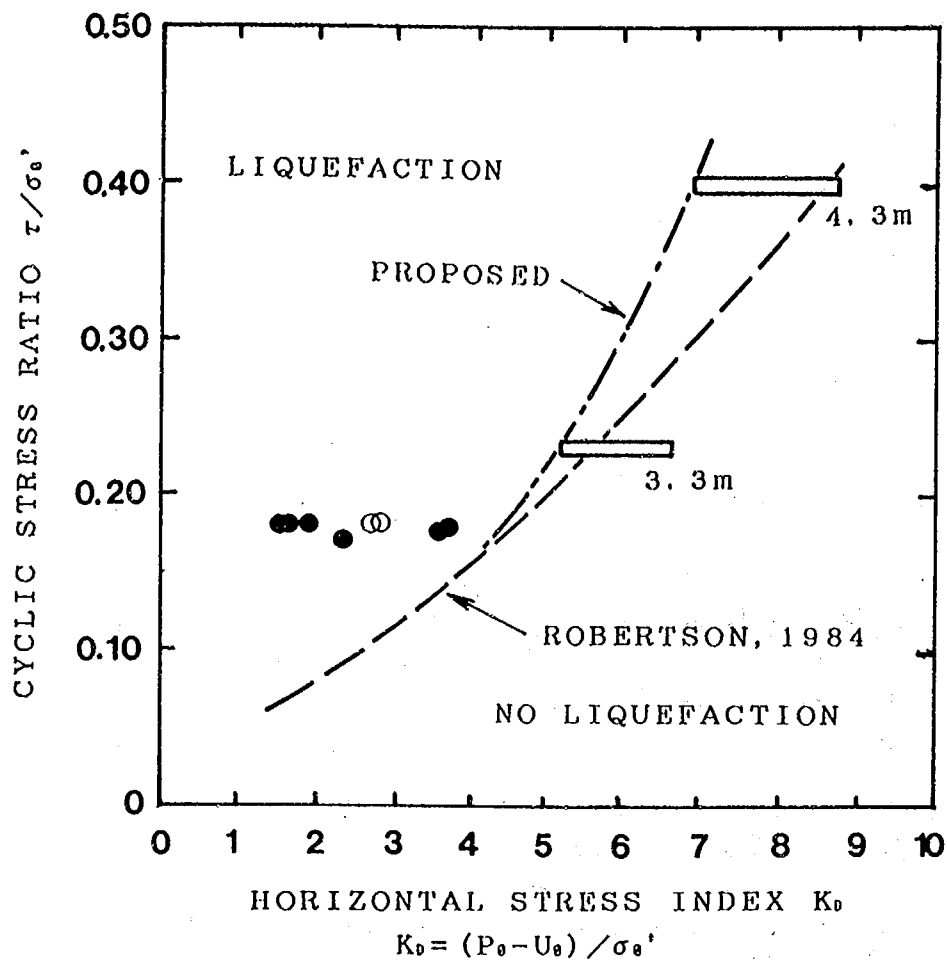


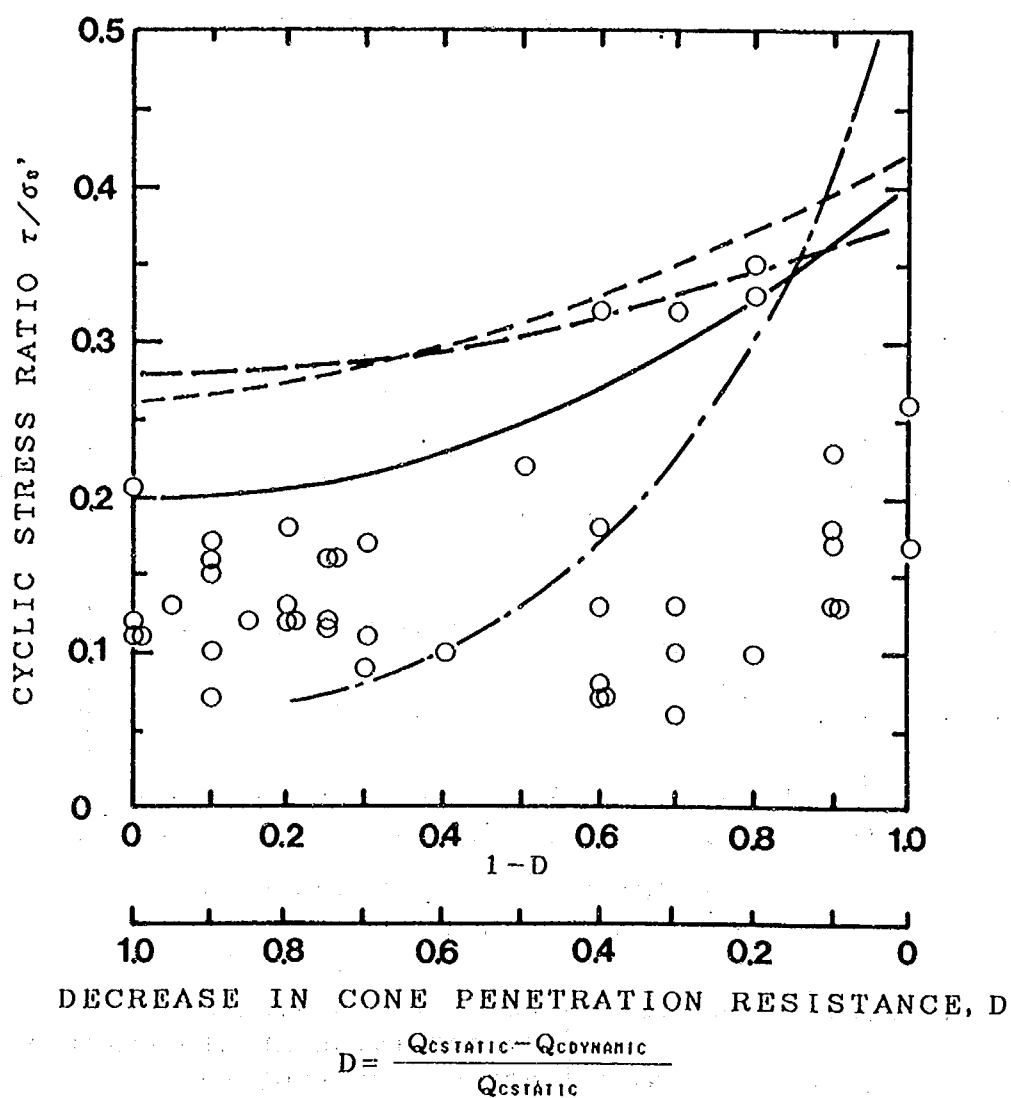
Figure 19 Comparison of 1983 and 1986 QC1 values - Site C - WD 0+585



KEY: ● LIQUEFACTION } FROM SITE B, AVERAGE K_0
 ○ CRITICAL $F_s \pm 1.0$ } IN SPT INTERVAL, LIQUEFACTION
 OCCURRENCE FROM SPT N_{100}

┌─┐ SITE C ESTIMATED CYCLIC STRENGTH FROM 1986
 CPT DATA USING Q_{c1} METHOD

Figure 20 Proposed correlation between liquefaction resistance under level ground conditions and dilatometer horizontal stress index



KEY: O D VALUE IN SPT INTERVAL AND CYCLIC STRENGTH FROM N160

$$\left. \begin{array}{l} \text{--- } \sigma'_v = 1.0 \text{ kgf/cm}^2 \\ \text{--- } \sigma'_v = 0.5 \text{ kgf/cm}^2 \end{array} \right\} \tau_{120} = \left[0.2 \cdot \sigma'_v{}^{1/2} \cdot (1-D)^2 \right] - \left(\frac{0.144}{\sigma'_v + 0.4} \right) + 0.3 \quad \text{SASAKI 85'}$$

$$\text{--- } R_{120} = 0.26 + 0.16 (1-D)^{1.5} \quad \text{SASAKI 84'}$$

--- ANTICIPATED TREND

Figure 21 Comparison of vibrocone penetration and cyclic strength from SPT N160 method

**SUMMARY OF RESEARCH ON THE EFFECTS OF TOPOGRAPHIC AMPLIFICATION
OF EARTHQUAKE SHAKING ON SLOPE STABILITY**

by

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March 1987

Preface

This report summarizes my research activities conducted at the Public Works Research Institute (PWRI) of the Ministry of Construction, located in Tsukuba Science City, Japan. The research, carried out from January to March of 1987, was funded through an appointment by the Science and Technology Agency of the Government of Japan. Dr. Yasushi Sasaki, Head of the Ground Vibration Division at PWRI, supervised the project.

The objective of my research at PWRI was to determine the effects of topographic amplification of earthquake shaking on slope stability, particularly in the case of large, deep-seated landslides. The massive debris avalanche from the south flank of Mt. Ontake triggered by 1984 Naganoken-Seibu earthquake was investigated as a possible case study of topographic amplification leading to slope failure. This required synthesis of research on several subjects:

1. A method was developed to estimate the relative dynamic performances of slopes having different dynamic stabilities and experiencing different levels of seismic shaking.

2. Findings from other investigations of topographic amplification were analyzed for comparison with each other and with findings from the present research. Data from seismic site-response experiments in Chile and Japan were examined in detail.

3. Results of shaking-table tests on silicon slope models, conducted at PWRI in 1987, were analyzed and compared with the results of the field studies mentioned above and to the Mt. Ontake debris avalanche.

4. Findings from the research described above were synthesized and applied to the Mt. Ontake debris avalanche to determine the effects, if any, of topographic amplification on the formation of the avalanche.

The research activities outlined above are summarized in four reports included herein as Parts 1-4, respectively. The reports were written somewhat independently of each other as the research progressed and are presented sequentially as they were written. Therefore, some statements in later reports supersede those made in earlier reports, because new information became available. The reports do, however, reference each other where appropriate (e.g. "see Part 3," etc.).

Page numbers are sequential from the beginning of Part 1 through the end of Part 4. Figures, tables, and equations, however, are numbered separately within each Part (for example, "figure 2.5" is figure 5 of Part 2, etc.); figures and tables for

each Part are collected at the end of that Part. References from all Parts are combined in a reference section at the end of the report.

Several people provided valuable help and support throughout this project. Dr. Y. Sasaki and Dr. T. Iwasaki of PWRI arranged for my visit and directed the research; Mr. T. Kuwabara of PWRI provided the results of his slope model tests and aided in their interpretation; Mr. Tamura of PWRI provided data from the Matsuzaki dense seismic array; Dr. K. Ishihara of Tokyo University helped me reconstruct his slope-stability model and provided data on dynamic soil strengths; discussions with Dr. Y. Kobayashi of Kyoto University and Dr. E. Harp of the U. S. Geological Survey aided in the evaluation of dynamic slope performance.

CONTENTS

PREFACE	55
PART 1: Use of the Newmark Analysis to Predict Dynamic Slope Performance	59
1.1 Introduction	59
1.2 Earthquake shaking intensity	59
1.3 Seismic slope stability	61
1.4 Evaluation of dynamic slope performance	63
Tables	67
Figures	70
PART 2: Description and Analysis of Preliminary Results of Seismic Site-Response Experiment Following the March 3, 1985 Central Chile Earthquake	72
2.1 Introduction	72
2.2 Comparisons of seismic responses	73
2.2.1 North-South versus East-West components	73
2.2.2 Ridge B-C versus ridge E-F	74
2.2.3 Station CBB versus station CBC	74
2.2.4 Station CBE versus station CBF	75
2.2.5 Comparison of all stations	75
2.2.6 Low-frequency versus high-frequency peak responses	76
2.3 Analysis of peak amplification	77
2.4 Conclusions	79
Tables	81
Figures	84
PART 3: Comparison of Results of Slope Model Tests with Field Studies of Topographic Amplification of Ground Shaking ...	92
3.1 Introduction	92
3.2 Description of model tests	92
3.3 Analysis of frequency-domain results	93
3.4 Spatial variation in acceleration	96
3.4.1 Variation along ridge crests	96
3.4.2 Variation along the maximum vertical cross section	97
3.4.3 Conclusions	98

3.5	Spatial variation in shear stress	98
3.5.1	Variation along ridge crests	99
3.5.2	Variation along the maximum vertical cross section	100
3.5.3	Conclusions	101
3.6	Comparison with results of Chile earthquake experiment	101
3.7	Comparison with data from the Matsuzaki dense seismic array ...	103
3.8	Summary and conclusions	104
	Tables	106
	Figures	109

PART 4:	Did Topographic Amplification of Earthquake Shaking Play a Role in Triggering the Mount Ontake Debris Avalanche?	153
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4.1	Introduction	153
4.2	Modeling slope stability	153
4.3	Comparison with similar slopes	155
4.4	Possible effects of topographic amplification	157
4.5	Summary and discussion	159
	Tables	161
	Figures	163

REFERENCES	169
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PART 1

Use of the Newmark Analysis to Predict Dynamic Slope Performance

1.1 INTRODUCTION

A method for comparing the dynamic responses of slopes having different stabilities and in different levels of earthquake shaking would find wide application in regional seismic hazard analysis. The most recent and comprehensive study on this subject is that of Wilson and Keefer (1986)¹⁾, who developed a method for predicting the areal limits of earthquake-induced landsliding from a given design earthquake. They used a four-step approach. First, they showed how to determine a slope's seismic stability as measured by its critical acceleration, the threshold ground acceleration required to initiate landslide movement. Second, they introduced the use of Arias intensity as a measure of earthquake shaking intensity and developed an equation that relates Arias intensity to earthquake magnitude and source distance. Third, they used the landslide displacement analysis developed by Newmark (1965)²⁾ to model dynamic slope response and estimated the amount of Newmark displacement necessary to cause failure. Fourth and finally, they determined the combination of critical acceleration and Arias intensity necessary to generate this amount of displacement and calculated the distance from the seismic source at which this Arias intensity would occur for their design earthquake.

The approach outlined above provides a valuable theoretical and practical framework for evaluation of seismic slope stability, but it suffers from two significant problems: it is somewhat unwieldy and difficult to apply, and its governing equations are based on several unjustified, though not unreasonable, assumptions. To overcome these problems, I use an approach similar to that of Wilson and Keefer (1986)¹⁾, but I derive the governing equations in a different manner and then develop a single equation easily applied to the evaluation of slope performance during earthquakes. This will involve (1) the development of a relationship relating earthquake shaking intensity to magnitude and source distance, and (2) the development of a relationship between seismic slope stability, earthquake shaking intensity, and dynamic slope performance.

1.2 EARTHQUAKE SHAKING INTENSITY

Earthquake shaking intensity as defined by Arias (1970)³⁾ has been shown to correlate well with earthquake damage in

general and landslides in particular (Wilson and Keefer, 1983⁴), 1986¹); Jibson, 1985⁵); Jibson and Keefer, in press⁶)). As defined by Arias (1970)³), this intensity (I_a) is directly proportional to the integral of the square of an acceleration time-history of an earthquake, which is simply the area enclosed by the time-domain strong-motion record. Note that an earthquake does not have an Arias intensity, but rather an earthquake strong-motion record has an Arias intensity, because I_a is measured directly from a given acceleration time-history. Because it is an integration of acceleration, Arias intensity has units of velocity and is normally expressed in meters per second. Arias intensity yields a reliable measure of the total shaking content of a strong-motion record and is thus appropriate for use in analysis of seismic slope stability. Use of Arias intensity as a measure of earthquake shaking is preferred over use of peak ground acceleration, typically used in slope-stability studies, because I_a depends not only on the ground acceleration, which may have a peak value of very high frequency and short duration that will not affect many slopes, but also on the duration of strong shaking.

From considerations of seismological theory, Wilson and Keefer (1986)⁴) suggested a relationship between Arias intensity, earthquake magnitude, and depth-corrected source distance having the following functional form for the mean data distribution:

$$\log(I_a) = a(M) - b \log(R) + c \quad (1.1)$$

where I_a is the Arias intensity; M is the moment magnitude as defined by Hanks and Kanamori (1979)⁷); R is the source distance corrected for focal depth; and a , b , and c are constants. They then assumed that $a = 1$ and $b = 2$, which assumes that the shaking intensity decreases as the square of the source distance; they further assumed a log-linear relationship between magnitude and Arias intensity in order to convert all the input data to a single equivalent magnitude. Although these assumptions may be reasonable as such, none was rigorously justified either theoretically or empirically; they were made solely for the purpose of reducing the problem to a bivariate linear regression.

The functional form of equation 1.1 is well supported by theory (Wilson and Keefer, 1986¹)). Therefore, to produce an equation relating Arias intensity to earthquake magnitude and source distance that does not depend on unjustified assumptions, I employ a multiple linear regression model that allows all of the constants (a , b , and c) in equation 1.1 to vary. This

permits the input data to determine the final form of the magnitude-distance equation. I use the same data set as Wilson and Keefer (1986)¹⁾ with the addition of a strong-motion record from the 1978 Tabas, Iran earthquake (table 1.1), included to add a large value of Arias intensity to the data set. The multiple regression analysis yielded the following equation:

$$\log(I_a) = 0.98(M) - 1.35 \log(R) - 4.90 \quad (1.2)$$

where I_a is in meters/second and R is in kilometers. The coefficient for M is nearly 1, which demonstrates that the data justify this assumption of Wilson and Keefer; the coefficient for $\log(R)$, however, differs significantly from 2 and thus indicates that the Arias intensity does not decrease as R^2 . This model yields a correlation coefficient (r) of 0.84, and is thus well fitted to the data. A comparative chi-square test shows that equation 1.2 is significantly better at predicting the observed values in the input data set than is the equation of Wilson and Keefer (1986)¹⁾. Thus, the theoretical justifications for the functional form presented by Wilson and Keefer have been preserved, but an equation better fit to the data that does not rely on unjustified assumptions has been developed to predict the variation in earthquake shaking intensity as a function of magnitude and source distance.

1.3 SEISMIC SLOPE STABILITY

Seismic slope stability traditionally has been evaluated by pseudostatic analysis, in which earthquake acceleration in conjunction with the landslide mass is treated as an explicit body force. By iteratively employing different input accelerations, a yield acceleration is determined at which the static factor of safety is reduced to 1.0, and any exceedance of this yield acceleration is defined as slope failure. As Wilson and Keefer (1983)⁴⁾ point out, this method is overconservative because it assumes that any exceedance of the yield acceleration results in failure. Experience shows, however, that earthquakes can produce transitory peaks of very high acceleration, sometimes exceeding 1 g, but that many slopes having yield accelerations much lower than the peak earthquake acceleration do not fail despite the fact that their yield accelerations have been exceeded by a considerable amount. Newmark (1965)²⁾ realized that short-duration, large accelerations can induce minor amounts of displacement in a landslide block without causing general failure; therefore, he developed a method for calculating the

displacement of a landslide block of known yield acceleration (or critical acceleration, as expressed by Newmark) when subjected to shaking represented by a given strong-motion record. Newmark (1965)²⁾ showed that for a homogeneous slope, the critical acceleration can be expressed as

$$A_c = (FS - 1)g \sin(\alpha) \quad (1.3)$$

where A_c is the critical acceleration in terms of g , the acceleration of gravity; FS is the static factor of safety, and α is the angle from the horizontal that the center of mass of the potential landslide block first moves. (For failure parallel to an infinite slope, α is the slope angle; for circular failure surfaces, α is the angle between a vertical line and a line connecting the center of mass of the landslide block and the center of the slip circle.) For nonhomogeneous slopes, yield acceleration may provide a more accurate index of a slope's dynamic stability.

As shown in figure 1.1, integration of those portions of a strong-motion record that exceed the critical acceleration of a potential landslide block produces a velocity profile of the block; a second integration produces a cumulative displacement profile of the landslide block. In this manner, a cumulative landslide displacement can be determined from a strong-motion record if the critical acceleration of the potential landslide block is known. Experience has shown that Newmark's method commonly underestimates the amount of actual landslide displacement, because in many slope materials, shear strength decreases during shearing, and thus the critical acceleration is continually decreasing as the slide block translates. Therefore, the results of Newmark's analysis do not necessarily predict the actual landslide displacement, but rather are a relative index of dynamic slope performance. Wilson and Keefer (1983)⁴⁾, however, did document an instance where the Newmark analysis very accurately predicted the actual measured displacement of a landslide triggered by an earthquake.

Because it is seldom possible to find a strong-motion record having the exact shaking intensity of interest to a particular problem, it is desirable to develop a general relationship between Newmark displacement (D_n), critical acceleration, and Arias intensity. To do this, I calculated Newmark displacements from a wide variety of strong-motion records and for a similar variety of critical accelerations. For the 10 strong-motion records listed in table 1.2, which have Arias intensities between

0.2 and 10 m/s, I conducted Newmark analyses for critical accelerations of 0.02, 0.05, 0.1, 0.2, 0.3, 0.4, and 0.5 g, the range of interest for most slope-stability problems. Figure 1.2 shows the results; critical acceleration is plotted as a function of Arias intensity and Newmark displacement, and best-fit lines are drawn through those data points corresponding to a given value of critical acceleration. The lines are nearly parallel as well as being fairly evenly spaced, and the best-fit lines for each critical acceleration have very high correlation coefficients (between 0.79 and 0.99), with the sole exception of $A_c = 0.2$ g ($r = 0.46$). This indicates a linear distribution of critical acceleration in relation to $\log(I_a)$ and $\log(D_n)$. To quantify this multivariate relationship, a multiple regression analysis using the following functional form was conducted:

$$\log(D_n) = a \log(I_a) + b(A_c) + c, \quad (1.4)$$

where D_n is the Newmark displacement; I_a is the Arias intensity; A_c is the critical acceleration; and a , b , and c are constants to be determined by the regression. The model produced by the regression analysis has a correlation coefficient (r) of 0.90, and the resulting equation is

$$\log(D_n) = 1.37 \log(I_a) - 6.36(A_c) + 1.57, \quad (1.5)$$

where D_n is in centimeters, I_a is in meters/second, and A_c is in g's. The high correlation coefficient indicates that this model accurately predicts the Newmark displacement from the input data. Figure 1.3 shows the contours of critical acceleration predicted by equation 1.5 and plotted as a function of $\log(I_a)$ and $\log(D_n)$. The upper dashed line represents $A_c = 0$, the theoretical and practical upper bound to the data; the lower dashed line represents $A_c = 0.5$ g, for which only two data points exist, and which thus provides a practical lower bound for this analysis. Using this model, any combination of critical acceleration and Arias intensity can be combined to estimate Newmark displacement, which thus eliminates the need to locate a strong-motion record that has the exact Arias intensity desired.

1.4 EVALUATION OF DYNAMIC SLOPE PERFORMANCE

When analyzing dynamic slope performance, we seldom know the exact shaking intensity of interest; rather, we normally model the effects of some actual or postulated earthquake of known magnitude and location. Therefore, an equation directly relating

Newmark displacement to earthquake magnitude and source distance, as well as to the critical acceleration of the slope, is most useful. Such an equation can be produced by substituting the right side of equation 1.2 for I_a in equation 1.5, which yields the following:

$$\log(D_n) = 1.37(0.98(M) - 1.35 \log(R) - 4.90) - 6.36(A_c) + 1.57,$$

and thus (1.6)

$$\log(D_n) = 1.34(M) - 1.85 \log(R) - 6.36(A_c) - 5.14, \quad (1.7)$$

where D_n is in centimeters, M is the moment magnitude, R is in kilometers, and A_c is in g's. The signs of the coefficients in this equation are intuitively consistent, in that they indicate that increasing magnitude produces increases landslide displacement, and that increasing source distance or critical acceleration results in decreased landslide displacement. This equation is very versatile, and can be solved for any of the four variables if the other three are known or postulated, as follows:

$$\log(R) = 0.72(M) - 0.54 \log(D_n) - 3.44(A_c) - 2.78, \quad (1.8)$$

$$M = 1.38 \log(R) + 0.75 \log(D_n) + 4.75(A_c) + 3.84 \quad (1.9)$$

$$A_c = 0.21(M) - 0.29 \log(R) - 0.16 \log(D_n) - 0.81 \quad (1.10)$$

In the case of Mt. Ontake (see Part 4), the magnitude and hypocenter of the earthquake are known, the critical accelerations of various slopes on the mountain can be calculated from the estimated static factors safety and the slope angles, and thus the Newmark displacement can be estimated for comparison of the dynamic performances of these slopes. Equations 1.7-1.10 can also be applied in any number of ways to problems of regional hazard analysis as well as to individual sites.

For some applications, it is more useful to have an equation in terms of peak ground acceleration (PGA) than earthquake magnitude and distance. Wilson (in press) developed a well constrained relationship between I_a , PGA , and the duration of strong ground shaking, T_d (defined by Dobry and others (1978)⁸) as the time required to build up the central 90 percent of the Arias intensity of an earthquake record):

$$I_a = 0.90(PGA^2)(T_d), \quad (1.11)$$

where I_a is in meters/second, PGA is in g's, and T_d is in second. Substituting the right side of equation 1.11 for I_a in equation 1.5 yields the following:

$$\log(D_n) = 2.74 \log(PGA) + 1.37 \log(T_d) - 6.36(A_c) + 1.51, \quad (1.12)$$

Thus, equation 1.12 can be used to estimate the Newmark displacement if the peak ground acceleration, shaking duration, and critical acceleration of the landslide are known or estimated.

Appraisal of dynamic slope performance from the Newmark displacement requires considerable judgment, however. Wilson and Keefer (1986)¹⁾ suggested using 10 cm of Newmark displacement as the threshold for failure (macroscopic cracking and damage to overlying structures) for coherent slides, and 2 cm for rock falls and other disrupted slides. The amount of actual landslide displacement, however, will depend on the properties of the landslide material. For slopes that will fail in tension and for slopes composed of very brittle materials or highly sensitive soils--those whose shear strength decreases by a large amount during shearing--a Newmark displacement of about 2 cm will probably result in catastrophic failure (infinite displacement). For moderately sensitive soils, large displacements will probably occur for Newmark displacements of about 5 cm. For nonsensitive or only slightly sensitive soils, the Newmark displacement probably corresponds approximately to the actual displacement, and, as Wilson and Keefer (1986)¹⁾ note, displacements of about 10 cm generally cause ground cracking and damage to overlying structures. Regardless of the relationship of the actual landslide displacement and that predicted by the Newmark analysis, for a given type of slope material, the Newmark displacement provides a valuable index to the relative dynamic performance of slopes having any range of static slope stabilities and in any level of earthquake shaking.

The amount of Newmark displacement required to cause catastrophic failure for a given type of slope material can be estimated by examining case studies of historical landslides triggered by earthquakes. If Newmark displacements for several such landslides can be calculated, they may indicate a minimum amount of Newmark displacement necessary to cause failure in a given type of slope material. Ishihara and Hsu (1986)⁹⁾ compiled data on several landslides triggered by the 1980 Irpinia, Italy and the 1984 Naganoken-seibu, Japan earthquakes. Table 1.3 lists the critical acceleration of each of the landslides and the

magnitude and source distance of the triggering earthquake. The Newmark displacement, D_n , was calculated using equation 1.7.

The landslides triggered by the Naganoken-seibu earthquake all formed in pumice, and 3 cm appears to be a reasonable lower bound of Newmark displacement required to cause catastrophic failure. The landslides triggered by the Irpinia earthquake all were reactivated slides in variegated clay; for these slides, 2 cm of Newmark displacement is a reasonable lower bound leading to general failure. These amounts of Newmark displacement agree well with the estimated critical displacements discussed previously.

Equations 1.7-1.10 and 1.12 reflect the well supported theoretical basis established by Wilson and Keefer (1986)¹⁾ for the relationship between earthquake magnitude, source distance, seismic slope stability, and dynamic slope response. These equations also are extremely easy to use, are rigorously constrained by empirical data, and are not based on unsupported assumptions.

Table 1.1 Magnitude-Distance-Arias Intensity Data
(from Wilson and Keefer, 1986)

Earthquake/Station	Moment Magnitude (M)	Source Distance (r), in km	Accele- ration (A)	Arias Intensity (I _a) in m/s
Kern Country/Taft -----	7.4	43	0.179	0.59
Parkfield/Station 2 -----	6.1	6.6 ¹	0.498	1.71
Parkfield/Station 5 -----	6.1	9.3 ¹	0.434	0.85
Lower California/El Centro -----	6.5	53	0.182	0.62
Imperial/EL Centro -----	7.0	12	0.348	1.93
San Fernando/Castaic -----	6.6	21	0.271	0.99
Eureka / Federal Building -----	6.5	25	0.257	0.710
San Fernando/Orion -----	6.6	20	0.255	1.28
Lytle Creek/Wrightwood --	5.3	14	0.198	0.14
Parkfield/Temblor -----	6.1	16	0.347	0.45
Ferndale/City Hall -----	5.6	25	0.237	0.105
San Fernado/Palmdale -----	6.6	34	0.113	0.334
Kern County/Pasadena -----	7.4	130	0.053	0.114
Hollister/City Hall -----	5.6	40	0.065	0.135
San Francisco/Golden Gate -----	5.3	11	0.105	0.05
San Francisco/State Building -----	5.3	17	0.085	0.052
San Francisco/Alex. Building -----	5.3	16	0.043	0.016
San Francisco/Oakland ---	5.3	26	0.04	0.010
Borrego/San Onofre -----	6.6	122	0.046	0.035
Kern County/(A005) -----	7.4	85	0.131	0.29
Kern County/(A006) -----	7.4	109	0.053	0.11
Borrego/(A020) -----	6.6	96	0.029	0.03
Long Beach/Vernon -----	6.3	53	0.133	0.23
San Jose/(A010) -----	5.8	10	0.102	0.075
Southern California/ (B023) -----	5.4	38	0.033	0.010
Wheeler Ridge/(B031) -----	5.9	43	0.068	0.042
Central California/ (U307) -----	5.0	6	0.057	0.05
Northern California/ (U308) -----	5.7	59	0.075	0.04
Torrance-Gardena/ (V316) -----	5.4	6	0.055	0.04
Southern California/ (V329) -----	5.0	6	0.167	0.100
Tabas, Iran -----	7.4	37	0.705	9.926

Table 1.2 Strong Motion Records for Newmark Analysis

No.	File Name	Earthquake Information	Duration (sec)	Time Increment (sec)	Arias Intensity (m/sec)	PGA (g)
1	PARKFL-2	1966 Parkfield, station 2	12	0.05	1.636	0.485
2	PARKFL-5	1966 Parkfield, station 5	11	0.05	0.953	0.447
3	ELCENTRO	1940 Imperial Valley El Centro station	34	0.05	1.722	0.293
4	TAFT	1952 Tehachapi Taft School	29	0.05	0.461	0.144
5	GOLETA	1978 Santa Barbara Goleta station	30	0.05	0.928	0.338
6	HILO-75	1975 Hilo	12	0.05	0.200	0.205
7	GLRY6-NE	1979 Coyote Lake Gilroy station 6 (050)	12	0.05	0.710	0.360
8	GLYCC250	1979 Coyote Lake Coyote Creek station (250)	11	0.05	0.245	0.206
9	PACO-SE	1971 San Fernando Pacoima Dam S16E	21	0.01	9.069	1.22
10	IRAN2	1978 Tabas, Iran	34	0.01	9.926	0.705

Table 1.3 Newmark displacements of landslides triggered by the 1980 Irpinia, Italy and 1984 Naganoken-Seibu, Japan earthquakes. Data on earthquake magnitudes and distances and on critical accelerations from Ishihara and Hsu(1986).

Landslide	Earthquake Magnitude	Source Distance (km)	Critical Acceleration (g)	Newmark Displacement (cm)
1980 Irpini, Italy Earthquake				
Buoninventre	6.5	8	0.210	3.66
Serra dell' Acquara	6.5	10	0.135	7.27
Pergola	6.5	10	0.170	4.35
Grassano	6.5	80	0.155	1.83
Andretta	6.5	18	0.095	4.40
1984 Naganoken-seibu, Japan Earthquake				
Ontake Headwall	6.8	10	0.27	2.54
Matsukoshi	6.8	1	0.32	86.46
Ontake Highland 1	6.8	3	0.30-0.32	11.33-15.18
Ontake Highland 2	6.8	3	0.27-0.34	8.45-23.56
Ontake Highland 3	6.8	3	0.40	3.51
Ontake Highland 4	6.8	3	0.40	3.51

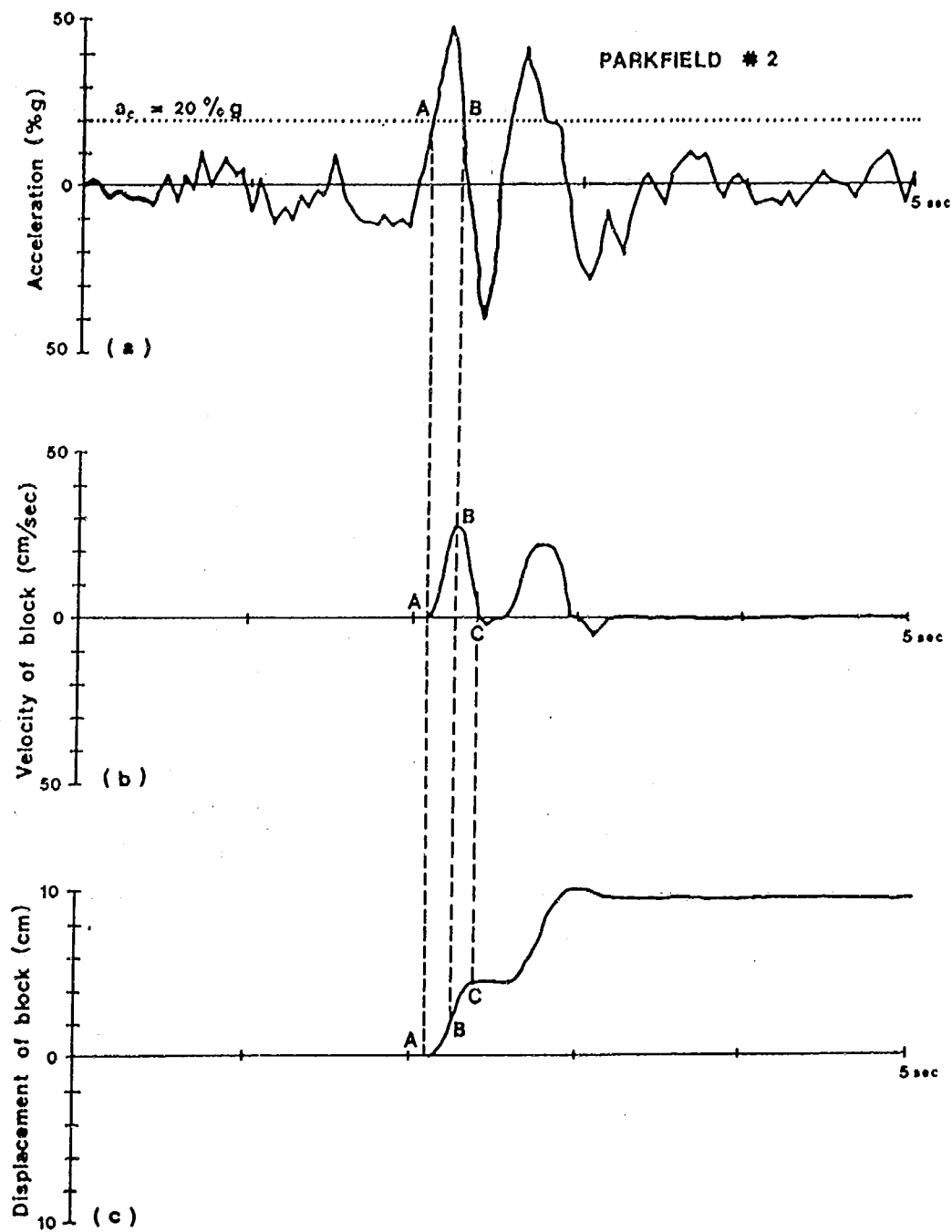


Figure 1.1 Demonstration of the Newmark-analysis algorithm:
 (a) strong-motion record with critical acceleration
 (dotted line) superimposed; (b) velocity of block versus
 time; and (c) displacement of block versus time.
 (from Wilson and Keefer, 1983).

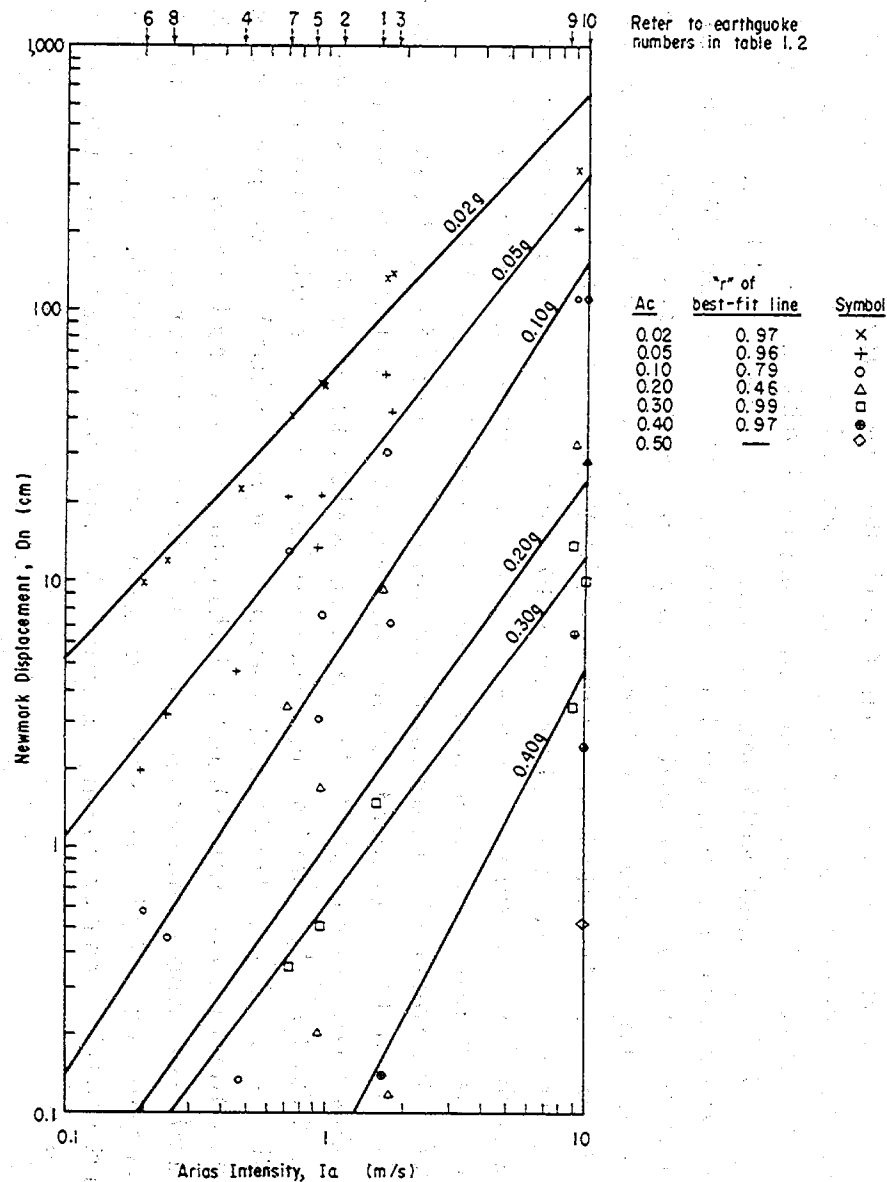


Figure 1.2 Best-fit lines of critical acceleration as a function of Newmark Displacement and Arias Intensity

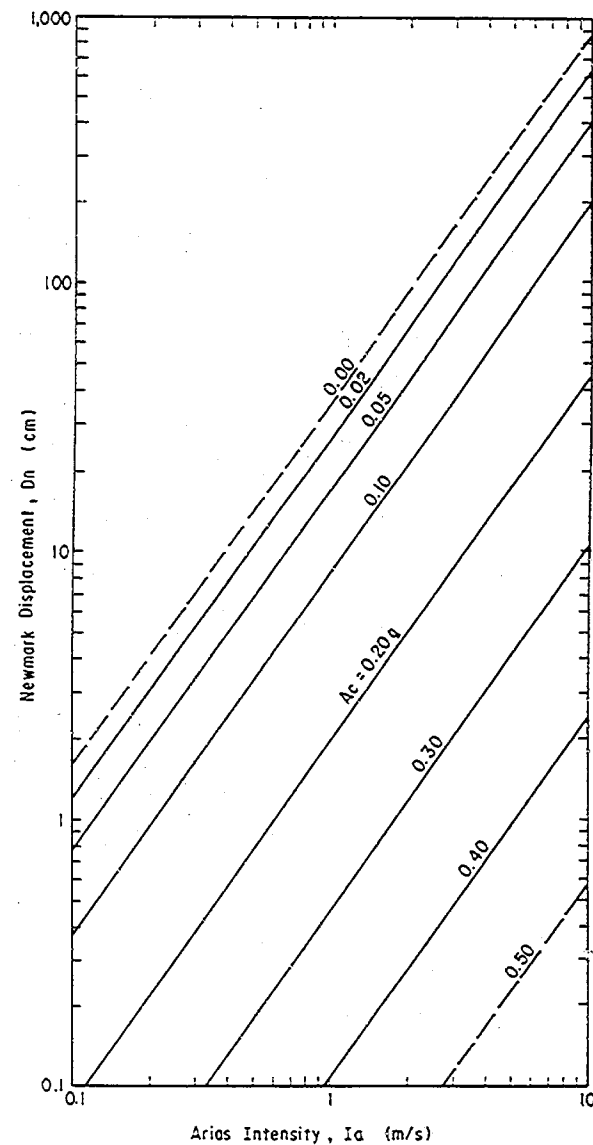


Figure 1.3 Lines of critical acceleration predicted by equation 1.5.

PART 2

Description and Analysis of Preliminary Results of Seismic Site-Response Experiment Following the March 3, 1985 Central Chile Earthquake

2.1 INTRODUCTION

Following the 3 March 1985 Central Chile earthquake ($M_s = 7.8$), personnel from the United States Geological Survey (USGS) deployed seismic instruments in the Canal Beagle subdivision of Vina del Mar, about 10 km east of Valparaiso (figure 2.1). The instruments were deployed along ridge crests and intervening valley bottoms to observe possible effects of topographic amplification of seismic shaking during aftershocks of the main earthquake (Celebi, 1986, in press)¹⁰.

Earthquake motions were recorded by eight portable General Earthquake Observation Systems (GEOS), which used two sets of three-component sensors consisting of force-balance accelerometers and velocity transducers. The only results available at present are from the velocity recordings (Celebi, 1986)¹¹. Ground motions were recorded at several sites in Canal Beagle (figure 2.2), which rests on Pleistocene sediments and decomposed granite. Ground motions at station CBA (figure 2.2), located in a valley bottom in Canal Beagle, were first compared with those recorded at a flat, hard-rock site (VAL in figure 2.1) near Valparaiso to determine the amplification effects owing to geology. All ground motions recorded at other stations in Canal Beagle were referenced to the motions at station CBA, and any differences in ground-motion response were attributed to the effects of topography. Stations CBB and CBC were located on one ridge (ridge B-C), stations CBE and CBF were located on a second ridge (ridge E-F), and station CBD was located on the hilltop from which both ridges emerge. All instruments were located on the ground floors of 1-5 story structures (table 2.1).

All graphical representations of amplification ratios contained in the present paper drawn by visually averaging the amplification ratios at least two seismic events as shown by Celebi (1986)¹¹. With one exception, ratios plotted for the same stations from different events showed similar responses, so the averaging process should introduce minimal error into the description and analysis of the test results. The single exception is station CBD, which showed responses having considerable variation; results for station CBD are thus uncertain.

2.2 COMPARISONS OF SEISMIC RESPONSES

Figure 2.3 shows the amplification ratio of station CBA relative to the flat, hard-rock site at station VAL. Celebi (1986)¹¹⁾ assumed that all variations in ground-motion response between these stations are due to geological conditions because the seismic source parameters and source distances for the two stations are about the same. Figure 2.3 indicates that the geological conditions at Canal Beagle cause amplification of frequencies from about 4-8 Hz and 9-10 Hz; no amplification or some deamplification occurs below 4 Hz and from 8-9 Hz.

Figure 2.4-2.8 show the responses of the two horizontal components of ground motion at each of the five stations located on ridges; all plots show the amplification ratio of the named station to station CBA. Figures 2.9 and 2.10 show the responses of all stations for the E-W and N-S components, respectively. All the responses are somewhat similar and display two major amplification peaks, one around 4 Hz, the other around 8 Hz. Table 2.2 summarizes the characteristics of these two dominant response peaks in the data, hereafter referred to as the low-frequency peak and the high-frequency peak. Table 2.2 lists the upper and lower frequency limits of the peaks (defined as the points between which the amplification ratio exceeds 2.0), the widths and center points of the peaks, the frequencies at which the greatest amplification occurs, and the peak amplification ratios.

Despite the overall similarity in the responses of all stations, some differences, mostly minor, are present. The following sections compare the spectral responses from the Canal Beagle experiment with respect to the following parameters: (1) north-south (N-S) versus east-west (E-W) components of ground motion at each site; (2) response on ridge B-C versus ridge E-F (figure 2.2); (3) response of station CBB versus CBC, and of station CBE versus CBF; (4) overall responses of stations CBB, CBC, CBD, CBE, and CBF; and (5) characteristics of the low-frequency response peaks compared to those of the high-frequency response peaks.

2.2.1 North-South Versus East-West Components

The ridges at Canal Beagle are aligned nearly east-west, so we might expect to see a systematic difference in the seismic responses between the two horizontal components of ground motion. Table 2.2 shows that few differences exist between the responses of the two horizontal components of ground motion. The following observations can be made:

1. For stations CBB and CBC, the low-frequency peak appears to be sharper and more well defined for the E-W component than for the N-S component, but this does not hold true for the other stations.

2. The center points of the response peaks for the E-W components at all stations are located at slightly lower frequencies than those for the N-S components. The offset is no greater than 0.8 Hz but is systematic throughout the data.

3. At the high-frequency peak, the amplification ratios for the N-S components are all greater than for the E-W components. At the low-frequency peak, variations in amplification ratios are inconsistent.

The systematic differences in the locations (in the frequency domain) and amplification ratios between N-S and E-W components suggests that these two components may respond somewhat differently to different aspects of the ridge geometries. However, the differences are small enough that little error is introduced by combining the horizontal ground-motion response into a single, average response (figure 2.11).

2.2.2 Ridge B-C Versus Ridge E-F

Two observations can be made regarding the response differences between ridge B-C and ridge E-F. First, the low-frequency peak of ridge B-C is located at a higher frequency than that of ridge E-F; the difference ranges from 0.2 to 1.6 Hz. Second, the low-frequency response peaks for the stations on ridge E-F are wider and not as well defined as those for ridge B-C. This may be due to geometrical differences between the ridges, as discussed subsequently.

2.2.3 Station CBB Versus Station CBC

Comparison of stations CBB and CBC will give insight into the variations of ground shaking on different portions of the same ridge. Station CBC is located along the ridge crest about halfway from the top to the base of the ridge; station CBB is located on the ridge crest about midway between station CBC and the base of the ridge. The following observations can be made:

1. Station CBC has much higher amplification ratios (15-24) than station CBB (8-9) at the low-frequency peak. At the high-frequency peak, amplification ratios are comparable.

2. At the low-frequency peak, station CBC has slightly broader response centered at slightly lower frequency than station CBB. At the high-frequency peak, the responses are similar, though that of CBC is very slightly broader.

3. Overall, the amplification of station CBC is generally greater than that of CBB. This observation is similar to that for the PWRI model tests that indicate that amplification is greatest near the midpoints of ridge crests (see Part 3).

2.2.4 Station CBE Versus Station CBF

Station CBE is located about where ridge E-F emerges from the main hill and is relatively higher on the ridge crest than is station CBC on ridge B-C. Station CBF is located farther down the ridge crest, downslope from the midpoint, at a point relatively higher on the crest than is station CBB on ridge B-C. The following observations can be made:

1. Both high- and low-frequency response peaks for station CBF are broader than those for CBE, but peaks for both stations are centered at the same frequencies.

2. Amplification ratios for station CBF at both high- and low-frequency response peaks are significantly greater than those for CBE: ratios of 13-16 versus 3-6 for the low-frequency peak, and ratios of 8-12 versus 3-5 for the high-frequency peak. This may occur because station CBF is closer to the midpoint of the ridge line than is station CBE, and because station CBE is located on the emerging portion of the ridge and is therefore more firmly anchored or buttressed to the main hill than is CBF.

2.2.5 Comparison of All Stations

If the two ridges are assumed to have identical ideal responses, then it is possible to compare all stations as if they were aligned progressively down a single ridge crest. In this case, station CBD would be at the top of the crest line, and, moving progressively downslope, would be stations CBE, CBC, CBF, and CBB. The ridges at Canal Beagle differ from those modeled at PWRI (see Part 3) in that they do not have a uniformly sloping crest line, but rather have relatively gently sloping upper surfaces and a steeply sloping nose or ridge front. Stations CBB and CBF are located on the noses or fronts of their respective ridges and therefore may experience similar amplifications at a level greater than might be expected from the model tests conducted on ridges having uniformly sloping crest lines.

The average horizontal responses of all stations (figure 2.11) show that at the high-frequency peak, stations CBB, CBC, and CBF all have peaks centered at the same frequency and having about the same amplitude. Each of these three stations is located either near the midpoint of the ridge crest or on the nose or front of the ridge, where amplification is expected to be

greatest. Stations CBD and CBE have significantly lower amplitude responses, and, perhaps surprisingly, the response of CBE is lower than that of CBD. This may be because station CBD is actually located on the midpoint of the much larger ridge from which the smaller ridges emerge (this ridge peaks at the closed 150-m contour and slopes downward to the northwest), so CBD may be responding to the effects of the larger ridge, rather than responding as if it were at the peak of one of the smaller ridges.

At the low-frequency peak, the responses are less uniform. Stations CBC and CBF have peak responses of similar amplitude, but they are centered at different frequencies (3.7 and 2.3 Hz, respectively). Stations CBB and CBC have a similar peak response, but the other stations appear to have either a very broad peak response or, more likely, two closely spaced peak responses at about 2 and 4 Hz. This significant difference in the nature of the low-frequency response may relate to differences in ridge geometries, as discussed subsequently. As with the high-frequency peaks, station CBE has the lowest amplitude response, CBD the next lowest.

Comparison of all stations indicates the following:

1. The stations distant from the point where the ridges connect to the main hill experience the greatest amplification, whether they be at the midpoint of the crest line or on the exposed fronts of the ridges.
2. Station CBD may be responding to the effects of the geometry of the larger ridge on which it is located.
3. Station CBE, located where ridge E-F connects with the main hill, had the lowest amplification ratios in all cases.

2.2.6 Low-Frequency Versus High-Frequency Peak Responses

The most notable difference between the high- and low-frequency response peaks is that the high-frequency peaks are invariably narrower and more well defined than the low-frequency peaks. This holds true for all stations; the mean width of the high-frequency peaks is 1.6 Hz, and the mean width of the low-frequency peaks is 3.0 Hz, almost twice as wide. The peak amplifications, however, are very similar for the two peaks; the mean peak amplification ratio for the high-frequency response is about 9, and for the low-frequency response it is about 11. The high-frequency peaks are all centered at about the same frequency, 8.3 Hz, and have a standard deviation about this mean of only 0.23. The low-frequency peaks are not centered so uniformly: the mean center frequency is 3.2 Hz, and the standard

deviation is 0.42, nearly twice as great as for high-frequency peaks.

Some of the low-frequency peaks, particularly those from stations CBD, CBE, and CBF, may actually contain two peaks, one at about 2.5 Hz and another at about 4.0 to 4.5 Hz. The broad, double-peaked shape of these responses, with a significant low point between peaks, suggests the possibility of two closely spaced but distinct peaks. This could account for the relatively broader, less consistently centered nature of the low-frequency response for these stations.

2.3 ANALYSIS OF PEAK AMPLIFICATION

Several researchers have modeled or documented topographic amplification of seismic waves (Boore, 1972¹²), 1973¹³); Bouchon, 1973¹⁴); Davis and West, 1973¹⁵); Rogers and Others, 1974¹⁶); Wong and Jennings, 1975¹⁷); Griffiths and Bollinger, 1979¹⁸); Brune, 1984¹⁹); Sasaki and Kuwabara, 1986²⁰), but none have been able to provide quantitative conclusions regarding how to predict which frequencies will be either amplified or deamplified. Davis and West (1973)¹⁵) stated that "the amount of amplification and periods at which it occurs vary with the size of the mountain and are probably a function of the relationship between the wavelengths of the incoming signal and the dimensions of the mountain." A similar observation was made by Bouchon (1973)¹³), who stated that "the effect of topography on surface motion appears to be very important when the wavelength is of the order of dimension of the anomaly..." Brune (1984)¹⁹) reported on model tests of seismic amplification and showed that amplification is maximum at ridge crests, for vertically incident shear waves, and that it varies according to the ratio of the incoming wavelength and the width of the ridge. Sasaki and Kuwabara (1986)²⁰) introduced a factor "I" to predict the resonant period of ridges:

$$I = HL / W(V_s), \quad (2.1)$$

where H , L , and W are the ridge height, length, and width, respectively, and V_s is the shear-wave velocity of the ridge material. They postulated that this factor I is about equal to the fundamental resonance period of ridges; any relationship of I to higher resonance modes is unclear.

Figure 2.12 shows a topographic map of the Canal Beagle area, and geometric constructions as recommended by Sasaki and Kuwabara (1986)²⁰) to determine the dimensions of the ridges are

shown. Dimensions are recorded in table 2.3, along with values of factor I for each ridge, for a shear-wave velocity of 700m/s for the Pleistocene sediments composing the ridges at Canal Beagle. For ridge B-C, $I = 0.29$ s, which corresponds to a resonance frequency of 3.5 Hz; for ridge E-F, $I = 0.25$ s, which corresponds to a resonance frequency of 4.1 Hz. Figure 2.11 and table 2.2 indicate that stations CBB and CBC, located on ridge B-C, have low-frequency amplification peaks centered at 3.6 Hz, which corresponds closely to the value predicted by factor I. Stations CBE and CBF, located on ridge E-F have low-frequency peaks centered near 3.0 Hz, but as explained above, this broad low-frequency peak may in reality be two closely spaced but distinct peaks (see figure 2.11). If this is the case, the higher frequency of these two minor peaks is centered near 4.1 Hz, again the frequency predicted by factor I. These results suggest that the fundamental resonance frequencies of ridges B-C and E-F are 3.5 and 4.1 Hz, respectively.

The other peak responses cannot easily be explained by factor I, which is an aggregate measure of the total ridge geometry and accounts for all the dimensions of the ridges simultaneously. If we examine single dimensions of the ridges individually as they relate to the incident seismic wavelength, however, we may be able to better understand the locations of the other response peaks. Bouchon (1973)²⁰⁾ and Davis and West (1973)¹⁵⁾ suggested that amplification probably occurs at wavelengths corresponding to the widths, heights, or lengths of topographic features. This hypothesis can be tested by setting the incident shear-wave length equal to each of these three dimensions for the two ridges and using the simple relationship

$$f = V_s / \lambda \quad (2.2)$$

where f is the frequency, V_s is the shear-wave velocity, and λ is the wavelength. The computed frequency can then be located on figure 2.11 to determine if it coincides with an amplification peak. Assuming a shear-wave velocity of 700 m/s for the Pleistocene sediments, frequencies corresponding to wavelengths equal to the ridge dimensions are shown in table 2.4. For ridge B-C, the height corresponds to a frequency of 8.2 Hz, very near the center of the high-frequency peaks of stations CBB and CBC (figure 2.11, table 2.2). The width of ridge B-C corresponds to a frequency of 3.5 Hz, which coincides with the centers of the low-frequency peaks for stations CBB and CBC; this frequency also coincides with the resonance frequency of ridge as estimated by

I. For ridge E-F, the height corresponds to a frequency of 7.8 Hz, close to the center of the high-frequency peaks of stations CBE and CBF; the ridge width corresponds to a frequency of 2.0 Hz, near the centers of the lower of the two low-frequency peaks of CBE and CBF.

These results appear to explain the difference in the nature of the low-frequency peaks of ridges B-C and E-F. Stations CBB and CBC have relatively narrow, well defined low-frequency response peaks, presumably because the overall resonance frequency of the ridge, predicted by factor I, coincides with the frequency of the wavelength-matching effect of the ridge width. Stations CBE and CBF have broader, more diffuse low-frequency peaks that appear to be the merging of two closely spaced but distinct peaks. This is explained by the overall resonance frequency from factor I, 4.1 Hz, being significantly different from the frequency of the matching wavelength of the ridge width, 2.0 Hz. These two frequencies correspond to the two minor peaks within the broad low-frequency peak for these two stations.

The frequencies corresponding to the lengths of both ridges do not coincide with amplification peaks from any of the stations (figure 2.11, table 2.2). This may be because the data are highly variable and not well constrained at such low frequencies for this experiment, or it could indicate that the long dimensions of topographic features are not as greatly affected by wavelength-matching as are the two shorter dimensions.

2.4 CONCLUSIONS

All of the stations show approximately similar amplification responses in that they have two frequency ranges where the amplification ratios exceed 2.0, one peak being centered around 3-4 Hz, the other around 8-9 Hz. Many of the minor differences discussed herein may be insignificant or at least unexplainable at present. The most significant difference in the amplification responses is that stations on ridge E-F have a broad, two-peaked response in the 3-4 Hz range, whereas stations on ridge B-C have narrower peaks in that range. The effects of resonance of the entire ridges, having frequencies predicted by factor I of Sasaki and Kuwabara (1986)²⁰, together with wavelength matching at frequencies corresponding to the heights and widths of the ridges, appear to explain the amplification responses of stations on both ridges.

The greatest amplification ratios were recorded at stations located near the midpoints of the ridge crest lines and on the exposed fronts of ridges. Amplification ratios exceeding 20 were

computed for some stations at some frequencies, and peak ratios exceeding 10 were common. Results of this experiment reported by Celebi (1986)¹¹⁾ show that ground motions on topographic ridges can be amplified by a large amount in frequency ranges related to the ridge geometry.

Table 2.1 Canal Beagle Experiment Stations (from Celebi, 1986).

Station	Description
<u>Site Stations:</u>	
CBA	At ground floor (no basement) of Type B* structure founded on a clear cut at canyon between two ridges. The structure and its twin next to it were not damaged. Geologic formation: sedimentary. (Building #1)
CBB	At ground floor of a Type B* structure on top of ridge where there is extensive damage. Geologic formation: Sedimentary and decomposed granite. (Building #7)
CBC	(Same as above, CBB.) (Building #12)
CRD	At ground floor of a Type A* (single story) structure. This part of Canal Beagle is on the main body of the hill crowned by the ridges. (699 Ventisquero Street - at corner of Canal Kivke Street)
CBE	At ground floor (no basement) of Type C* (all 5 stories) structure located on top of emerging portion of the ridge. (Building #4 - Edificio Thomson)
CBF	At ground floor (no basement) of Type C* (all 5 stories) structure located at the top of the ridge. (Building #15 - Edificio Hyatt)

Reference Stations:

- VAL, On a concrete pedestal at the University of Santa Maria. Same location as the SMA station of the Chilean strong motion network. The site is amphibolite and granite gneiss formation.

* There are three types of structures in Canal Beagle. Type A structures are single and two-story buildings on top of the hill, whereas Type B structures are four-story buildings on one ridge and Type C structures are five-story buildings on another ridge. Two of the Type B structures are in a canyon at the entrance to the subdivision.

Table 2.2 Characteristics of amplification peaks.

LOW-FREQUENCY PEAK							HIGH-FREQUENCY PEAK						
STATION/ COMP.	LOWER LIMIT (Hz)	UPPER LIMIT (Hz)	WIDTH (Hz)	CENTER POINT (Hz)	PEAK POINT (Hz)	AMPLIFICATION RATIO	STATION/ COMP.	LOWER LIMIT (Hz)	UPPER LIMIT (Hz)	WIDTH (Hz)	CENTER POINT (Hz)	PEAK POINT (Hz)	AMPLIFICATION RATIO
CBB/E-W	2.9	4.4	1.5	3.7	3.8	9x	CBB/E-W	7.5	8.8	1.3	8.2	8.2	8x
CBB/N-S	2.3	5.5	3.2	3.9	4.1	9x	CBB/N-S	7.4	9.1	1.7	8.3	8.3	18x
CBB/AVE	2.5	4.8	2.3	3.7	3.9	8x	CBB/AVE	7.4	9.1	1.7	8.3	8.3	11x
CBC/E-W	1.9	4.6	2.7	3.3	3.7	24x	CBC/E-W	6.8	9.0	2.2	7.9	8.2	10x
CBC/N-S	1.8	4.8	3.0	3.3	4.3	15x	CBC/N-S	7.4	9.3	1.9	8.4	8.2	15x
CBC/AVE	2.0	4.8	2.8	3.4	3.7	15x	CBC/AVE	7.2	9.1	1.9	8.2	8.2	11x
CBD/E-W	1.3	4.1	2.8	2.7	3.4	6x	CBD/E-W	7.7	8.8	1.1	8.3	8.2	6x
CBD/N-S	1.4	5.1	3.7	3.3	2.4	10x	CBD/N-S	7.8	9.5	1.7	8.7	8.4	8x
CBD/AVE	1.4	4.3	2.9	2.9	1.9	6x	CBD/AVE	7.8	9.6	1.8	8.7	8.4	5x
CBE/E-W	1.3	4.1	2.8	2.7	2.3	4x	CBE/E-W	7.7	8.3	0.6	8.0	8.1	3x
CBE/N-S	2.0	4.6	2.6	3.3	4.4	6x	CBE/N-S	7.6	9.1	1.5	8.4	8.4	5x
CBE/AVE	1.9	4.5	2.6	3.2	2.6	3x	CBE/AVE	7.6	8.9	1.3	8.3	8.0	4x
CBF/E-W	1.0	4.4	3.4	2.7	2.2	16x	CBF/E-W	7.2	9.0	1.8	8.1	7.8	8x
CBF/N-S	1.3	5.6	4.3	3.5	2.4	14x	CBF/N-S	7.5	9.2	1.7	8.4	8.3	12x
CBF/AVE	1.3	4.6	3.3	3.0	2.3	13x	CBF/AVE	7.4	9.1	1.7	8.3	7.9	9x

Table 2.3 Ridge dimensions and "I" values (shear-wave velocity 700 m/s)

Ridge	Height (m)	Width (m)	Length (m)	Factor I (sec)	Frequency (Hz)
B-C	85	200	475	0.29	3.5
E-F	90	345	660	0.25	4.1

Table 2.4 Frequencies corresponding to wavelengths equal to ridge dimensions (shear-wave velocity 700 m/s)

Ridge	Dimension	Value (m)	Frequency (Hz)
Ridge B-C	Height	85	8.2
	Width	200	3.5
	Length	475	1.5
Ridge E-F	Height	90	7.8
	Width	345	2.0
	Length	660	1.1

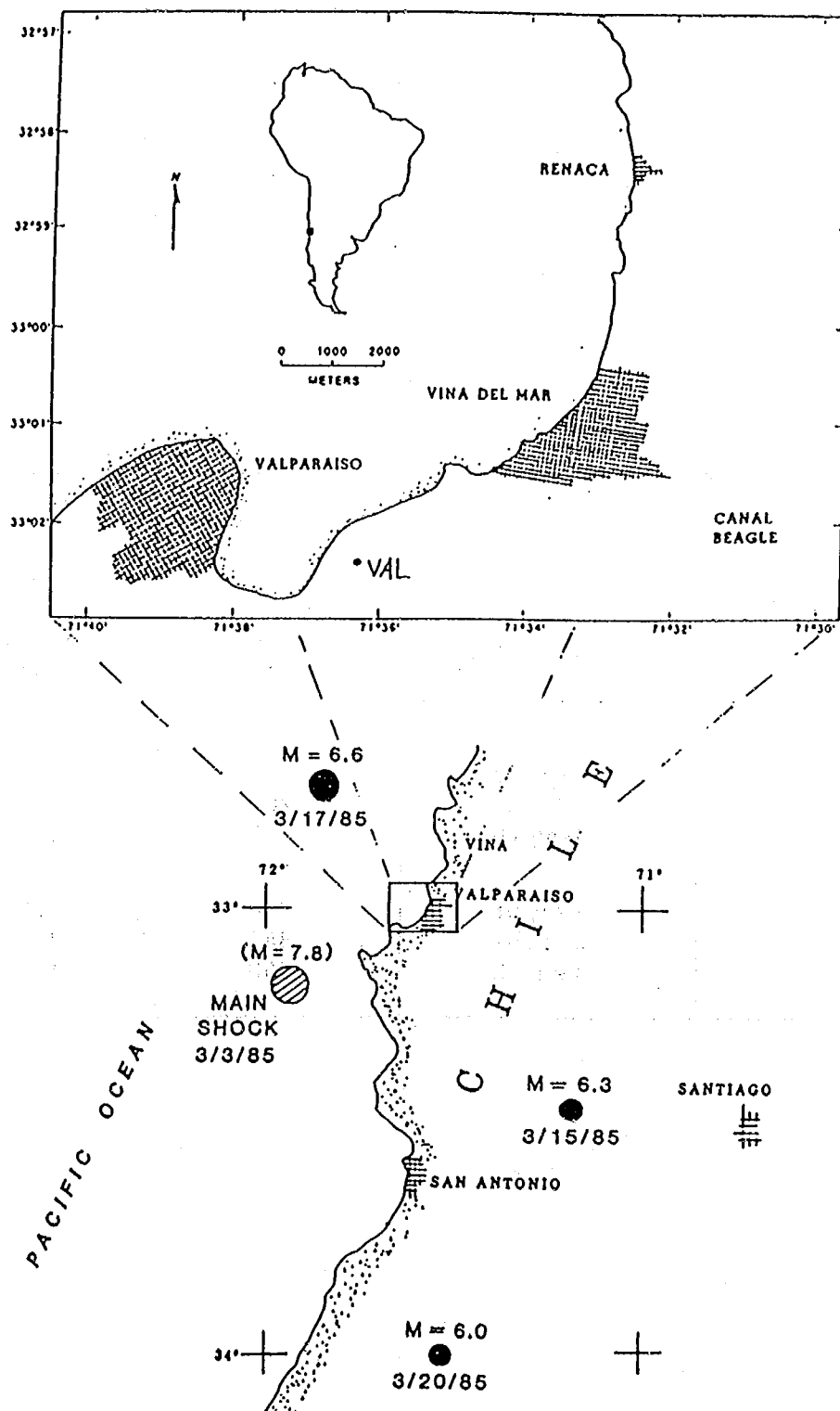


Figure 2.1 Location map showing main shock and largest aftershocks. (adapted from Celebi, 1986).

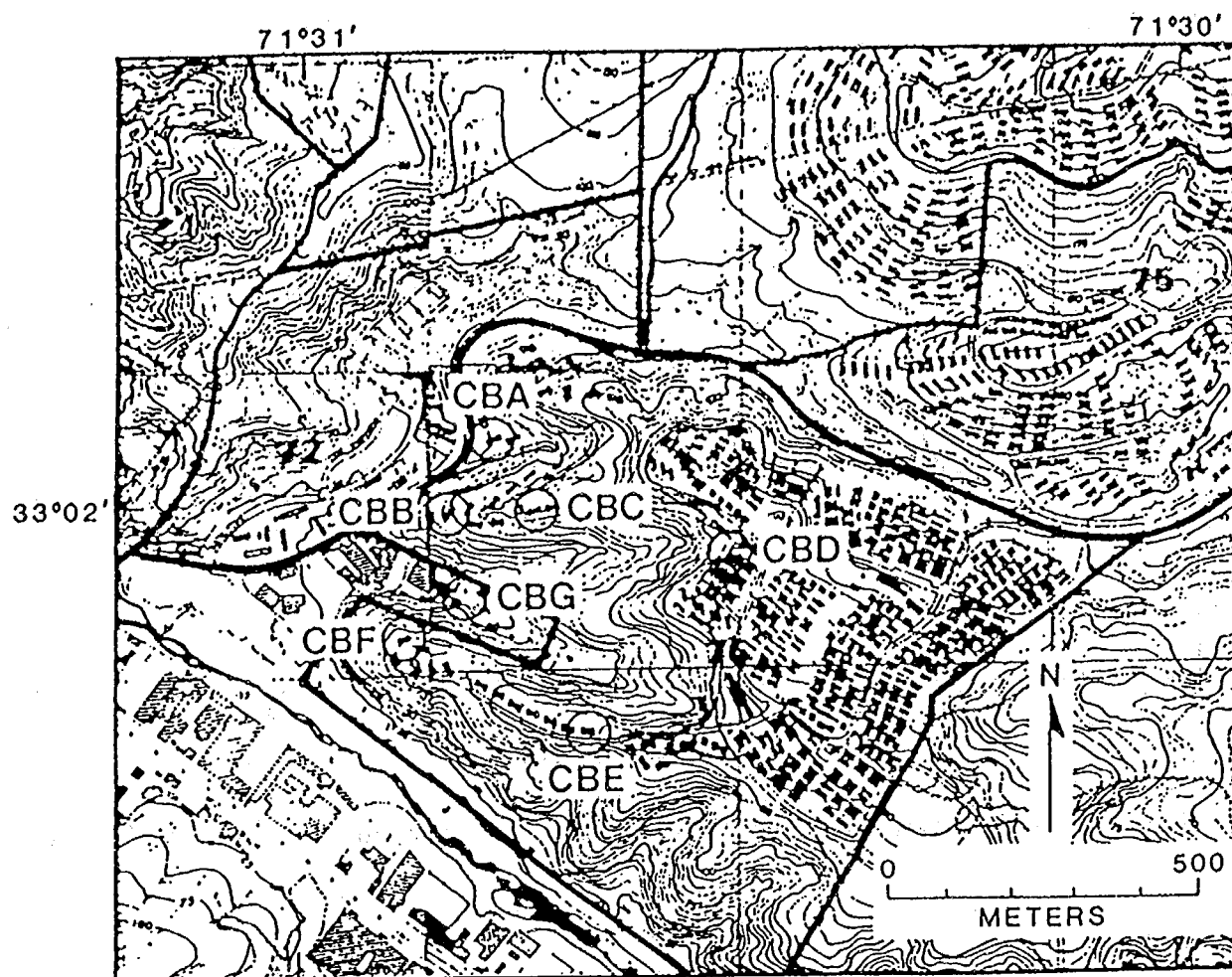


Figure 2.2 Detailed topography of Canal Beagle. The stations of the Canal Beagle site are indicated. Also a general scale and the latitudes and longitudes are shown (from Celebi, 1986).

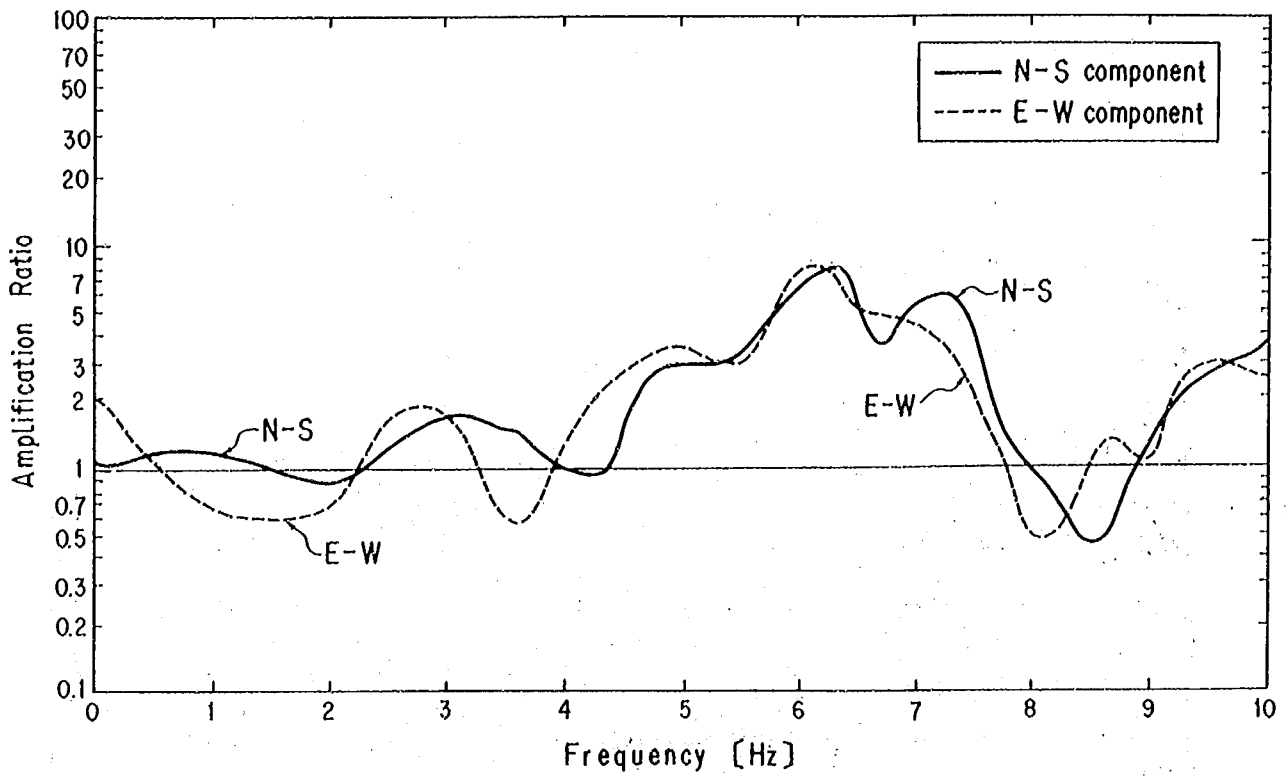


Figure 2.3 Amplification Ratio of Station CBA relative to Station VAL (CBA/VAL)

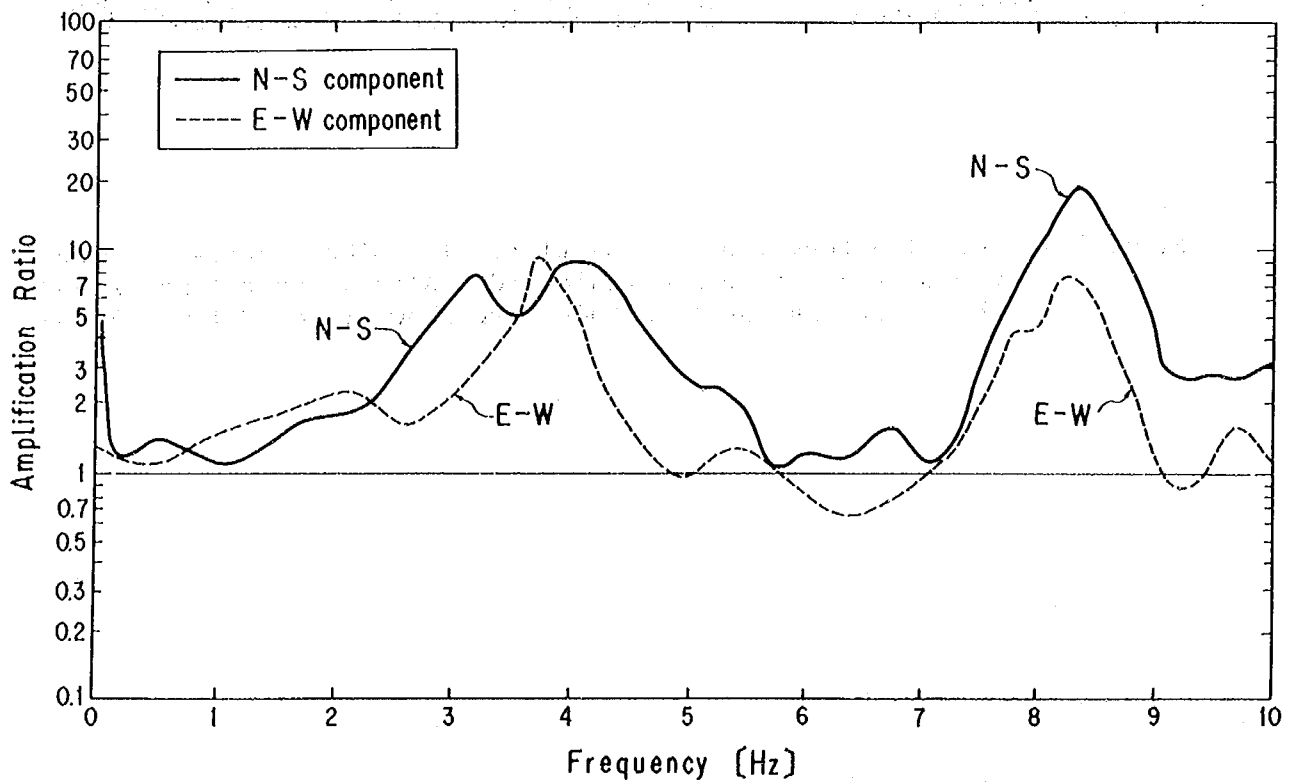


Figure 2.4 Amplification Ratio of Station CBB relative to Station CBA (CBB/CBA)

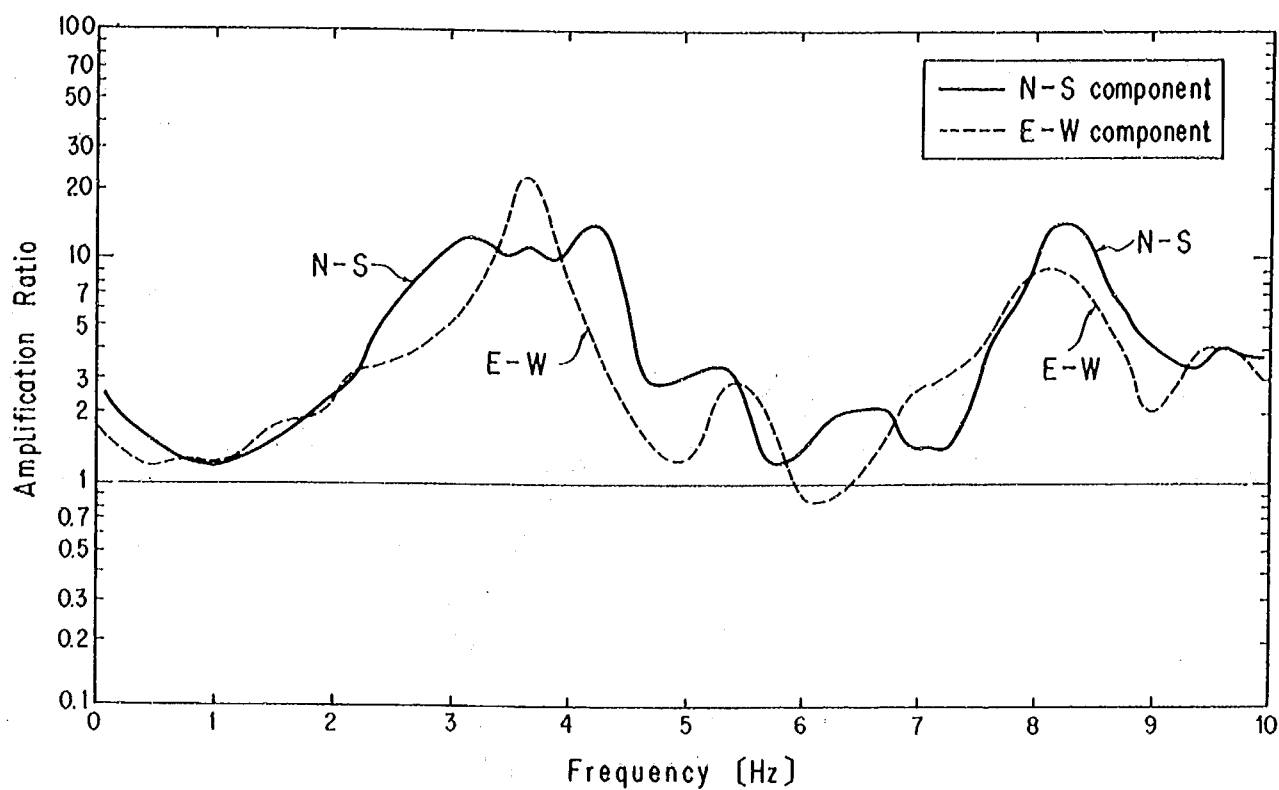


Figure 2.5 Amplification Ratio of Station CBC relative to Station CBA (CBC/CBA)

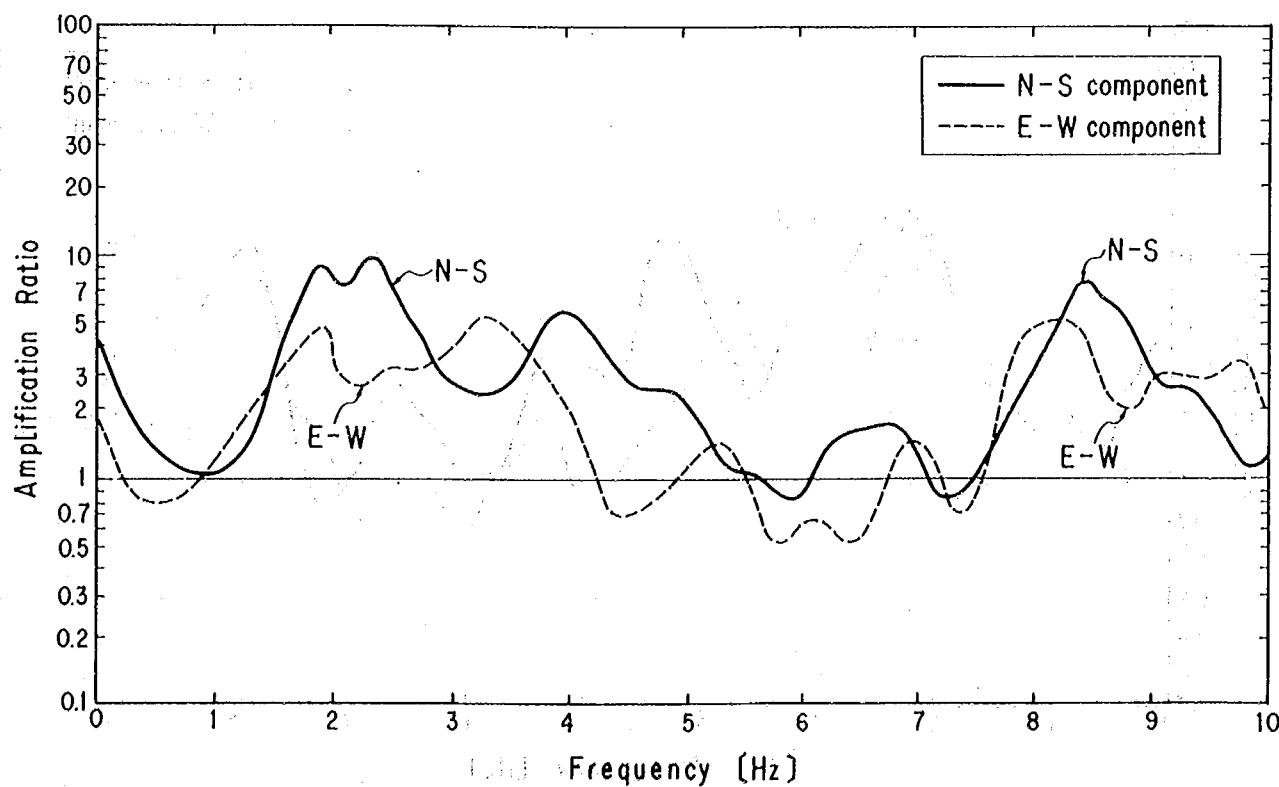


Figure 2.6 Amplification Ratio of Station CBD relative to CBA (CBD/CBA)

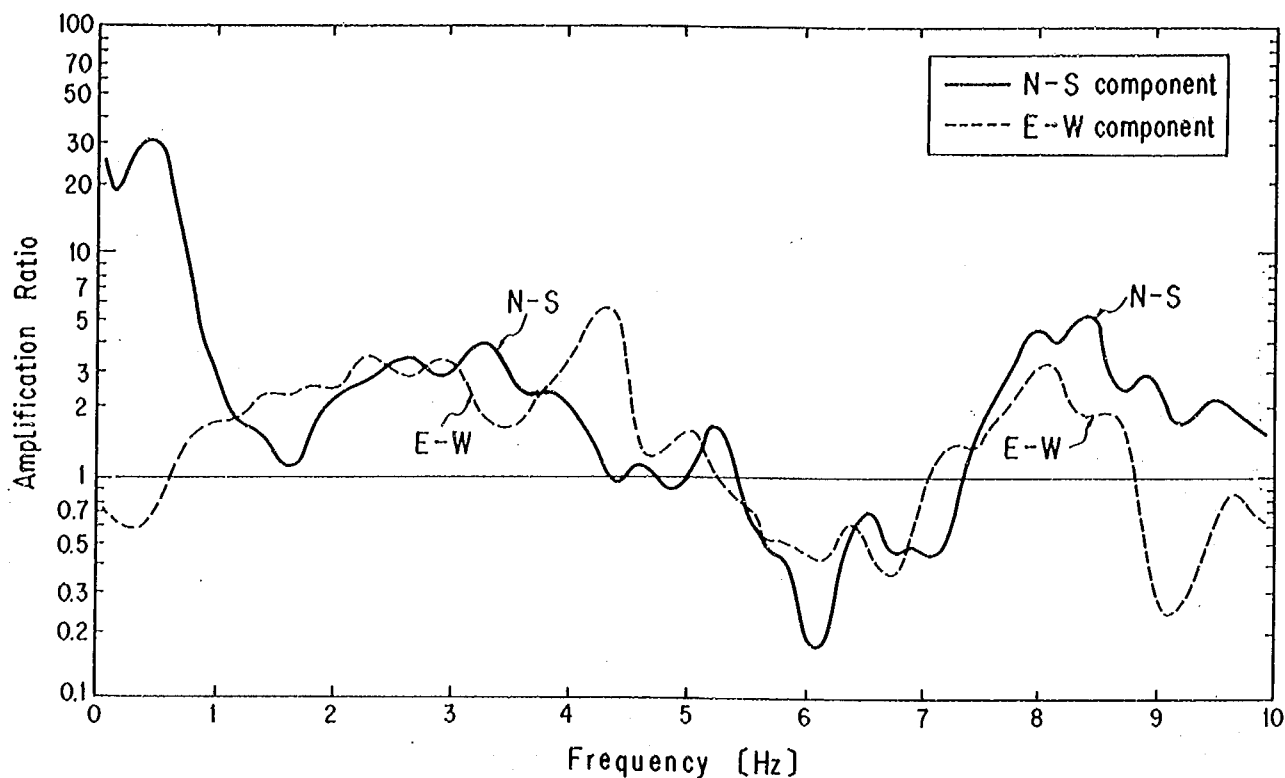


Figure 2.7 Amplification Ratio of Station CBE relative to CBA (CBE/CBA)

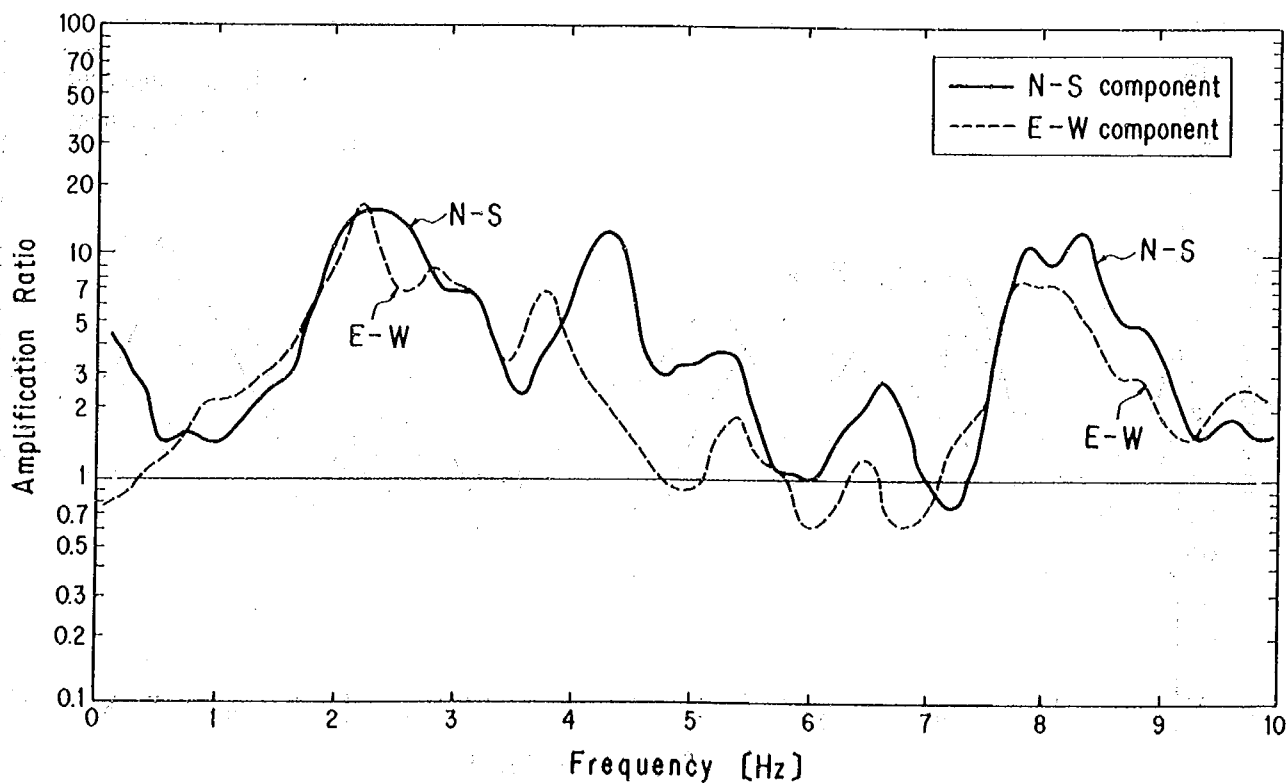


Figure 2.8 Amplification Ratio of Station CBF relative to CBA (CBF/CBA)

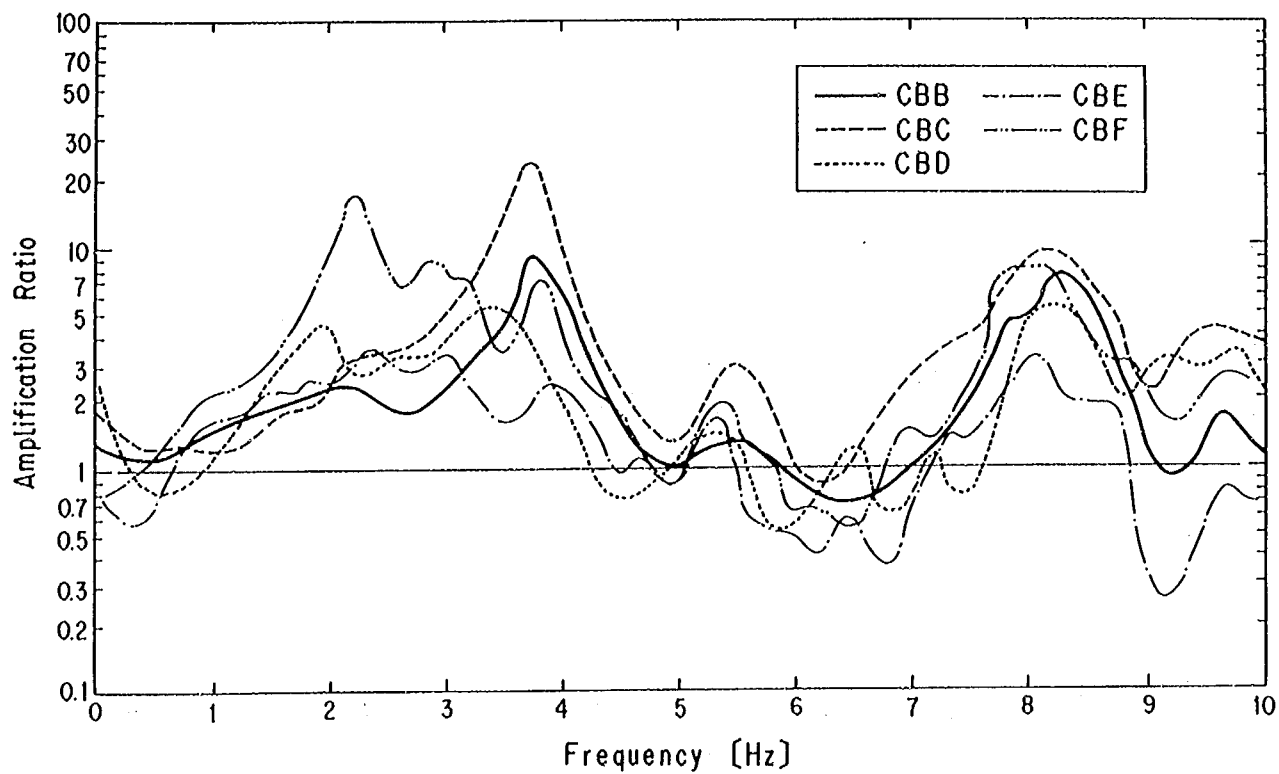


Figure 2.9 Amplification Ratio of East-West Components

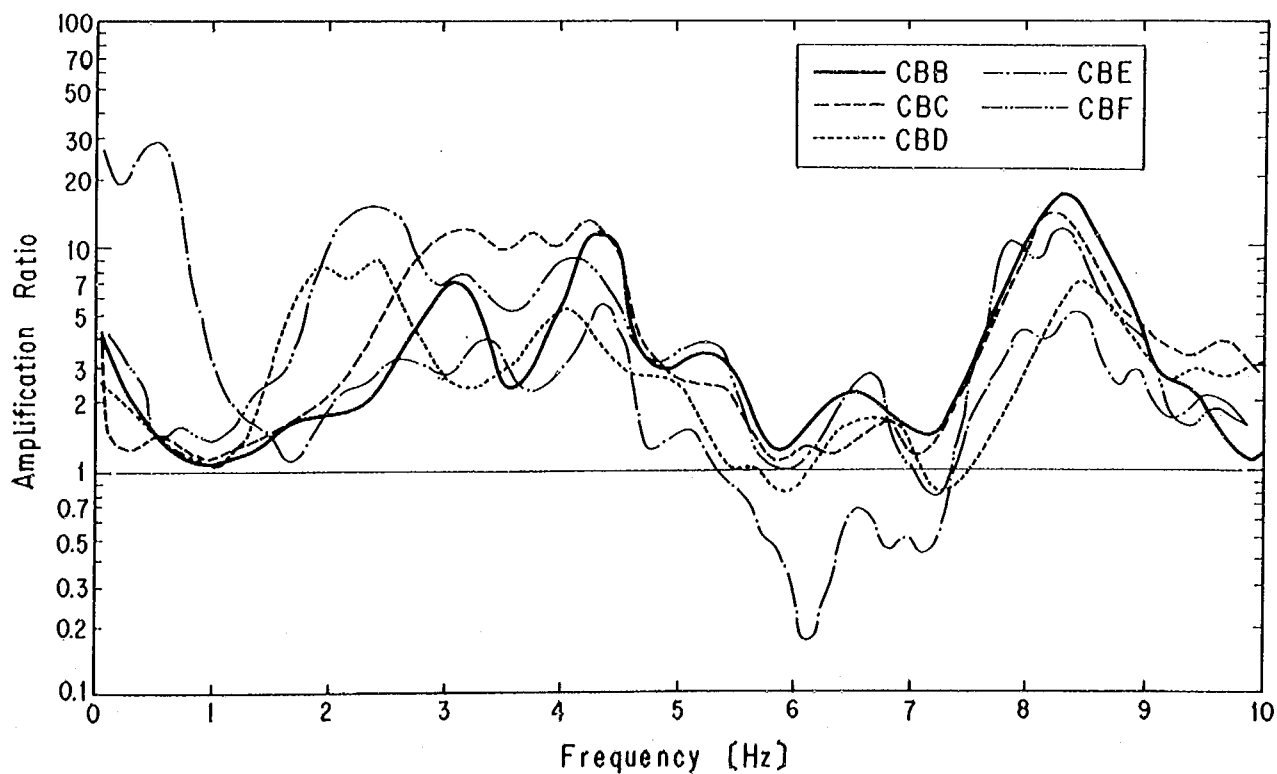


Figure 2.10 Amplification Ratio of North-South Components

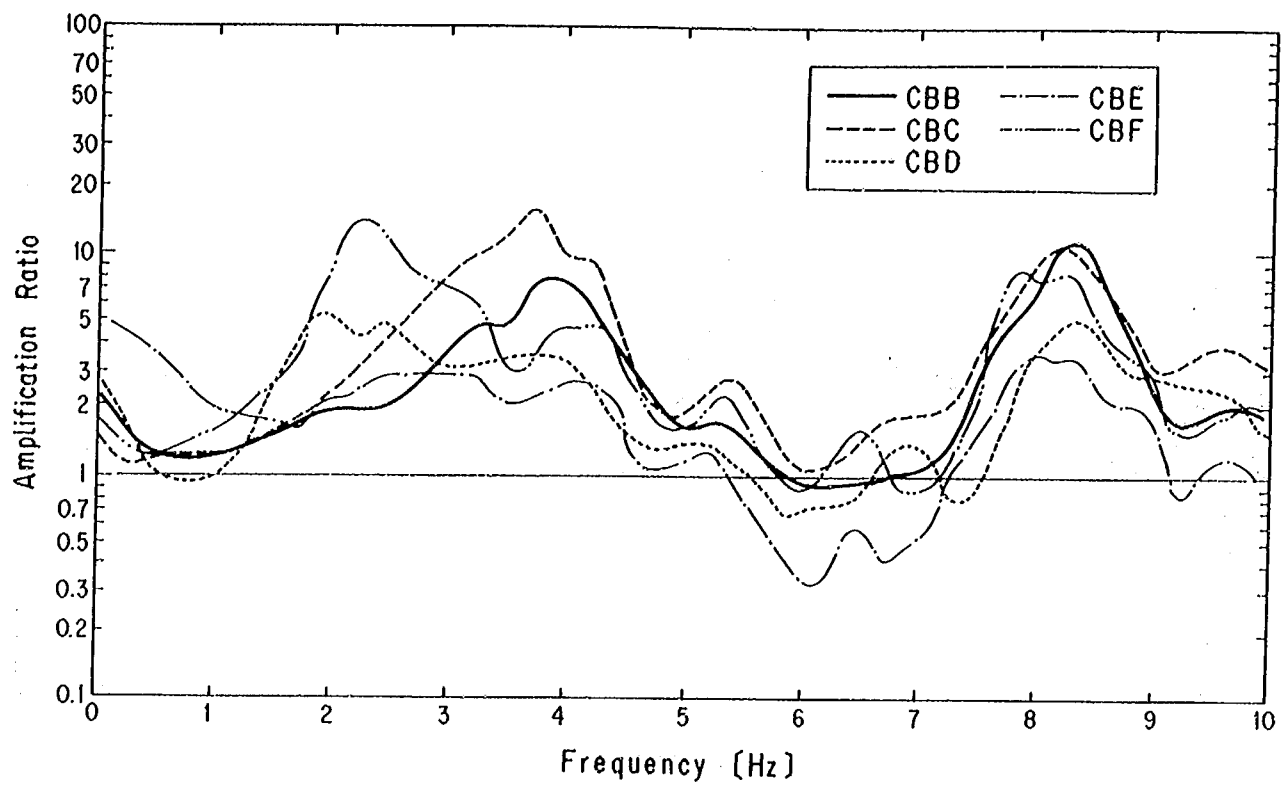


Figure 2.11 Average Amplification Ratio of Horizontal Components

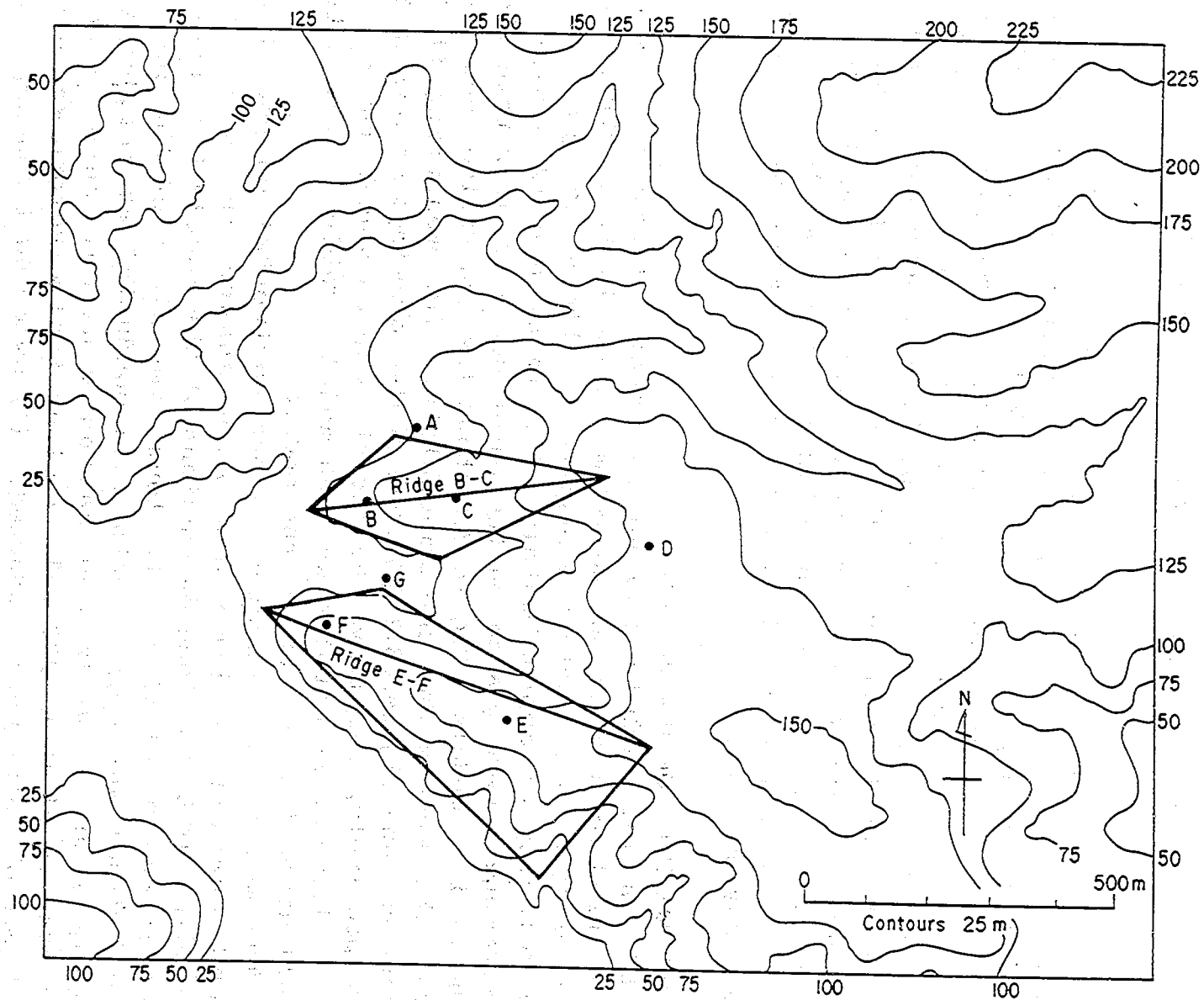


Figure 2.12 Geometric constructions to measure ridge dimensions.

PART 3

Comparison of Results of Slope Model Tests with Field Studies of Topographic Amplification of Ground Shaking

3.1 INTRODUCTION

In January and February 1987, a series of model tests were performed at the Public Works Research Institute (PWRI), Tsukuba Science City, Japan, to observe the effects of topography on the amplification of earthquake ground motion. The present paper (Part 3) compares the results of the model tests with results from the seismic site-response experiment conducted after the 1984 Central Chile earthquake (Celebi, 1986, in press; Part 2) and with data from the dense seismic array at Matsuzaki, Japan. What follows is not a rigorous analysis of the model-test results but rather a summary of those results for comparison with the field-study results.

3.2 DESCRIPTION OF MODEL TESTS

Six slope models having different geometries were manufactured for testing on the 6x8-m shaking table at PWRI. The models were constructed of transparent silicon having a shear-wave velocity of approximately 1237 mm/s, and 12 miniature accelerometers were placed on the outer surfaces or within each of the silicon models. Figure 3.1 shows the generalized geometry of all the models and the nomenclature for measuring their geometries, and figures 3.2-3.7 show scaled drawings of models 1-6, respectively, including the locations and channel numbers of the accelerometers mounted on each model. Table 3.1 lists the overall dimensions of each of the models.

The models were exposed to one horizontal component of ground shaking at time and were shaken in directions perpendicular to the long dimensions of the models (transverse shaking) and parallel to the long dimensions of the models (longitudinal shaking). Tests were conducted at frequencies ranging from 3-60 Hz and having constant zero-to-peak acceleration of about 200 gal. A sinusoidal waveform was used. Two types of tests were conducted: (1) sweeping-frequency tests where the frequency of the ground motion increased 1 Hz/s from 3-60 Hz, and (2) constant-frequency tests conducted at 0.5-Hz intervals from 3-60 Hz. Test results were summarized in the frequency domain by plotting the ratio of the recorded acceleration for each channel to the input acceleration measured on the rigid base of the shaking table.

3.3 ANALYSIS OF FREQUENCY-DOMAIN TEST RESULTS

Figures 3.8-3.13 show the frequency-domain acceleration ratios for transverse shaking recorded on all accelerometers for models 1-6, respectively. All models displayed similar responses: the fundamental resonance frequency is about 14-15 Hz, and the response peaks for this resonance mode differ by no more than 1.5 Hz between models. Second-mode peaks of smaller amplitude are less consistently located, but are generally between 20-25 Hz. Figures 3.14-3.18 show frequency-domain acceleration ratios for longitudinal shaking for models 1-5, respectively. The models all had fundamental resonance frequencies of about 16-17 Hz and secondary peaks centered between 30-35 Hz.

The similarity of the responses of model slopes having significantly different geometries indicates that the fundamental resonance frequency is controlled by one or more of the inter-related geometric parameters held constant for all models. These parameters include the maximum vertical cross-sectional height or thickness (figure 3.1, dimension T); the 45° angle of the rigid backslope to which the model is attached; and, more approximately, the slope length measured along the crestline (figure 3.1, dimension S). Thus, overall slope height, width, and horizontal length (figure 3.1, dimensions H , W , and L , respectively), the dimensions related by Sasaki and Kuwabara (1986)²⁰ (see Part 2) through a factor "I" to predict the fundamental resonance frequency, appear unrelated to the fundamental resonance frequency in the model tests. This observation is confirmed by calculating values of factor I for each of the models. Table 3.2 shows I-values calculated in several different ways for each of the models. The fundamental resonance period of the models as measured by the shaking-table tests ranges from 0.067 to 0.071 s. None of the values listed in table 3.2 are close to this range, and most differ by a large amount.

To detect any minor response differences between slope models, I compared the responses of the six models channel by channel for the case of transverse shaking. Figure 3.19-3.30 show the responses for channel 3-14, respectively, as located on model 1 (different, but analogous, channels were plotted in some cases for models 3 and 6 because transducers on these models were arranged somewhat differently). Two types of systematic response differences were investigated: (1) differences in responses between models having different crestline slope angles (figure 3.1, angle α) when the angle of the side slopes (figure 3.1,

angle β) is constant, which requires comparison of models 1, 2, 3, and 6; and (2) differences in responses between models having different side-slope angles when the crestline angle is constant, which requires comparison of models 1, 4, and 5.

To document any systematic effects of increasing crestline angle on the fundamental resonance frequency, I examine figures 3.19-3.30 and recorded those instances where the model peak responses increased or decreased in frequency consistently with increasing crestline angle (crestline angle increases in the model sequence 3-2-1-6). Observations are summarized in table 3.3. Channels 6, 12, 13, and 14 (figures 3.22, 3.28-3.30) showed slight but systematic decreases in the fundamental resonance frequency with increasing crestline angle; channels 9 and 10 (figures 3.25, 3.26) showed slight increases in the fundamental resonance frequency with increasing crestline angle. The remaining channels showed no systematic response differences. For the secondary response peaks, only channels 10 and 11 showed systematic differences: the peak frequency increased slightly with increasing crestline angle. Thus, even these minor response differences vary inconsistently.

Examination of the effects of increasing the side-slope angle (model sequence 5-1-4) shows that channels 6, 7, and 9 (figures 3.22, 3.23, 3.25) all display slight systematic decreases in the fundamental resonance frequency with increasing side-slope angle. The other channels show no systematic differences. For the secondary response peak, channels 6, 7, and 9 show decreases, and channel 4 (figure 3.20) shows an increase in the resonance frequency with increasing side-slope angle. Thus, in the majority of cases where systematic differences are present, increasing either crestline angle or side-slope angle results in very slight decreases in the fundamental and secondary resonance frequencies. It must be remembered, however, that most channels showed no systematic differences between the models, and that those differences that do exist are slight and not entirely consistent. One reason for this inconsistency is that each accelerometer is measuring the response of a different part of the resonating model.

Systematic variations in the amplification ratio were also investigated (table 3.3). For the case of increasing crestline angle, channels 9, 10, 12, 13, and 14 (figures 3.25, 3.26, 3.28-3.30) all showed systematic increases in the amplification ratio; channel 6 (figure 3.22) showed a systematic decrease in the ratio. For the secondary response peaks, channel 10 showed an increase, and channel 11 (figure 3.27) showed a decrease in

amplification ratio with increasing crestline angle. Other channels showed no systematic variations in either response peak.

For the case of increasing side-slope angle, channel 6 (figure 3.22) showed an increase and channels 7 and 9 (figures 3.23, 3.25) decreases in amplification ratio. For the secondary response peaks, channels 4, 6, 7, 9, and 11 (figures 3.20, 3.22, 3.23, 3.25, 3.27) all showed systematic increases in amplification ratio with increasing side-slope angle.

The variations in amplification ratio with changing slope geometry are of much more significant magnitude than the variations in frequency. whereas peak response frequencies vary by no more than 1.5 Hz between models, amplification ratios for a given channel location vary between models by factors as great as 2. The majority of the data suggest that increasing the crestline angle tends to increase the amplification ratio for the fundamental resonance mode; increasing the side-slope angle tends to decrease the amplification ratio for the fundamental resonance peak, but it tends to increase the ratio for the secondary response peak. The data, however, are not entirely consistent.

The most significant and consistent factor affecting the magnitude of the amplification ratio is the vertical distance from the rigid model base to the accelerometer. Figure 3.31 shows amplification ratio plotted against accelerometer height for all channels having a response peak at the fundamental resonance frequency (14-15 Hz). The solid line is the "best-fit" from a standard linear regression; it has a correlation coefficient (r) of 0.93 and is thus well fitted to the data. The amplification ratio for an accelerometer height equal to zero is 0.64, which is reasonably close to the value of 1.0 intuitively expected at the rigid base of the model. The dashed line in figure 3.31 is constrained to have an amplification ratio of 1.0 at zero accelerometer height and was visually fitted to the data. Thus, the model tests confirm theory and intuition that amplification of ground motion is maximum at the highest points of ridges. These results also agree with results from Brune (1984)¹⁹⁾ that showed a consistent increase in the amplification ratio from the base to the apex of a ridge model similar in shape to those tested at PWRI.

In most cases, for a given accelerometer height, accelerometers located along the longitudinal axis of the models displayed greater amplifications than accelerometers located along the flanks of the ridges. This phenomenon was not consistent throughout the data, however.

3.4 SPATIAL VARIATION IN ACCELERATION

One way in which topographic amplification of ground motion can lead to ground failure is by increasing the inertial forces acting to drive potential landslide blocks downslope. This effect can be examined by plotting the spatial variation in acceleration for each of the models. This section discusses the variation in acceleration ratios between (1) accelerometers located along the model ridge crests and (2) accelerometers aligned vertically along the centerlines of the maximum vertical cross-sections.

3.4.1 Variation Along Ridge Crests

For each of the model cases (1-6), I plotted the acceleration ratio for each accelerometer located on the crestline as a function of its proportional distance along the crestline from the base to the top of the ridge. Data are plotted for each frequency at which any of the accelerometers along the crestline had peak responses. Data from transverse as well as from longitudinal ground vibration are plotted as figures 3.32-3.37 and 3.38-3.42, respectively. No data from case 6 for longitudinal shaking is available. Figure 3.43 and 3.44 summarize the data for transverse and longitudinal ground shaking, respectively, by showing acceleration-ratio plots for the three major response peaks (peak 1, peak 2, peak 3) for each of the six cases. The frequency ranges of these peaks is indicated on the figures.

Figures 3.43 shows that the peak-1 responses of each of the models subjected to transverse ground shaking are essentially identical: a single acceleration-ratio maximum of 5-7 centered between 0.4 and 0.6 (of the way from the base to the top of the crestline). This shows that the maximum acceleration in the fundamental resonance mode is at the center of the crestline. Examining cases 3-2-1-6 shows the effects of increasing the crestline angle, which appears to cause the acceleration maximum to be centered successively lower along the crestline. Increasing the side slope angle (cases 5-1-4) has no discernible effect.

The peak-2 responses are less consistent. Cases 3 and 6, at the extremes of the range of crestline angle, show a distinctive two-peaked response centered near the one-third points along the crestline. Case 2 has a broad, single-peaked response centered at 0.5. Cases 1, 4, and 5 have responses intermediate between those previously described, but are similar to each other in that the peak response is at about 0.3. These latter three cases have

the same crestline angle but differing side-slope angles, and the plots suggest that increasing the side-slope angle increases the peak acceleration ratio from about 3.8 to about 5.4. On average, acceleration ratios for peak 2 are less than those for peak 1.

All of the plots for peak 3 show similar two-peaked shapes centered close to the one-third points. No systematic variations in the responses with differing crestline angle or side slope-angle are evident. Acceleration ratios, on average, are roughly the same as those for peak 2.

Figure 3.44 shows peak responses from longitudinal ground shaking for cases 1-5. The peak-1 plots are very similar to those of figure 3.43: single-peak responses centered near the midpoint of the crestline. As is the case for transverse ground shaking, increasing the crestline angle (cases 3-2-1) moves the peak response lower along the crestline. Peak acceleration ratios range from about 4.5 to 6.5.

The peak-2 plots in figure 3.44 show two-peaked responses that are asymmetrical, the peak lower along the crestline generally having the greatest acceleration ratio. Maximum acceleration ratios range from about 2 to 4, and increasing the crestline angle appears to cause a decrease in the peak response.

Only cases 1, 4, and 5 yielded valid data in the frequency range of peak 3. All cases show two-peaked responses having very low (less than 2) acceleration ratios.

These data for both transverse and longitudinal ground shaking show that the maximum acceleration ratios occur between about 0.2 and 0.8 of the way from the base to the top of the crestline. Between these points, however, locations of acceleration maxima vary considerably. In the fundamental mode (peak 1) the maxima are near the crestline midpoint; in higher frequency modes (peaks 2 and 3) the maxima are located near the one-third points, but locations can vary significantly. Acceleration ratios for peak 1 tend to be the greatest. In the case transverse ground shaking, acceleration ratios for peaks 2 and 3 are slightly less than those for peak 1; in the case of longitudinal shaking, they are much less than those for peak 1.

3.4.2. Variations Along the Maximum Vertical Cross Section

Acceleration ratios for accelerometers located along the vertical line bisecting the maximum vertical cross section were plotted in a manner similar to that explained in the previous section. Only data for transverse ground shaking was recorded. Figures 3.45-3.50 show plots for cases 1-6, respectively; data were plotted from frequencies at which any of the accelerometers

along the vertical section had peak responses. Figure 3.51 summarizes these data by showing plots of the three major response peaks for each of the six cases.

The peak-1 acceleration ratios range from 4-7, and in all cases the acceleration increases from the base to the top of the ridge, though not linearly. No significant differences attributable to changing ridge geometry are evident.

The peak-2 acceleration ratios, ranging from about 1.5 to 4, are generally less than those for peak 1. The response shape is distinct from that of peak 1 but is similar for each of the six cases. As in the previous case, maximum accelerations are at the ridge top. Increasing the crestline angle appears to decrease the maximum acceleration response; while increasing the side-slope angle tends to increase the maximum acceleration response.

The peak-3 acceleration ratios are much lower than those of peaks 1 and 2; ratios are all less than 2.5, and most are less than 2.0.

3.4.3 Conclusions

The model-test results summarized in this section indicate that inertial forces within a ridge are greatest along the crestline near its midpoint. Significant amplification of ground shaking occurs along the central 60 percent of ridge crests, but response peaks in different frequencies and from ridges having different geometries are located in different places within this region. Amplification of ground motion is invariably greatest at the ridge crest as compared to points within the ridge vertically below the crest. These findings suggest that inertial forces will have the greatest effect on potential landslides near the surface (shallow landslides) on the central parts of ridge crests. This conclusion is substantiated by my observations, as well as those in several reports, that topographic amplification of earthquake shaking commonly results in (1) shattered ridge crests covered by churned surficial soils and (2) shallow, disrupted landslides in surficial material.

3.5 SPATIAL VARIATION IN SHEAR STRESS

In addition to increasing inertial forces, differential amplification of ground shaking induces increased shear stresses within a ridge. The greater the difference in acceleration between adjacent elements of soil, the greater the induced shear stress. Thus, spatial differences in the amplification ratio are an index of induced shear stress. To measure this effect, I calculated and plotted the absolute value of the first derivative

of the acceleration-ratio plots shown in figure 3.43, 3.44, and 3.51. The derivative was calculated as the ratio of the difference in acceleration ratio between two adjacent points to the proportional distance separating those two points. This point-to-point calculation eliminates the bias in the approximated curves drawn in the figures. The derivative is the ratio of two dimensionless numbers and is referred to herein as the "Shear-Stress Index" because it is a semiquantitative index of the induced shear stress within the ridge.

3.5.1 Variations Along Ridge Crests

Figure 3.52 shows the distribution of relative induced shear stress along the crestlines of the six models for each of the three major response peaks in the case of transverse ground shaking. For peak 1, shear stress is least at the extremities of the crestlines and is greatest in the central portion. In the case of increasing crestline angle (cases 3-2-1-6), the maximum shear stress tends to decrease, but its location varies within the central 60 percent of the crestline. The shear-stress distribution for cases 5, 1, and 4, which have identical crestline angles and increasing side-slope angles, have similar shapes, but the magnitude of the shear-Stress Index varies inconsistently with the differing geometries. The maximum values of the Shear-Stress Index range from about 15 to 23.

The plots for peak 2 mostly show shear-stress maxima near the extremities of the crestlines and minima in the central portions. Case 3 is notable exception, which shows a constant induced shear stress across the entire central part of the crestline. Maximum values of the Shear-Stress Index for peak 2 range from about 9 to 18, somewhat lower than those for peak 1. Increasing the side-slope angle tends to increase the shear stress, but the spatial distribution remains unchanged. Increasing the crestline angle has no systematic effect.

Plots for peak 3 show inconsistent shear-stress distributions: some are shaped similarly to those from peak 1, others to those from peak 2. The maximum values of the Shear-Stress Index range from about 12 to 29, the latter being the greatest value calculated for transverse shaking. No consistent variations in the shear-stress distribution can be correlated with changing crestline or side-slope angle.

The induced shear stress is of similar magnitude for each of the three peaks despite fact that the acceleration ratios for peaks 2 and 3 were much lower than those for peak 1. Thus, the higher frequency response peaks are as significant as the

fundamental response peak in inducing shear stress along the ridge crest in the case of transverse ground shaking.

Figure 3.53 shows the induced shear-stress distribution for the case of longitudinal ground shaking. The distributions for peak 1 are similar to those in figure 3.52 in that the ridge extremities have shear-stress minima. All of the plots also show a local shear-stress minimum near the ridge-crest centers between two maxima. Maximum values of the shear-Stress Index range from 13-17, lower than those for transverse shaking. No systematic differences related to ridge geometry are evident.

For the peak-2 data, the induced shear stress tends to be concentrated in the central portion of the crestlines. Maximum values of the Shear-Stress Index range from about 5 to 20, and most of the values are much lower than are those for peak 1. No differences related to ridge geometry are evident.

For peak 3, only cases 1, 4, and 5 produced valid data. The Index for case 1, having a maximum value of about 20, is the greatest calculated for longitudinal shaking and is located at the center of the crestline. The shear stresses induced for cases 4 and 5 are minimal.

The distribution of induced shear stress for longitudinal shaking shows that shear stress tends to be concentrated in the central parts of the ridges, but not in a unique area. The induced shear stresses decrease from peak 1 through peak 2 to peak 3, such that for peak 1, the stresses are similar to those induced from transverse shaking, but for peak 3, the stresses are insignificant in most cases.

3.5.2 Variation Along the Maximum Vertical Cross Section

Figure 3.54 shows the distribution of induced shear stress along the vertical line bisecting the maximum vertical cross section of the models. The scale of the plotted Shear-Stress Index is 40 percent of that in figures 3.52 and 3.53, so the magnitude of induced shear stress in the vertical section is much smaller than it is along the ridge crest. The maximum Shear-Stress Index in the vertical section is about 8, compared to a maximum of about 30 along the ridge crest.

The data for peak 1 indicate that, except for case 4, the shear stress is maximum near the center of the ridge. The effects of increasing crestline angle (cases 3-2-1-6) are a systematic increase in maximum induced shear stress. Maximum values range from about 5 to 8.

The peak-2 data generally show shear-stress minima in the ridge centers and maxima at the ridge base or top. Increasing

crestline angle appears to correlate with decreasing shear stress. Maximum values of Shear-Stress Index range from about 1.5 to 6.

The data for peak 3 show very small induced shear stress, Index values ranging from about 1 to 4. No systematic pattern in shear-stress distribution is discernible.

3.5.3 Conclusions

Figure 3.55 shows the sum of the Shear-Stress Index for the three major response peaks depicted in all cases shown in figures 3.52-3.54. This summation shows the overall distribution of induced shear stress over all frequency ranges tested. For transverse shaking, figure 3.55 shows that the shear stress is least at the extremities of the crestline, but that no unique area within the central 60-80 percent of the ridge crest consistently displays a shear-stress maximum. For longitudinal shaking, the overall shear stress is less than that for transverse shaking, and is similarly randomly distributed within the central parts of the ridge crest. The shear stresses in the maximum vertical cross section are of much lower magnitude than those along the crestline, and no significant variation along the vertical cross section is evident.

The preceding evidence indicates that shear stresses induced from all three response peaks, both individually and collectively, are distributed more or less randomly along the entire central portion of the ridge crest and along the entire vertical cross section; no single part of the ridge can be identified as consistently having anomalously high induced shear stress. Shear stresses induced along the ridge crest are much greater than those induced along the maximum vertical cross section, which agrees with findings from the previous section on spatial variation in acceleration that deformation and consequent ground failure induced by topographic amplification will tend to be concentrated near the ground surface.

3.6 COMPARISON WITH RESULTS OF CHILE EARTHQUAKE EXPERIMENT

The resonance frequencies observed in the results of the Chile earthquake experiment (Celebi, 1986, in press¹⁰); Part 2) appeared to relate consistently both to the frequency predicted by factor I of Sasaki and Kuwabara (1986)²⁹ and to frequencies corresponding to wavelengths equal to the height and width of the ridges. As discussed previously, the model responses do not occur at the frequencies predicted by factor I (table 3.2), and table 3.4 shows that frequencies corresponding to wavelengths

equal to the dimensions of the models as listed in table 3.1 likewise do not correlate with the observed resonance frequency (14-15 Hz) of the models.

Two general reasons for the lack of agreement between the Chile earthquake experiment results and the results of the PWRI model tests are possible. First, differences between the natural ridges in Chile and the model ridges tested may be so pronounced that they behave in fundamentally different ways and thus respond to incident ground motion differently. Such differences might include (1) geometric differences between the models and the Chile ridges, (2) the properties and behavior of the ridge materials, (3) possible complex subsurface geologic structure in the Chilean ridges, (4) effects of three components of random ground shaking versus one component of steady-state shaking, or (5) scale effects in the model tests.

The second possible reason for the lack of agreement is misinterpretation of one or both of the data sets. The model tests indicate that the fundamental resonance frequency is probably related to slope length (measured parallel to the crestline), maxima vertical ridge thickness, and (or) the geometry of the rigid base to which the model is attached. Examination of these factors for the Chilean ridges, which had responses similar to one another, shows that (1) the "rigid base geometry" of the two ridges is probably similar because they lie adjacent to one another in an area of relatively uniform geology, so it is reasonable to assume that the subsurface geologic structure within and beneath the two ridges is similar; (2) the maximum height or thickness of the ridges is similar; and (3) the slope lengths differ somewhat, but, as with all such measurements, the beginning and ending points of the slopes are somewhat arbitrarily determined. Also, for the Chile ridges, the ridge height and ridge thickness as measured in the models (figure 3.1) are identical because the "rigid base" on which the ridges rest is probably nearly horizontal. Thus, the Chilean ridges and the PWRI model ridges may be responding to similar parameters, and the resonant frequencies predicted by factor I and by wavelength-matching effects may have been coincidental in the case the Chile data.

The effects of scale must be accounted for in the model tests for appropriate comparison of results with actual ridges. Sasaki and Kuwabara (1986)²⁰ suggest using a scaling relationship based on a model of shearing in a vertical plane. Using their relationship, I plotted (figure 3.56) model frequency as function of a scaling ratio, κ , and the frequency of an actual

ridge. For the most likely range of shear-wave velocities of the Chile ridge material (500-700 m/s), the two main resonance frequencies for the model tests (14 and 22 Hz) are plotted as a function of κ , the ratio of the ridge thickness to the model thickness, and the frequency of the ridge. The resonance peaks of the Chile ridges predicted by the model tests using this scaling relationship are 13-19 Hz and 22-30 Hz. The observed resonance peaks in the Chile data were centered at about 4 and 8 Hz. This indicates that (1) the scaling relationship used is inappropriate for the type of deformation experienced in the models or the Chile ridges, (2) the models and the Chile ridges deformed in significantly different ways, or (3) the differences in the geometries of the models and the Chile ridges are so great that they behave fundamentally differently.

Overall trends in the amplification ratios for the Chile data and the PWRI model tests are quite similar. Accelerometers located nearest to the crestline midpoints and the points of maximum ridge thickness on the models uniformly recorded the greatest accelerations (figures 3.8-3.18). This result is similar to the results from the Chile experiment, which showed that stations near the crestline midpoints and on exposed ridge faces recorded the greatest accelerations. The magnitudes of the acceleration ratios are also similar for the two data sets. Ratios from the Chile experiment, though reaching peak values between 20 and 30, typically ranged from about 2 to 8. Amplification ratios from the PWRI model tests were as great as 6.8 and generally ranged from about 2 to 5. The two ridges instrumented in Chile had fairly similar geometries, so it is impossible to observe any effects of changing ridge geometry on amplification of ground shaking.

3.7 COMPARISON WITH DATA FROM MATSUZAKI DENSE SEISMIC ARRAY

PWRI has deployed several seismometers in a dense array in Matsuzaki on the western Izu Peninsula, as shown in figure 3.57. The array is located on a long ridge having a gently sloping (less than 10°) upper part and a steeply sloping ($15-25^\circ$) face. Station 1 is located where the ridge abruptly steepens and is thus on the thickest portion of the ridge on an exposed promontory. Stations 2-5 are spaced along the steep lower part of the ridge face to the base of the ridge. Peak ground accelerations (PGA) from strong motion records from five earthquakes are recorded in table 3.5 for stations 1-5. Figure 3.58 shows the ratio of the PGA recorded at each station to the PGA recorded at station 1 plotted versus the elevations of the

stations above the ground surface at station 5. The ratios decrease significantly from the maximum ratio of 1.0 for station 1 on the ridge crest to a minimum mean ratio of 0.36 for station 5 at the base of the ridge. Thus, the Matsuzaki data agree with observations from the Chile data and from the model tests that amplification is maximum on the thickest parts of ridges and on exposed ridge faces. Amplification ratios as great as about 3 (1/0.36) at station 1 compared to station 5 at Matsuzaki are in the same range as amplifications from the other data sets.

3.8 SUMMARY AND CONCLUSIONS

The PWRI slope models all displayed similar resonance responses when vibrated in the 3-60 Hz frequency range. For ground shaking transverse to the length of the ridges, the fundamental resonance frequency is 14-15 Hz, and secondary resonance peaks are centered between 20 and 25 Hz. For ground shaking longitudinal to the ridge length, the fundamental resonance frequency is 16-17 Hz, and the secondary resonance peak is centered between 30 and 35 Hz. The models had differing overall heights, widths, and horizontal lengths; maximum vertical thicknesses (maximum vertical cross-sectional heights), slope lengths measured parallel to the crestline, and rigid model-base geometries were the same for all models. The similarity of responses between models indicates that these latter factors that were held constant govern the fundamental resonance frequency of the models and that the factors that varied between models do not significantly affect the fundamental resonance frequency. This finding differs markedly from the findings of previous studies that used a factor I , related to overall slope height, width, and horizontal length, to predict the fundamental resonance frequency. This contradiction may be due to fundamental differences in the behaviors of the model ridges and the natural ridges studied previously, or possibly to misinterpretation of one or more of the data sets.

Plots of the spatial variation of acceleration along the ridge crestlines and along the maximum vertical cross section indicate that accelerations are greatest along the central 60-80 percent of the crestline, but that no unique part of the crestline experiences anomalously high accelerations. A similar result is obtained from plots of the induced shear-stress distribution: maximum shear stress is distributed somewhat randomly along the central 80 percent of the crestline. No particular part of the ridge could be identified as being especially susceptible to deformation or ground failure as a

result of inertial forces related to amplification of acceleration or to induced shear stress related to differential amplification. Plotting the distribution of acceleration ratio and induced shear stress over the entire range of frequencies tested (at 1-2 Hz intervals) would more completely illustrate space- and frequency-dependent variations in these parameters.

The magnitude of the amplification ratios in the model tests agree well with data from the Chile earthquake experiment and from the Matsuzaki dense seismic array. Amplification in all cases is greatest along ridge crests and on exposed ridge faces and decreases towards the bases of slopes and the on anchored or buttressed portions of slopes. Maximum amplification ratios along ridge crests generally range from about 2-8. Amplification ratios at points lower on the ridge crest or flanks--or deeper within the ridges--are much lower and appear to decrease as a linear function of height above the base of the ridge.

These findings are in general agreement with my experience and with documented field observations of apparent topographic amplification effects along ridges in past earthquakes. The commonest reported observation is that of surficial material along ridge crests being highly disrupted and churned or of boulders and other objects resting on the ground surface having been overturned or otherwise disturbed. Rock falls, debris slides, and other shallow, disrupted landslides have likewise been reported along ridges that may have experienced topographic amplification. For the most part, observations of the effects of topographic amplification of earthquake shaking are of near-surface effects. Topographic amplification decreases rapidly toward the bases of ridges; therefore, the effects of such amplification on deep-seated landslides having failure surfaces near the bases of ridges is probably much smaller than the maximum effects present along the ridge crests.

Table 3.1 Dimensions of slope models as defined in figure 3.1
(L, H, W, S, and T in mm; α and β in degrees)

CASE	L	H	W	S	T	α	β
1	852	397	424	940	212	25	45
2	915	333	424	974	212	20	45
3	1,081	290	424	1,119	212	15	45
4	852	397	244	940	212	25	60
5	852	397	734	940	212	25	30
6	866	500	424	1,000	212	30	45

Table 3.2 Values of factor I computed in various ways (H, L, W, T, and S as defined in figure 3.1; V is the shear-wave velocity of ridge material, 1237 mm/s)

	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
$I_a = HL/VW$	0.645	0.581	0.598	1.121	0.373	0.826
$I_b = HS/VW$	0.712	0.618	0.619	1.236	0.411	0.953
$I_c = TL/VW$	0.344	0.370	0.437	0.598	0.199	0.350
$I_d = TS/VW$	0.380	0.394	0.452	0.660	0.219	0.404
$I_e = H^2/VW$	0.301	0.211	0.160	0.522	0.174	0.477
$I_f = T^2/VW$	0.086	0.086	0.086	0.149	0.050	0.086

Table 3.4 Frequencies corresponding to wavelengths equal to the ridge dimensions listed in table 3.1
(values in Hz; shear-wave velocity is 1237 mm/s)

Dimension	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
L	1.45	1.35	1.14	1.45	1.45	1.43
H	3.12	3.71	4.27	3.12	3.12	2.47
W	2.92	2.92	2.92	5.07	1.69	2.92
S	1.32	1.27	1.11	1.32	1.32	1.24
T	5.83	5.83	5.83	5.83	5.83	5.83

Table 3.3 Systematic acceleration responses. A "+" indicates a systematic increase in resonance frequency or amplification ratio; a "-" indicates a systematic decrease. No symbol is shown where no systematic increase or decrease is present.

Increasing crestline slope angle	Resonance Frequency				Amplification Ratio				Model sequence 3 → 2 → 1 → 6
	Channel	Figure 3.-	1st Peak	2nd Peak	Channel	Figure 3.-	1st Peak	2nd Peak	
Increasing side-slope angle	3	14			3	14			Model sequence 5 → 1 → 4
	4	15			4	15			
	5	16			5	16			
	6	17	-		6	17	-		
	7	18			7	18			
	8	19			8	19			
	9	20	+		9	20	+		
	10	21	+	+	10	21	+	+	
	11	22		+	11	22		-	
	12	23	-		12	23	+		
	13	24	-		13	24	+		
	14	25	-		14	25	+		
Increasing side-slope angle	3	14			3	14			Model sequence 5 → 1 → 4
	4	15		+	4	15		+	
	5	16			5	16			
	6	17	-	-	6	17	+	+	
	7	18	-	-	7	18	-	+	
	8	19			8	19			
	9	20	-	-	9	20	-	+	
	10	21			10	21			
	11	22			11	22		+	
	12	23			12	23			
	13	24			13	24			
	14	25			14	25			

Table 3.5 Peak ground accelerations (PGA), amplification ratios, and station elevations (relative to ground surface at station 5) for five earthquakes recorded at the Matsuzaki dense seismic array.

STATION	COMPONENT	ELEVATION (m)	PGA (gal)	RATIO
Earthquake of 9/13/85				
1	N-S	167	106.4	1.0
2	N-S	148	86.0	0.808
4	N-S	48	78.4	0.737
5	N-S	0	77.9	0.732
5	N-S	-5	32.1	0.302
1	E-W	167	101.4	1.0
2	E-W	148	70.0	0.690
4	E-W	48	47.1	0.464
5	E-W	0	87.1	0.859
5	E-W	-5	26.3	0.259
1	U-D	167	65.8	1.0
2	U-D	148	26.4	0.401
4	U-D	48	23.9	0.363
5	U-D	0	24.8	0.377
5	U-D	-5	18.2	0.277
Earthquake of 10/4/85				
1	N-S	167	21.8	1.0
2	N-S	148	12.8	0.587
4	N-S	48	5.9	0.271
1	E-W	167	20.6	1.0
2	E-W	148	9.7	0.471
4	E-W	48	8.1	0.393
1	U-D	167	8.1	1.0
2	U-D	148	4.9	0.605
4	U-D	48	2.4	0.296
Earthquake of 10/18/85				
1	N-S	167	43.2	1.0
2	N-S	148	21.1	0.488
4	N-S	48	29.3	0.678
5	N-S	-5	15.9	0.368
1	E-W	167	34.7	1.0
2	E-W	148	22.4	0.646
4	E-W	48	14.3	0.412
5	E-W	-5	10.8	0.311
1	U-D	167	26.5	1.0
2	U-D	148	14.3	0.540
4	U-D	48	8.0	0.302
5	U-D	-5	6.7	0.253
Earthquake of 6/24/86				
1	N-S	167	22.3	1.0
2	N-S	148	10.5	0.471
3	N-S	98	8.8	0.395
4	N-S	48	6.9	0.309
5	N-S	-5	2.9	0.130
1	E-W	167	18.6	1.0
2	E-W	148	10.9	0.586
3	E-W	98	9.8	0.527
4	E-W	48	6.7	0.360
5	E-W	-5	3.5	0.188
1	U-D	167	11.5	1.0
2	U-D	148	9.9	0.861
3	U-D	98	5.5	0.478
4	U-D	48	6.1	0.530
5	U-D	-5	3.4	0.296
Earthquake of 11/22/86				
1	N-S	167	49.6	1.0
2	N-S	148	30.6	0.617
5	N-S	0	18.1	0.365
5	N-S	-5	11.3	0.228
1	E-W	167	32.2	1.0
2	E-W	148	24.4	0.758
5	E-W	0	16.9	0.525
5	E-W	-5	9.3	0.289
1	U-D	167	19.7	1.0
2	U-D	148	12.3	0.624
5	U-D	0	7.8	0.396
5	U-D	-5	5.4	0.274

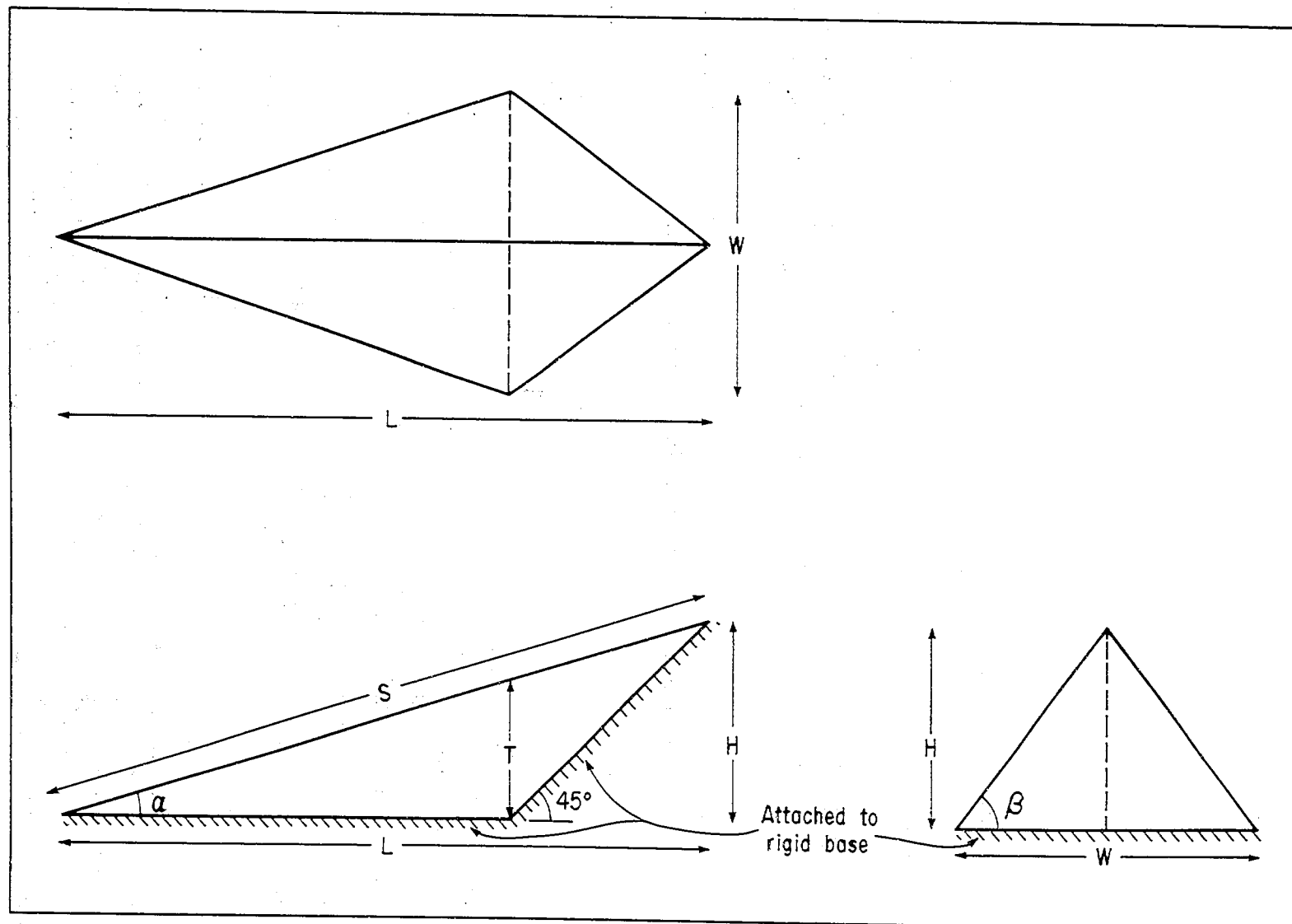


Figure 3.1 Generalized geometry of model slopes showing dimension, nomenclature.

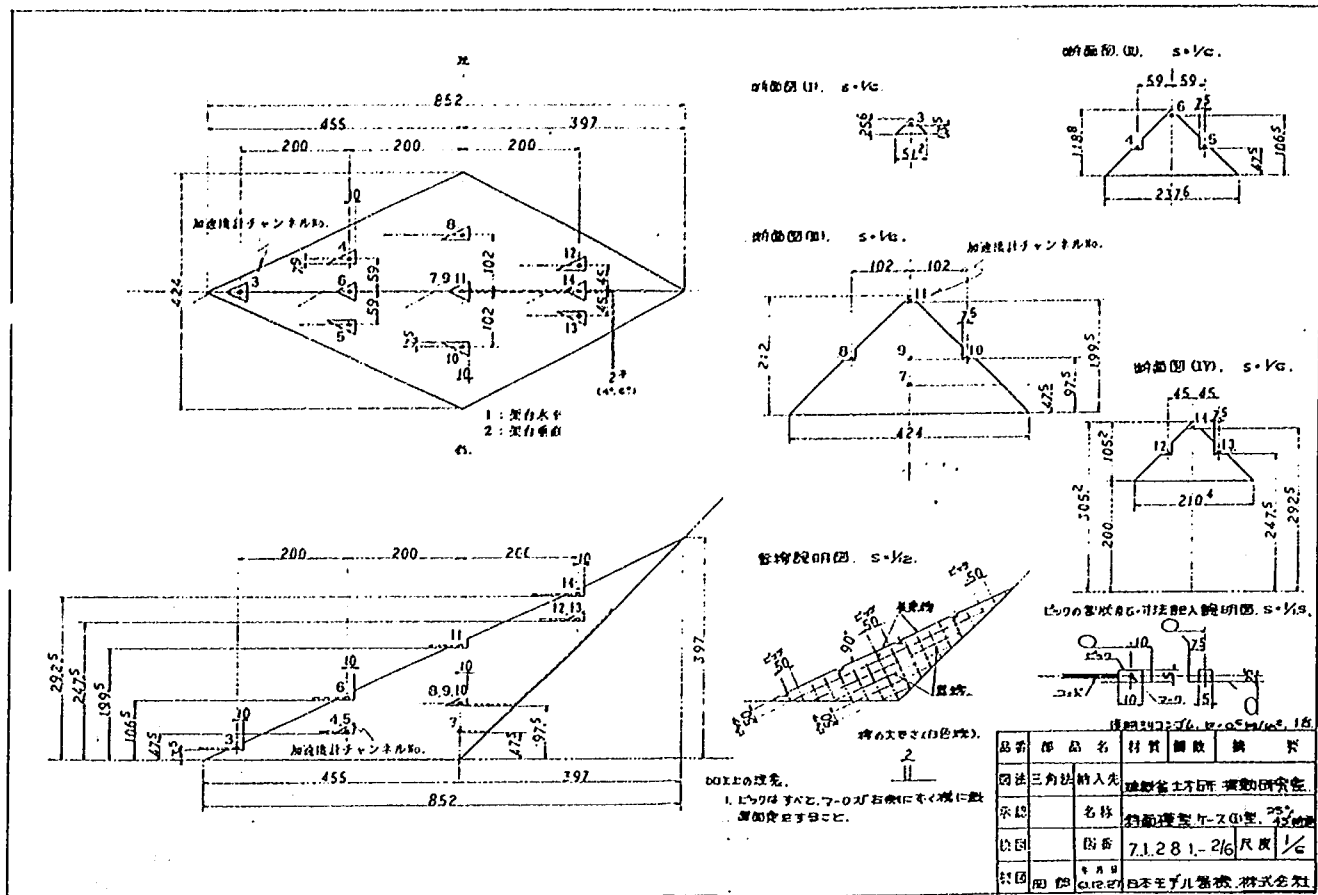


Figure 3.2 Scale Drawing of Case-1 Model

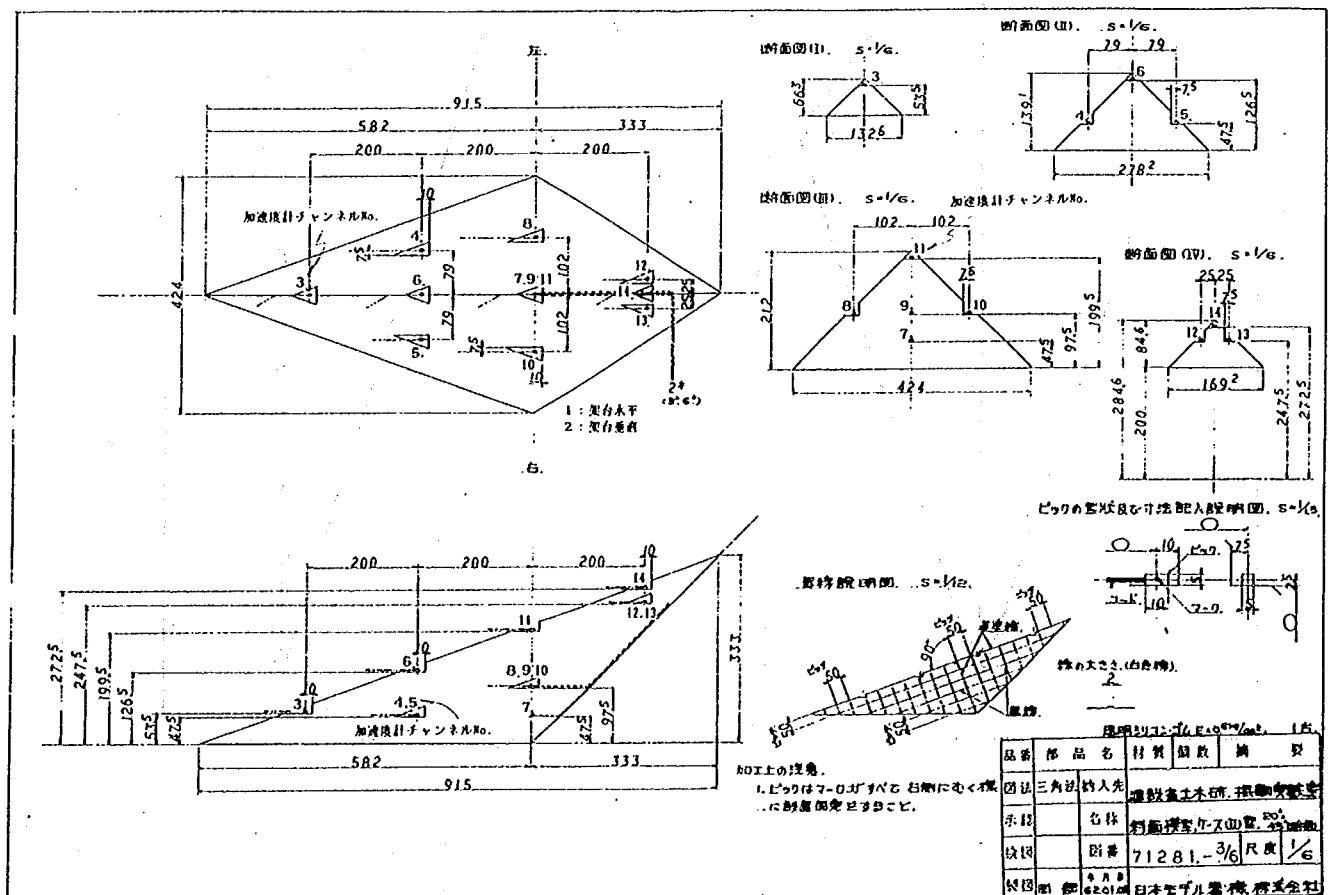


Figure 3.3 Scale Drawing of Case-2 Model

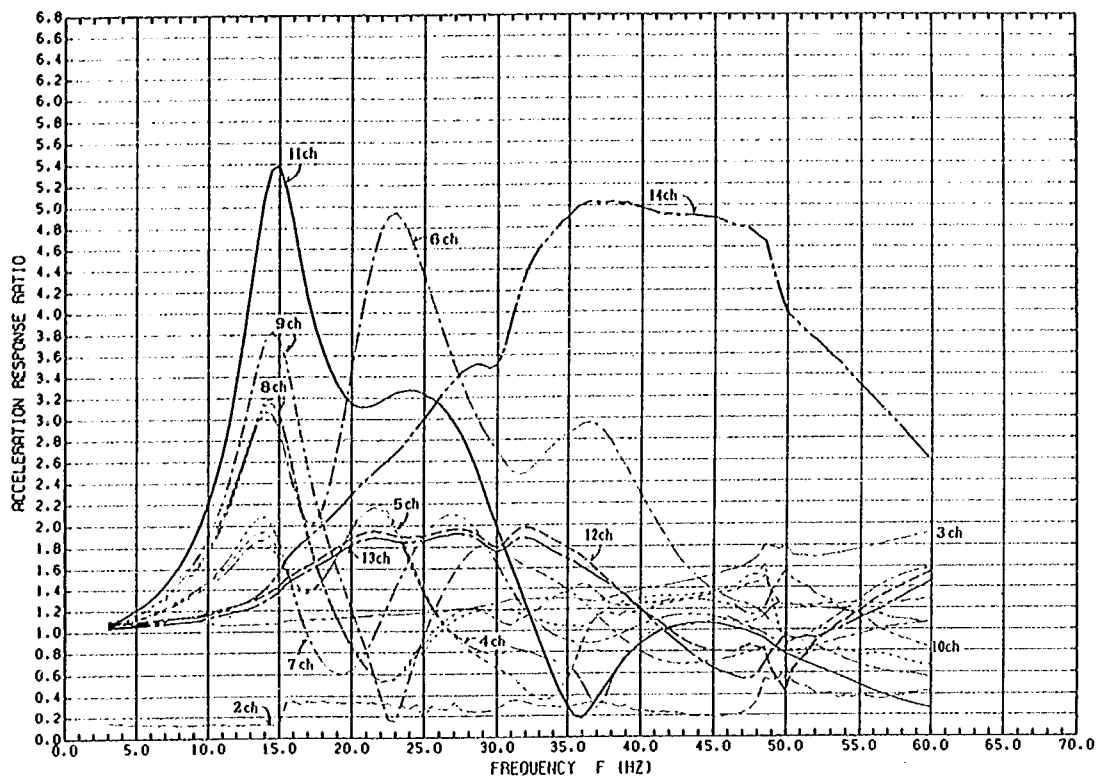


Figure 3.8 Acceleration response for case-1 model, transverse shaking.

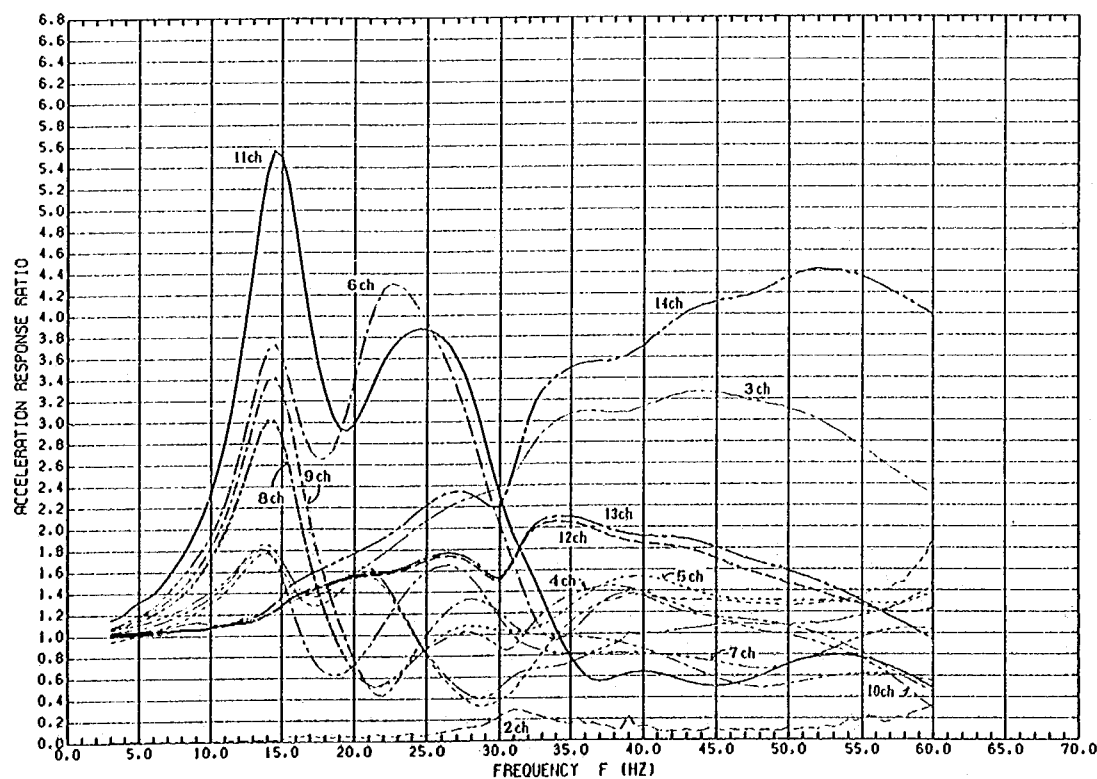


Figure 3.9 Acceleration response for case-2 model, transverse shaking.

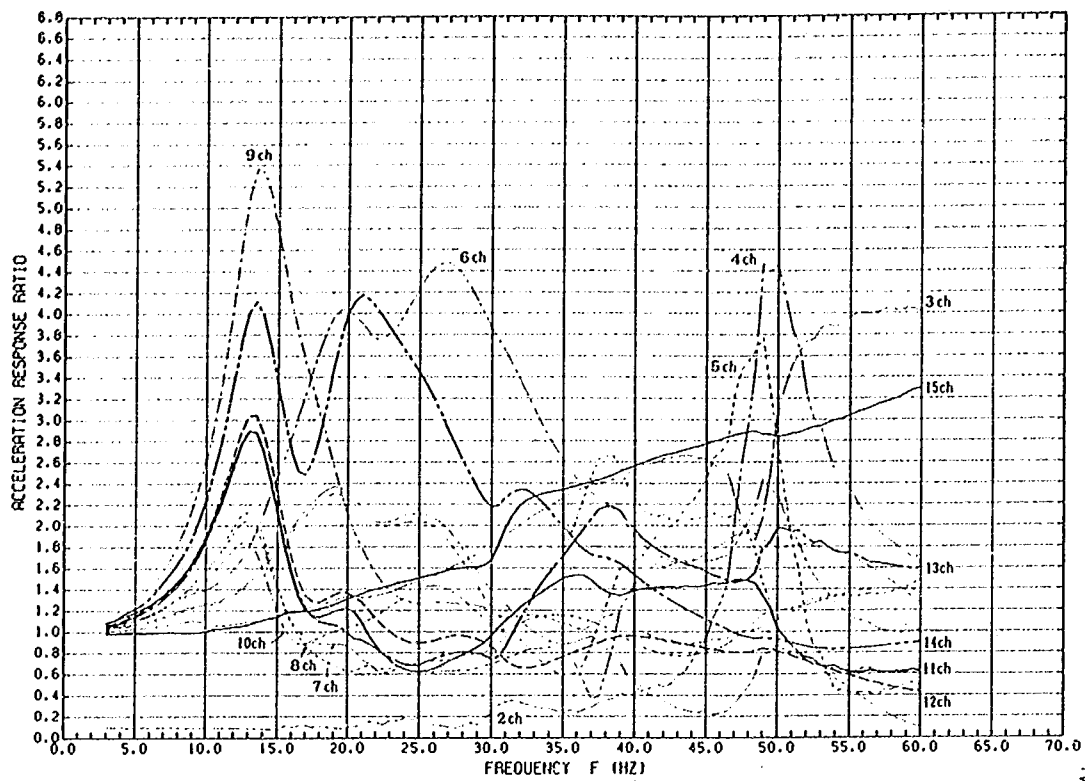


Figure 3.10 Acceleration response for case-3 model, transverse shaking.

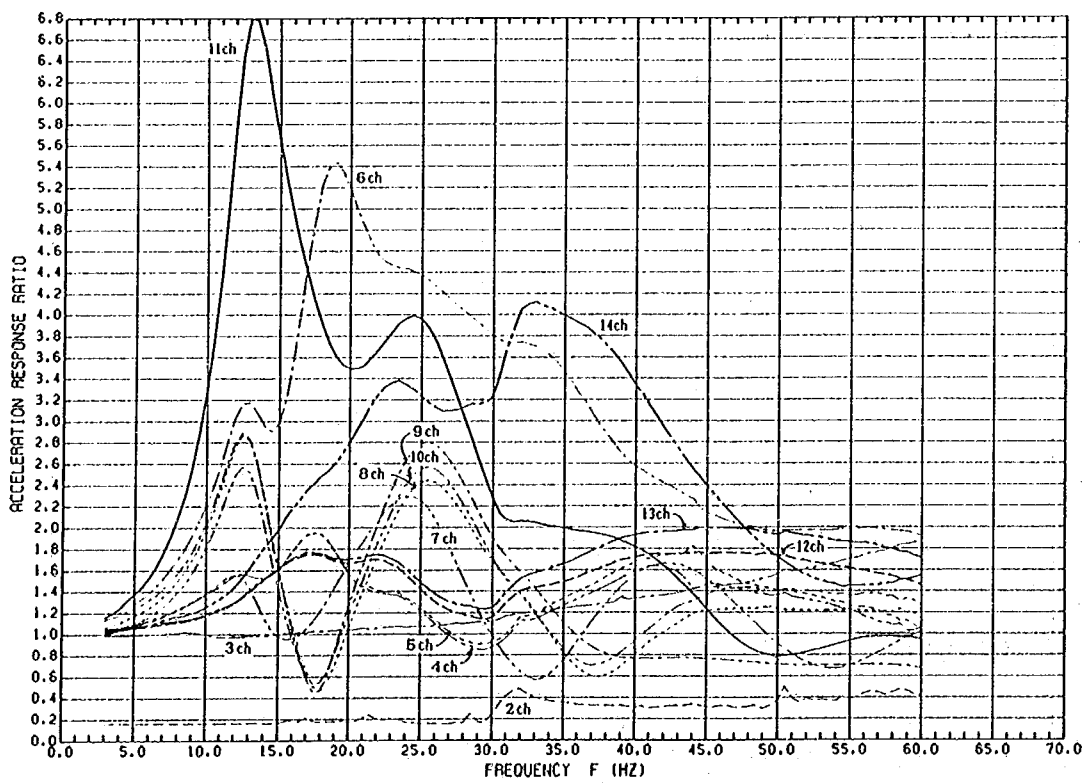


Figure 3.11 Acceleration response for case-4 model, transverse shaking.

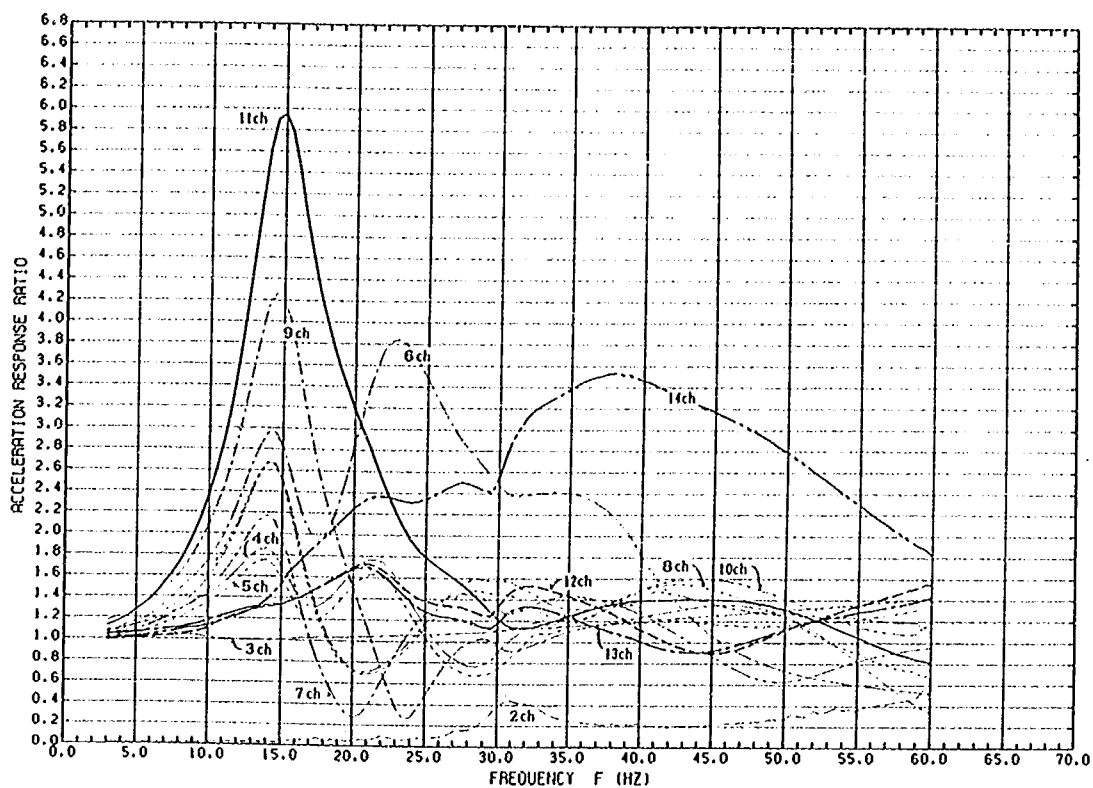


Figure 3.12 Acceleration response for case-5 model, transverse shaking.

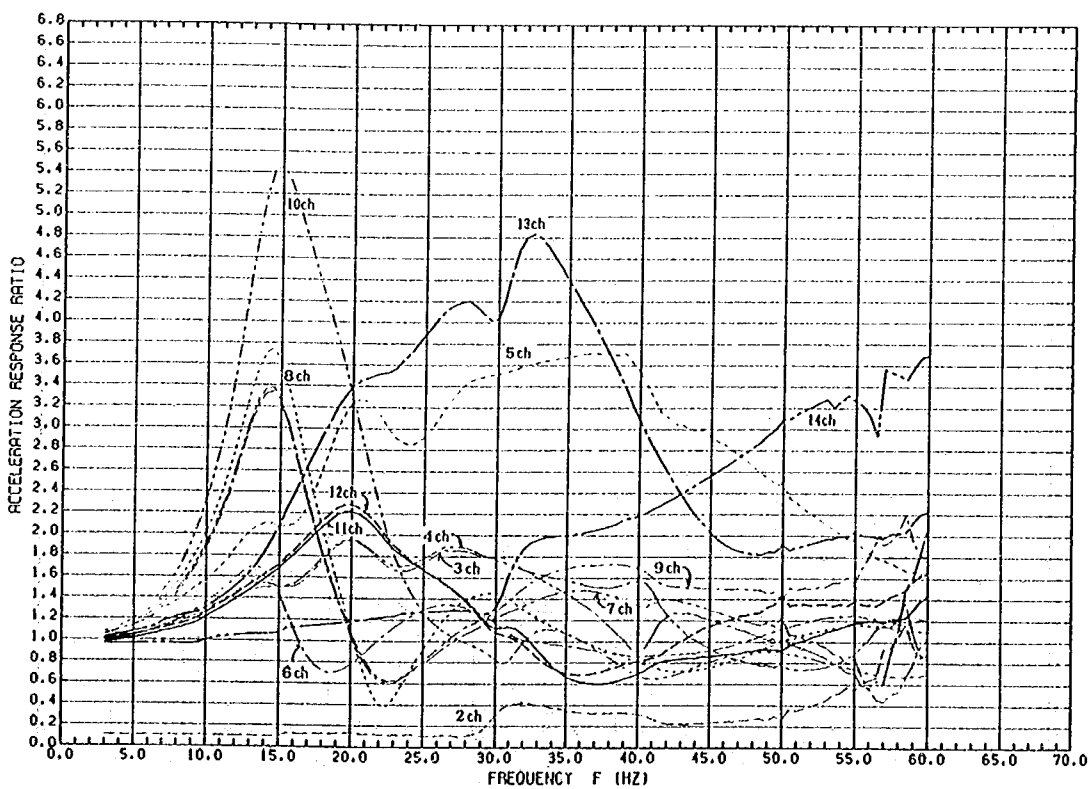


Figure 3.13 Acceleration response for case-6 model, transverse shaking.

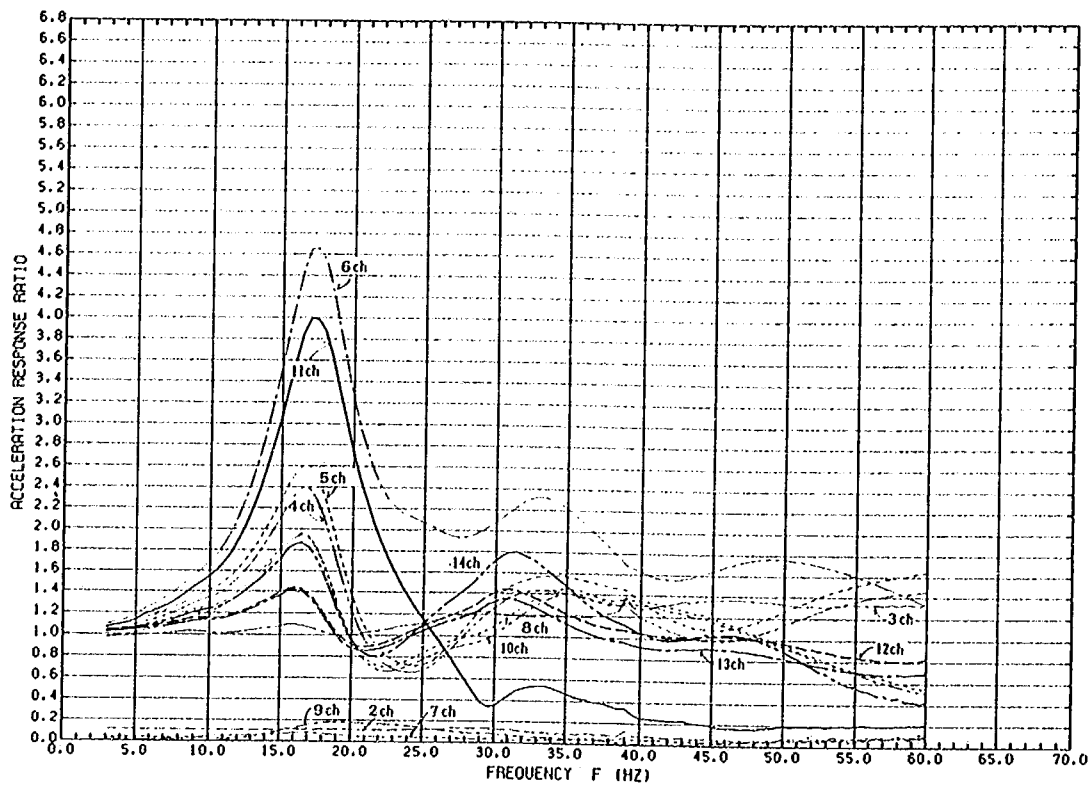


Figure 3.14 Acceleration response for case-1 model, longitudinal shaking.

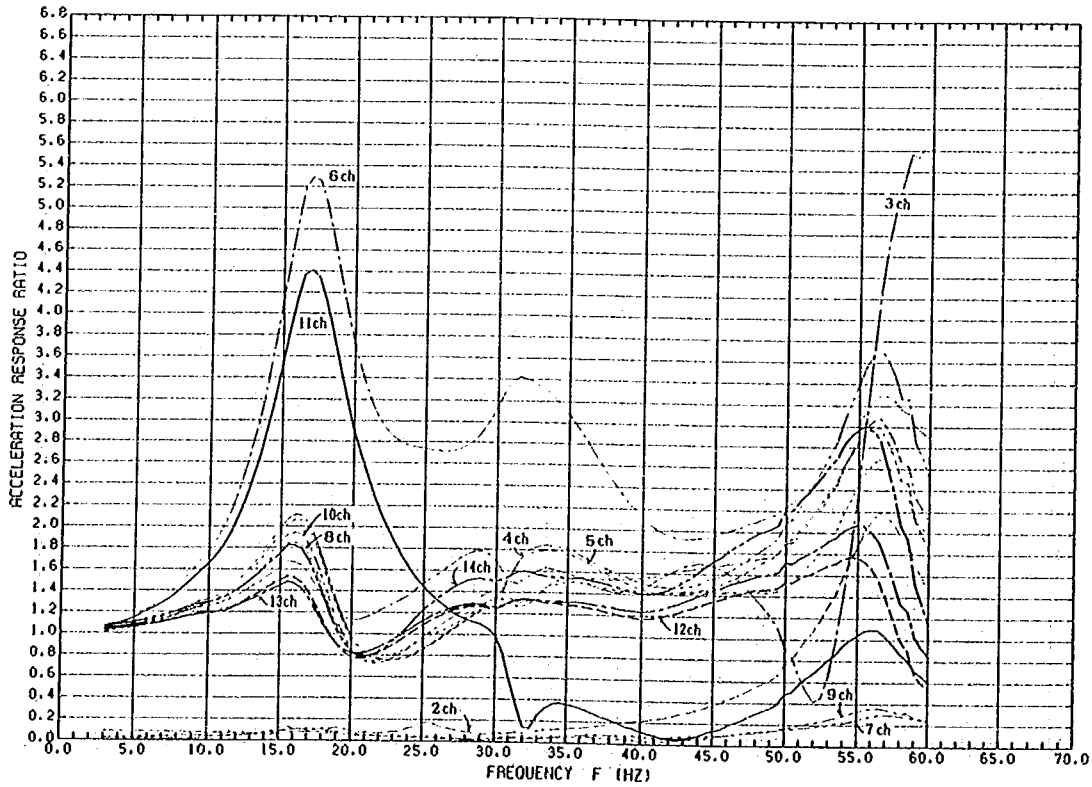


Figure 3.15 Acceleration response for case-2 model, longitudinal shaking.

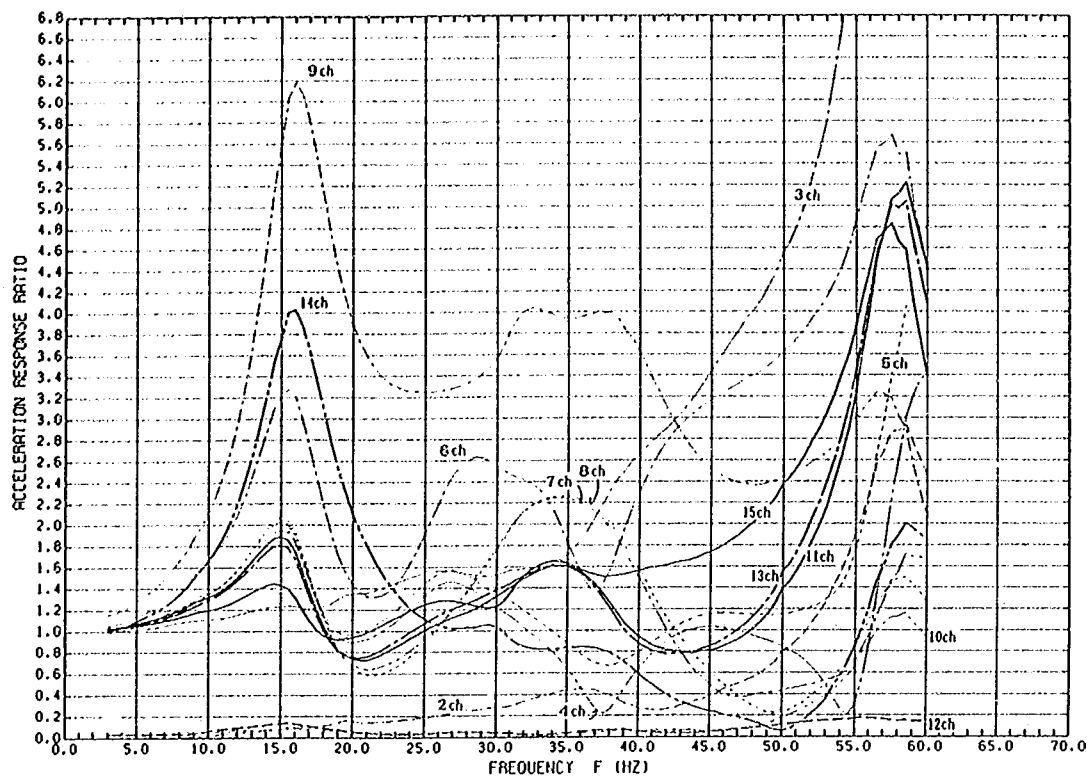


Figure 3.16 Acceleration response for case-3 model, longitudinal shaking.

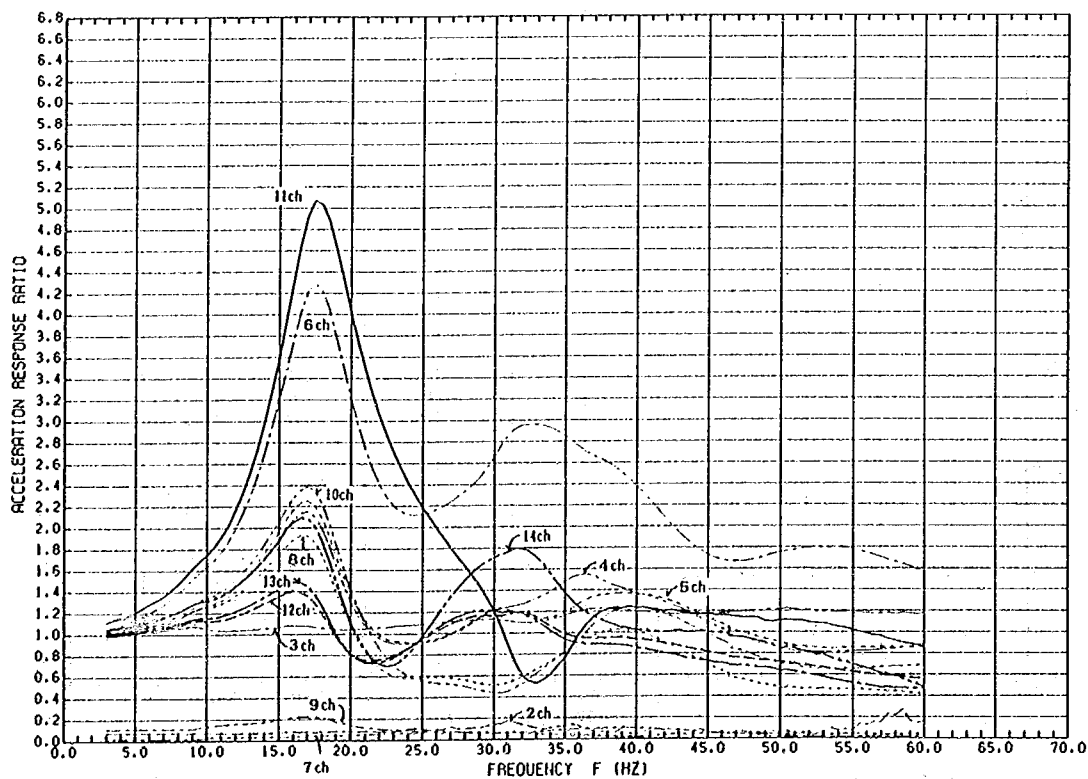


Figure 3.17 Acceleration response for case-4 model, longitudinal shaking.

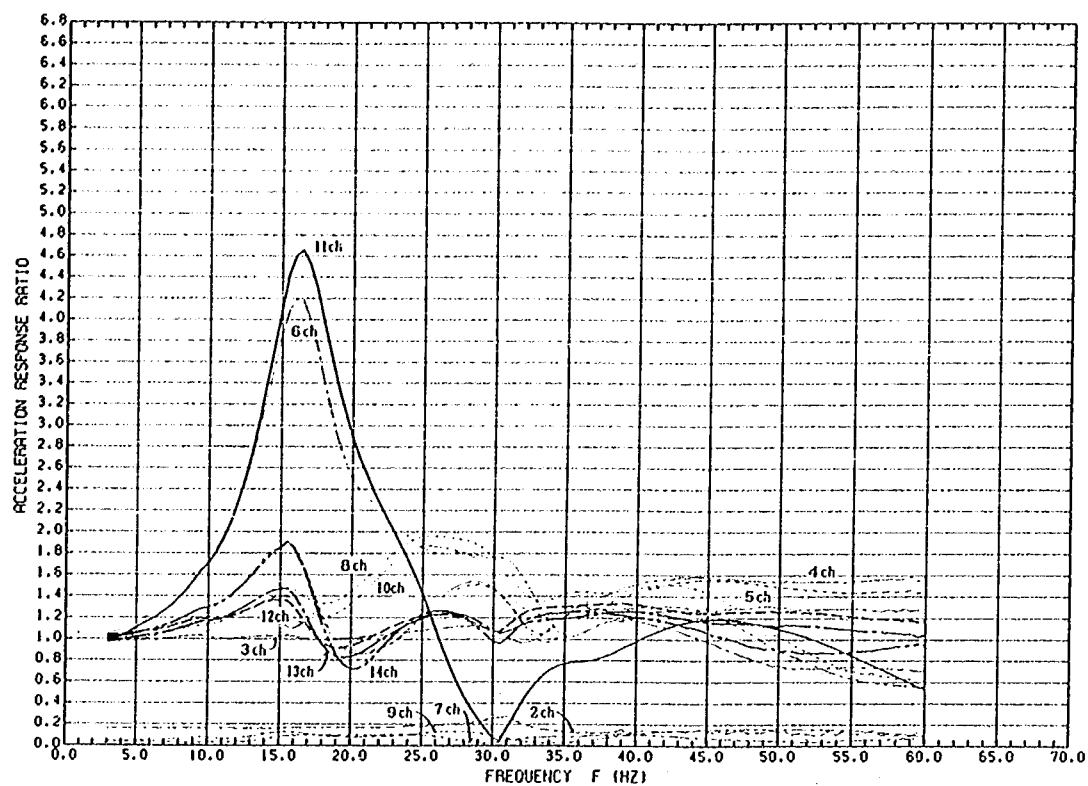


Figure 3.18 Acceleration response for case-5 model, longitudinal shaking.

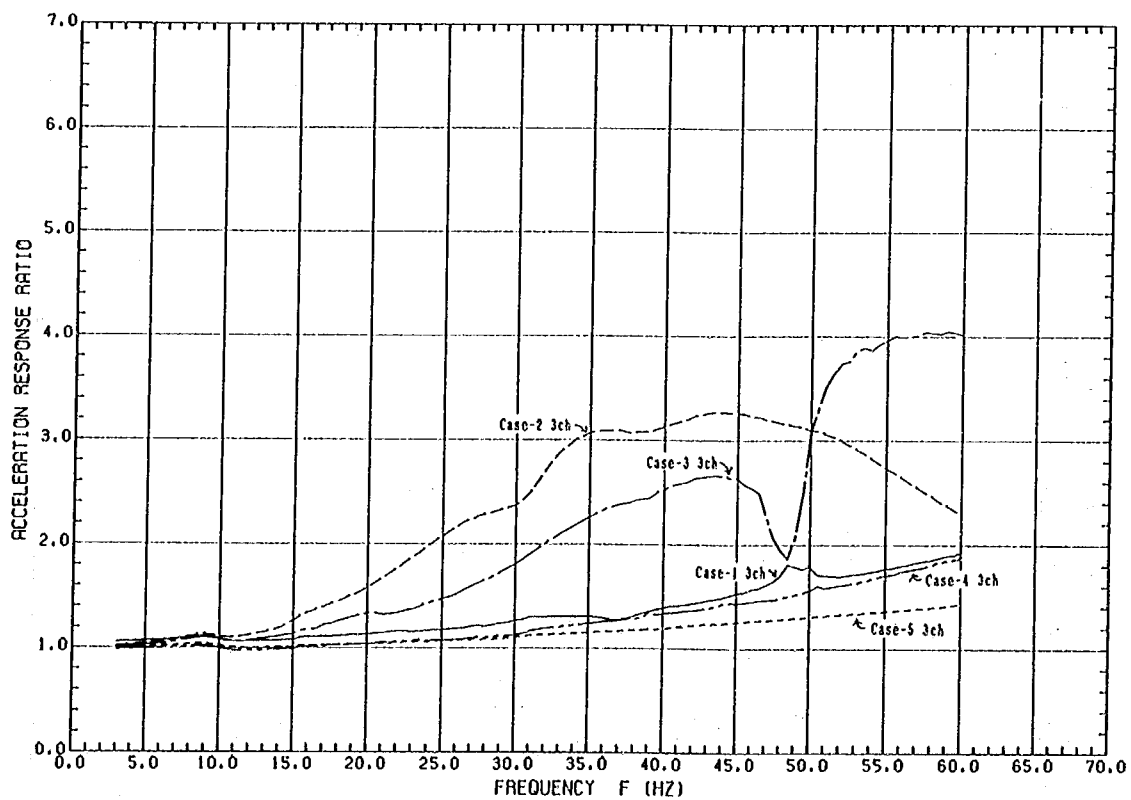


Figure 3.19 Acceleration response of channel 3, transverse shaking.

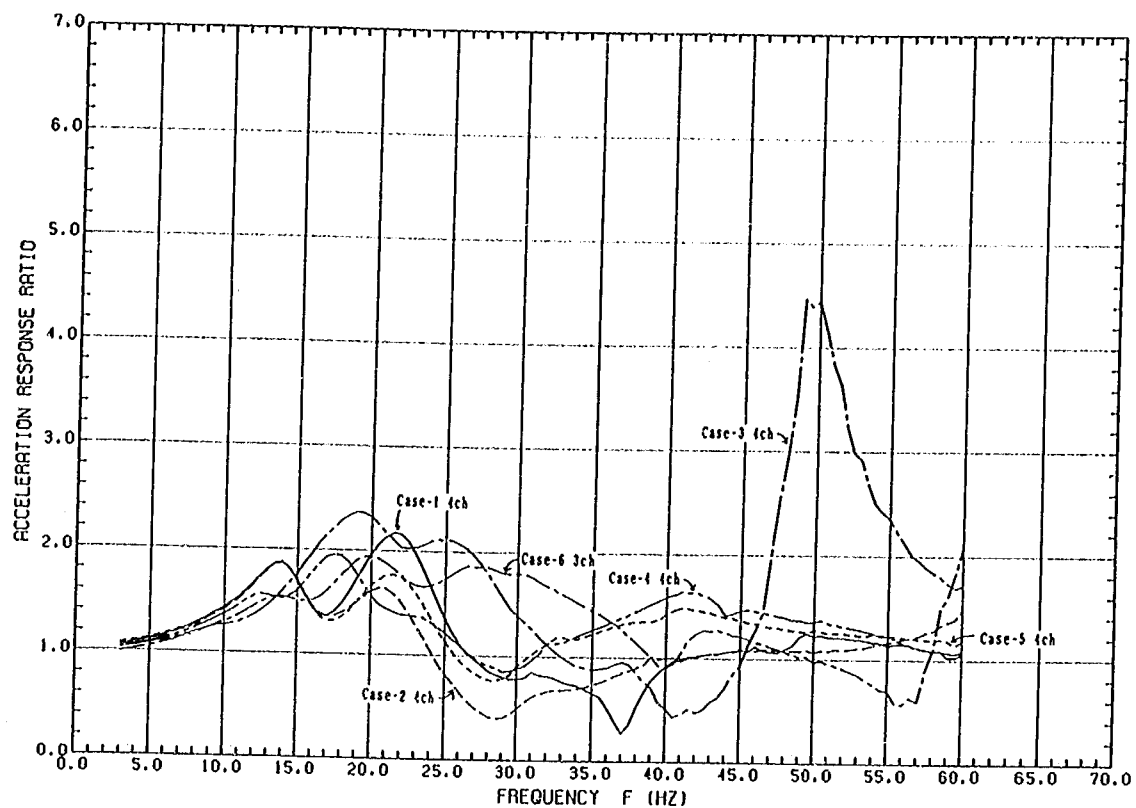


Figure 3.20 Acceleration response of channel 4 (ch. 3, case 6), transverse shaking.

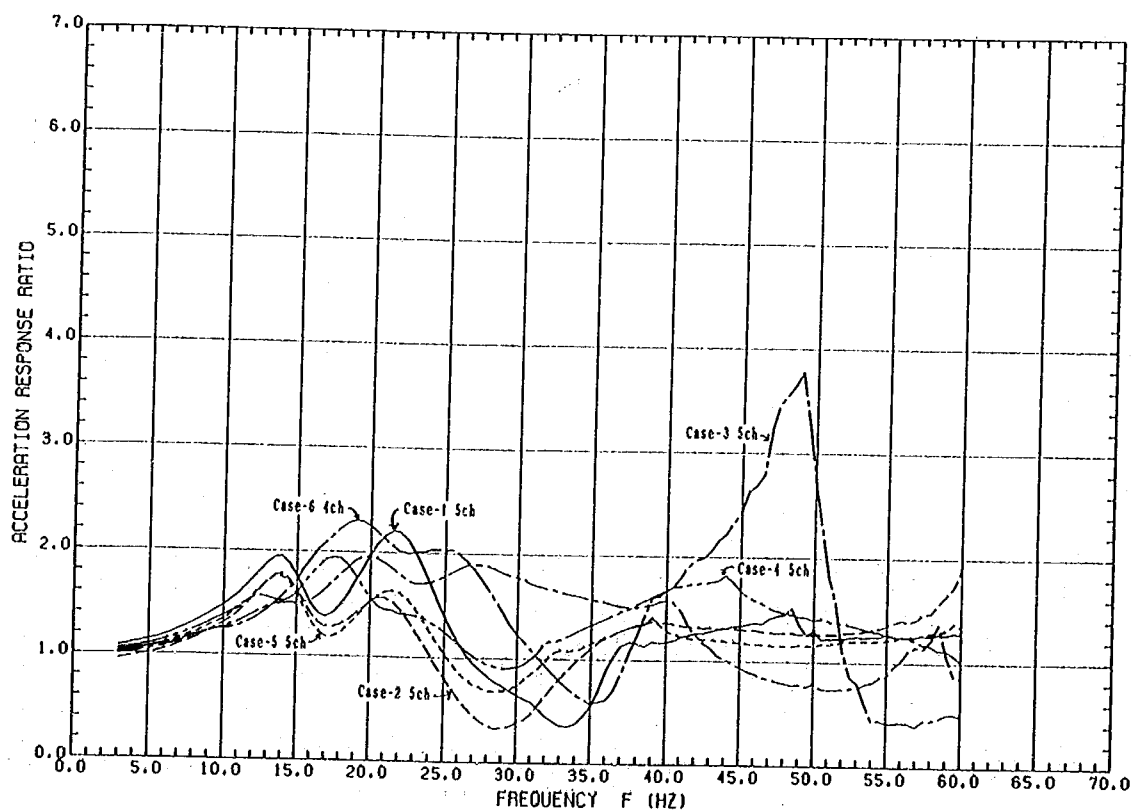


Figure 3.21 Acceleration response of channel 5 (ch. 4, case 6), transverse shaking.

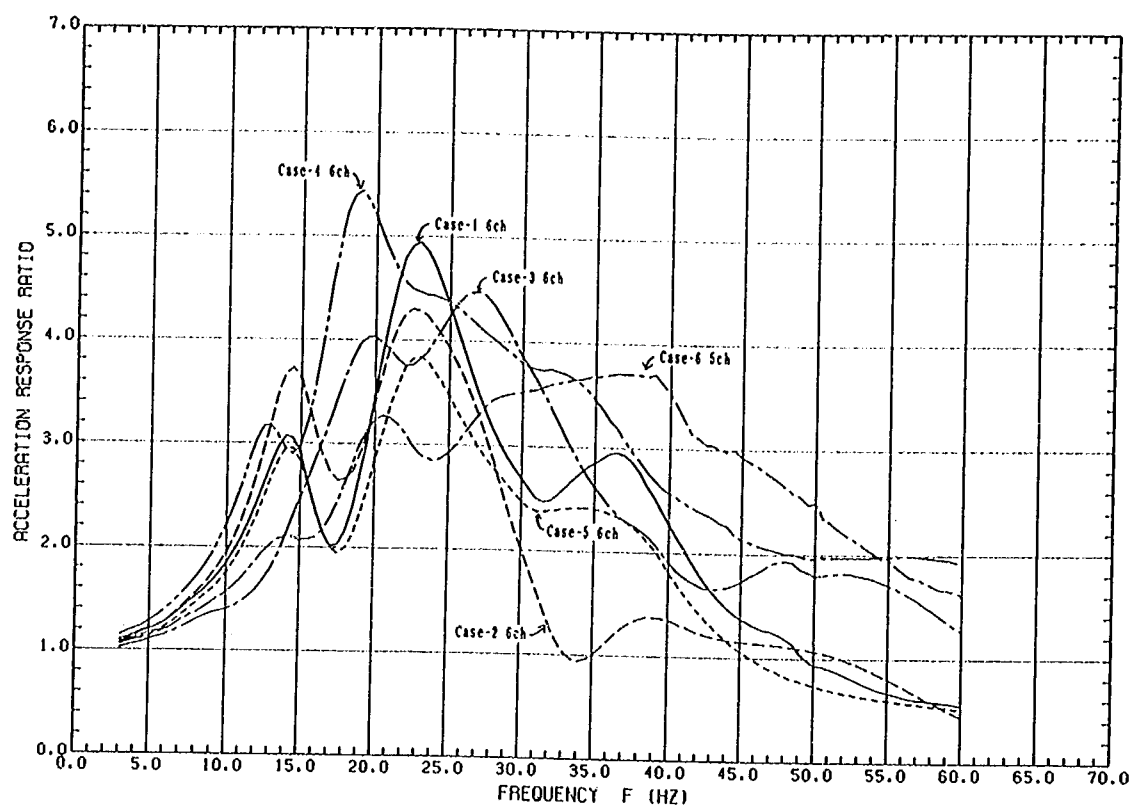


Figure 3.22 Acceleration response of channel 6 (ch. 5, case 6), transverse shaking.

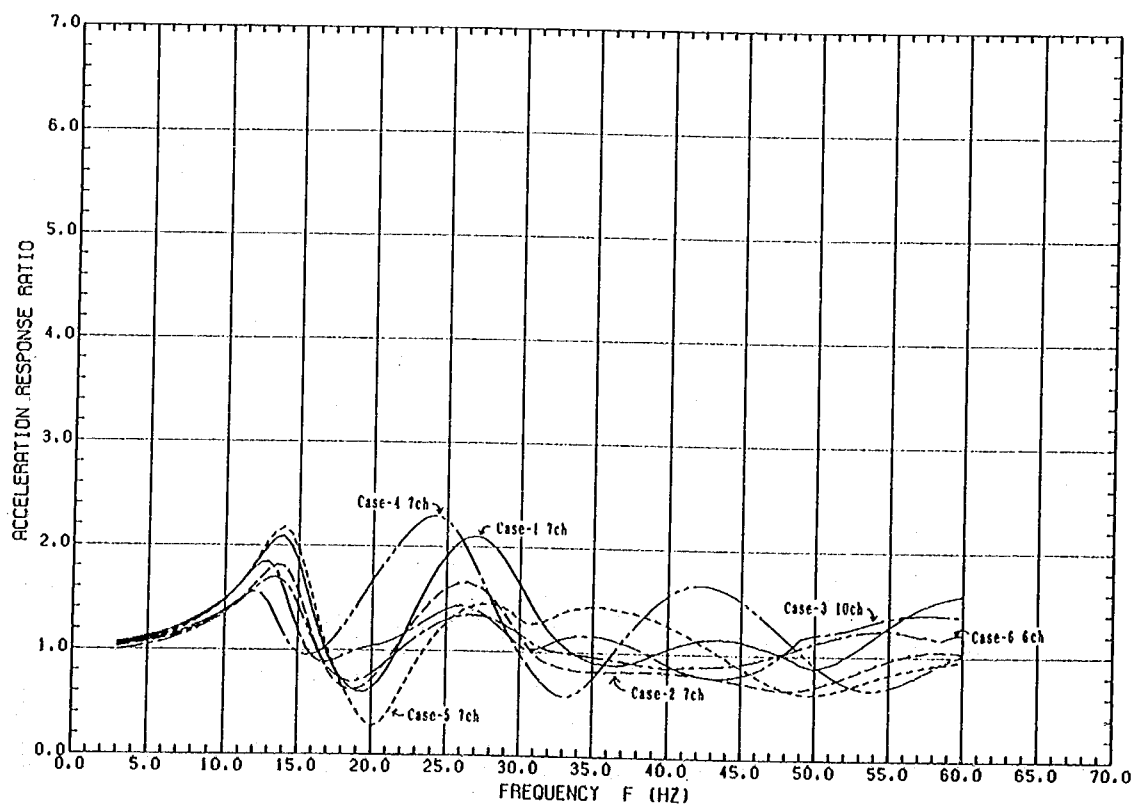


Figure 3.23 Acceleration response of channel 7 (ch. 10, case 3; ch. 6, case 6), transverse shaking.

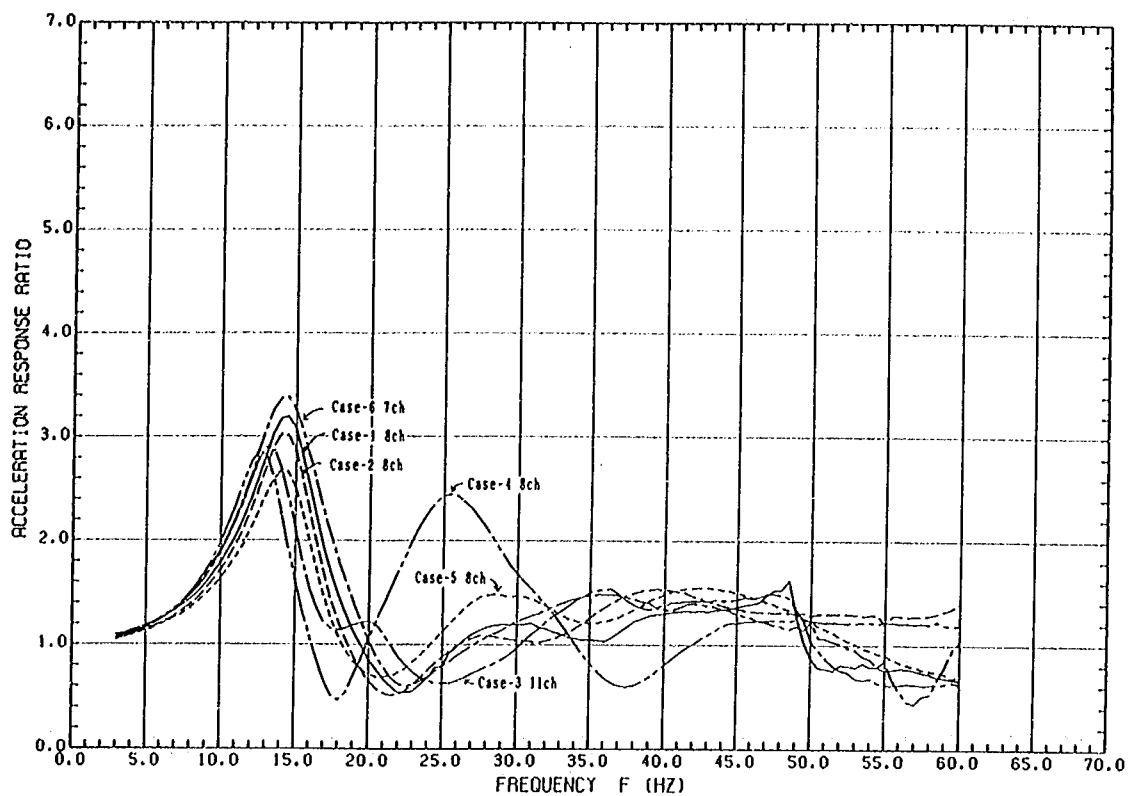


Figure 3.24 Acceleration response of channel 8
(ch. 11, case 3; ch. 7, case 6), transverse shaking.

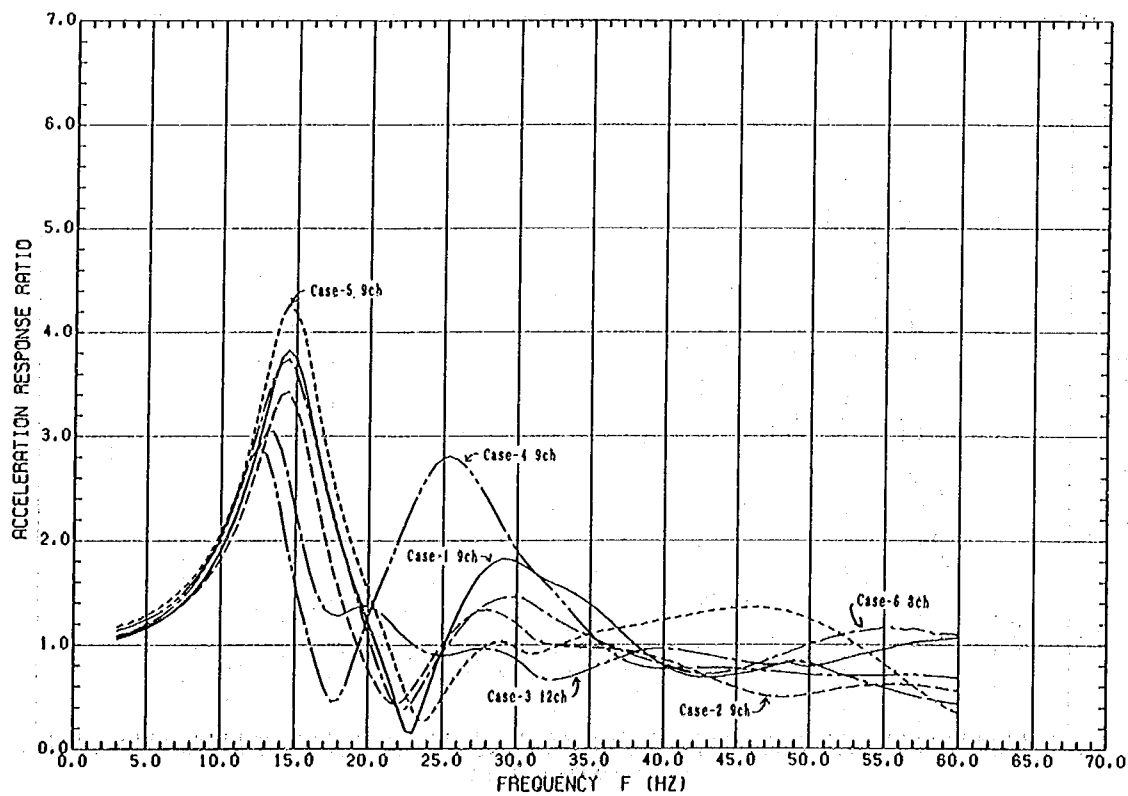


Figure 3.25 Acceleration response of channel 9
(ch. 12, case 3; ch. 8, case 6), transverse shaking.

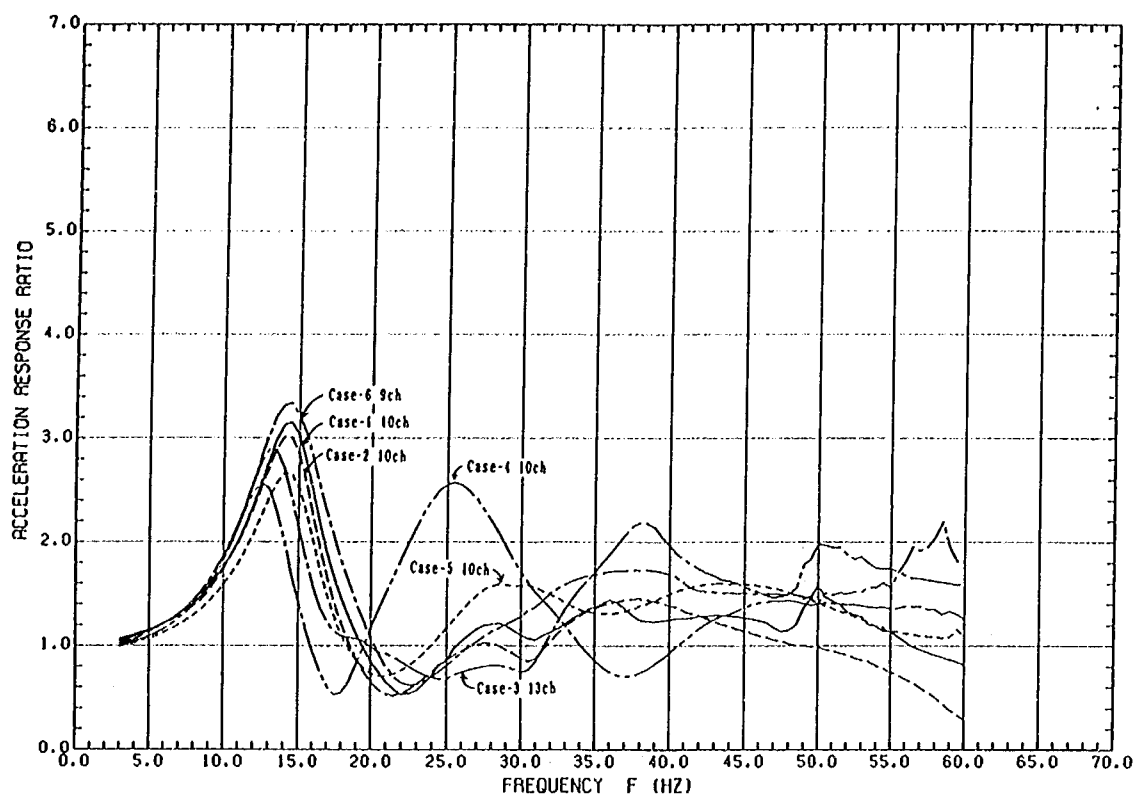


Figure 3.26 Acceleration response of channel 10
(ch. 13, case 3; ch. 9, case 6), transverse shaking.

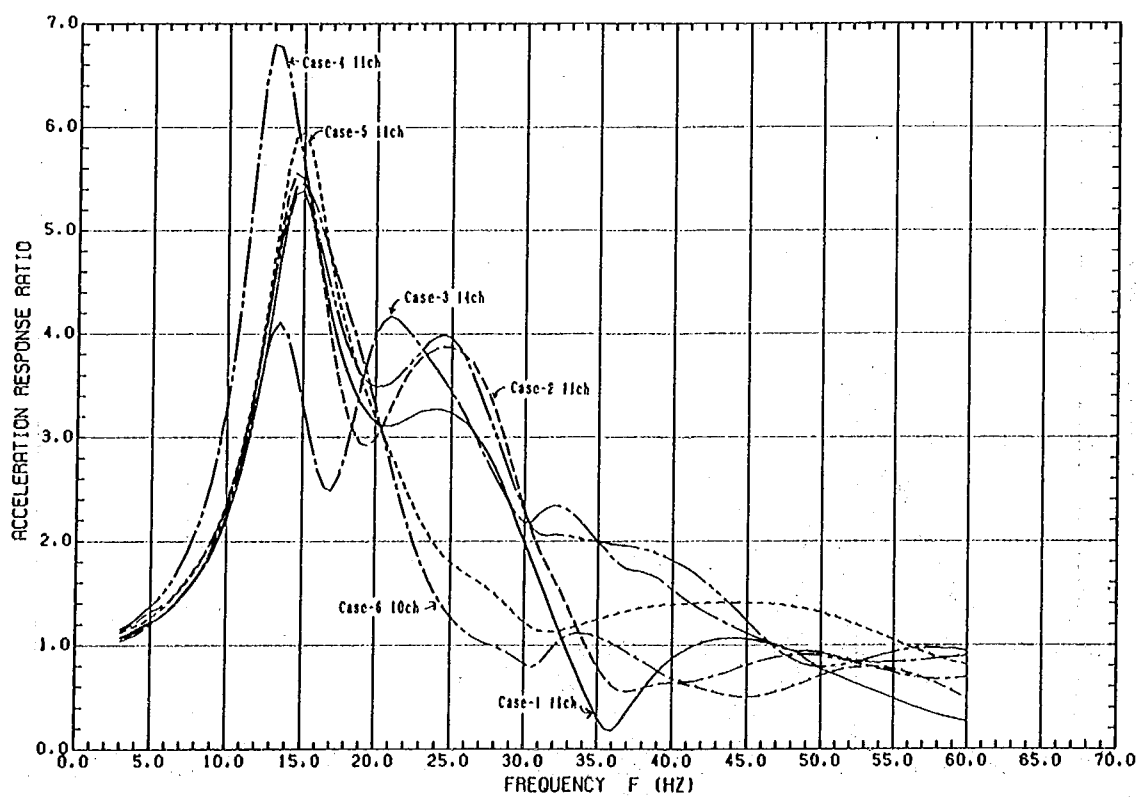


Figure 3.27 Acceleration response of channel 11
(ch. 14, case 3; ch. 10, case 6), transverse shaking.

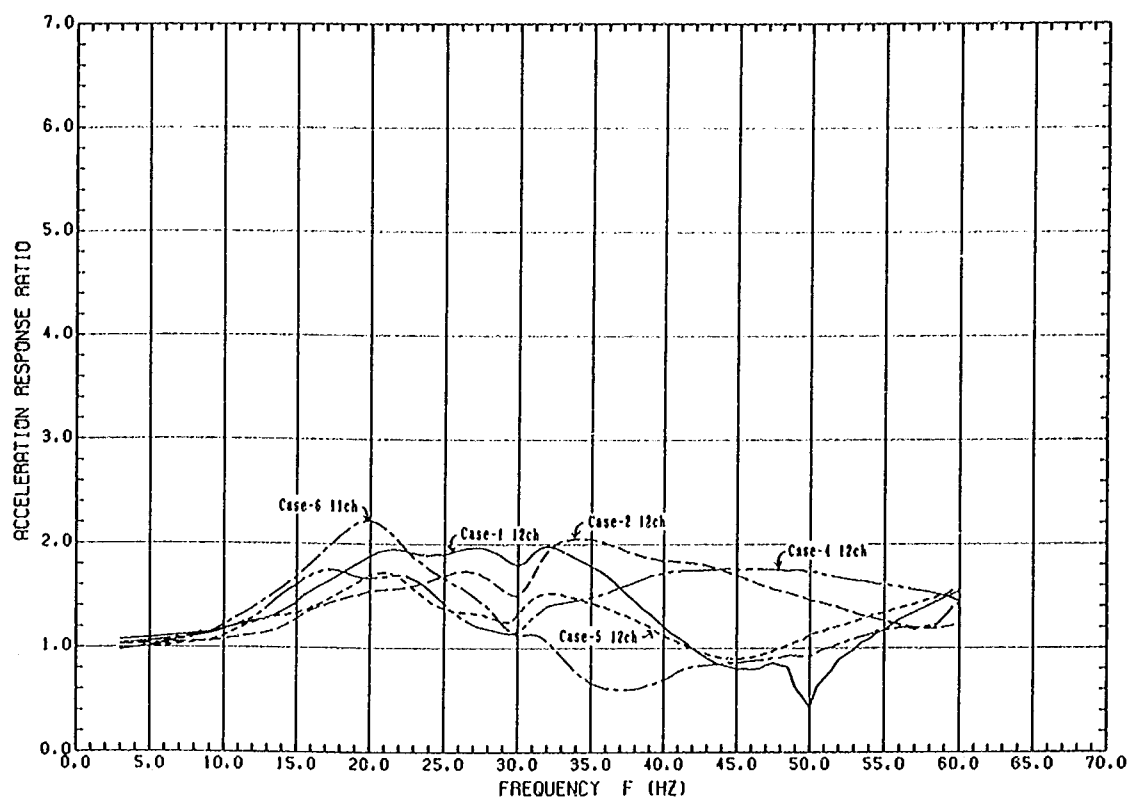


Figure 3.28 Acceleration response of channel 12
(ch. 11, case 6), transverse shaking.

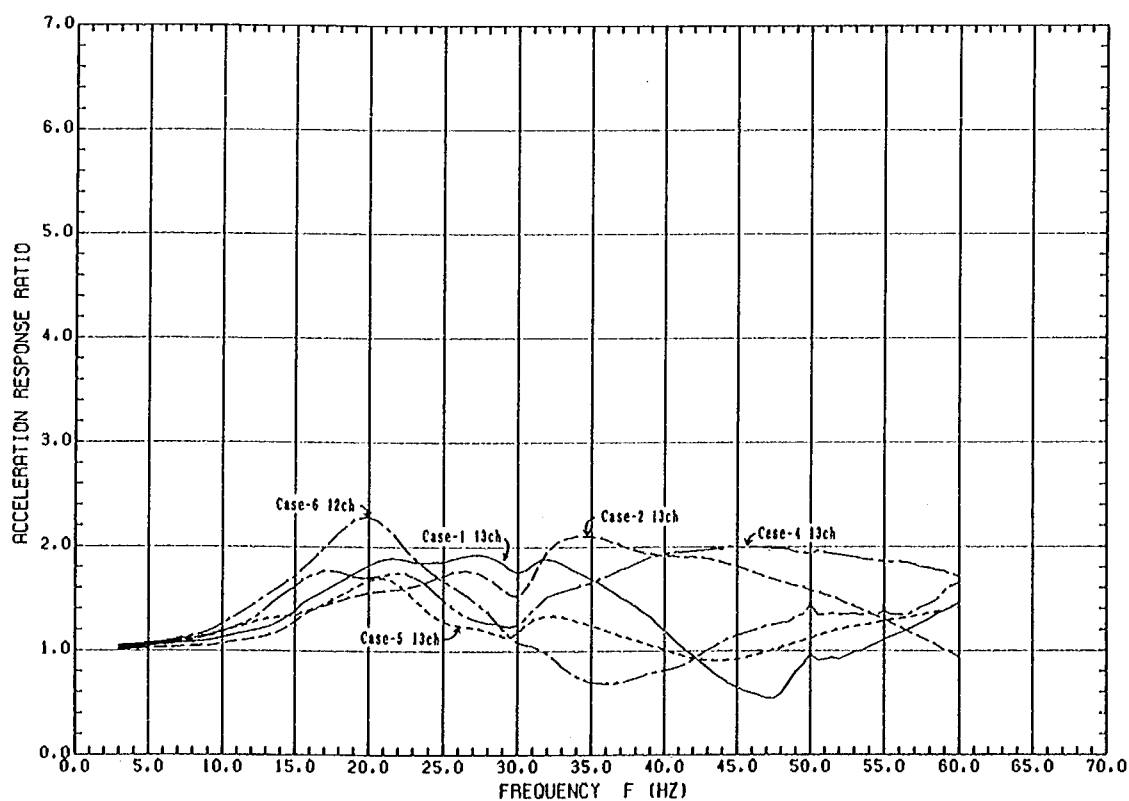


Figure 3.29 Acceleration response of channel 13
(ch. 12, case 6), transverse shaking.

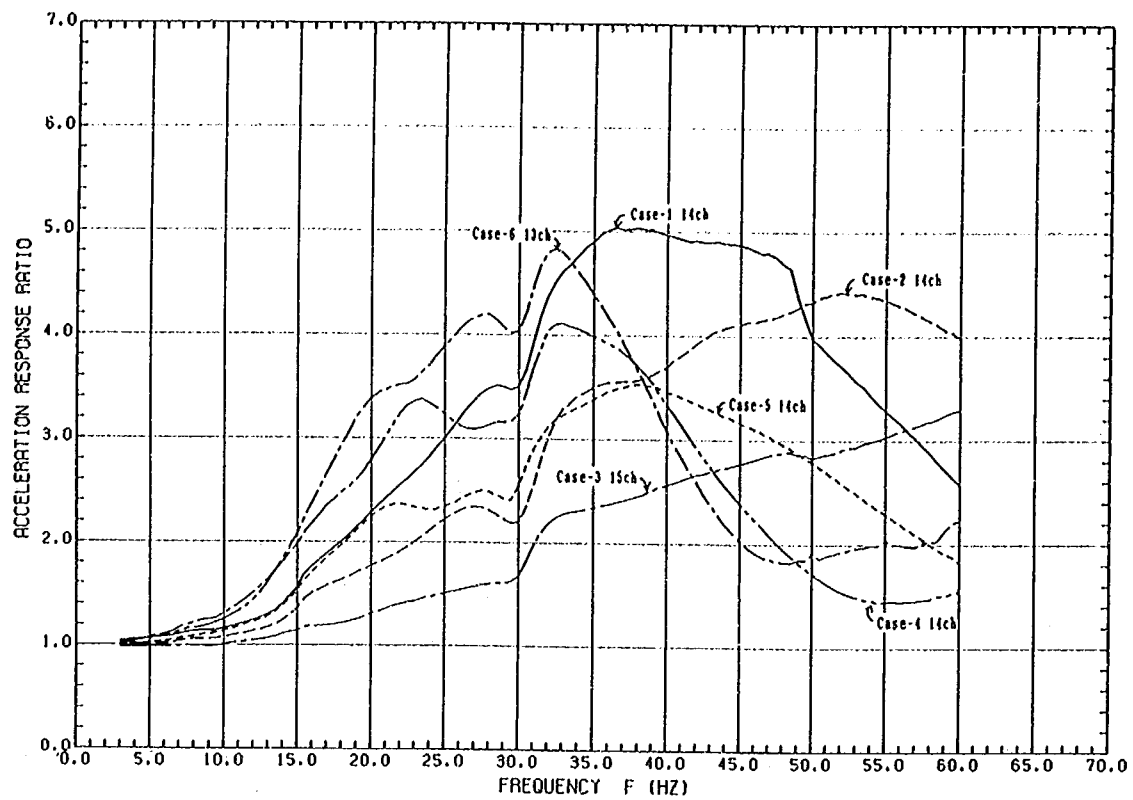


Figure 3.30 Acceleration response of channel 14
(ch. 15, case 3; channel 13, case 6), transverse shaking.

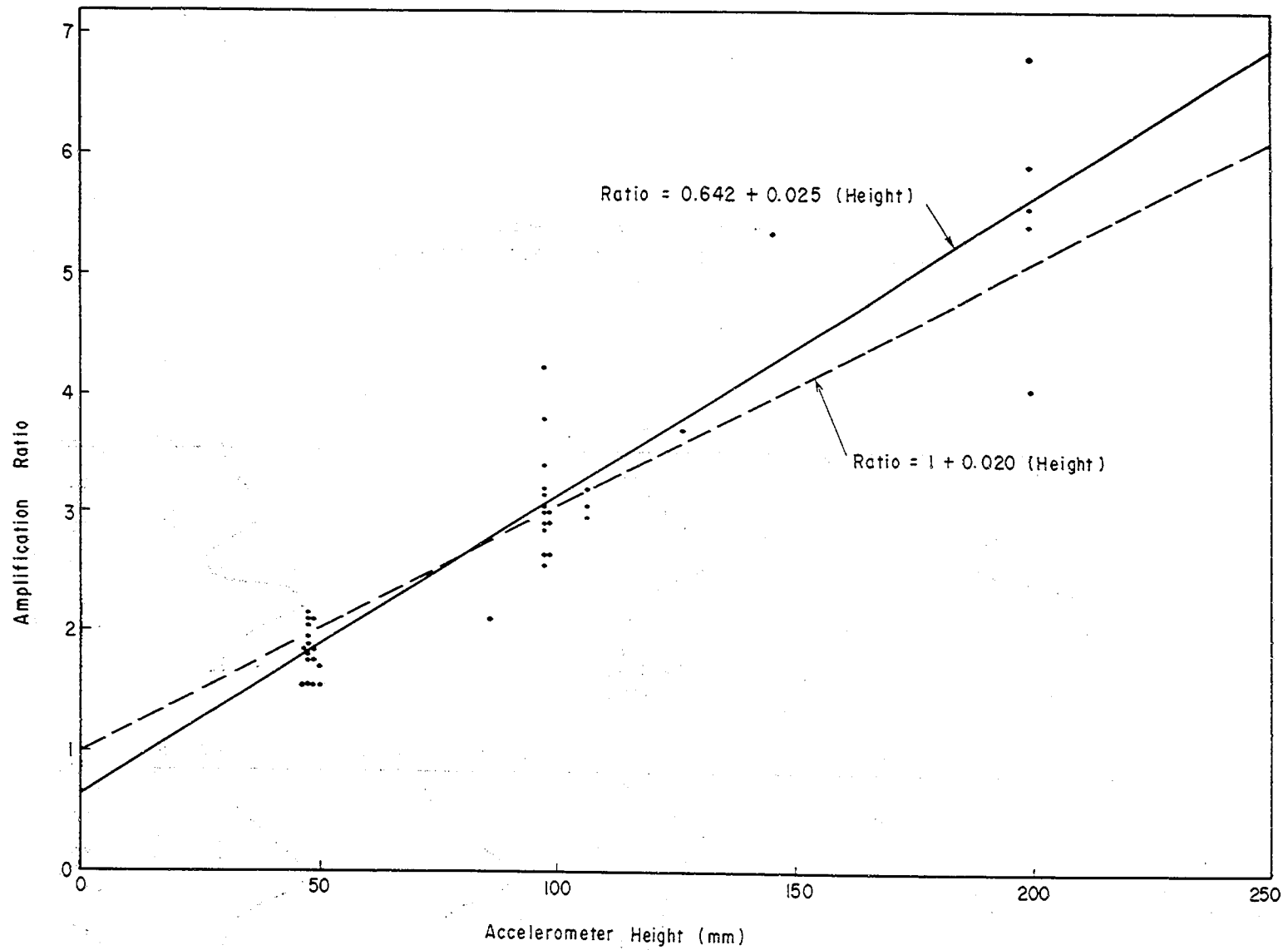


Figure 3.31 Accelerometer height versus Amplification ratio.

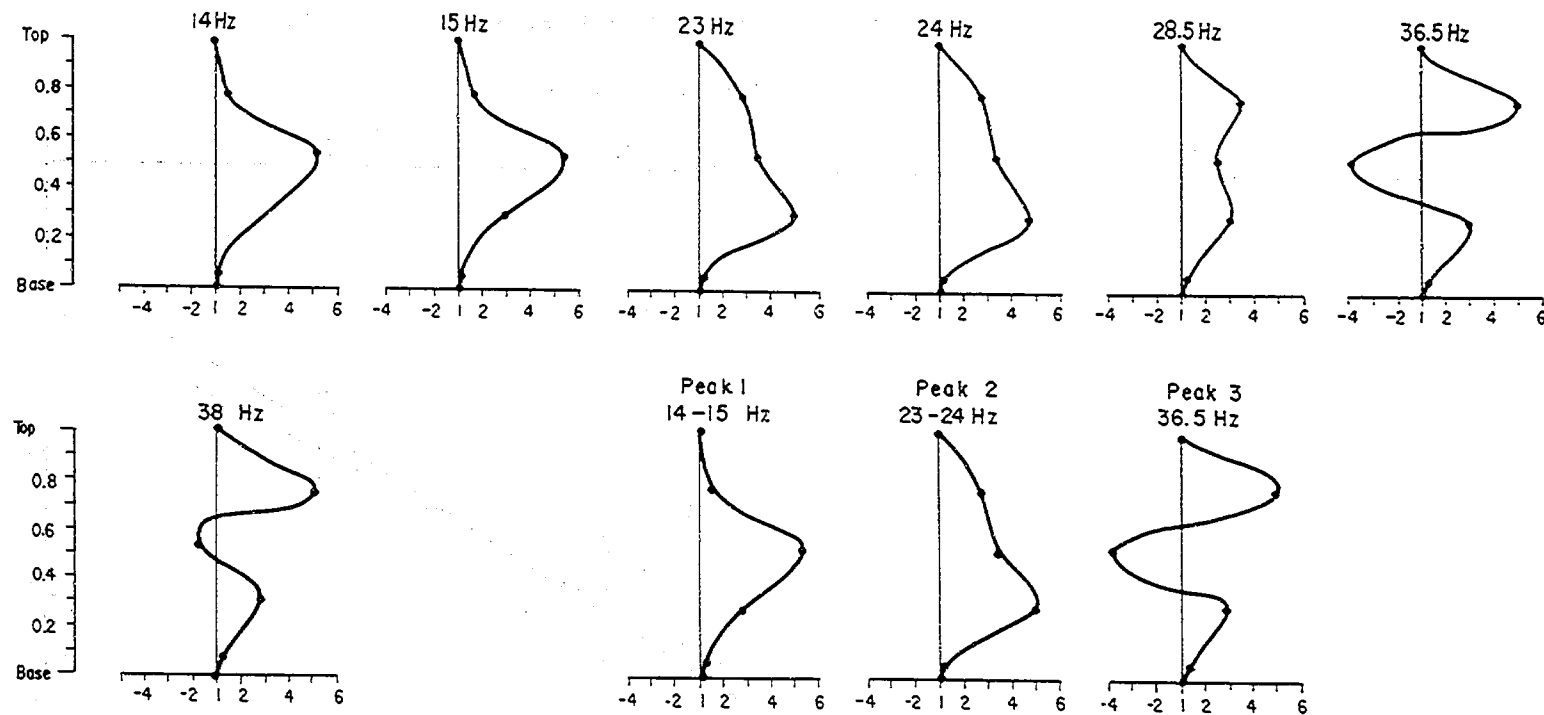


Figure 3.32 Acceleration ratio plotted along ridge crestline, case 1, transverse shaking.

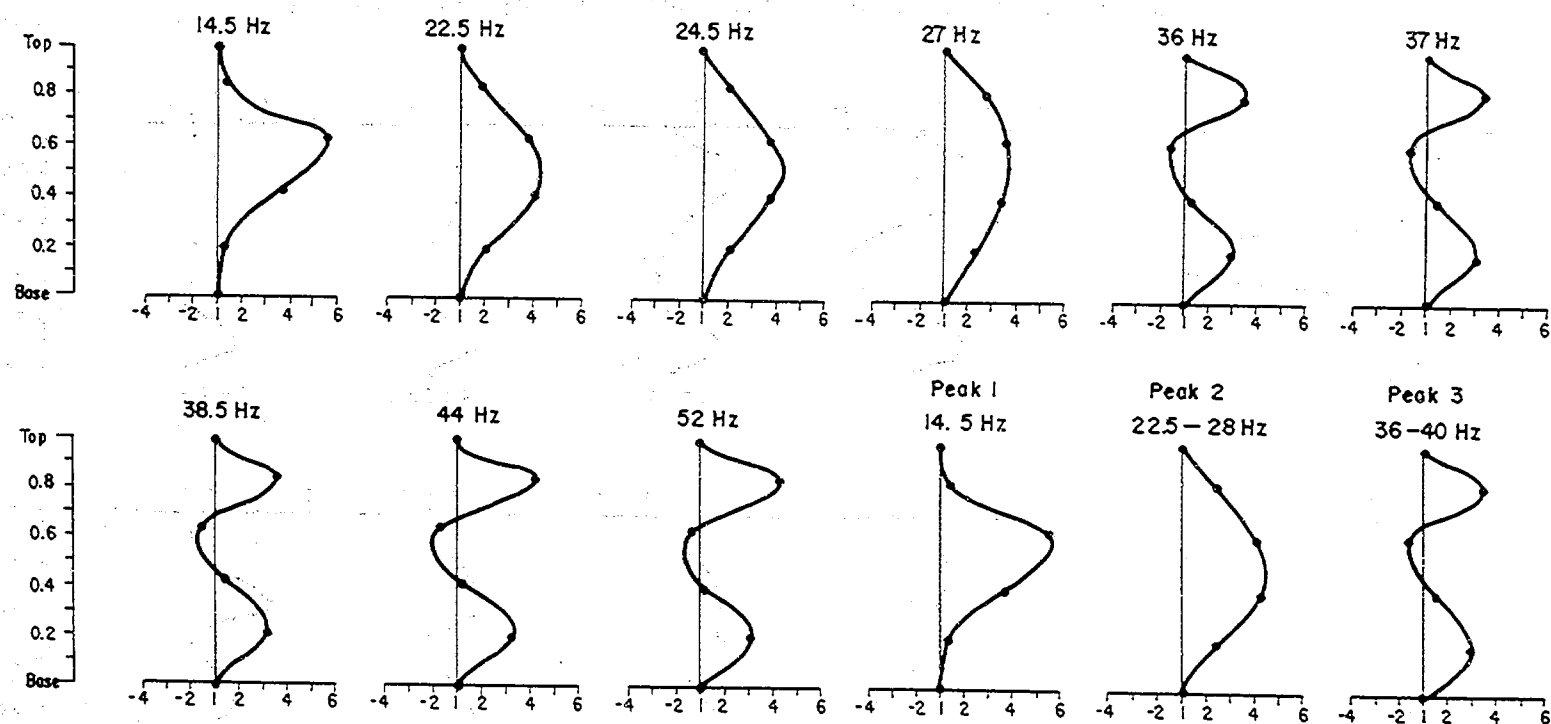


Figure 3.33 Acceleration ratio plotted along ridge crestline, case 2, transverse shaking.

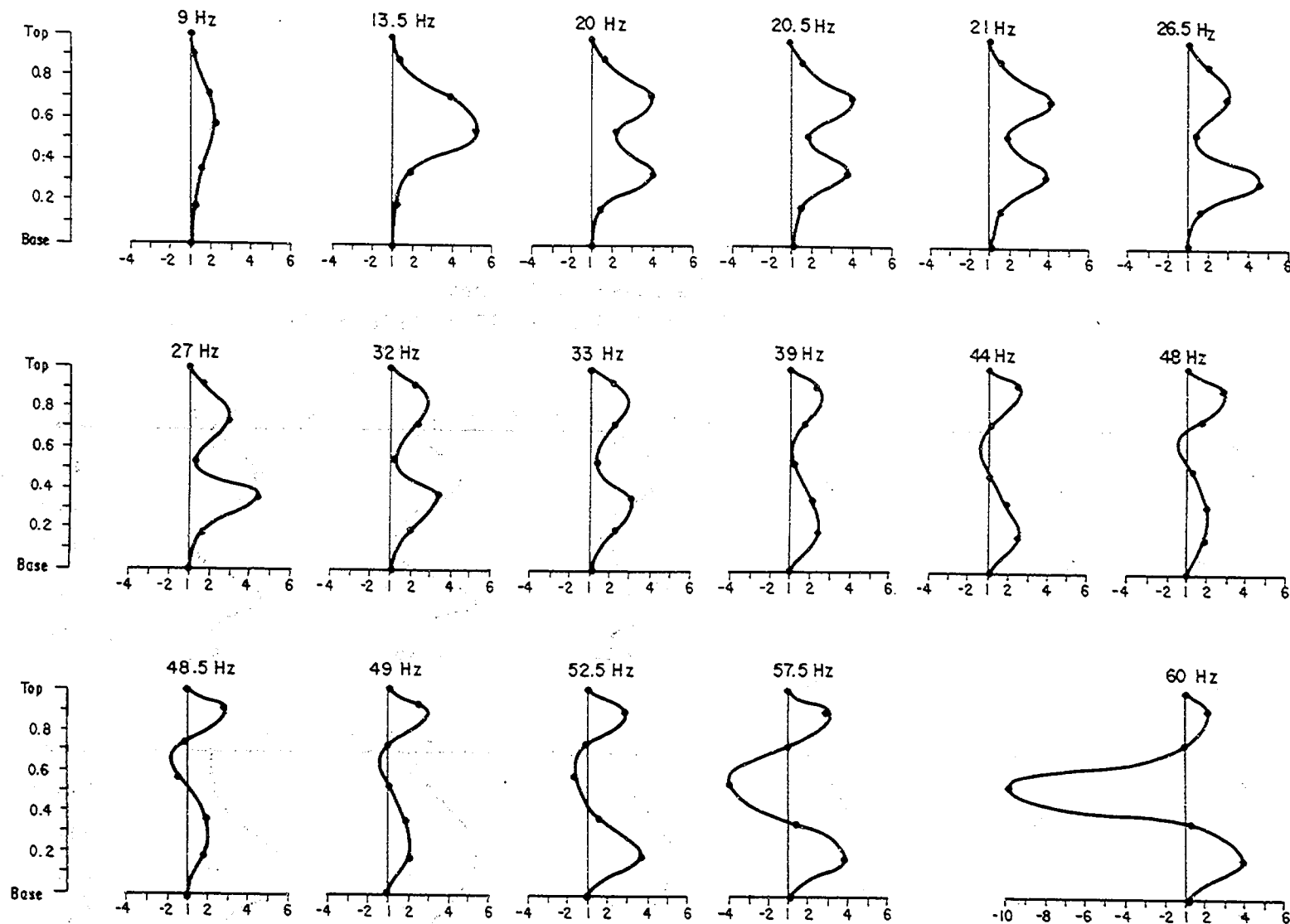


Figure 3.34 Acceleration ratio plotted along ridge crestline, case 3, transverse shaking.

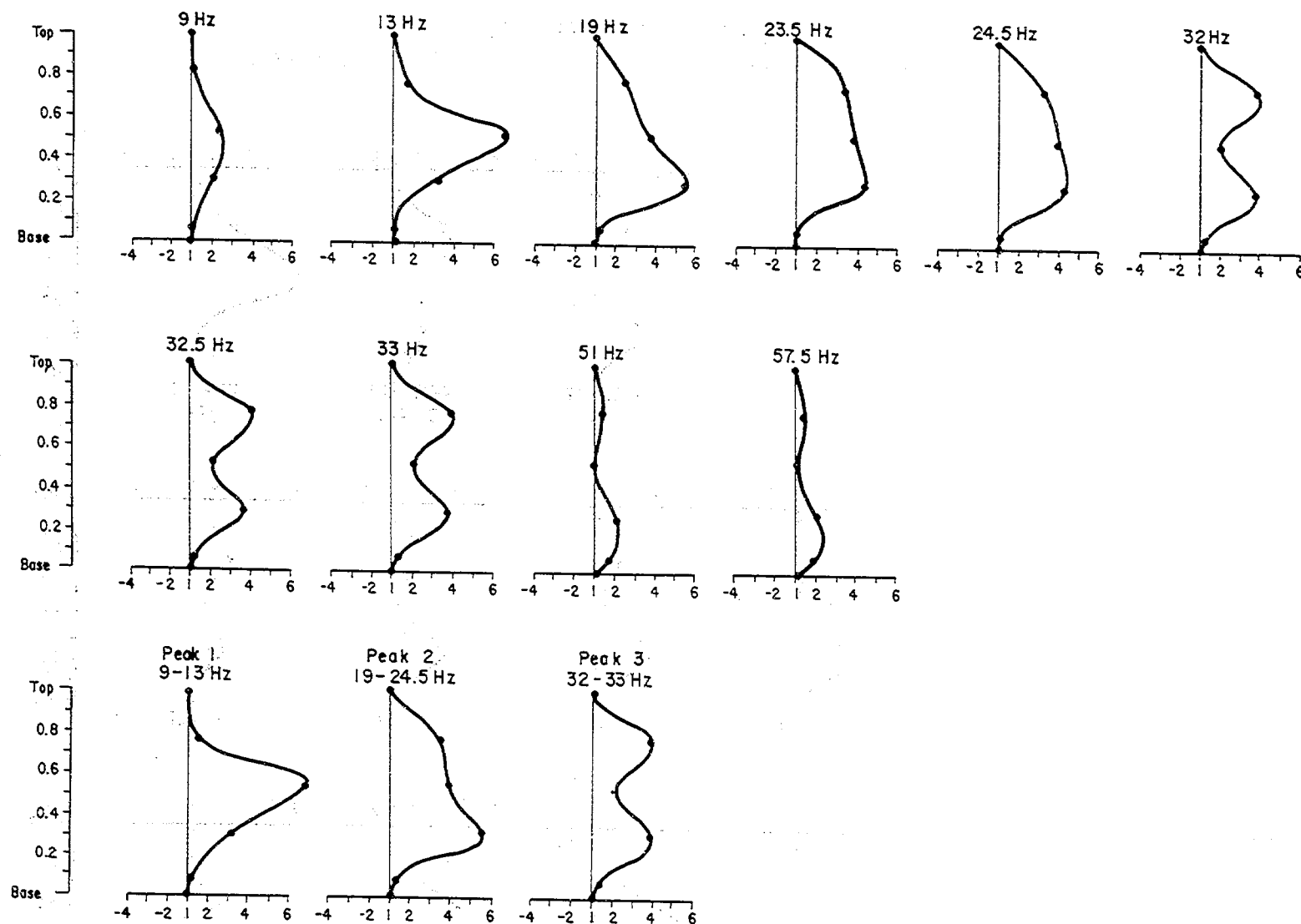


Figure 3.35 Acceleration ratio plotted along ridge crestline, case 4, transverse shaking.

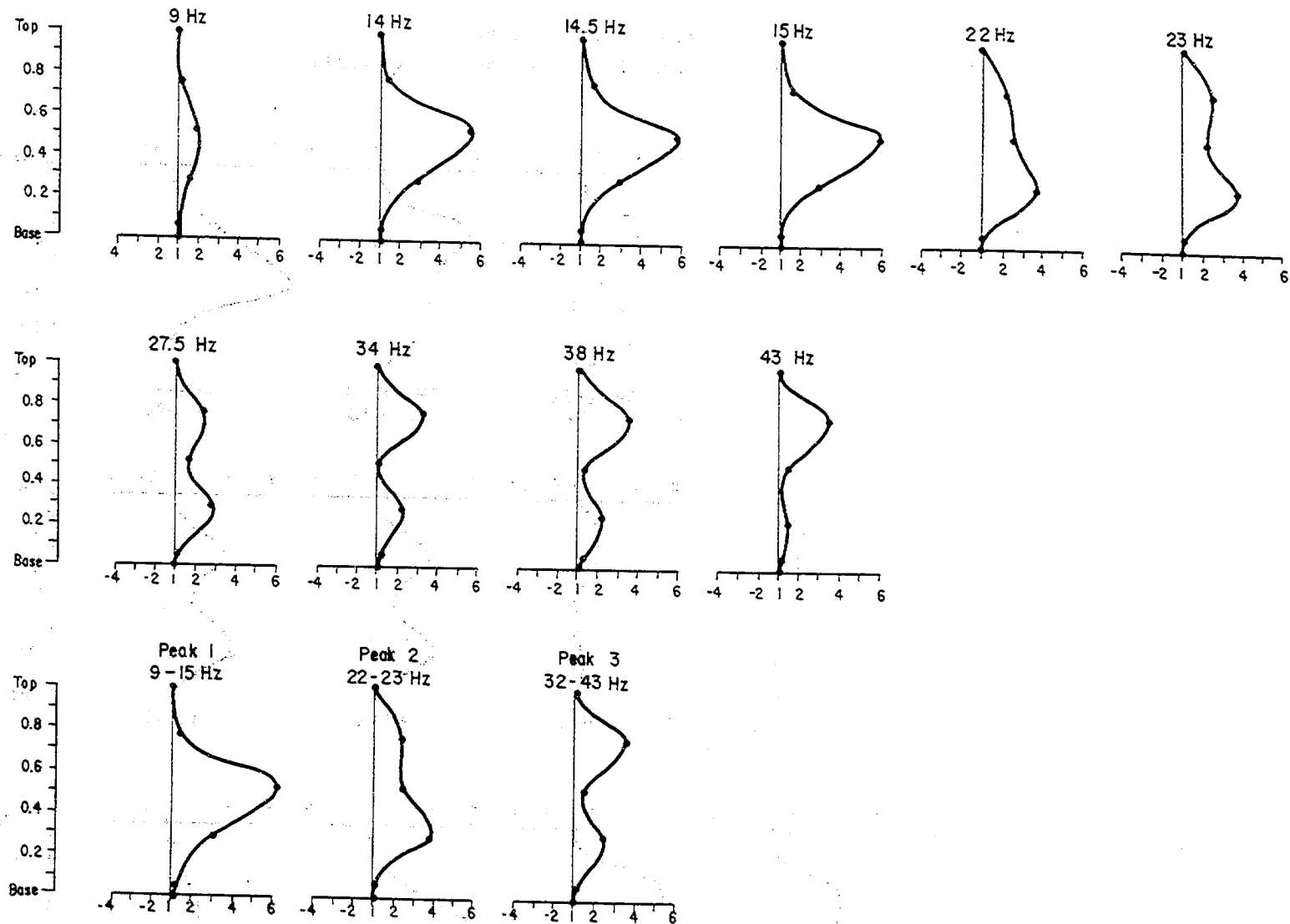


Figure 3.36 Acceleration ratio plotted along ridge crestline, case 5, transverse shaking.

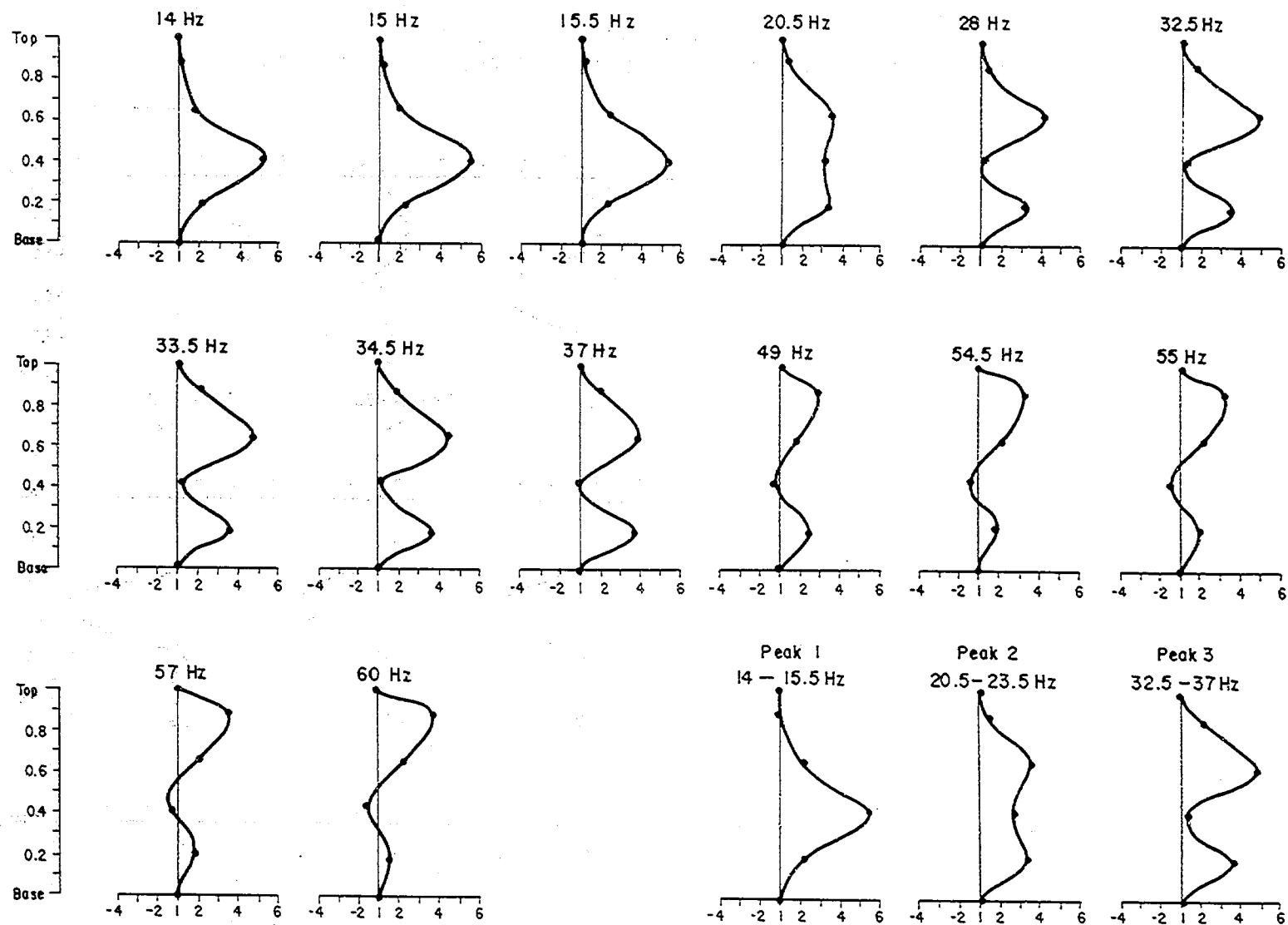


Figure 3.37 Acceleration ratio plotted along ridge crestline, case 6, transverse shaking.

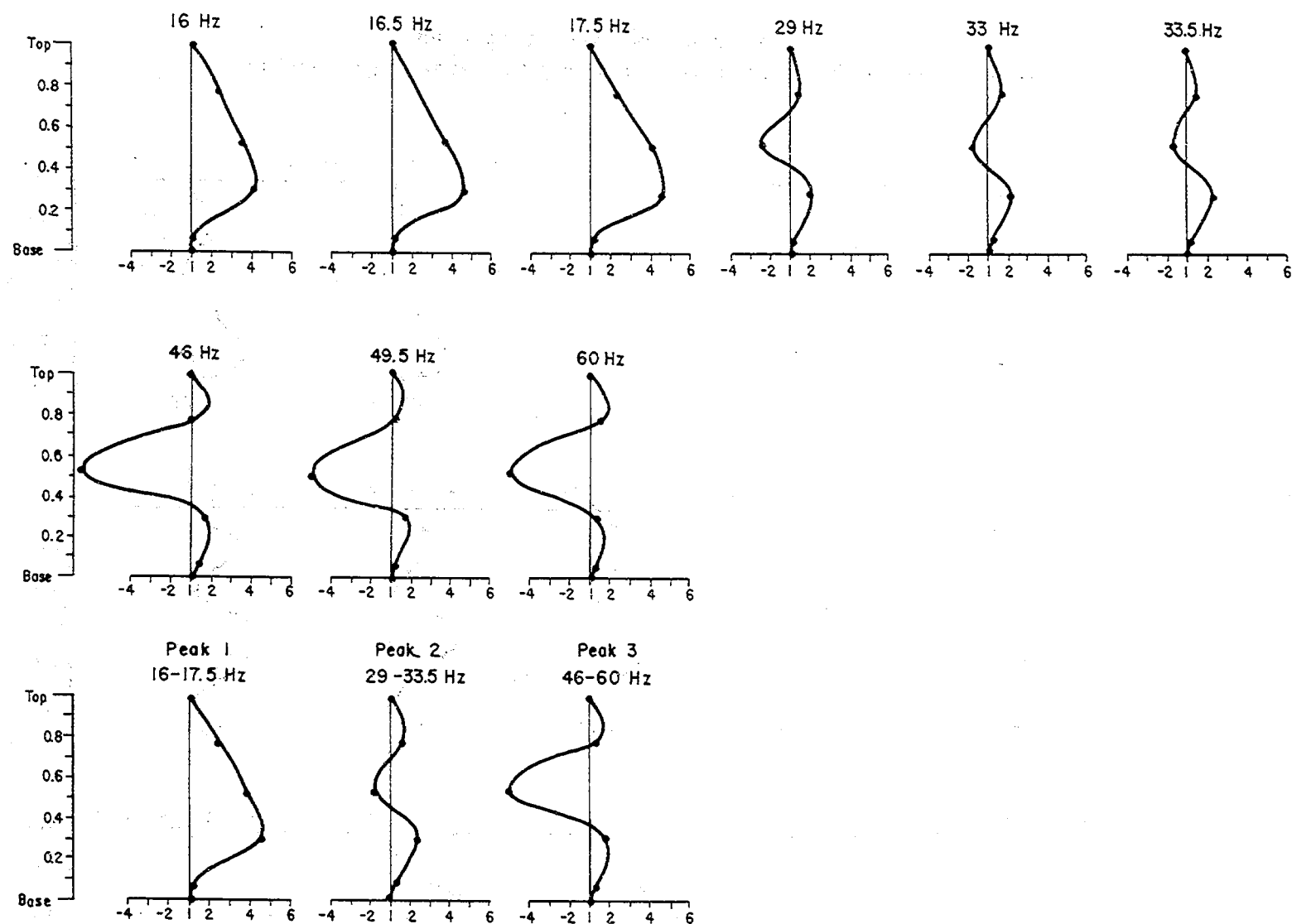


Figure 3.38 Acceleration ratio plotted along ridge crestline, case 1, longitudinal shaking.

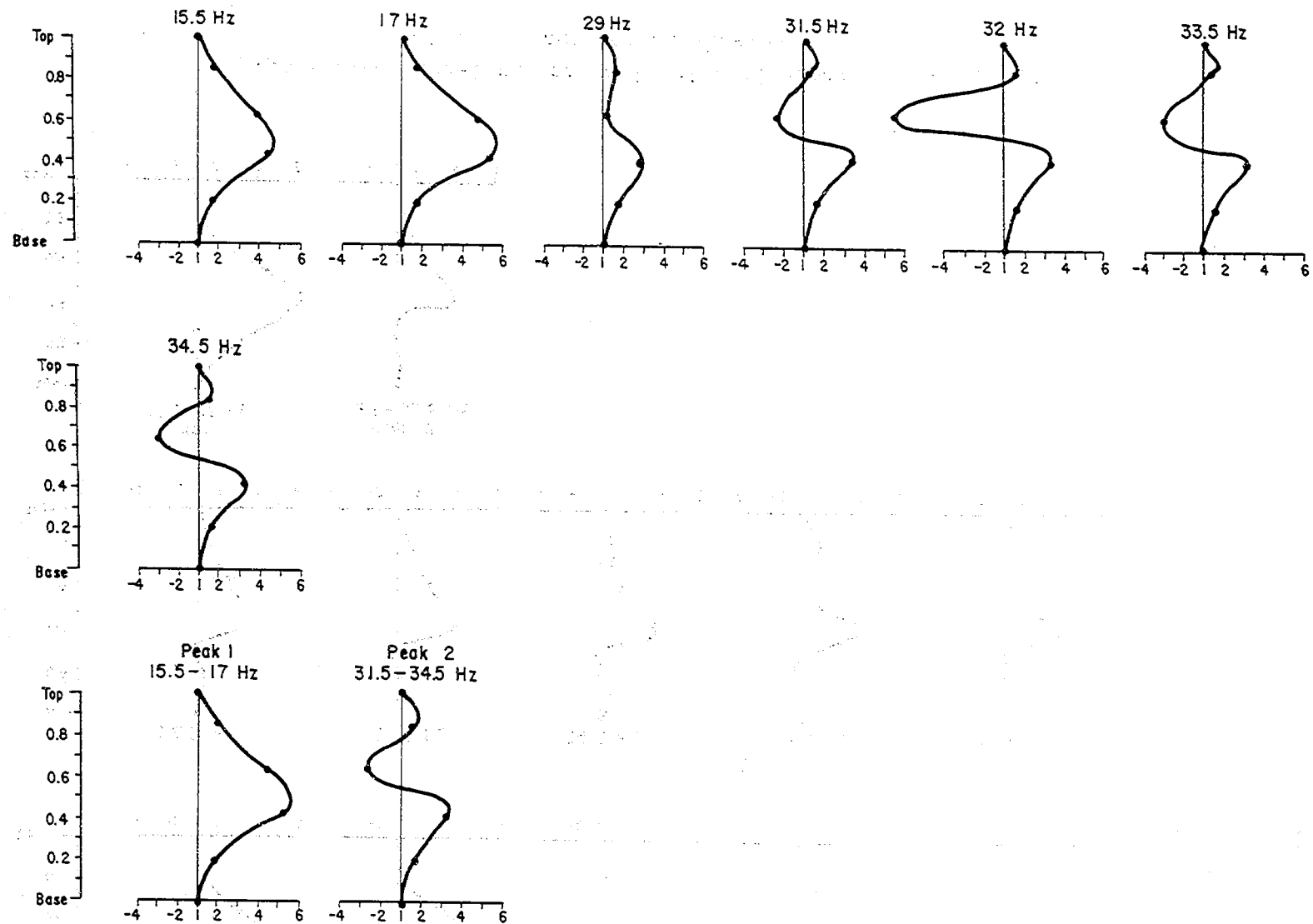


Figure 3.39 Acceleration ratio plotted along ridge crestline, case 2, longitudinal shaking.

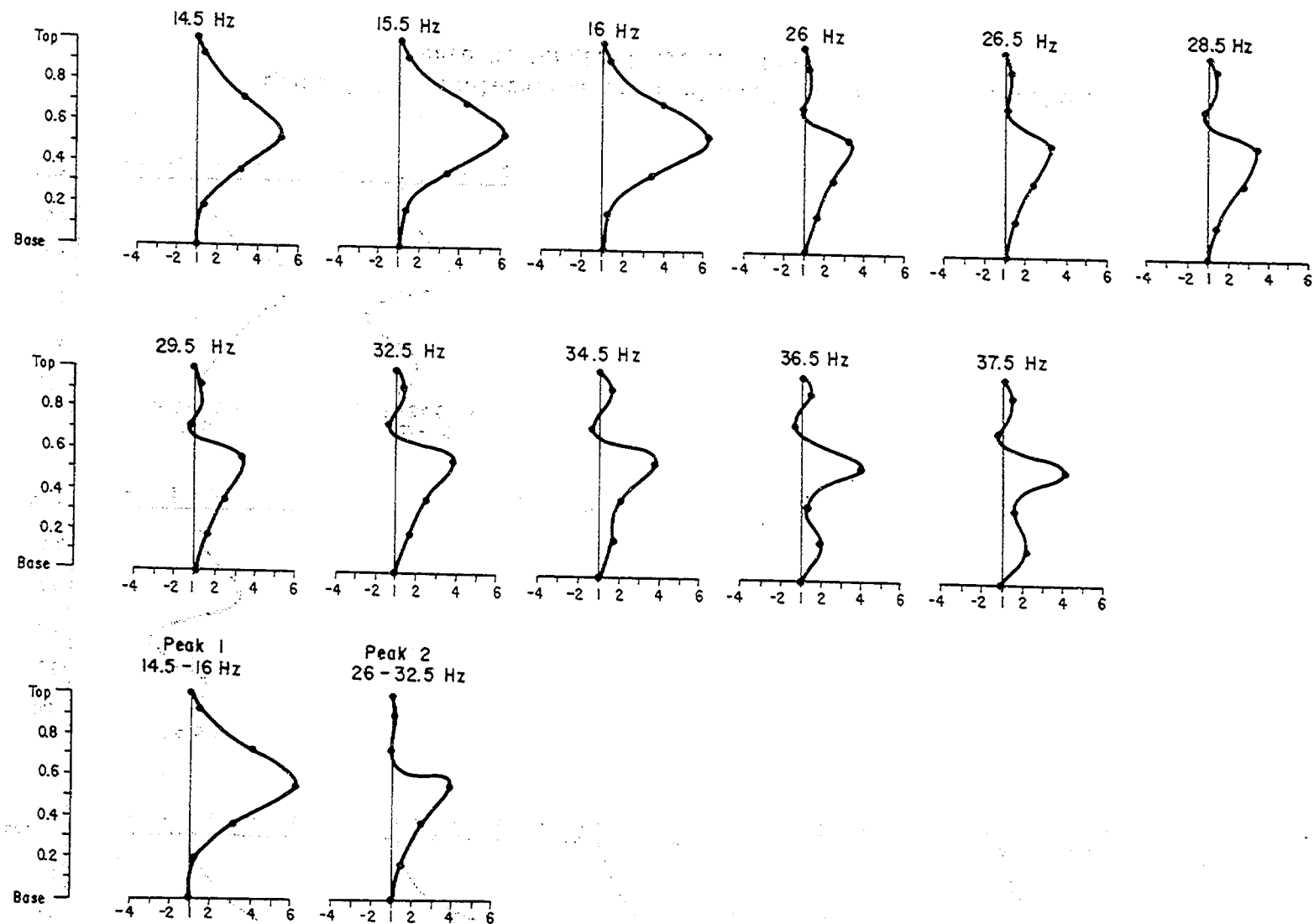


Figure 3.40 Acceleration ratio plotted along ridge crestline, case 3, longitudinal shaking.

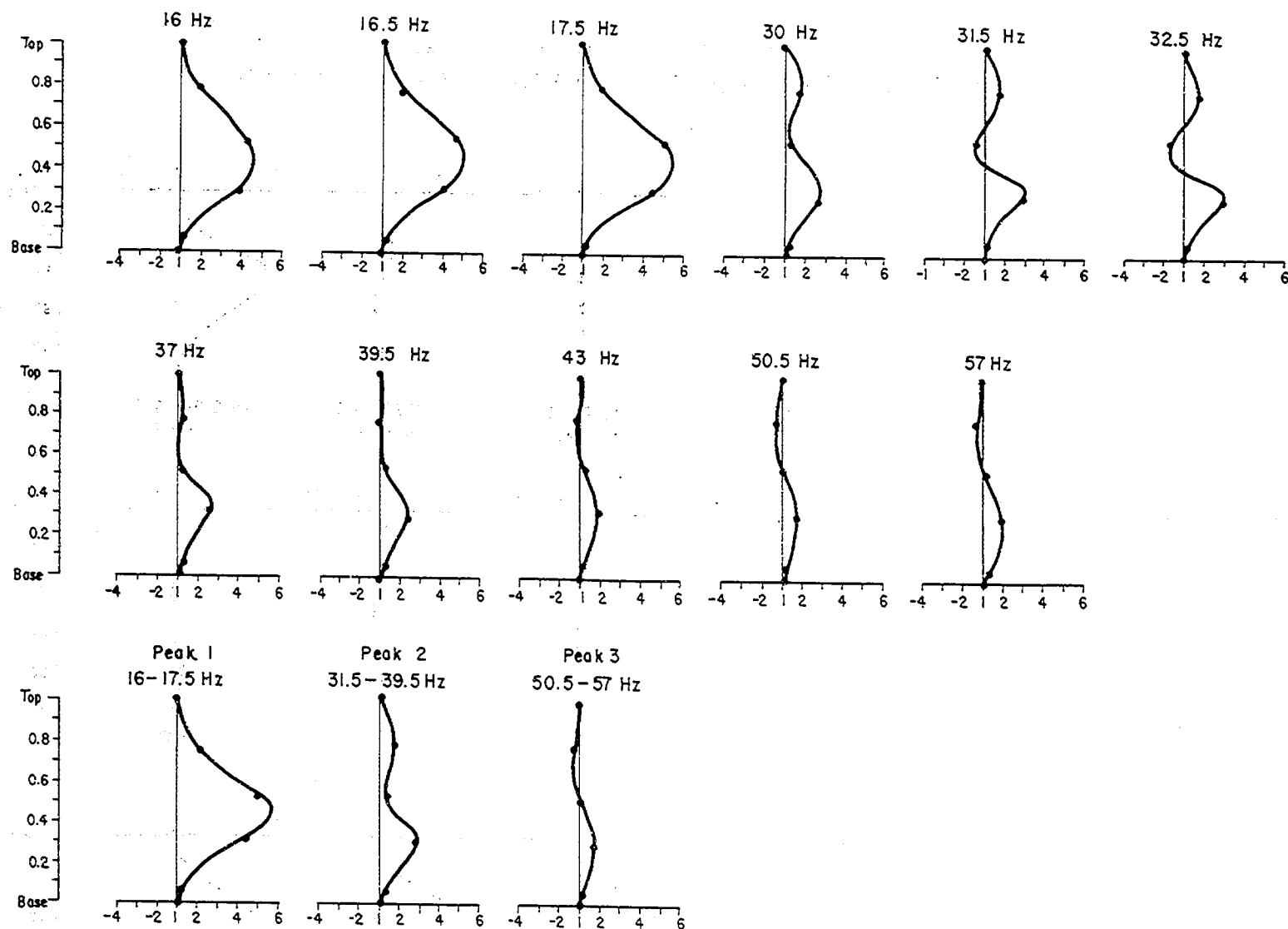


Figure 3.41 Acceleration ratio plotted along ridge crestline, case 4, longitudinal shaking.

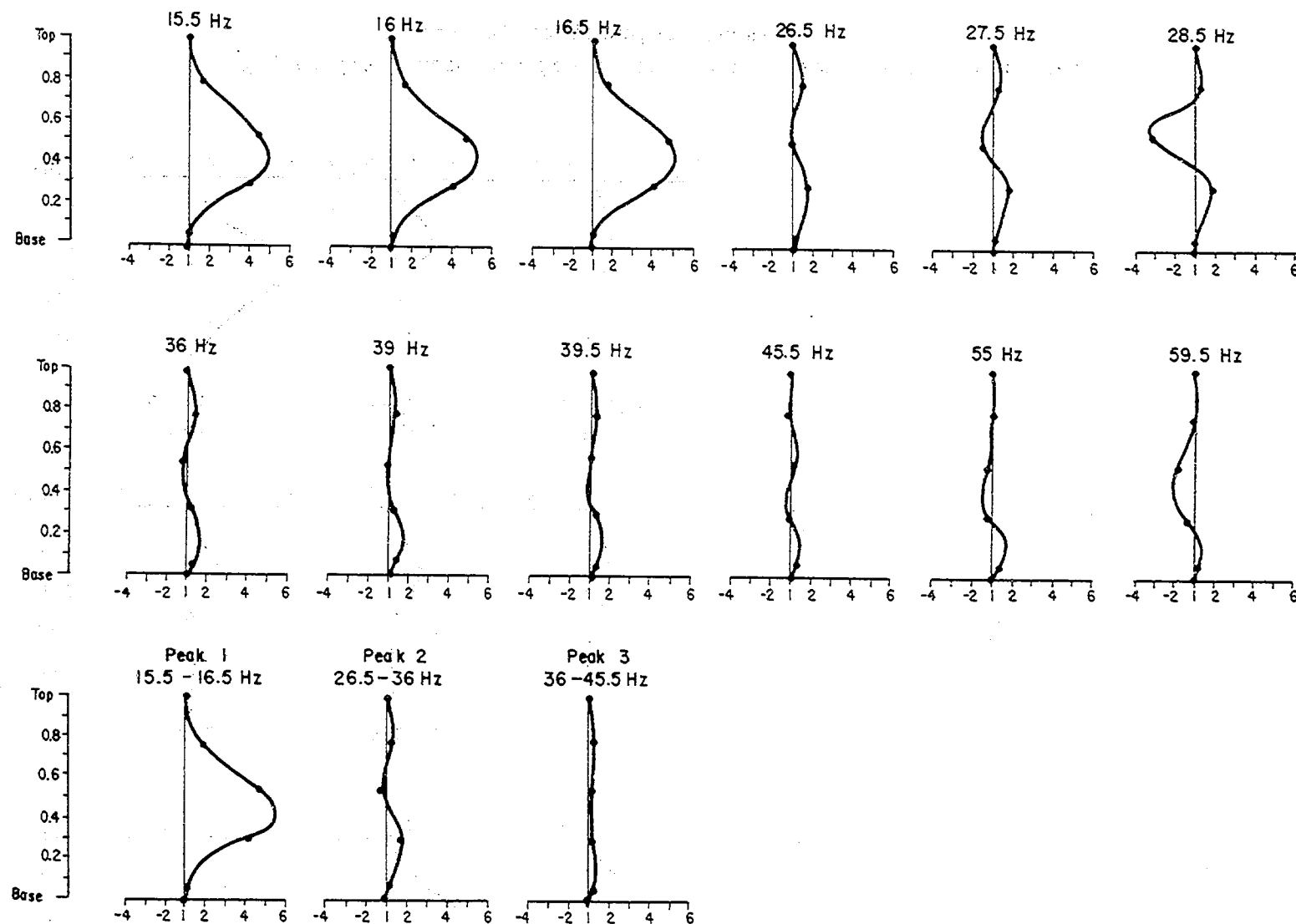


Figure 3.42 Acceleration ratio plotted along ridge crestline, case 5, longitudinal shaking.

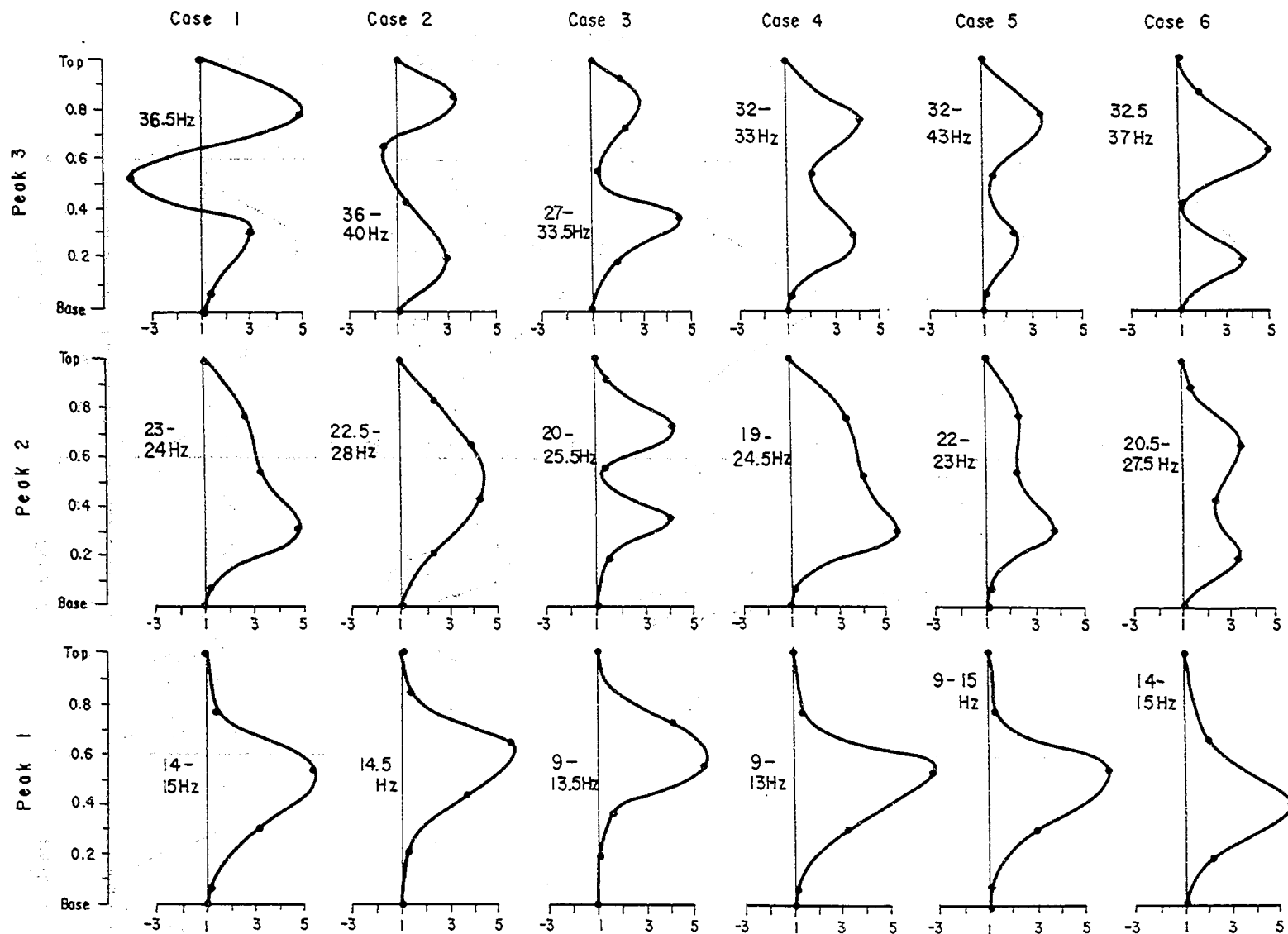


Figure 3.43 Acceleration ratio plotted along ridge crestline for three major response peaks, transverse shaking.

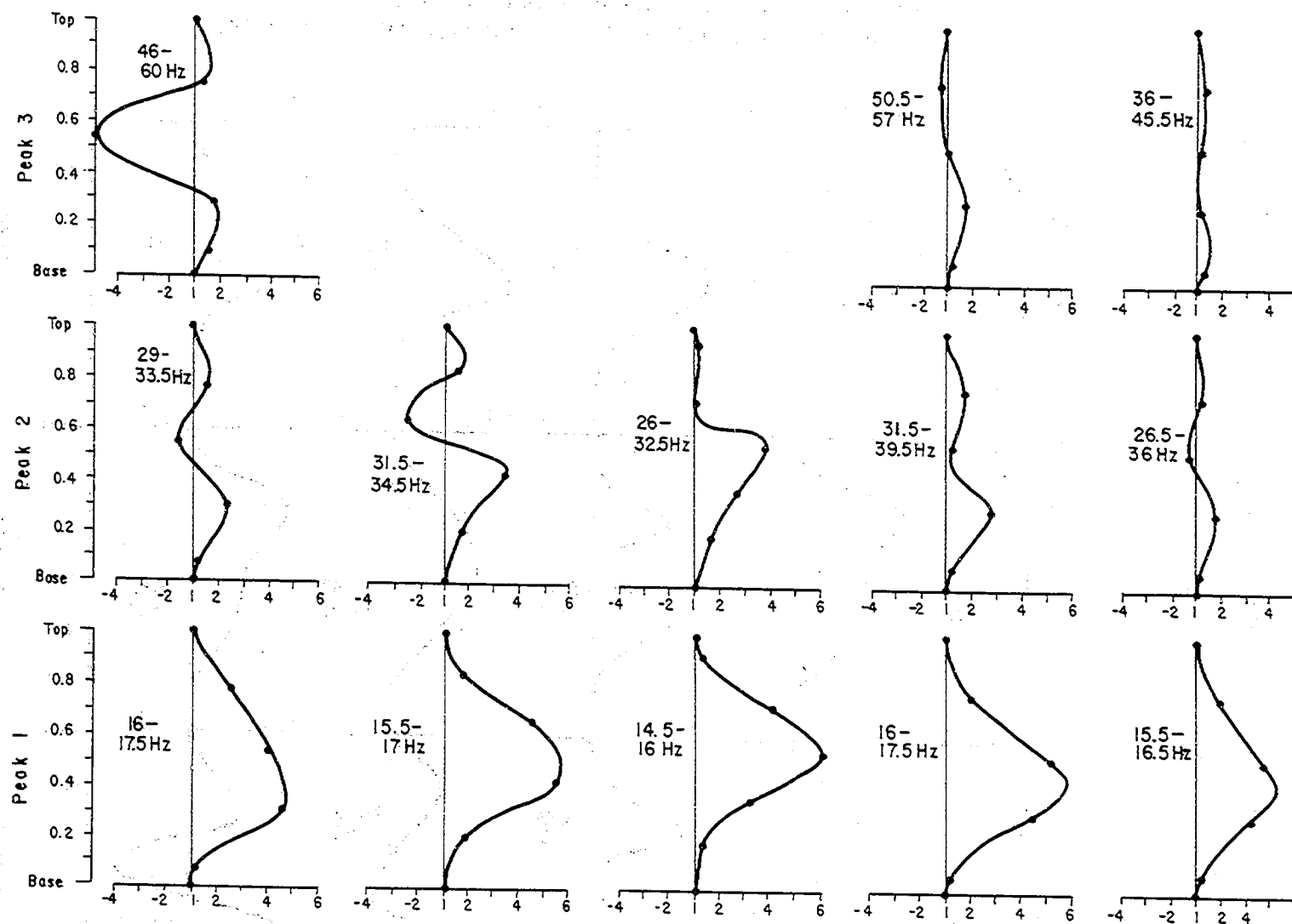


Figure 3.44 Acceleration ratio plotted along ridge crestline for three major response peaks, longitudinal shaking.

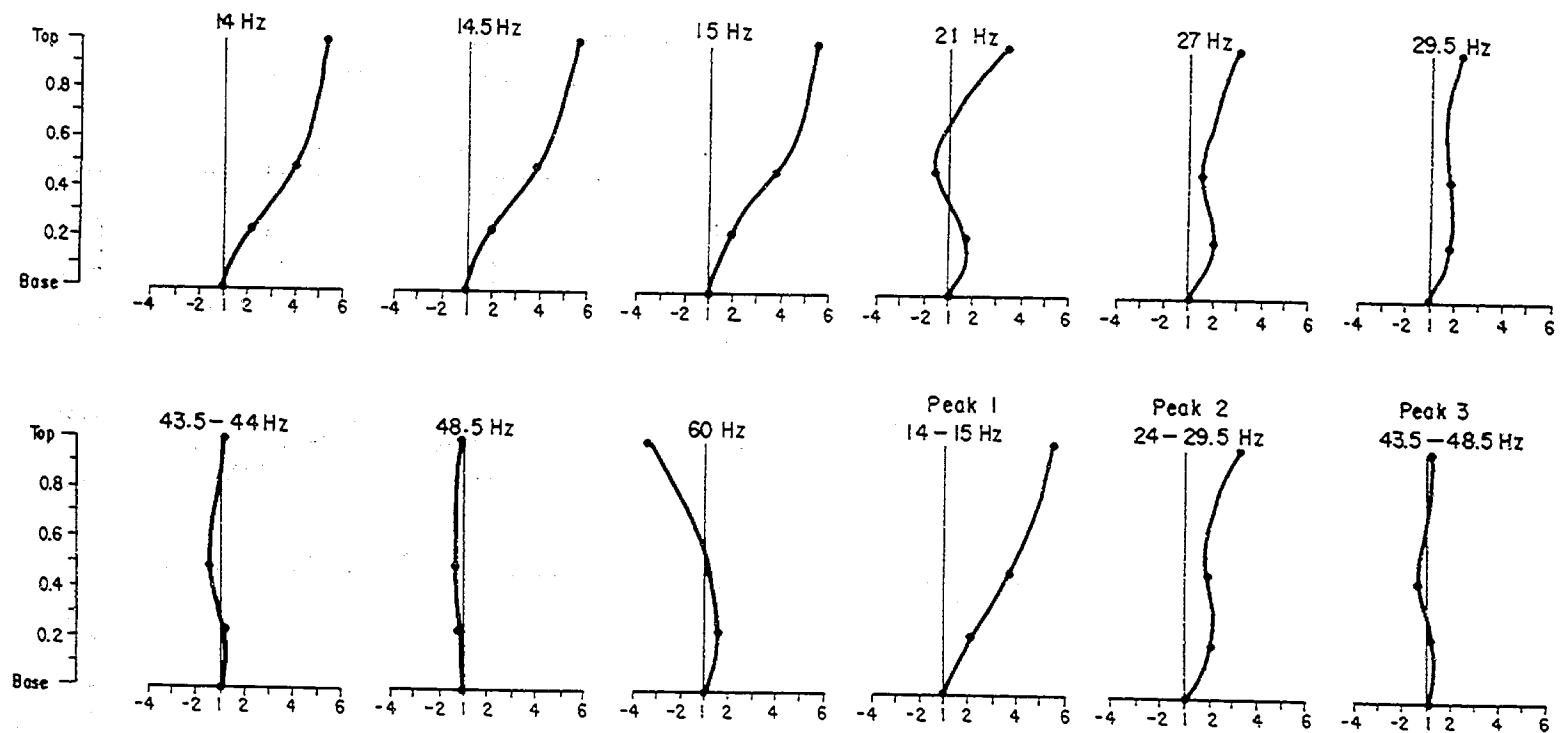


Figure 3.45 Acceleration ratio plotted along maximum vertical cross section, case 1, transverse shaking.

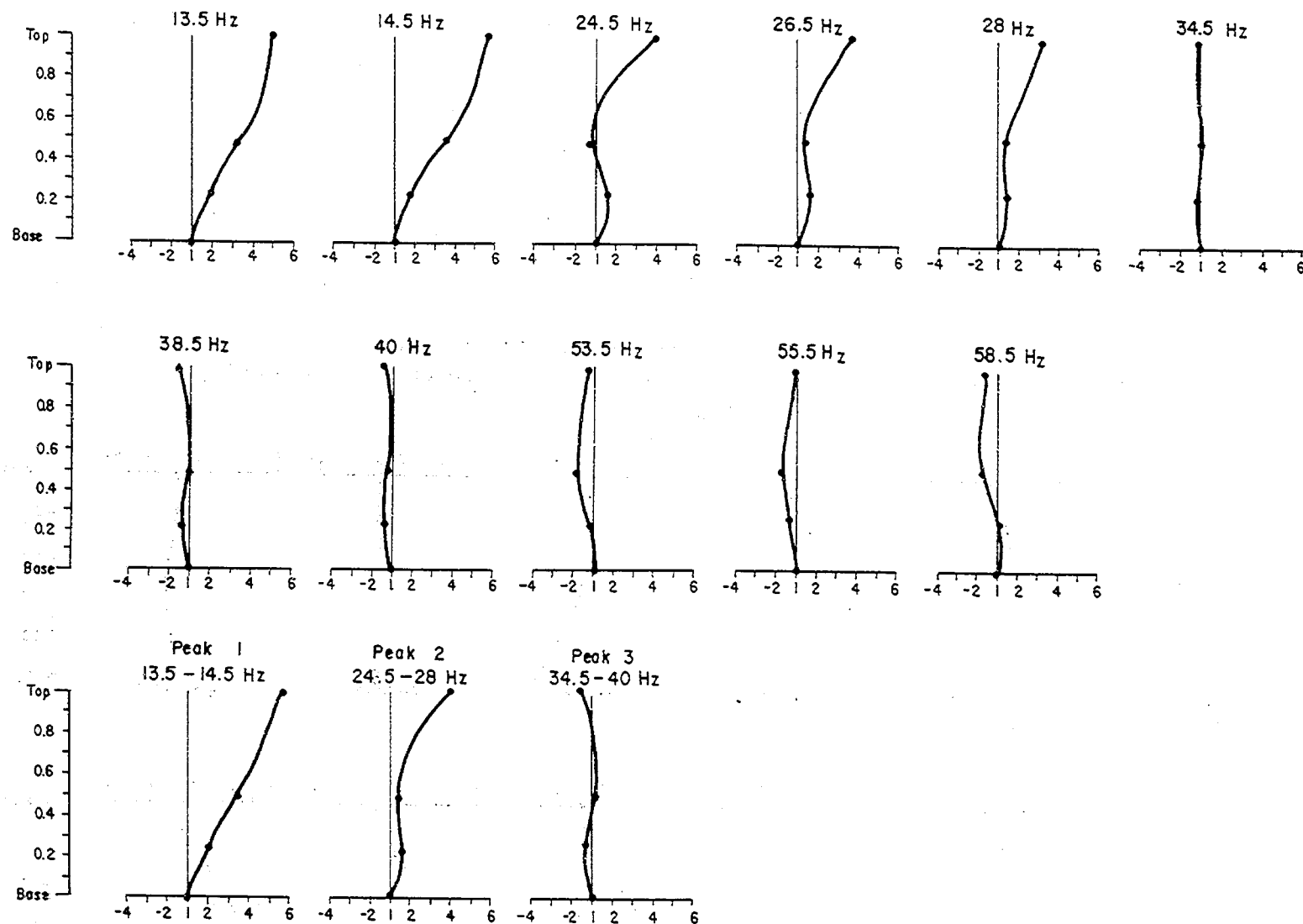


Figure 3.46 Acceleration ratio plotted along maximum vertical cross section, case 2, transverse shaking.

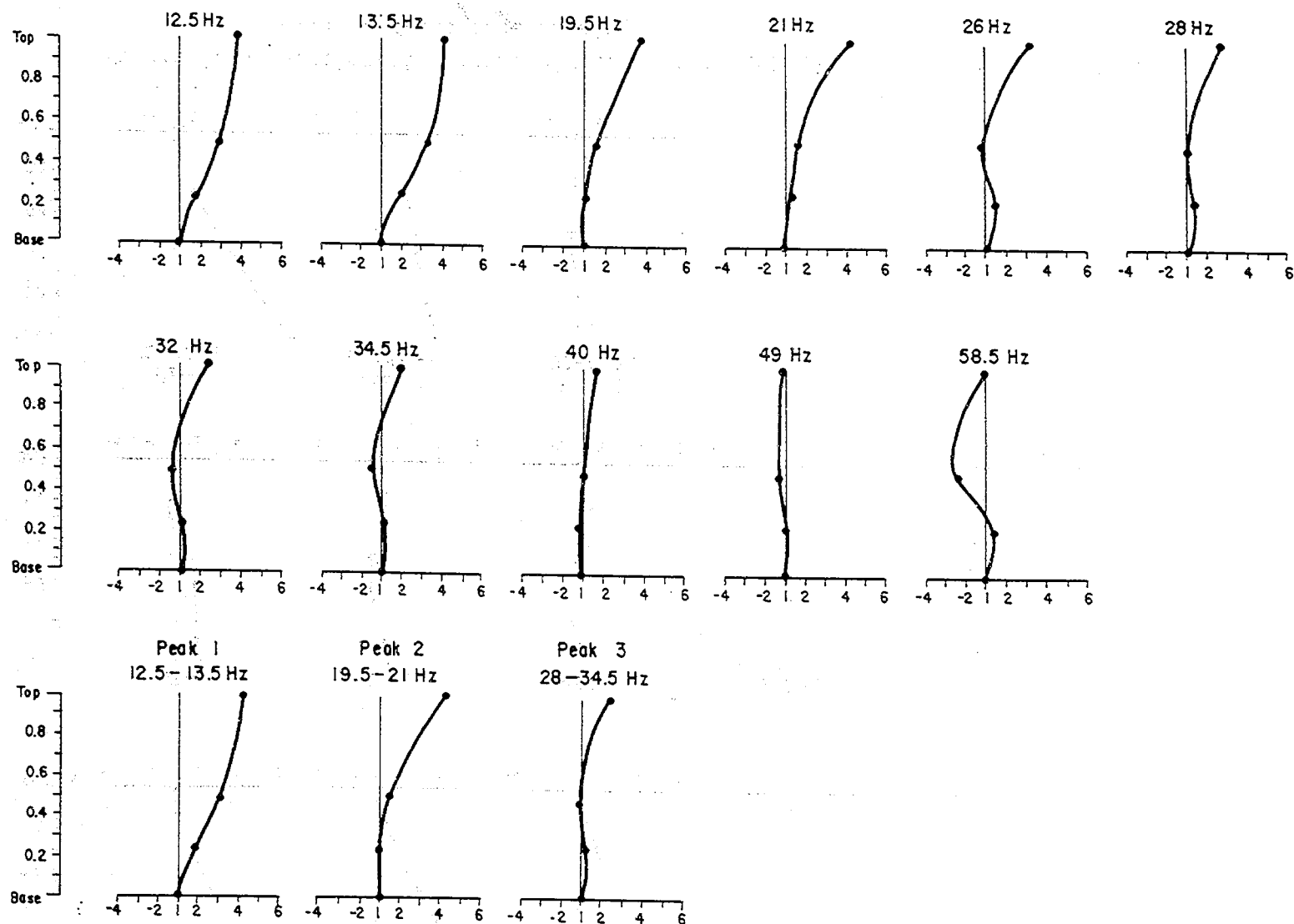


Figure 3.47 Acceleration ratio plotted along maximum vertical cross section, case 3, transverse shaking.

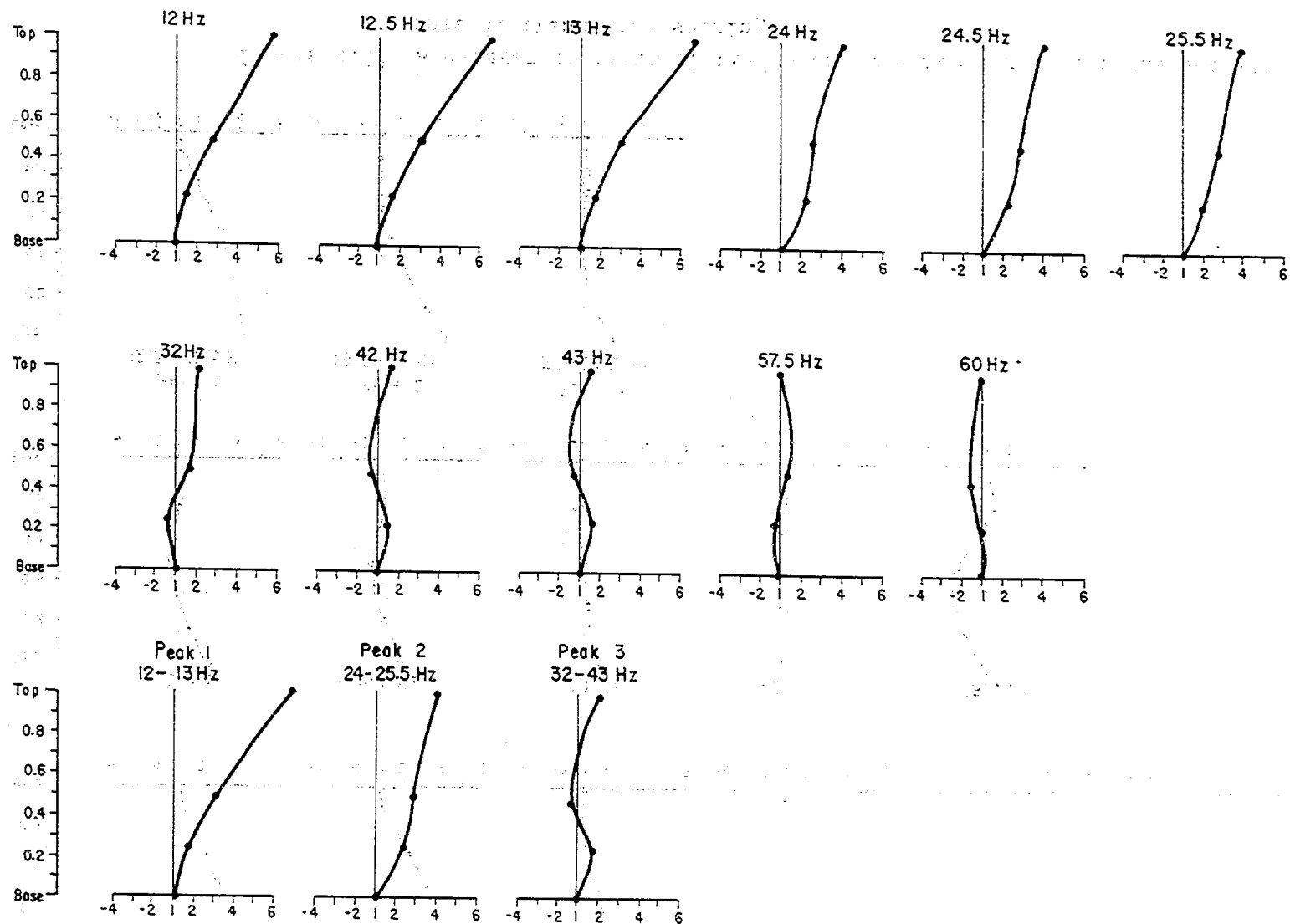


Figure 3.48 Acceleration ratio plotted along maximum vertical cross section, case 4, transverse shaking.

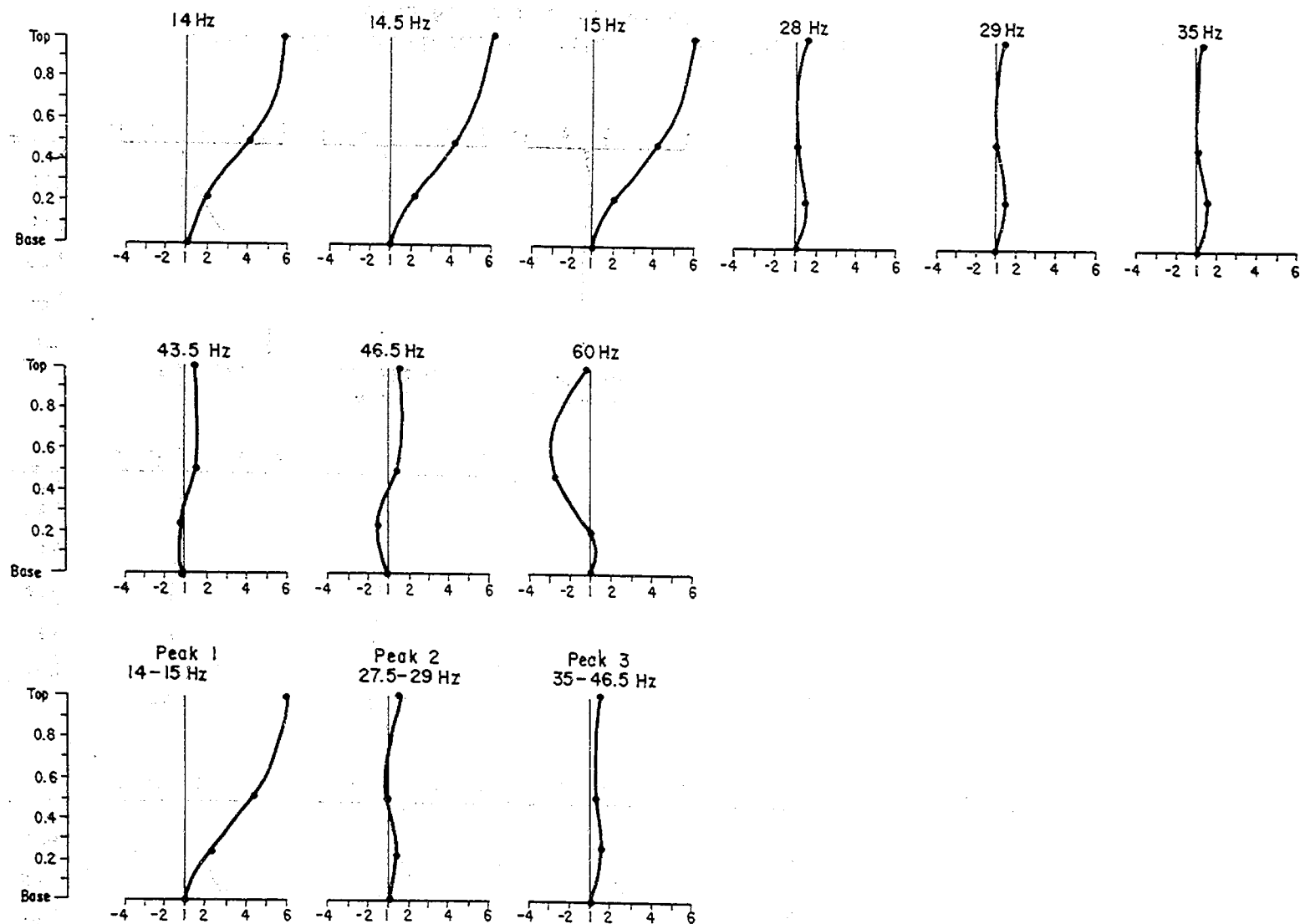


Figure 3.49 Acceleration ratio plotted along maximum vertical cross section, case 5, transverse shaking.

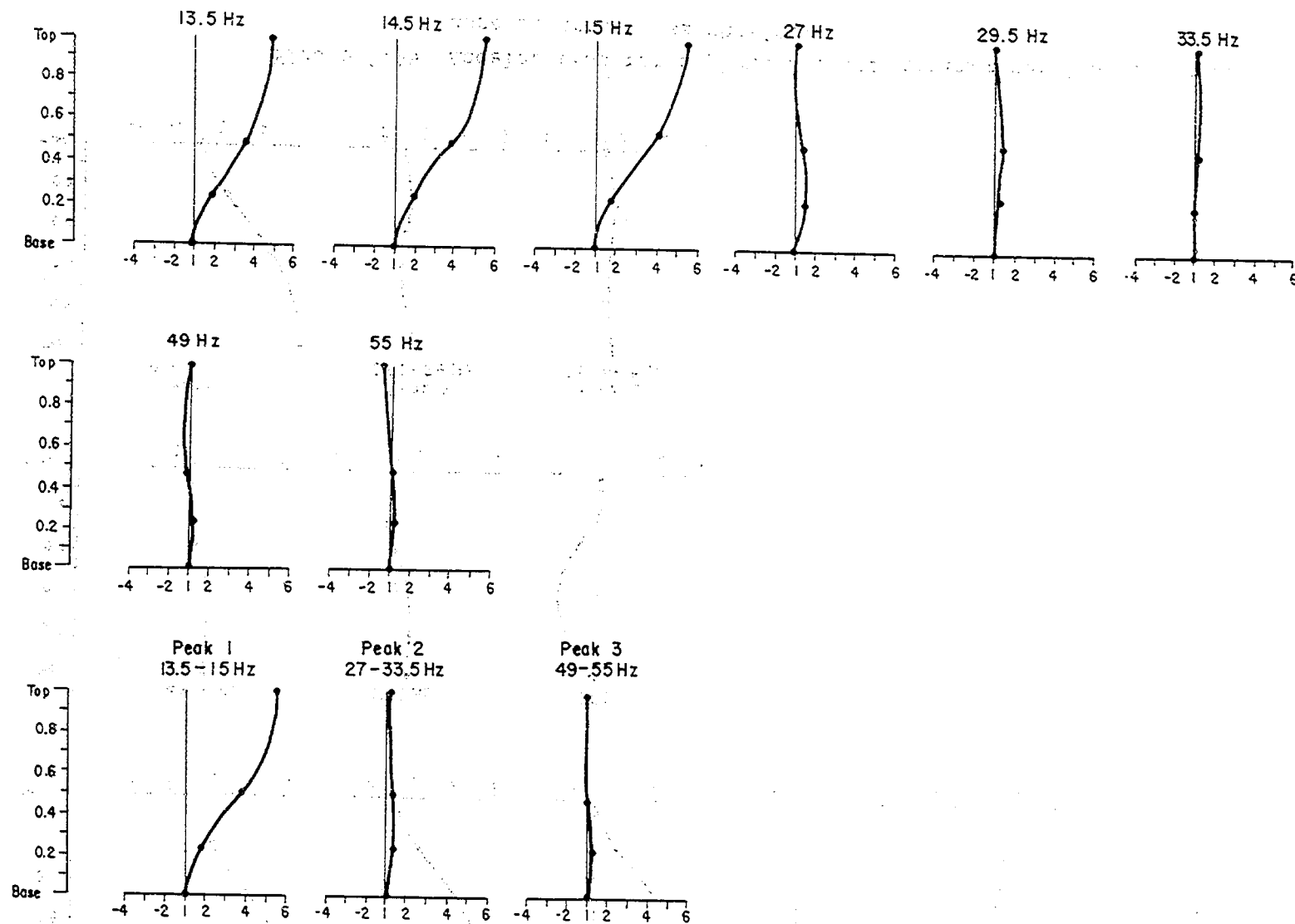


Figure 3.50 Acceleration ratio plotted along maximum vertical cross section, case 6, transverse shaking.

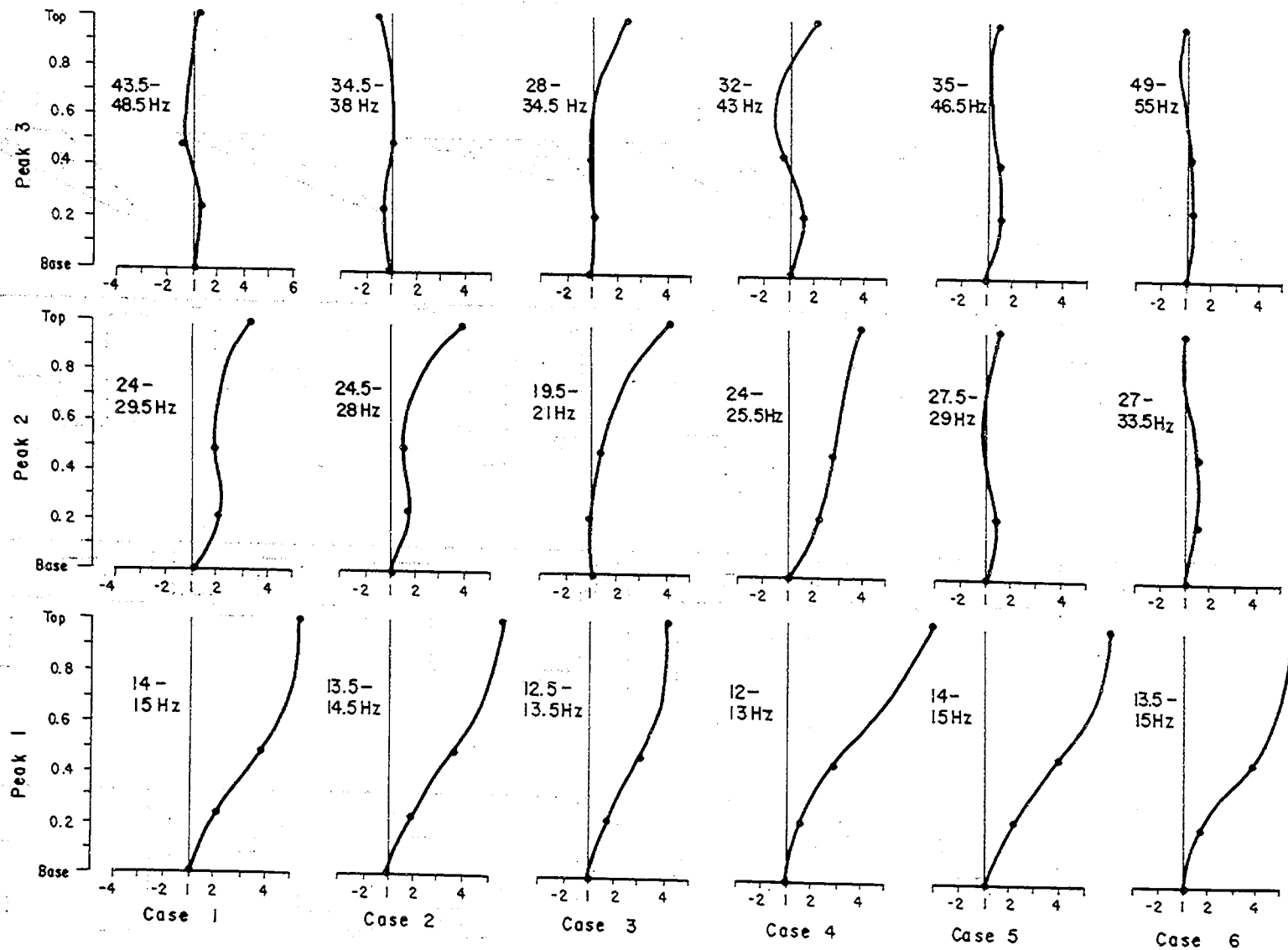


Figure 3.51 Acceleration ratio plotted along maximum vertical cross section for three major response peaks, transverse shaking.

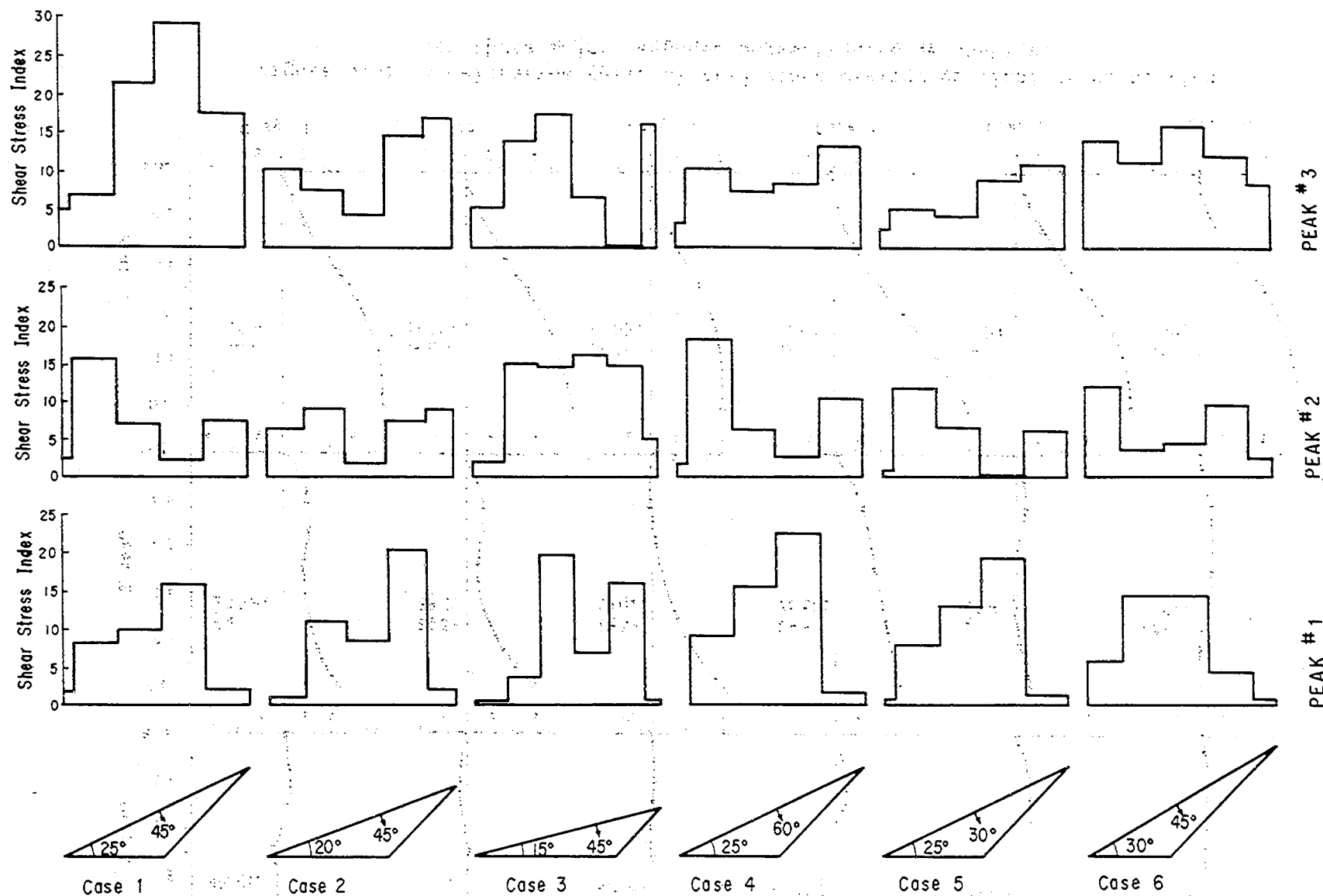


Figure 3.52 Shear-stress Index plotted along ridge crestline, transverse shaking.

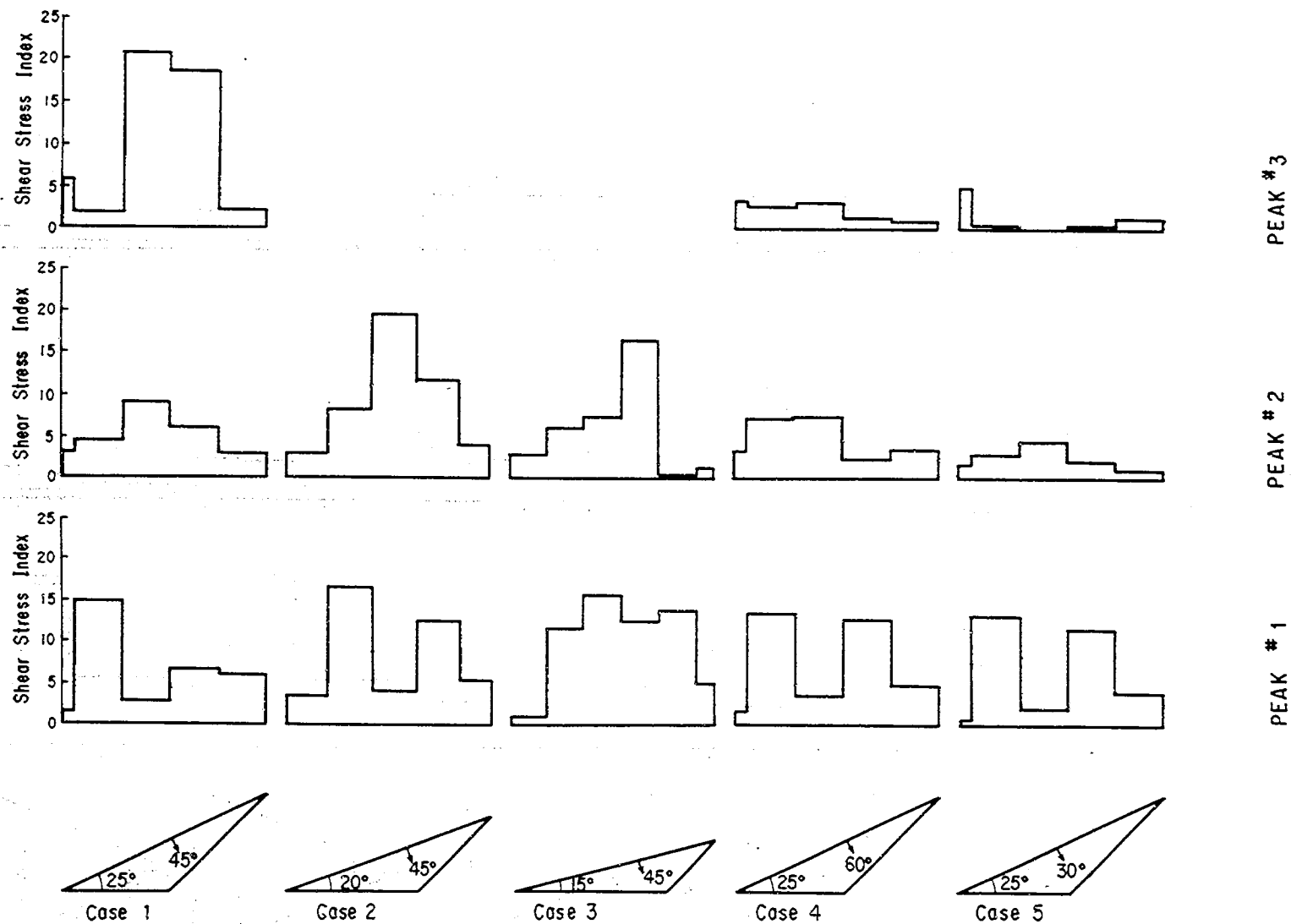


Figure 3.53 Shear-stress Index plotted along ridge crestline, longitudinal shaking.

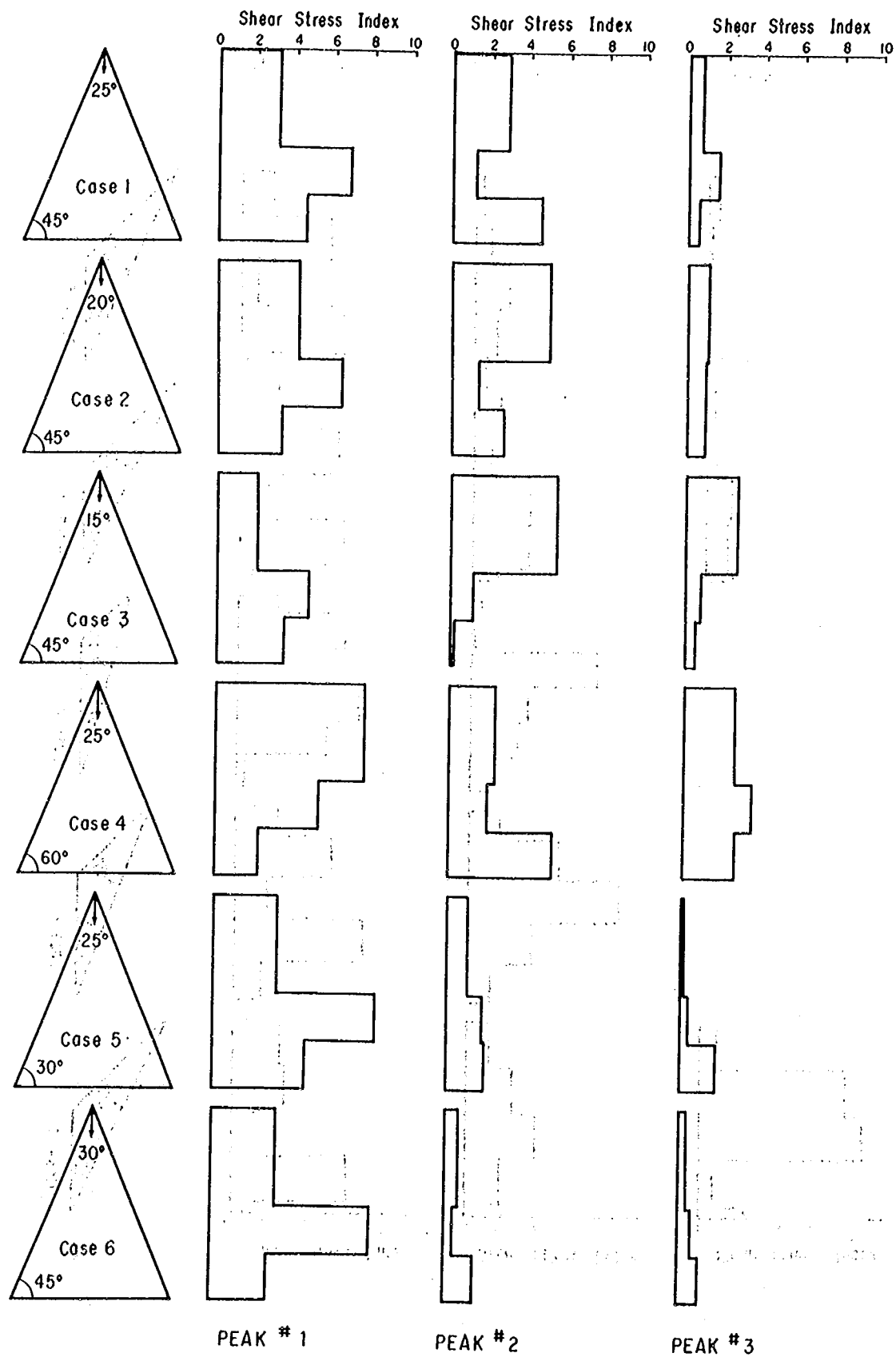


Figure 3.54 Shear-stress Index plotted along maximum vertical cross section, transverse shaking.

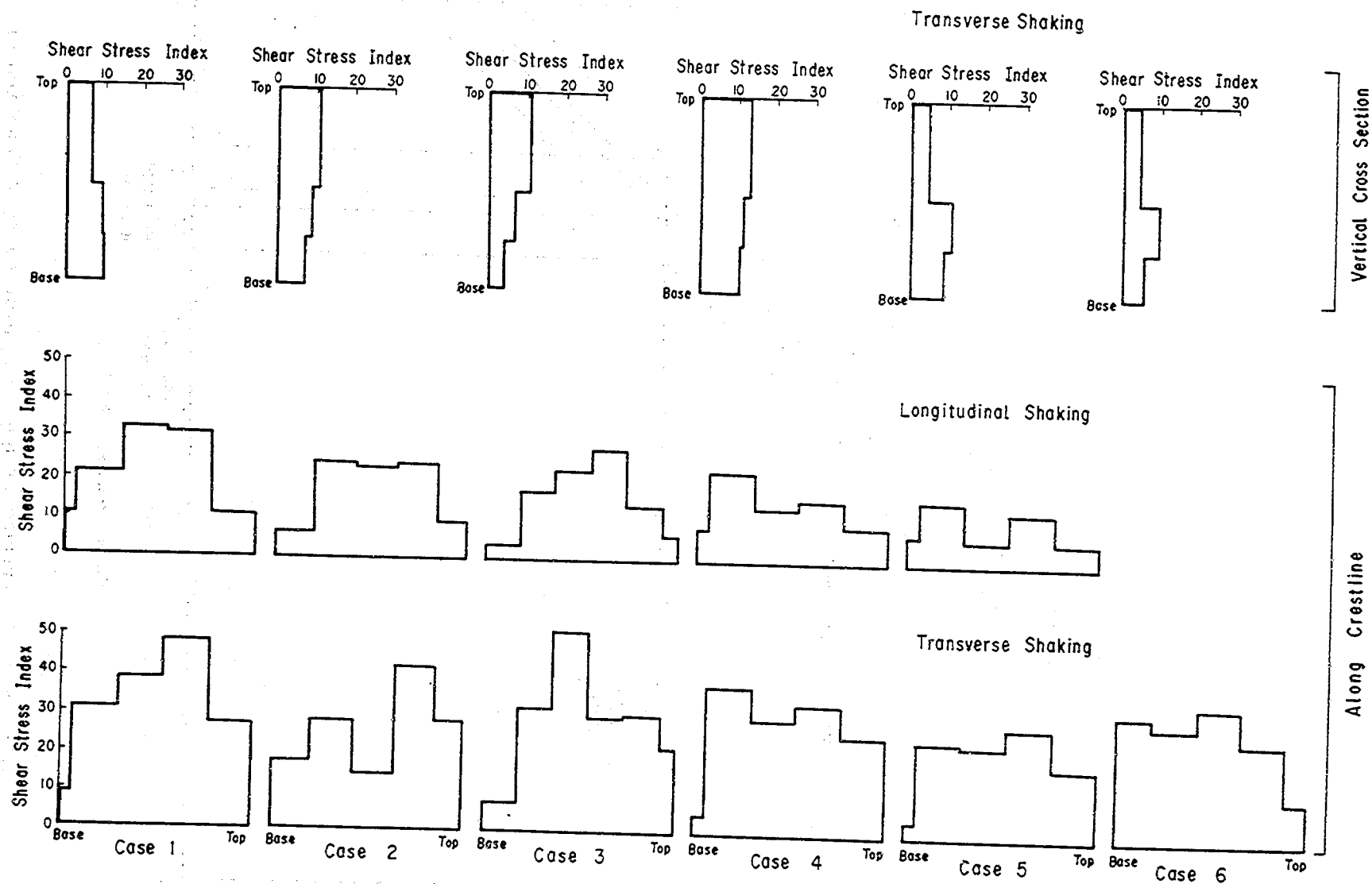


Figure 3.55 Sum of Shear stress Index for three major response peaks shown in figures 52-54.

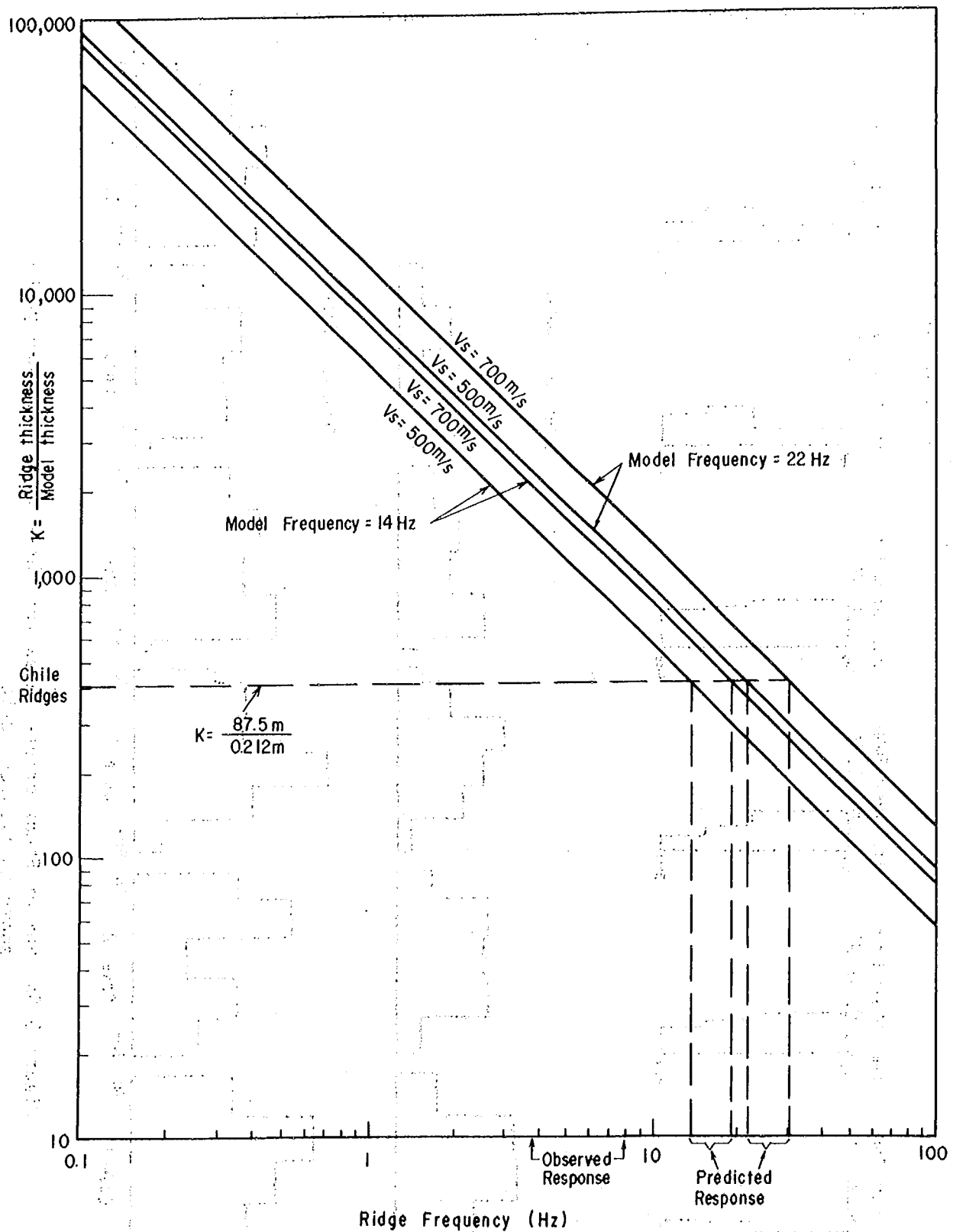
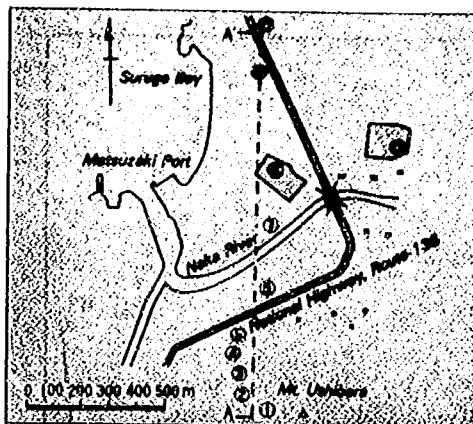
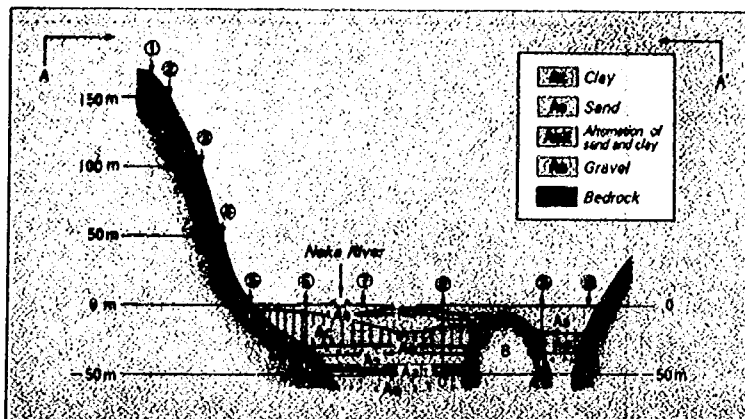


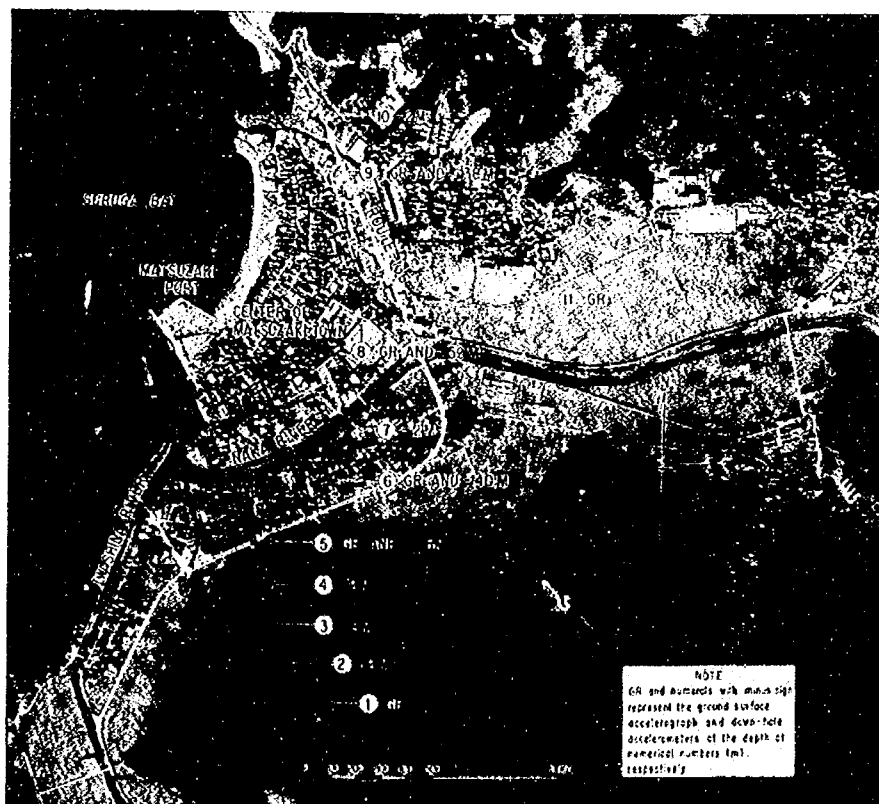
Figure 3.56 Model resonance frequencies plotted as a function of scaling factor "K" and ridge frequency.



Map Showing Deployment of Accelerographs at Matsuzaki Site



Longitudinal Section of A-A' Traverse Line



List of Locations of Accelerographs Installed	
Point	Locations Installed
①	Ground surface
②	Ground surface (-2m)
③	Ground surface (-2m)
④	Ground surface (-2m)
⑤	Ground surface, -5m
⑥	Ground surface, -40m
⑦	Ground surface (-2m)
⑧	Ground surface, -52m
⑨	Ground surface, -38m
⑩	Ground surface (-2m)
⑪	Ground surface

Figure 3.57 Matsuzaki dense seismic array.

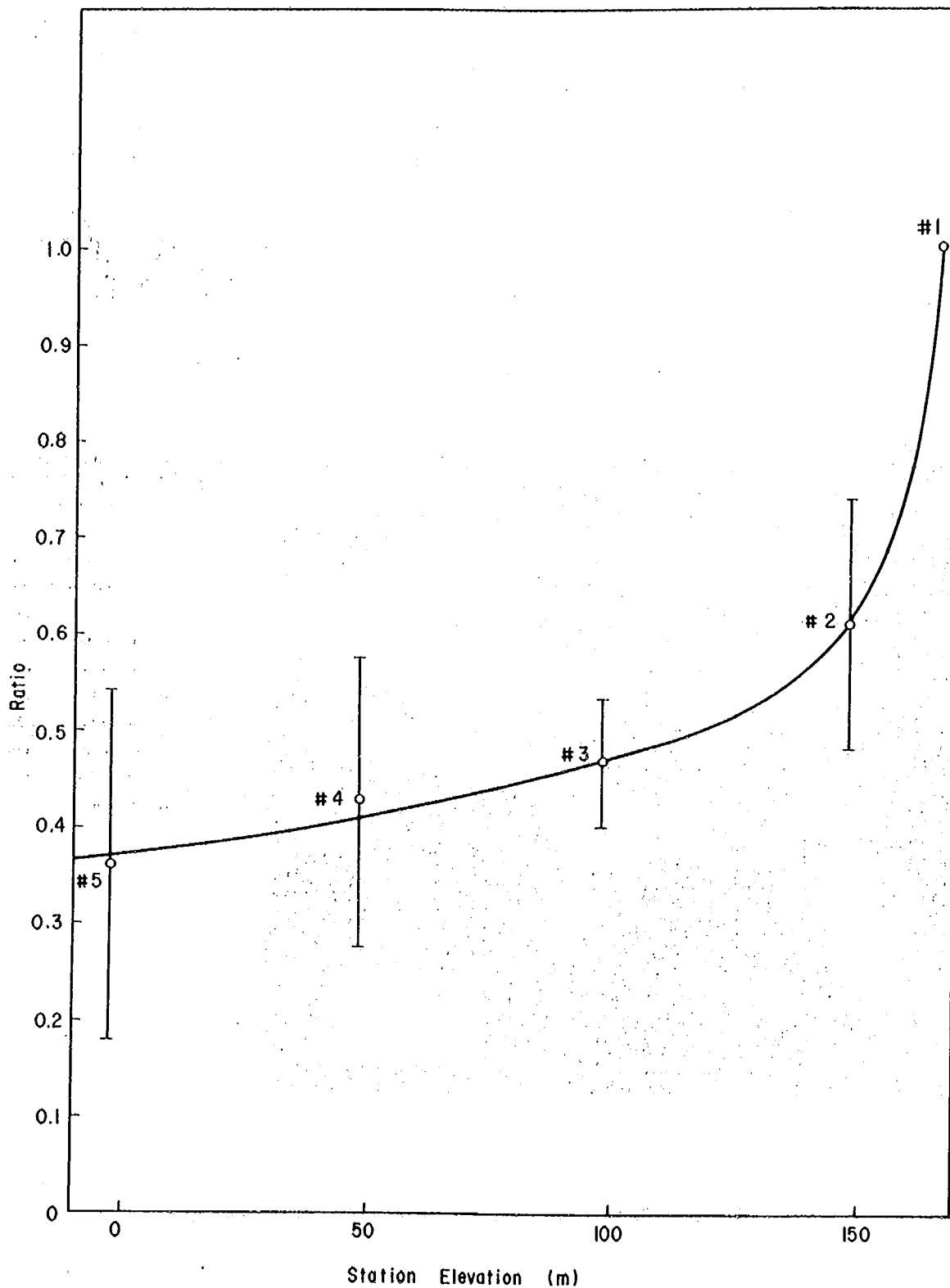


Figure 3.58 Ratio of stations' PGA to PGA of station 1 versus elevation of station above ground level at station 5. Mean $\pm$ one standard deviation plotted.

PART 4

Did Topographic Amplification of Earthquake Shaking Play a Role in Triggering the Mount Ontake Debris Avalanche?

4.1 INTRODUCTION

On September 14, 1984, a large ($M=6.8$) earthquake struck the Nagano Prefecture in central Japan. Several landslides were triggered by the quake, the largest of which was a debris avalanche consisting of about 35 million m^3 of volcanic soil and rock, which initiated at an elevation of 2550 m on the south flank of Mt. Ontake. The debris traveled a total distance of about 12 km and caused at least 15 fatalities. The Mt. Ontake debris avalanche (hereafter referred to as the Ontake slide) has been described and analyzed by Okusa and others (1985)²¹), Tanaka and others (1985)²²), Ishihara and others (1986)²³), and Ishihara and Hsu (1986)⁹).

The mass that slid had formed a prominent ridge before the earthquake triggered its failure; the ridge had an average slope of about 25° . Reports of historical earthquakes indicate that such ridges in the epicentral areas of earthquakes have experienced topographic amplification of ground shaking resulting in anomalously large ground motions and consequent secondary ground failure. The discussion that follows centers on the estimated dynamic response of mass that failed, both with and without the possible effects of topographic amplification, as well as the responses of nearby ridges having similar morphologies. Findings of research on (1) use of the Newmark displacement analysis to evaluate dynamic slope performance, (2) documented topographic amplification during other earthquakes, and (3) shaking-table tests on model slopes, summarized in Parts 1-3 of the present report, are used to address the question of whether or not topographic amplification of earthquake shaking played a significant role in the formation of the Mt. Ontake debris avalanche.

4.2 MODELING SLOPE STABILITY

Ishihara and others (1986)²³) conducted static and pseudo-static stability analyses of the Ontake slide. Figure 4.1 shows their geometric model (hereafter referred to as Ishihara's model) analyzed using an equilibrium equation similar to that proposed by Janbu (1955)²⁴). I constructed an identical slope-stability model using Ishihara's geometric construction and material properties in order to observe the sensitivity of the model to

changes in certain parameters, as discussed below.

Ishihara's equilibrium equation ignores interslice forces; in my model I added the empirical factor, related to the curvature of the slip surface, suggested by Janbu (1973)²⁵⁾ to account for the interslice forces. Addition of this factor increases the static factor of safety slightly.

No water table is assumed in Ishihara's model, and thus no effects of pore pressure on the failure surface are present. In the week preceding the earthquake, however, 165 mm of rain fell in the epicentral area (Ishihara and others, 1986²³⁾; Tanaka and others, 1985²²⁾). The material overlying the slip surface of the Ontake slide was highly fractured and permeable andesite lava, and the slip surface consisted of relatively impermeable pumice predominantly composed of silt-sized particles. Thus, much of the antecedent rainfall probably percolated through the fractured andesite and formed a perched water table on the less permeable pumice. The pumice that failed had water contents greater than 100 percent, well above the liquid limit of the material, and Ishihara and others (1986)²³⁾ reported springs in the lower part of the failure surface. Tanaka and others (1985)²²⁾ state that concentration of ground water on the pumice layer probably contributed to failure. Therefore, it appears likely that the pumice layer that failed was saturated, or nearly so, and that a water table at least 1-2 meters above the slip surface was present in the slide mass at the time of the earthquake.

Static factors of safety using dynamic strength parameters (indicated in figure 4.1) were calculated for a potential slide mass having a water table parallel to the failure surface for a variety of water-table heights. Dynamic strengths were employed because the stability models well be used to determine the critical acceleration of the potential slide mass--that earthquake ground acceleration required to initiate movement of the potential slide block. Figure 4.2 shows that the static factor of safety (for dynamic strength conditions) is 1.90 in the absence of a water table; a water table perched just 7.3 m above the failure surface, however, reduces the static factor of safety to 1.00. If static soil strengths are used, an even lower water table reduces factor of safety to 1.00, because dynamic shaking increases the cohesion of the material and does not affect on the friction angle. A concentration of a few meters of ground water on the relatively impermeable pumice as a result of very heavy antecedent rainfall is not an unlikely occurrence; therefore, the static factor of safety of 1.90 must considered an upper bound, and the factor of safety may have been only slightly greater than

1.00 immediately before the earthquake. Lack of quantitative information regarding the ground-water conditions in the slope before the earthquake makes it impossible to more accurately establish the static factor safety for the Ontake slide.

To estimate the dynamic response of the Ontake slide, we must determine the critical acceleration of the slide mass. For the range of water-table levels of interest, horizontal ground accelerations required to reduce the factor of safety to 1.0 were determined using a standard pseudostatic approach on the model in figure 4.1. Figure 4.3 shows the relationship of critical acceleration to water-table level. Where the water table is below the slip surface, the critical acceleration is about 0.32 g; a water table of about 7.3 m reduces the critical acceleration to zero, the case of static slope failure.

4.3 COMPARISON WITH SIMILAR SLOPES

Only one of several morphologically similar ridges on Mt. Ontake failed during the 1984 earthquake; therefore, some unique combination of factors must have led to failure of the Ontake slide. Several factors could have contributed to failure:

1. The geometry and location of the Ontake slide were controlled by the presence of a relatively weak pumice layer. The extent of this pumice layer beneath other ridges in the area is unknown.

2. Smaller scale landsliding had been occurring at the toe of the Ontake slide for some time, primarily as a result of stream erosion and ground-water seepage. This condition probably reduced the stability of the Ontake slide relative to other ridges in the area.

3. The ridge that failed in the Ontake slide may have been particularly susceptible to amplification of ground shaking because of some unique geometric properties or azimuthal alignment.

4. The ridge that failed was on the flank of Mt. Ontake nearest to the fault rupture surface and thus probably experienced the strongest ground shaking.

The pumice layer that failed in the Ontake slide has been mapped in many locations on and around Mt. Ontake (Ishihara and others, (1986)²³). For the purpose of comparison of the stability of adjacent ridges, it may be reasonable to assume that the pumice blankets the area and is present beneath all the ridges in the vicinity of the Ontake slide. This assumption is undoubtedly inaccurate in many instances, but the lack of detailed subsurface geologic information in the area precludes

analysis of geologic variation between ridges.

Several ridges in the vicinity of the Ontake slide were identified and measured for comparison of dynamic slope performance during the 1984 earthquake. Ridges geometries were measured as recommended by Sasaki and Kuwabara (1986)²⁰). Also, several linear profiles along the steepest parts of these ridges were analyzed. Table 4.1 lists the geometric properties of the ridges and profiles. Also listed in table 4.1 are the distances from the ridges and profiles to the fault rupture surface of the 1984 Naganoken-seibu earthquake. The extent of the fault rupture surface was determined by the locations of aftershocks, and the rupture surface was assumed to extend to the ground surface owing to the very shallow (1-2 km) focal depth of the main shock and reports of possible minor fault rupture at the surface.

If a potential failure mechanism identical to that of the Ontake slide is assumed for the other ridges and profiles analyzed, the critical acceleration of each slope becomes a multivariate function of slope angle and water-table level. Pseudostatic stability calculations of critical acceleration for a variety of slope angles and water-table levels (using the model geometry in figure 4.1) are summarized in figure 4.4. The relationships shown can be expressed as

$$A_c = 0.822 - 0.020(\alpha) - 0.044(h) \quad (4.1)$$

where A_c is the critical acceleration of the potential landslide block in g's, α is the slope angle in degrees, and h is the height of the water table above the potential slip surface in meters. This model is nearly perfectly fit to the data: the correlation coefficient is 1.00. Equation 4.1 is not a general relationship between A_c , α , and h but applies only to the model geometry in figure 4.1.

Critical accelerations were calculated using equation 4.1 for the slope angles listed for each of the ridges and profiles in table 4.1. Values of h equal to zero (no water table) and 2 m were employed to yield a range of critical accelerations encompassing the most likely situation. The relative dynamic performance of these slopes was estimated using Newmark displacement as follows:

$$\log(D_n) = 1.34(M) - 1.85 \log(R) - 6.36(A_c) - 5.14 \quad (4.2)$$

where D_n is the Newmark displacement in centimeters, M is the moment magnitude of the earthquake, R is the earthquake source

distance in kilometers, and A_c is the critical acceleration of the landslide in g's (this is equation 1.7, Part 1).

Table 4.2 lists the critical accelerations and Newmark displacement for the slopes on Mt. Ontake that were analyzed as well as for the Ontake slide, and figure 4.5 plots the displacements as a histogram. The ridge that failed as the Ontake slide has the greatest Newmark displacement, 2.23 to 8.11 cm, of all the ridges (A-O) analyzed; the greatest Newmark displacement calculated for ridges A-O is 1.14 to 4.15 cm (ridge G), only half as great as that for the Ontake slide. For the profiles (1-23), only two had Newmark displacements greater than the Ontake slide; profile 4 (2.82 to 10.22 cm) and profile 7 (4.86 to 17.62 cm). Thus, even if the unfavorable geologic conditions leading to the Ontake slide are assumed to exist beneath all the ridges in the area, the ridge that failed is among the most susceptible to dynamic slope failure as estimated by the Newmark analysis (figure 4.5). The close proximity of the Ontake slide to the fault rupture and the relatively steep ridge on which it formed rendered this slope highly susceptible to dynamic slope failure.

4.4 POSSIBLE EFFECTS OF TOPOGRAPHIC AMPLIFICATION

Results of the Newmark analysis described above indicate that the ridge that failed as the Ontake slide had Newmark displacements between 2.23 and 8.11 cm, for water tables below, and 2 m above, the slip surface, respectively. Findings from Part I indicate that, for the slope materials at Mt. Ontake, about 3 cm of Newmark displacement is the approximate threshold above which slope failure occurs. Therefore, if the water table was below the slip surface, the Newmark displacement of about 2 cm is somewhat below the estimated threshold for slope failure; any significant level of ground water above the slip surface would result in Newmark displacements above this threshold for failure. Therefore, some amount of topographic amplification of ground shaking may have occurred and contributed to the failure of the Ontake slide.

The effects of amplification of ground shaking can be observed through the application of the following equation:

$$\log(D_n) = 2.74 \log(PGA) + 1.37 \log(T_d) - 6.36(A_c) + 1.51, \quad (4.3)$$

where D_n is the Newmark displacement in centimeters, PGA is the peak ground acceleration g's, T_d is the duration of strong ground shaking (defined by Dobry and others (1978)⁸) as the time

required to build up the central 90 percent of the Arias intensity) in seconds, and A_c is the critical acceleration of the landslide in g's (this is equation 1.12, Part 1). T_d can be estimated using a three-step approach:

1. Calculate the Arias intensity at the site using the following relationship:

$$\log(I_a) = 0.98(M) - 1.35 \log(R) - 4.90, \quad (4.4)$$

where I_a is the Arias intensity in meters/second, M is the moment magnitude, and R is the earthquake source distance in kilometers (this is equation 1.2, Part 1). At the Ontake slide, the Arias intensity calculated using equation 4.4 is 4.12 m/s.

2. Estimate the PGA at the site using the attenuation relationship for medium ground proposed by Kawashima and others (1984)²⁶):

$$PGA = 232.5 \times 10^{0.216m} / (\Delta + 30)^{-1.218}, \quad (4.5)$$

where PGA is the peak ground acceleration in gals, m is the earthquake magnitude, and Δ is the epicentral distance in kilometers. The PGA calculated using equation 4.5 for the 1984 earthquake at the Ontake slide is about 0.39 g.

3. Calculate the duration from the following equation proposed by Wilson (in press)²⁷):

$$I_a = 0.90(PGA^2)(T_d), \quad (4.6)$$

where I_a is the Arias intensity in meters/second, PGA is the peak ground acceleration in g's, and T_d is the duration in seconds (this is equation 1.11, Part I). Such a calculation yields a T_d of about 30 seconds, a reasonable duration.

By holding the duration constant, Newmark displacements for a range of peak ground accelerations and critical accelerations can be calculated using equation 4.3. Figure 4.6 shows contours of critical acceleration plotted as a function of Newmark displacement and peak ground acceleration for a T_d of 30 seconds. The maximum critical acceleration for no water table, 0.322 g, and the critical acceleration for a water table 2 m above the slip surface, 0.234 g, for the Ontake slide are shown. Amplification ratios for a PGA of 0.39 g are shown along the upper horizontal axis. For the case where A_c is 0.322 g and PGA is 0.39 g, the Newmark displacement is about 2.2 cm, which agrees with that previously calculated using equation 4.2. To have a

Newmark displacement of 3 cm, the threshold for slope failure, at a critical acceleration of 0.322 g requires a peak ground acceleration of about 0.43 g, which corresponds to an amplification ratio of only 1.1. Thus, even in the most stable possible slope conditions, when no water table is present, only a very minor amount of topographic amplification of ground shaking would have been necessary to trigger failure. Indeed, the uncertainty inherent in this procedure because of model assumptions and approximations suggests that the role of topographic amplification in the failure of the Ontake slide is doubtful at best and probably was not required to cause failure.

4.5 SUMMARY AND DISCUSSION

Comparison of the dynamic performances of several ridges on Mt. Ontake similar to the one that failed suggest that, even if the unfavorable geologic conditions at the Ontake slide were present everywhere, the Ontake slide was more susceptible to failure than almost any other slope on the mountain: only two of the 38 ridges and profiles analyzed yielded greater Newmark displacements. This indicates that the ridge that failed did so because it was most susceptible to failure in the conditions present during the 1984 earthquake. Had many or most of the other ridges yielded greater Newmark displacements than the Ontake slide, this could have provided evidence that extraordinary conditions such as amplified ground shaking had occurred.

At present, there is no way to determine if the ridge that failed was uniquely susceptible to topographic amplification because of its geometry, azimuthal alignment, internal structure, or material properties. Results of the PWRI model tests, in their current form (see Part 3), cannot be used for such an analysis; further analysis of the model-test results may make such a determination possible in the future.

The amount of Newmark displacement calculated for the Ontake slide in the most stable slope condition, when no water table is present, is only slightly below the threshold or critical displacement necessary to cause failure; in the presence of even a few decimeters of hydrostatic head on the slip surface, this critical displacement is exceeded. Thus, results from the Newmark analysis indicate that the maximum amplification of ground shaking required to cause failure in the most stable slope condition is only 1.1, an amount probably much less than the uncertainty introduced in the analysis by model assumptions and approximations.

The Ontake slide had a maximum thickness of about 150 m, and the slip surface formed along a weak pumice layer deep within the ridge near its base. The results of the PWRI model tests (Part 3) indicate that amplification of ground shaking deep within a ridge ranges from 1.0 (no amplification) at the base of the ridge to no more than about 2.0 in the center of the maximum vertical cross section. Shear stresses induced by differential accelerations within a ridge are greatest along the ridge crest and are minimal deep within a ridge. These results suggest that possible amplification that may have affected the slip surface of the Ontake slide was no greater than about 1.5 and probably was less than 1.2.

Did topographic amplification of round shaking contribute to the failure of the Mt. Ontake debris avalanche? Probably not. None of the evidence indicates that amplification was necessary to cause failure, that the ridge that failed was uniquely susceptible to amplification, or that significant amplification would have occurred. Evidence from the model tests suggests that any effects of amplification near base of such a ridge would have been minimal.

The location and geometry of the Ontake slide were entirely controlled by the geologic structure within the ridge, and the critical acceleration of the slide block cannot be determined uniquely without knowing the ground-water conditions at the site. Also, no strong-motion records were written near the site, so estimates of the shaking are necessarily crude. These observations, in concert with the findings summarized above, indicate that the Ontake slide is not a valid case study of topographic amplification of ground shaking. Seismically induced slope failures in homogeneous slopes--such as earth dams, natural or man-made levees, or natural soil slopes--may better illustrate the effects of topographic amplification of earthquake shaking on slope stability.

Table 4.1 Characteristics of ridges and profiles on Mt.Ontake

Ridge or Profile	Length (m)	Height (m)	Width (m)	Slope Angle (°)	Focal Distance (km)
Ridges					
A	2690	920	1625	19	6.7
B	3050	1280	800	23	7.3
C	2075	750	550	20	8.0
D	1650	650	600	22	8.5
E	1225	270	650	12	8.8
F	1950	650	1100	18	10.2
G	1500	690	875	25	10.2
H	1900	670	1475	19	11.8
I	3060	1170	750	21	7.9
J	2610	610	2265	14	7.8
K	1675	600	625	20	9.4
L	1975	700	475	20	10.4
M	1975	460	625	13	12.9
N	2775	1010	1150	20	11.8
O	2050	820	400	22	11.5
Profiles					
1	1265	450		20	7.1
2	1475	600		20	7.3
3	1250	500		22	8.2
4	800	400		27	8.6
5	675	250		20	8.8
6	1065	350		18	10.5
7	515	300		30	10.3
8	675	350		27	11.9
9	1000	400		22	11.7
10	975	400		22	12.0
11	890	300		19	13.7
12	800	400		27	11.8
13	650	350		28	11.8
14	1425	450		18	12.0
15	1315	400		17	12.1
16	690	250		20	12.5
17	575	250		24	12.5
18	915	350		21	11.8
19	1125	550		26	11.1
20	1065	450		23	10.4
21	850	350		22	9.4
22	1425	350		14	9.0
23	1465	600		22	7.9

Table 4.2 Critical accelerations and Newmark displacements
for ridges and profiles on Mt. Ontake

Ridge or Profile	NO WATER TABLE		WATER TABLE 2 M ABOVE BASE	
	Critical Acceleration (g)	Newmark Displacement (cm)	Critical Acceleration (g)	Newmark Displacement (cm)
Ontake Slide	0.322	2.23	0.234	8.11
Ridges				
A	0.442	0.43	0.354	1.56
B	0.362	1.18	0.274	4.29
C	0.422	0.41	0.334	1.50
D	0.382	0.67	0.294	2.41
E	0.582	0.03	0.494	0.12
F	0.462	0.15	0.374	0.53
G	0.322	1.14	0.234	4.15
H	0.442	0.15	0.354	0.55
I	0.402	0.57	0.314	2.06
J	0.542	0.07	0.454	0.27
K	0.422	0.31	0.334	1.12
L	0.422	0.26	0.334	0.93
M	0.562	0.02	0.474	0.08
N	0.422	0.20	0.334	0.73
O	0.382	0.38	0.294	1.38
Profiles				
1	0.422	0.52	0.334	1.87
2	0.422	0.49	0.334	1.78
3	0.382	0.71	0.294	2.58
4	0.282	2.82	0.194	10.22
5	0.422	0.35	0.334	1.26
6	0.462	0.14	0.374	0.51
7	0.222	4.86	0.134	17.62
8	0.282	1.54	0.194	5.60
9	0.382	0.37	0.294	1.34
10	0.382	0.35	0.294	1.28
11	0.442	0.14	0.354	0.51
12	0.282	1.57	0.194	5.69
13	0.262	2.10	0.174	7.63
14	0.462	0.11	0.374	0.40
15	0.482	0.08	0.394	0.29
16	0.422	0.18	0.334	0.66
17	0.342	0.59	0.254	2.12
18	0.402	0.27	0.314	0.98
19	0.302	1.31	0.214	4.75
20	0.362	0.61	0.274	2.23
21	0.382	0.55	0.294	2.00
22	0.542	0.06	0.454	0.21
23	0.382	0.76	0.294	2.76

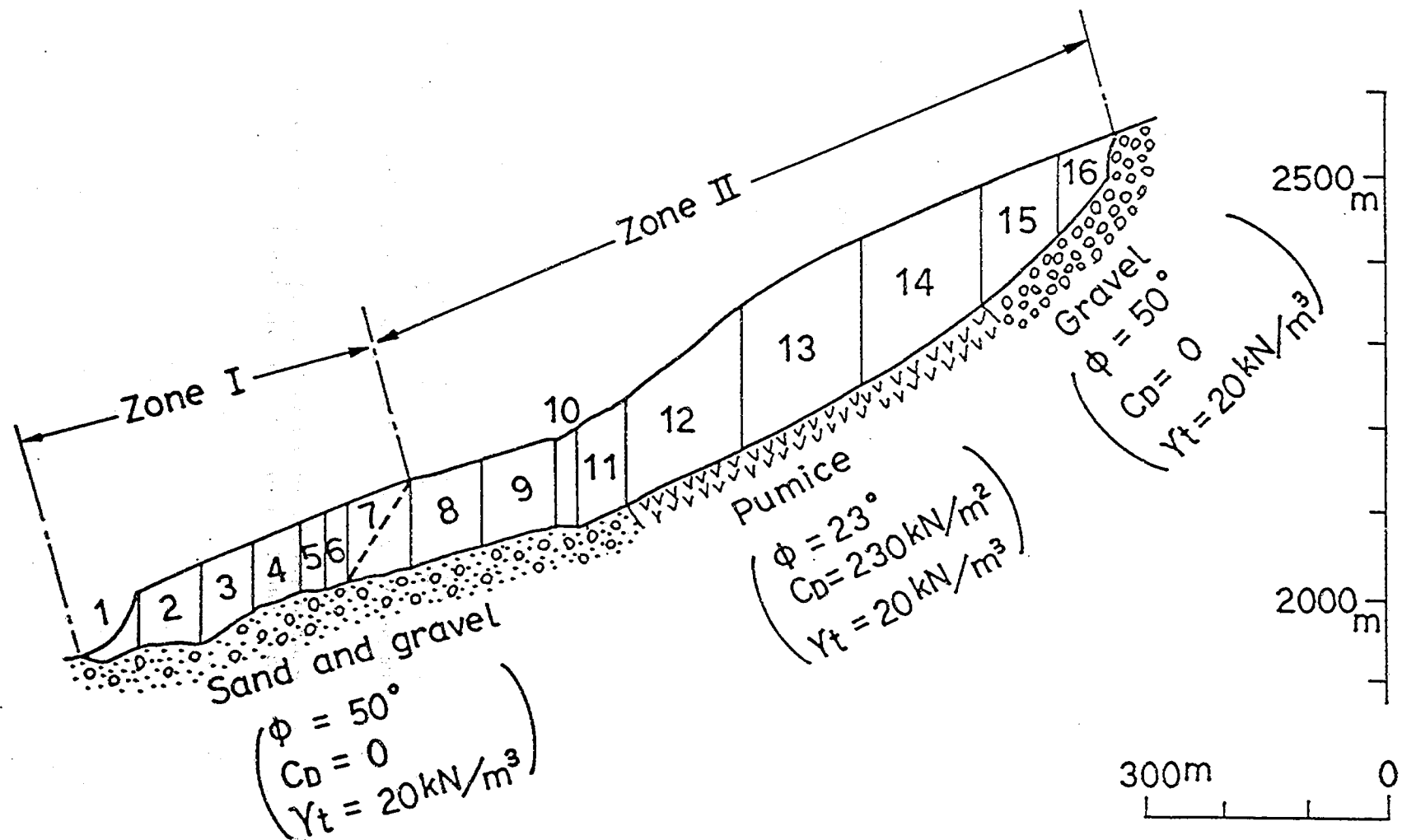


Figure 4.1 Slope-Stability model proposed by Ishihara and others (1986) for Mt. Ontake debris avalanche.

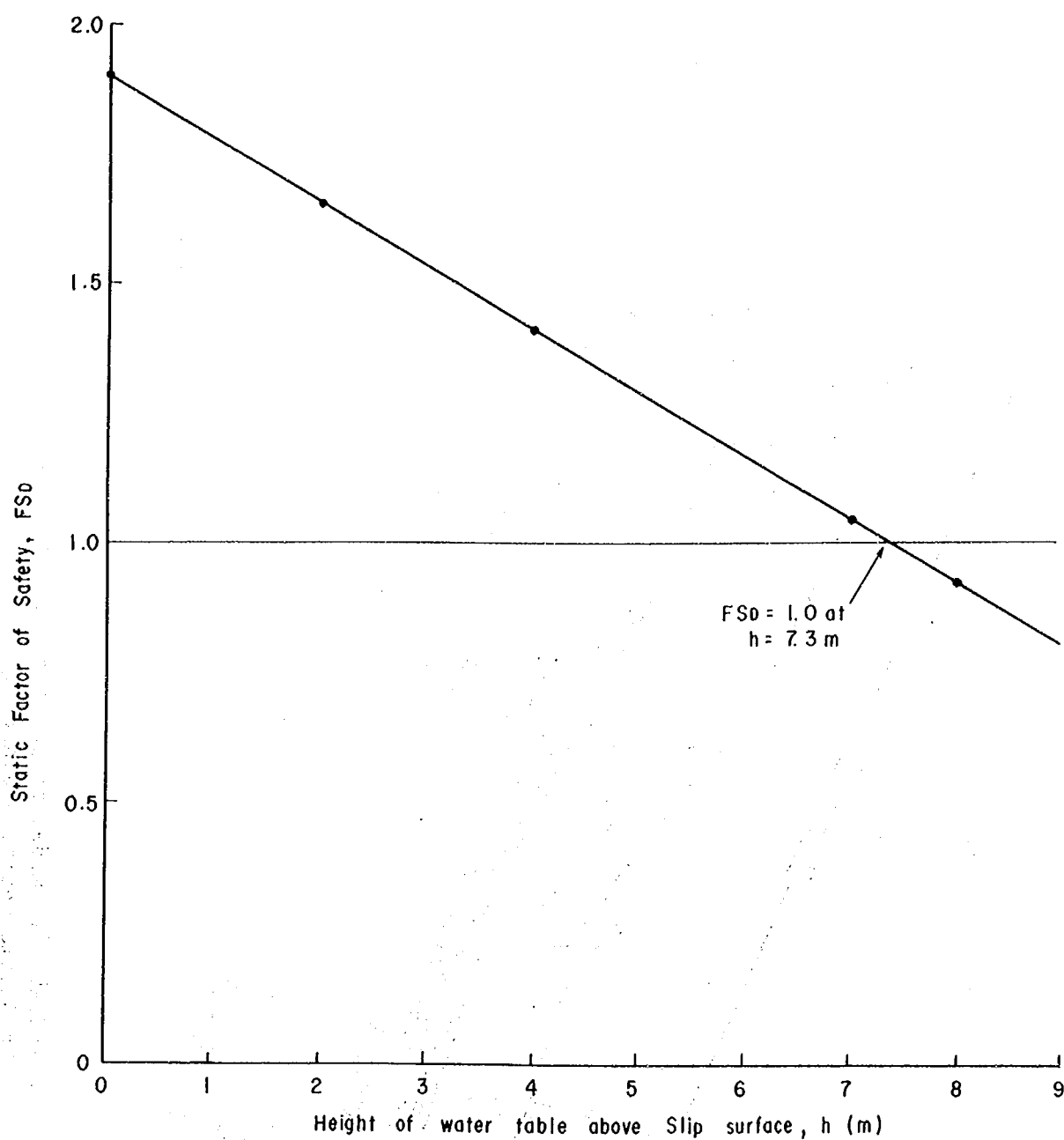


Figure 4.2 Static factor of safety for dynamic strength as a function of water-table level.

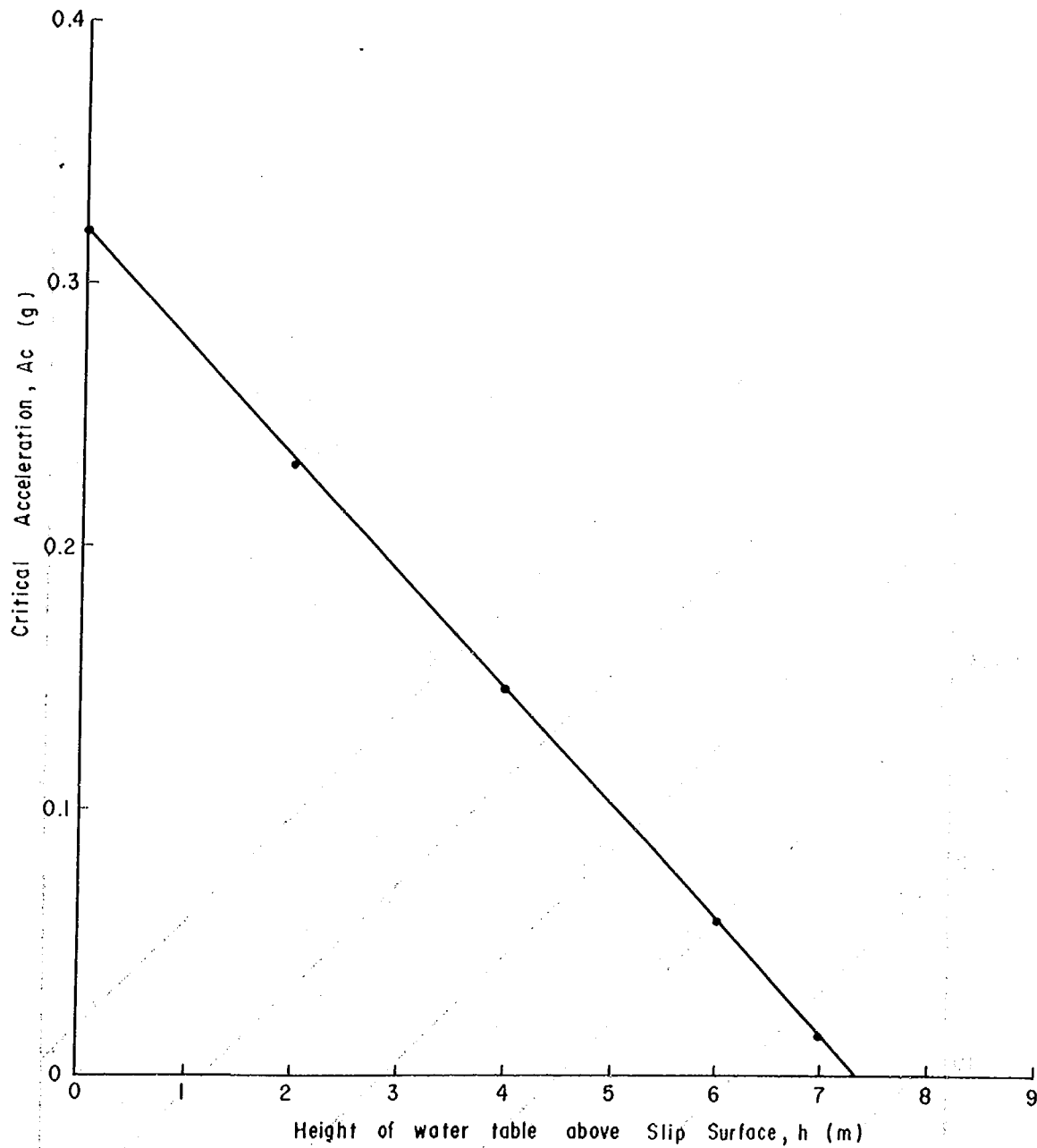


Figure 4.3 Critical acceleration as a function of water-table level.

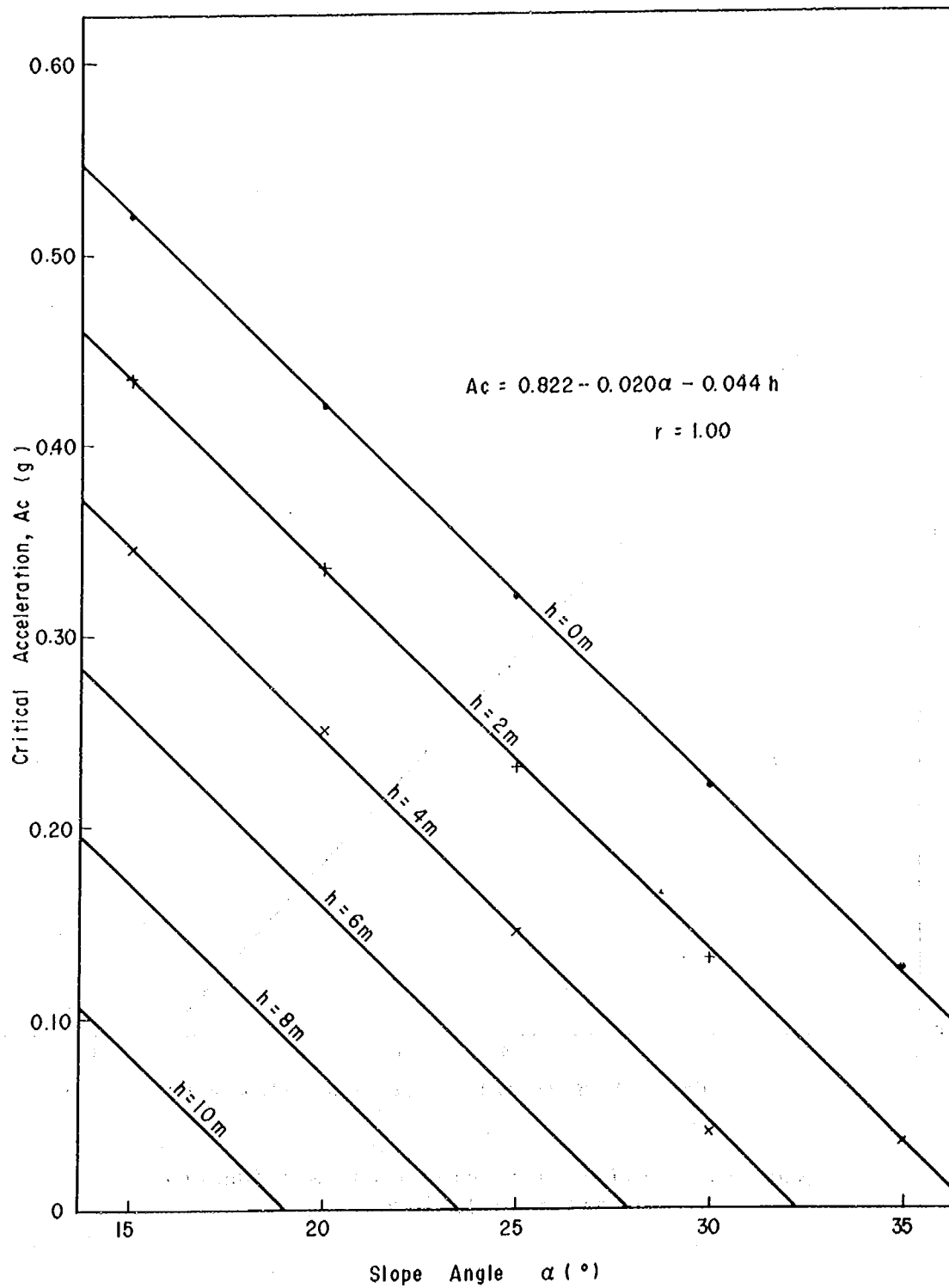


Figure 4.4 Relationship between critical acceleration (A_c), slope angle (α), and water-table level (h) for the Ontake slide.

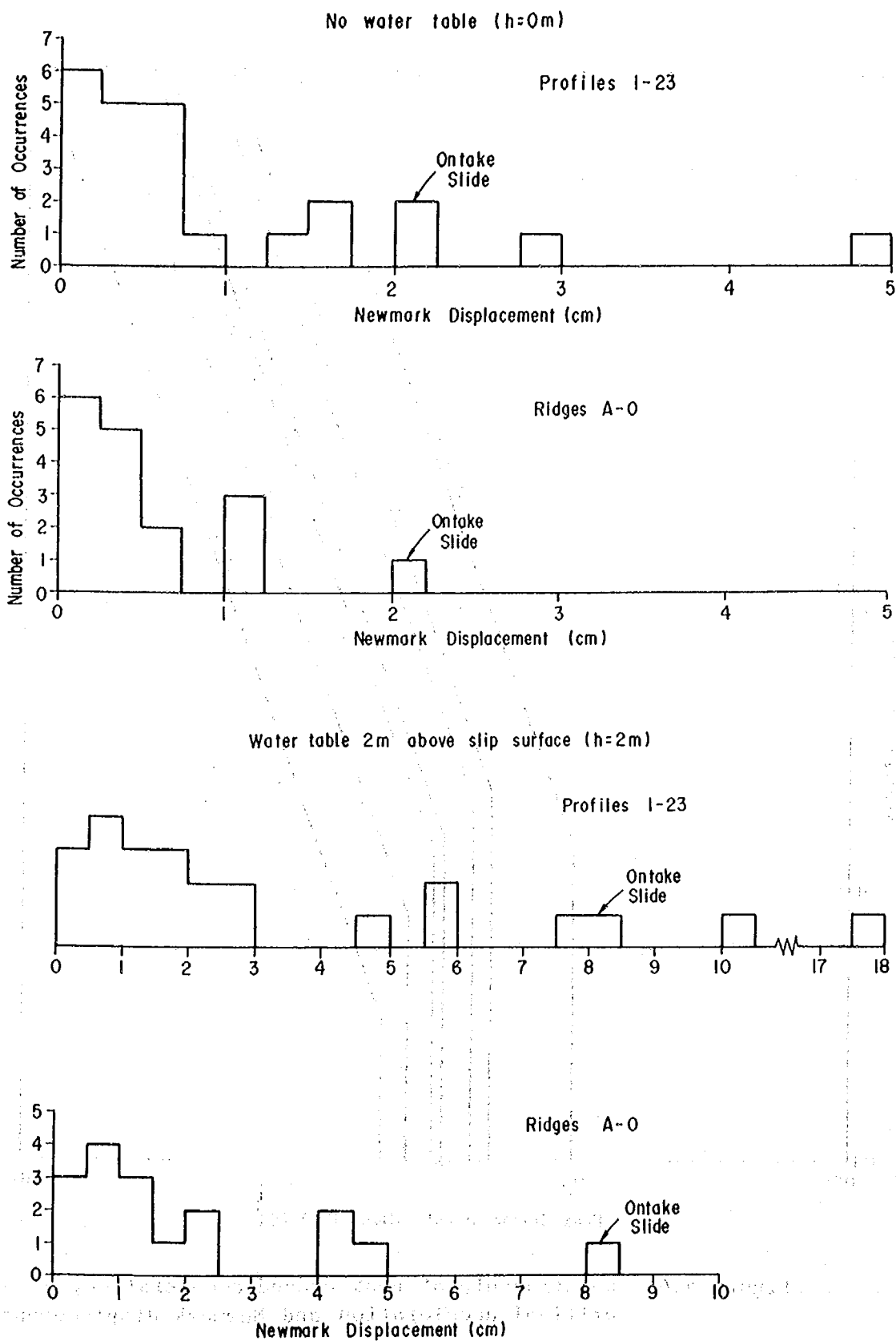


Figure 4.5 Histograms of Newmark displacement

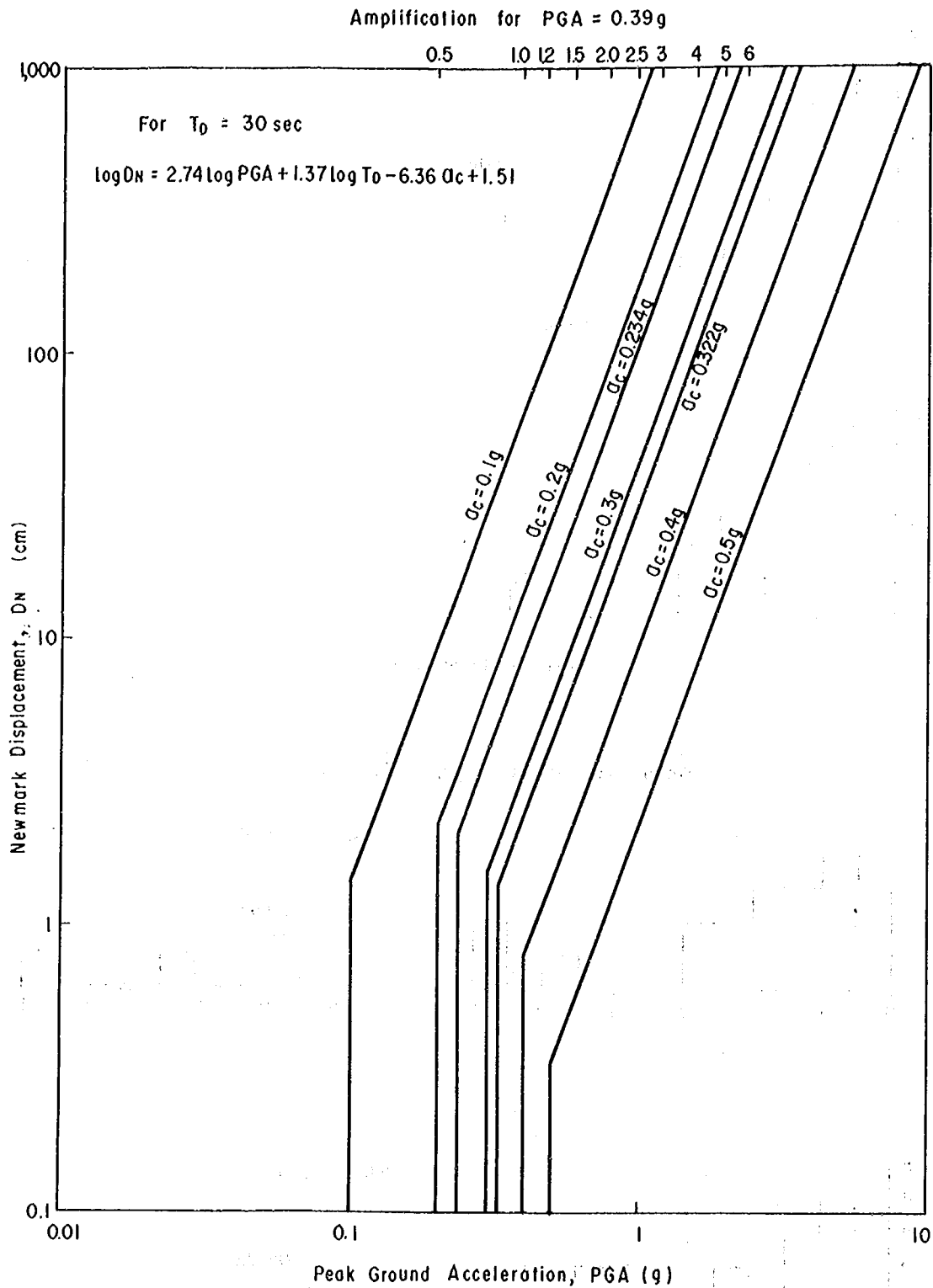


Figure 4.6 Relationship of peak ground acceleration, critical acceleration and Newmark displacement

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ATTENUATION OF PEAK GROUND MOTIONS
FROM
MEXICO EARTHQUAKES OF SEPTEMBER 19 AND 21, 1985

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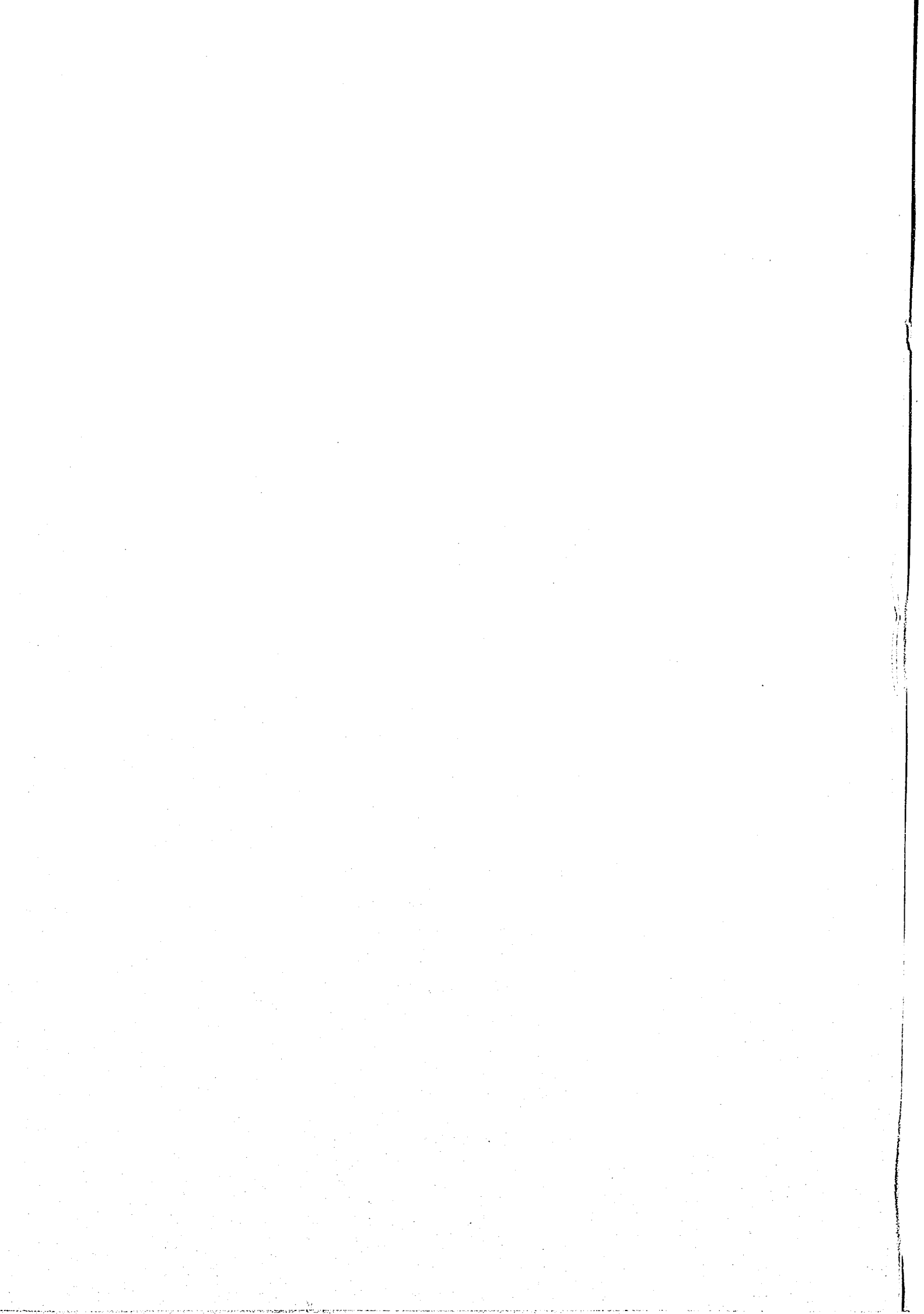
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ATTENUATION OF PEAK GROUND MOTIONS
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MEXICO EARTHQUAKES OF SEPTEMBER 19 AND 21 , 1985

Abstract

This work presents the result of the regression analysis of 15 horizontal strong motion acceleration records that were obtained at 14 stations from the Guerrero accelerograph array , installed along the Pacific coast of Mexico. Each pair of orthogonal horizontal components was taken into account , to select the bigger peak ground motion in the horizontal plane , as well as the maximum value of absolute acceleration response spectra with damping of 5% of critical on the horizontal plane . The records were obtained from the network of digital accelerograph installed on rock , for the purpose of recording strong ground motion on bedrock from large subduction thrust earthquakes . Notwithstanding, in order to validate the data used in this analysis , a field review and study of the subsoil conditions should be made. By means of multiple regression analysis, empirical formulae of attenuation of maximum peak ground motions , acceleration , velocity , displacement , as well as absolute acceleration response spectra with 5% damping ratio were obtained and compared with the results obtained from the multiple regression analysis performed at the Public Works Research Institute , with the records of 67 free field sites in Japan , from 90 earthquakes.

CONTENTS

Abstract

1. Introduction
 2. Strong motion data analyzed
 3. Attenuation of the maximum peak ground motion
 4. Attenuation of absolute acceleration response spectra
 5. Conclusions
- Appendix

1. Introduction

In determining the peak ground motion at a given site, for estimating the seismic effects to be considered in design of civil engineering structures, the attenuation of ground motion with distance from the origin of the earthquake is of great importance and there has been much uncertainty about whether attenuation rates vary significantly in different parts of the world.

The importance of this kind of determination is because it can be expected to provide a realistic basis for assessing the characteristics of earthquake ground motions for wide range of soil conditions associated with the frequency characteristics and intensities of ground motion.

2. Strong motion data analyzed

The data analyzed was the one obtained from the main shock of magnitude $M_s=8.1$, and largest aftershock $M_s=7.5$ from September 19 and 21, 1985 respectively, along the Pacific coast of Mexico, in front of the states of Michoacan and Guerrero.

The data used in this analysis was obtained by means of the following instruments:

- 9 DCA-333 Terra technology digital accelerographs, $\pm 2g$
- 18 DSA-1 Kinematic digital accelerographs, $\pm 2g$

3. Attenuation of the maximum peak ground motion

The factors considered in this analysis are the followings:

3.1 Seismic conditions:

- a) Magnitude M
- b) Epicentral distance D (km)

3.2 Soil properties:

Observation sites are on Rock, as for the information at hand for this analysis.

3.3 Characteristic variables of acceleration records

- a) a_{max} (gal) : Absolute peak acceleration. That is the largest value of two perpendicular horizontal motions
- b) v_{max} (cm/sec): Peak velocity. That is the largest value of two perpendicular horizontal motions.
- c) d_{max} (cm) : Maximum displacement. That is the largest value of displacement of two perpendicular horizontal values recorded for each site.

In order to perform this kind of analysis, there are some items that should be considered before regression analysis, among which are the followings:

- Combination of two horizontal conditions, to find out the real peak value in time domain for horizontal conditions.
- Instrumental correction.

- Classification of ground conditions.

For this work, any one of these items were taken into account, because of the kind of information at hand. It is very important to complete this information and validate the results given here, as shall be explained in what follows.

The characteristics of $a_{\max}$, $v_{\max}$ and $d_{\max}$ are represented by a expression of the form:

$$a_{\max} = (D+D_0)^a \cdot 10^b$$

$$v_{\max} = (D+D_0)^a \cdot 10^b$$

$$d_{\max} = (D+D_0)^a \cdot 10^b$$

respectively, where each "a" and "b" value can change for each ground condition in the Japanese equations, while for the equations obtained for the Mexico earthquakes, it remains to perform the analysis for different magnitudes M and different ground conditions.

In this analysis, D was substituted for $D+D_0$, where D_0 was taken as 30 km, but it may be important to consider some other values of D_0 , and compare among the Japan and Mexico ground motion characteristics.

For the Mexico earthquakes, it was found that only one magnitude could be considered, the one corresponding to the main shock ($M_s=8.1$), given that because the lack of information for the largest aftershock, as well as for the smaller epicentral distance.

It should be considered for this kind of analysis, the combination of two horizontal components in time domain, as well as the necessary instrumental corrections which were not taken into account for this work.

In table 1 of the appendix, it is shown the formulated regression equations and multiple correlation coefficients obtained.

From these graphs obtained from the regression analysis of the Mexico earthquake, and presented in the appendix. In the plotting of values obtained for $a_{\max}$ for different epicentral distances, it can be seen how is it that the curve ($a_{\max}-D$) obtained, has lower values up to 200 km than the corresponding Japanese curve for rock formation and that for larger D values, the difference becomes quite small.

After plotting the values of $a_{\max}$, maximum acceleration obtained at epicentral distances, equal to 400 km or even larger values of D, corresponding to the accelerations obtained on the state of "Puebla" and "Mexico City", it can be seen how it is that for the data obtained, corresponding to rock formation, the agreement is quite good (specially for the stations of Tacubaya, CV and Puebla), while, for the values obtained and corresponding to soft subsoils of lacustrine, volcanic origin of Mexico City, the amplification characteristics can be seen very clearly.

It can be seen how, for that information, at $D \geq 400$ km, $a_{\max}$ has a value of about 35 gals, while for soft subsoils, the value

of a_{max} can reach up to 170 gals, what makes it clear an amplification effect of about 5, after the corresponding attenuation generated at $D \pm 400$ km.

For the $(D-v_{max})$ values obtained for Mexico, it can be seen how is it that the values of v_{max} vary close to the corresponding to v_{max} of Japan, but with greater values of v_{max} , up to $D \pm 200$ km, given that after that epicentral distance, the attenuation curve of v_{max} behaves more similar to the corresponding to softer soils of the Japanese curves.

Here again, it can be seen the amplification effect for the soft subsoils of Mexico City. All of this, according to the data obtained from the 19 Sept. Mexico earthquake.

For the attenuation characteristics of d_{max} , the variation corresponding to Mexico earthquake is much more similar to the corresponding to soft subsoils of the Japan curves of attenuation.

On the other hand, for the attenuation characteristics of a_{max} , v_{max} and d_{max} , obtained for the largest aftershock ($M_s=7.5$), the lack of data corresponding to small values of epicentral distance D is responsible of the quite great disagreement with the Japanese attenuation equations.

4. Attenuation of absolute acceleration response spectra

Earthquake response spectra of strong-motion records are important in civil engineering works, because they account for two principal features:

- Frequency characteristics
- Intensities of ground shaking and structures

The first is characterized by the spectral shape and the second by the spectral amplitude.

In order to get the attenuation characteristics of absolute acceleration response spectra of the Mexico earthquakes, and compare them with the Japanese attenuation characteristics, an analysis of multiple regression analysis was performed. The following general attenuation equation was obtained.

$$S_A(T, 0.05) = (D+30)^a * 10^b$$

Different values of "a" and "b" were obtained for different values of T . The corresponding empirical equations are summarized on Table 2 of the appendix.

It is quite important to point out that this study should be completed, taking into account the comments stated in chapter 3 of this work. Besides it should be included a detailed study of the effect of different values of damping "h", on the absolute response spectra attenuation characteristics.

The empirical correlation equations obtained for T : 0.1, 0.15, 0.2, 0.3, 0.5, 0.7, 1.0, 2.0 and 3.0 seconds, are summarized on Table 2 and plotted simultaneously with the Japanese attenuation characteristics, for the same periods, including the curves corresponding to rock, diluvium and alluvium, from where it can be seen how the acceleration response

spectra have bigger values for the rock formation, corresponding to smaller period values ($T = 0.1 - 0.15$ sec.), comparing with the values of softer subsoils and it is gradually decreasing for periods of vibration of the order of 0.2 sec. and 0.3 sec. For values of periods of vibration (T), greater than 0.5 sec., the greater values of acceleration response spectrum corresponding to softer subsoils, like alluvium and diluvium.

For Mexico earthquake, the acceleration response spectrum corresponding to small values of period of vibration ($T = 0.1 - 0.15$ sec.), it is quite similar to the corresponding for alluvium for the Japanese attenuation characteristics.

For greater values of period of vibration, the acceleration response spectrum has attenuation characteristics that can be considered to vary between rock and softer subsoils, but closer to the corresponding to rock, for the Japanese attenuation characteristics.

From the accelerographs presented in the appendix, corresponding to station CALE, with an epicentral distance $D = 21$ km. and N00E comp., as well as the corresponding to SCT1 at Mexico City, for N90W component, epicentral distance $D = 400$ km, it is very important to point out the following characteristics:

CALE	N00E	(on rock)
	$D = 21$ km	
	$a_{\max} = 138.491$ gal	
	with mainly short period component	
SCT1	OA N90W	(on lacustrine montmorillonitic clay of volcanic origin)
	$D = 400$ km	
	$a_{\max} = -167.918$ gal	
	mainly long period vibration components	

From here, it can be seen the high destructive effects of the Mexico 19 september, 1985 earthquake on the soft subsoils of Mexico City, even the epicentral distance of 400 km. In their corresponding acceleration response spectrum, it can be seen the difference in dominant periods of vibration for the two stations selected as representing the characteristic seismic behavior. It should be said here that this problem is nowadays been studied in Mexico, in order to determine the different amplification effects in different parts of the City, according to the seismic-dynamic characteristics of the subsoil and topographic differences detected, for rational civil engineering works.

5. Conclusions

The empirical equations of attenuation of maximum peak ground motion ($a_{\max}$, $v_{\max}$, and $d_{\max}$) are summarized in table 1.

The empirical equations for absolute acceleration response spectral amplitude are summarized in table 2.

On the figures included in this report, as an appendix, the graphs of ($D - a_{\max}$), ($D - v_{\max}$), ($D - d_{\max}$) and ($D - S_A$)

)for characteristics of vibration for T : 0.1, 0.15, 0.2, 0.3, 0.5, 0.7, 1.0, 1.5, 2.0 and 3.0 seconds are handled as functions of D (epicentral distance). On all of these graphs are included the Japanese and Mexican attenuation characteristics, from where the following conclusions can be stated:

- The main shock ($M_s=8.1$) has a good agreement with the Japanese attenuation equations.
- For the largest aftershock, it was found a lack of agreement, due mainly to the absence of information near the epicenter.
- In order to complete this work and think about being able to get empirical attenuation equations to be used in Mexico, it is necessary to perform the following works:
 - a)Determine soil conditions in a detailed form, at each recording site, classifying the different site conditions, according to geotechnical features, including soil profile and shear wave velocity, obtained by means of soil borings all along the network lines.
 - b)It is quite important to go on accumulating more ground motion information, to complete this study with earthquakes of different magnitudes.
 - c)It should be considered the combination of two horizontal components in time domain, to get the largest ground motion characteristics that can be present in any direction.
 - d)The recording instruments should be analyzed in its performance, so that their limitations could be well known and doing so, be able to make the corresponding instrumental corrections.
 - e)Pay a very special attention to the local amplification effects that can be present for and all along each civil structure, for what it will be necessary to complete this study for different ground conditions.

Appendix

Table 1 Empirical Equations of
Attenuation of Maximum Peak
Ground Motion

(a) Main Shock

	Soil Condition	Empirical Equation	Correlation Coefficient
a_{max} [gal]	Rock	$a_{max} = (D+30)^{-0.95421} \times 10^{4.0627}$	-0.794
v_{max} [cm/s]	Rock	$v_{max} = (D+30)^{-0.87883} \times 10^{2.9635}$	-0.799
d_{max} [cm]	Rock	$d_{max} = (D+30)^{-0.92415} \times 10^{2.5934}$	-0.742

(b) Largest Aftershock

	Soil Condition	Empirical Equation	Correlation Coefficient
a_{max} [gal]	Rock	$a_{max} = (D+30)^{-2.6291} \times 10^{7.9347}$	-0.905
v_{max} [cm/s]	Rock	$v_{max} = (D+30)^{-2.3737} \times 10^{6.2223}$	-0.913
d_{max} [cm]	Rock	$d_{max} = (D+30)^{-2.8689} \times 10^{6.6734}$	-0.944

Table 2 Empirical Equations of
Attenuation of Acceleration
Response Spectrum

(a) Main Shock

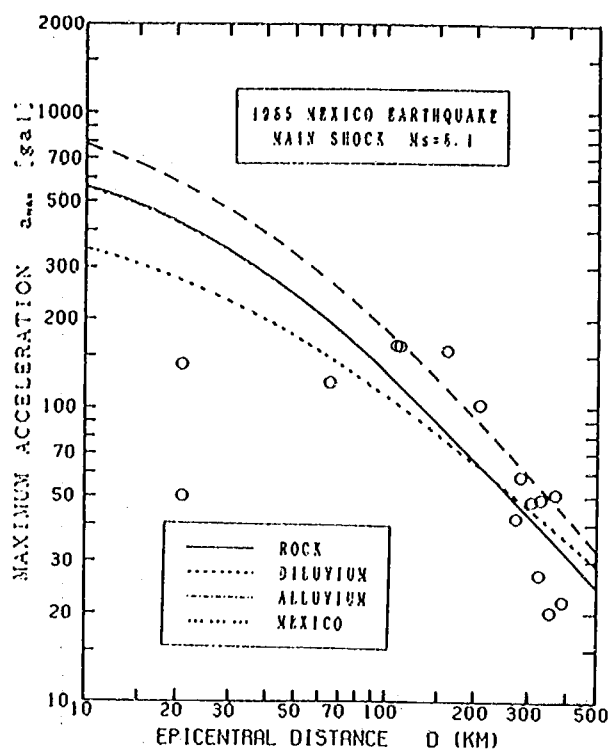
Natural Period[sec]	Soil Condition	Empirical Equation	Correlation Coefficient
T=0.1	Rock	$S_a = (D+30)^{-1.1785} \times 10^{4.8024}$	-0.763
T=0.15	Rock	$S_a = (D+30)^{-1.1652} \times 10^{4.8473}$	-0.730
T=0.2	Rock	$S_a = (D+30)^{-1.1757} \times 10^{4.8945}$	-0.756
T=0.3	Rock	$S_a = (D+30)^{-1.0859} \times 10^{4.6890}$	-0.808
T=0.5	Rock	$S_a = (D+30)^{-1.0885} \times 10^{4.6951}$	-0.829
T=0.7	Rock	$S_a = (D+30)^{-0.85438} \times 10^{4.1068}$	-0.683
T=1.0	Rock	$S_a = (D+30)^{-0.80475} \times 10^{3.8859}$	-0.850
T=1.5	Rock	$S_a = (D+30)^{-0.74147} \times 10^{3.5495}$	-0.792
T=2.0	Rock	$S_a = (D+30)^{-0.92214} \times 10^{3.8792}$	-0.791
T=3.0	Rock	$S_a = (D+30)^{-0.86805} \times 10^{3.4252}$	-0.783

Table 2 Empirical Equations of
Attenuation of Acceleration
Response Spectrum

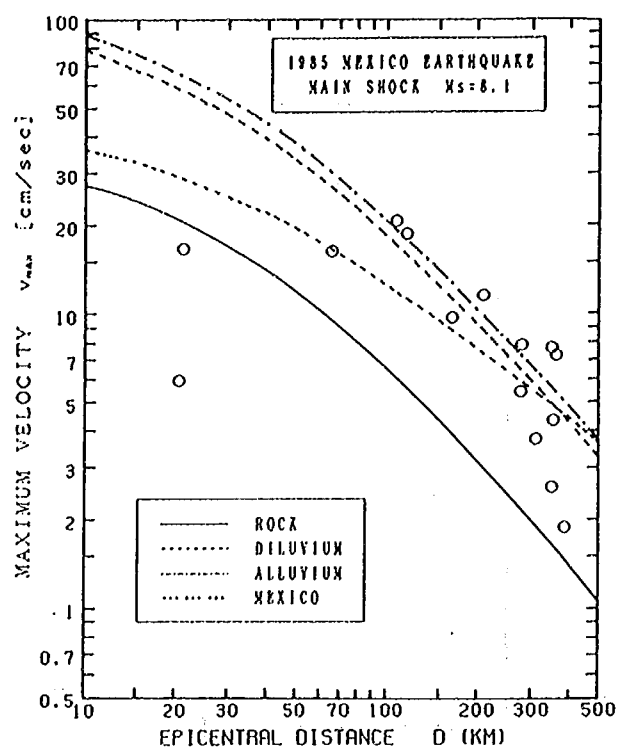
(b) Largest Aftershock

Natural Period [sec]	Soil Condition	Empirical Equation	Correlation Coefficient
T=0.1	Rock	$S_a = (D+30)^{-2.9336} \times 10^{8.8861}$	-0.830
T=0.15	Rock	$S_a = (D+30)^{-2.3930} \times 10^{7.7460}$	-0.779
T=0.2	Rock	$S_a = (D+30)^{-2.4127} \times 10^{7.7553}$	-0.848
T=0.3	Rock	$S_a = (D+30)^{-2.4716} \times 10^{7.9228}$	-0.957
T=0.5	Rock	$S_a = (D+30)^{-2.2247} \times 10^{7.2551}$	-0.881
T=0.7	Rock	$S_a = (D+30)^{-1.7353} \times 10^{6.0757}$	-0.801
T=1.0	Rock	$S_a = (D+30)^{-2.2542} \times 10^{7.0960}$	-0.920
T=1.5	Rock	$S_a = (D+30)^{-2.3471} \times 10^{6.9553}$	-0.838
T=2.0	Rock	$S_a = (D+30)^{-2.1481} \times 10^{6.3268}$	-0.907
T=3.0	Rock	$S_a = (D+30)^{-2.8758} \times 10^{7.7928}$	-0.964

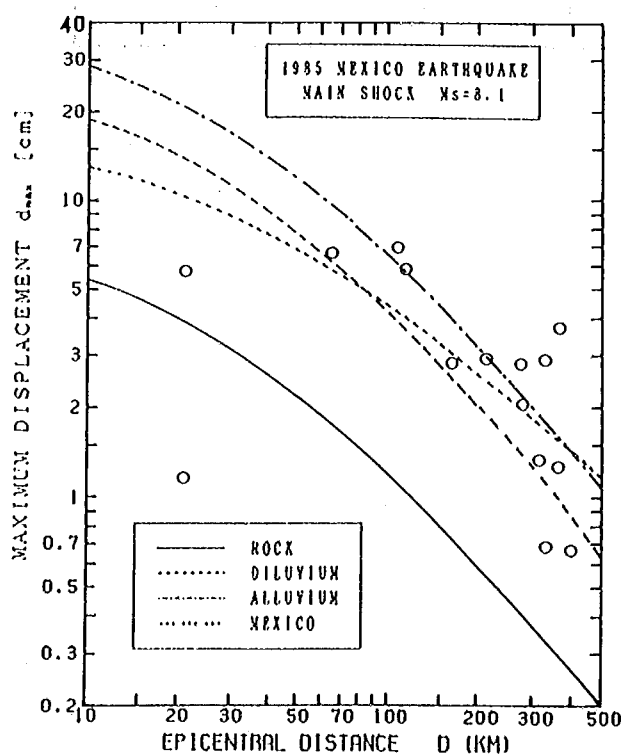
Attenuation of the Maximum Acceleration (Main Shock)



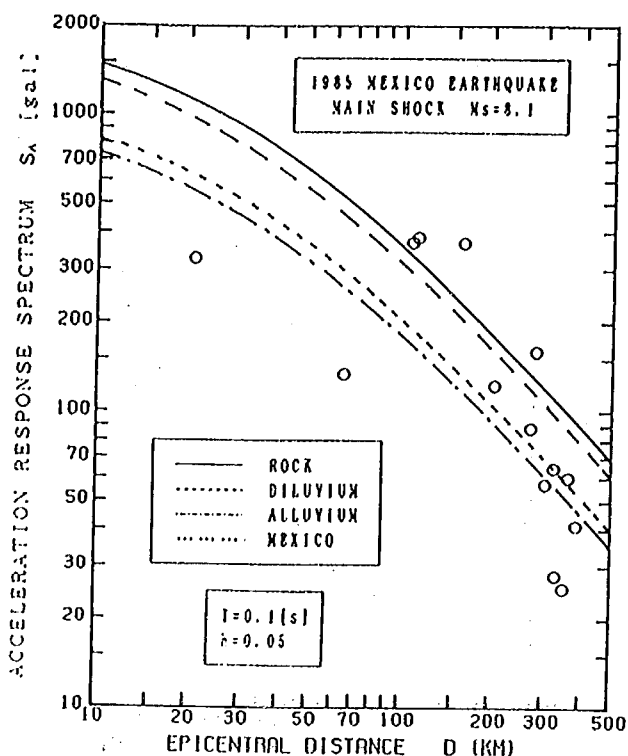
Attenuation of the Maximum Velocity (Main Shock)



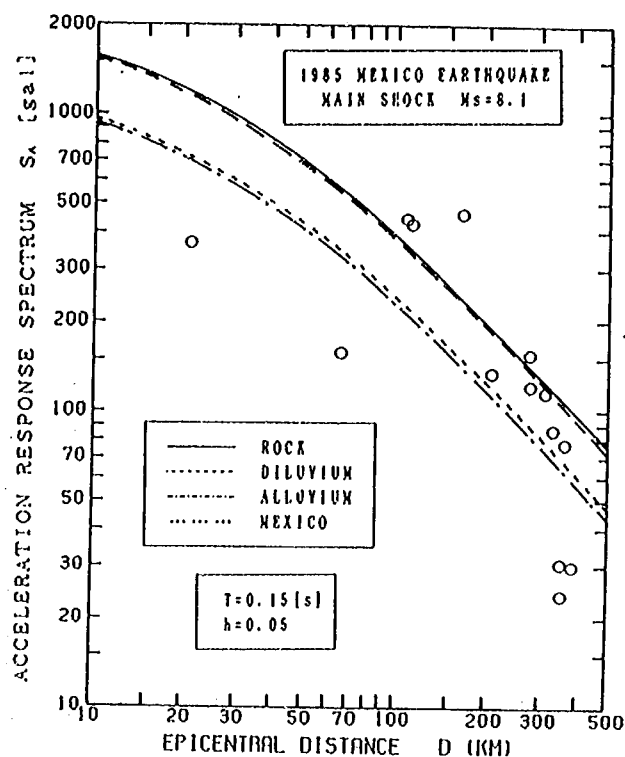
Attenuation of the Maximum Displacement (Main Shock)



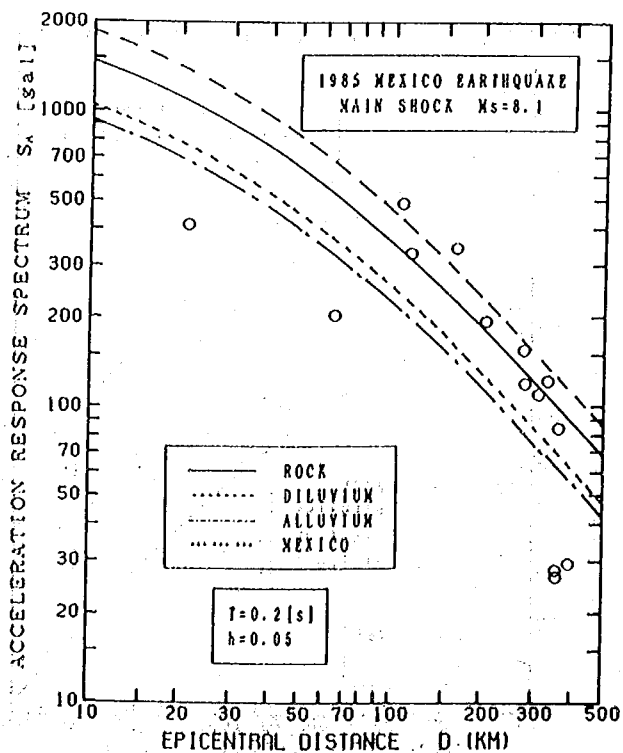
Attenuation of the Acceleration
Response Spectrum
($T=0.1$ (sec), Main Shock)



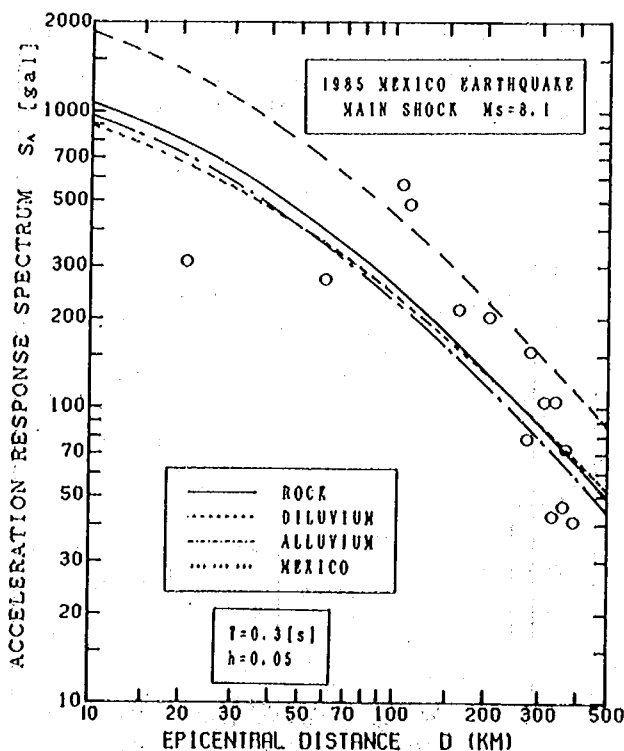
Attenuation of the Acceleration
Response Spectrum
($T=0.15$ (sec), Main Shock)



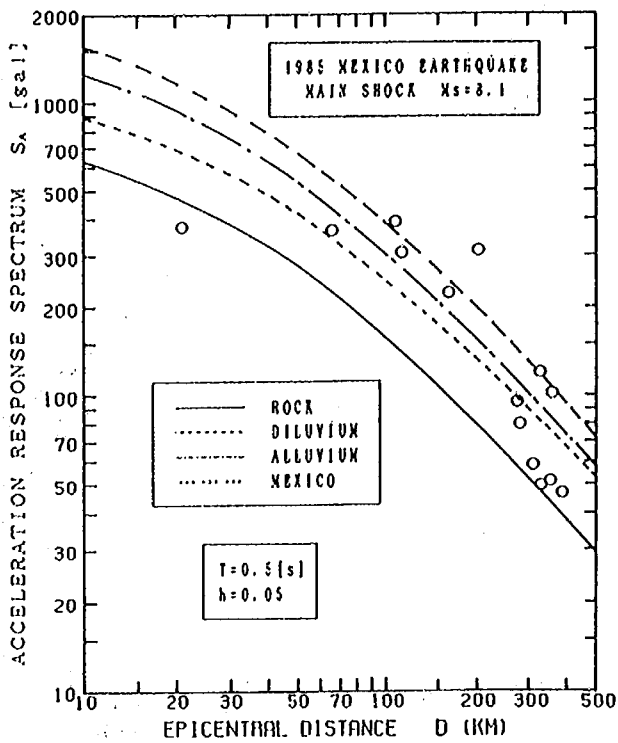
Attenuation of the Acceleration
Response Spectrum
($T=0.2$ (sec), Main Shock)



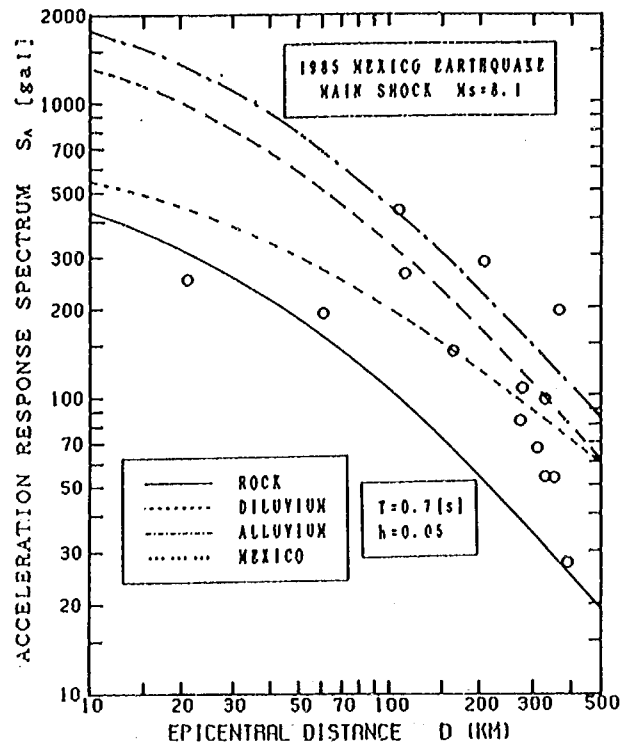
Attenuation of the Acceleration
Response Spectrum
($T=0.3$ (sec), Main Shock)



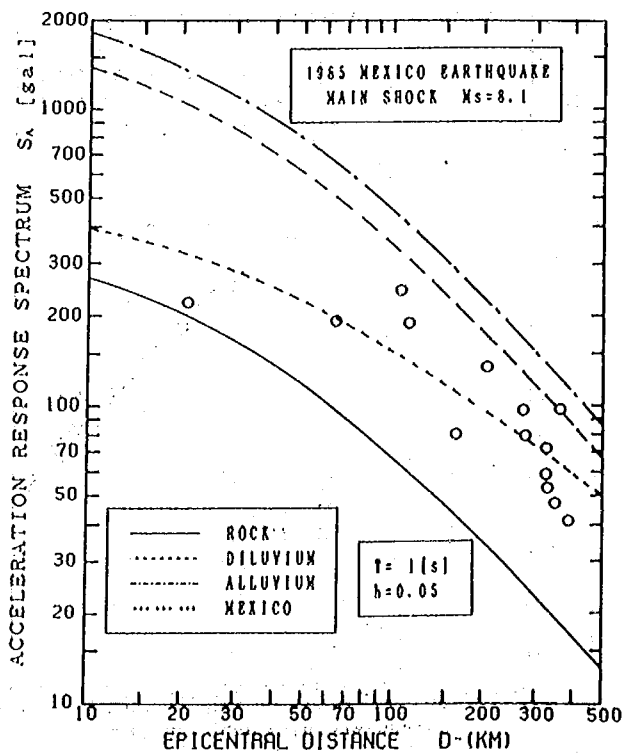
Attenuation of the Acceleration
Response Spectrum
($T=0.5$ (sec), Main Shock)



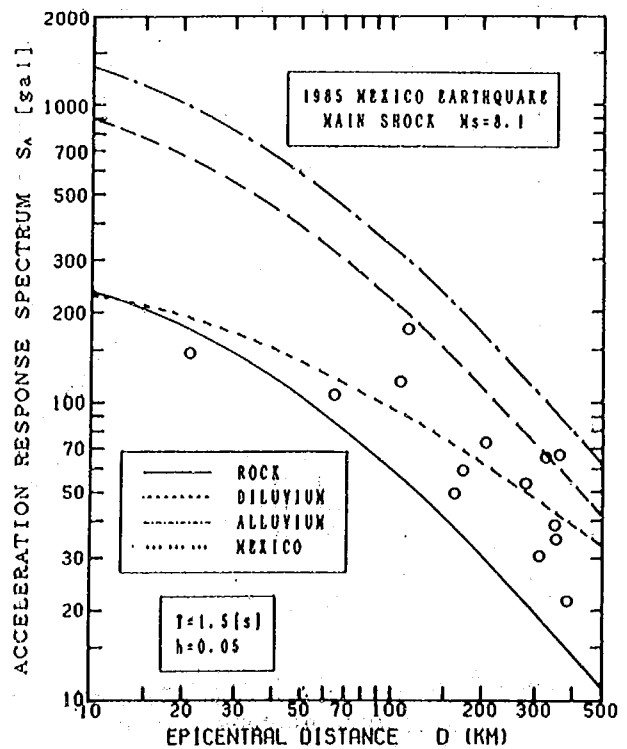
Attenuation of the Acceleration
Response Spectrum
($T=0.7$ (sec), Main Shock)



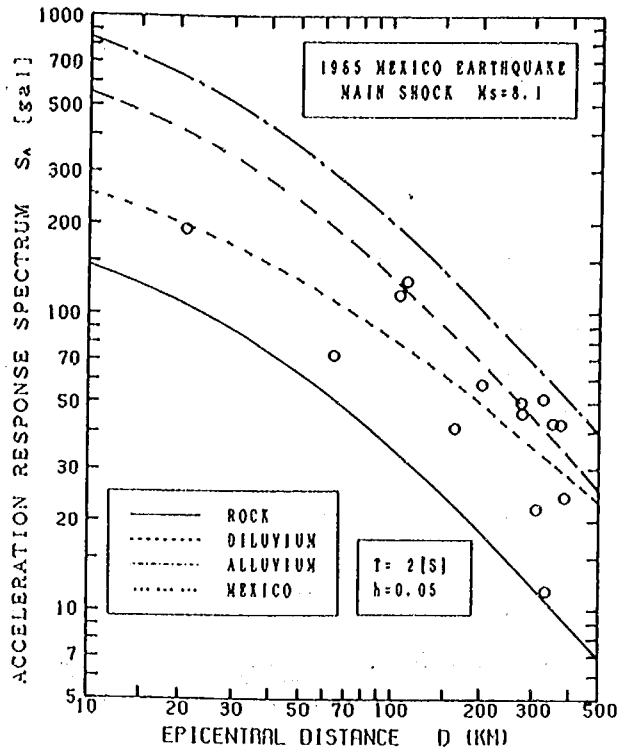
Attenuation of the Acceleration
Response Spectrum
($T=1$ (sec), Main Shock)



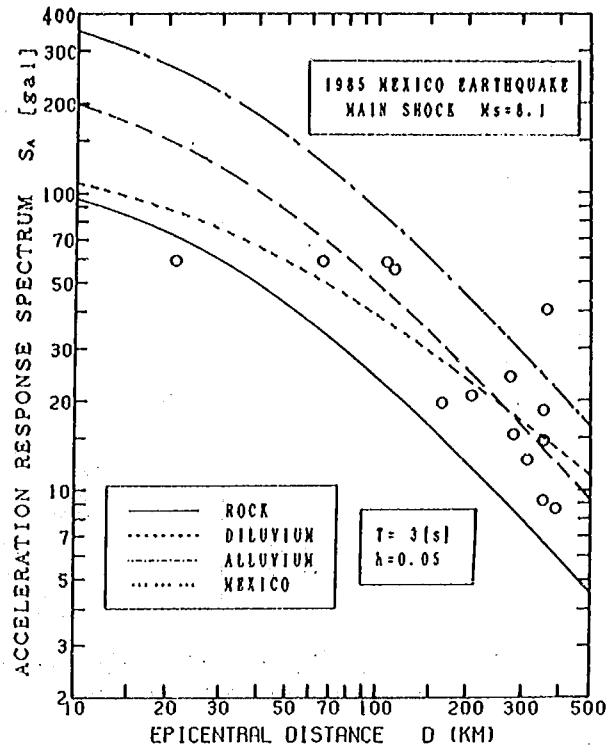
Attenuation of the Acceleration
Response Spectrum
($T=1.5$ (sec), Main Shock)



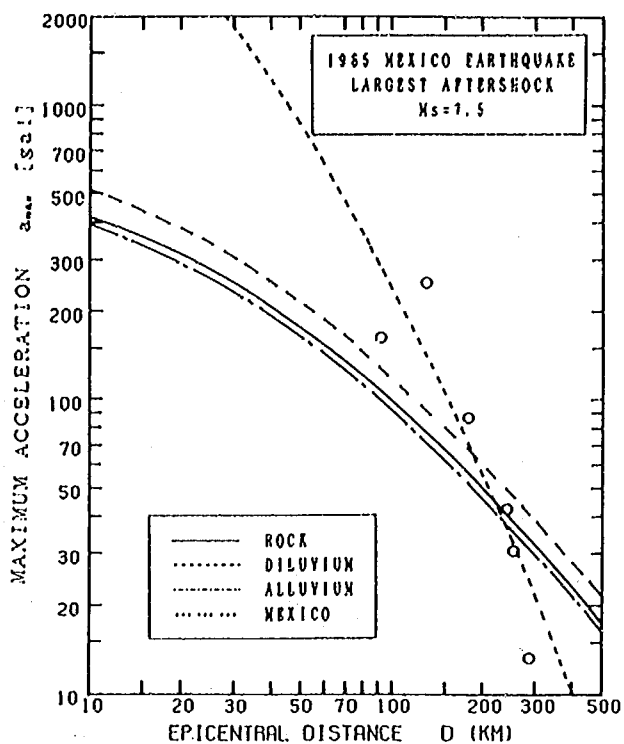
Attenuation of the Acceleration
Response Spectrum
($T=2$ [sec], Main Shock)



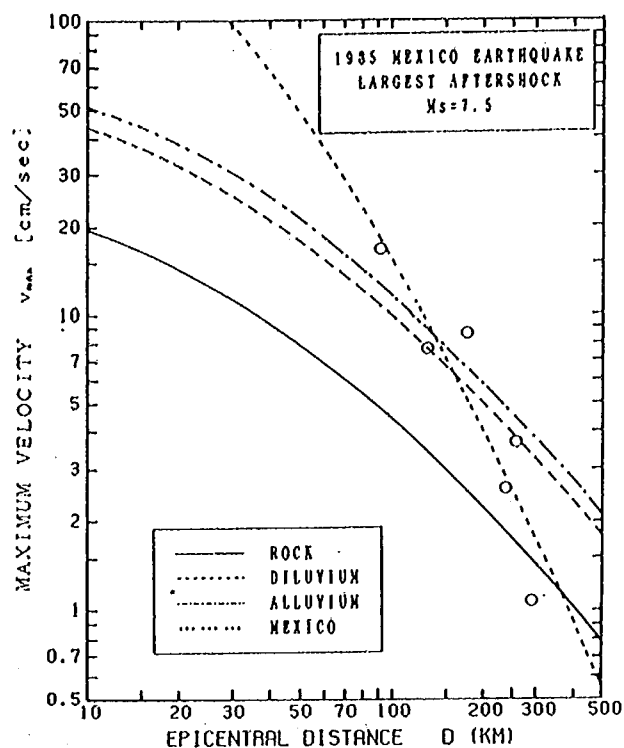
Attenuation of the Acceleration
Response Spectrum
($T=3$ [sec], Main Shock)



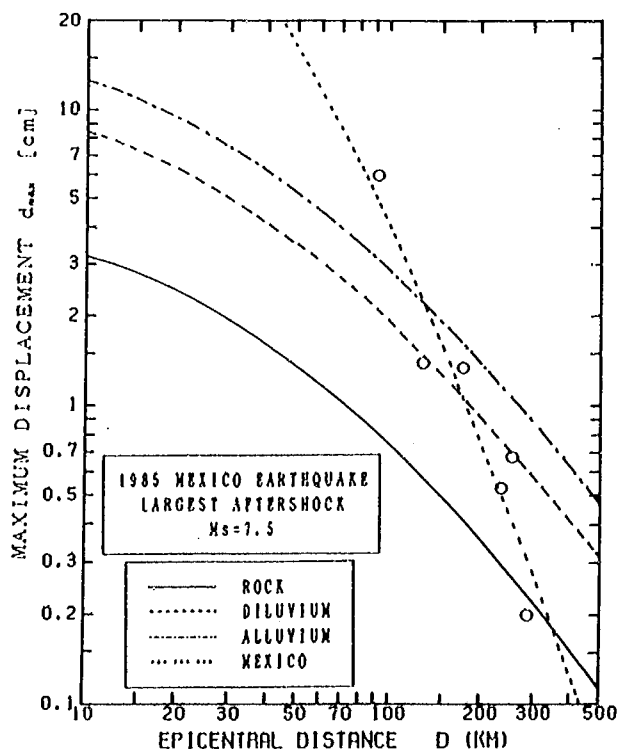
Attenuation of the Maximum Acceleration (Largest Aftershock)



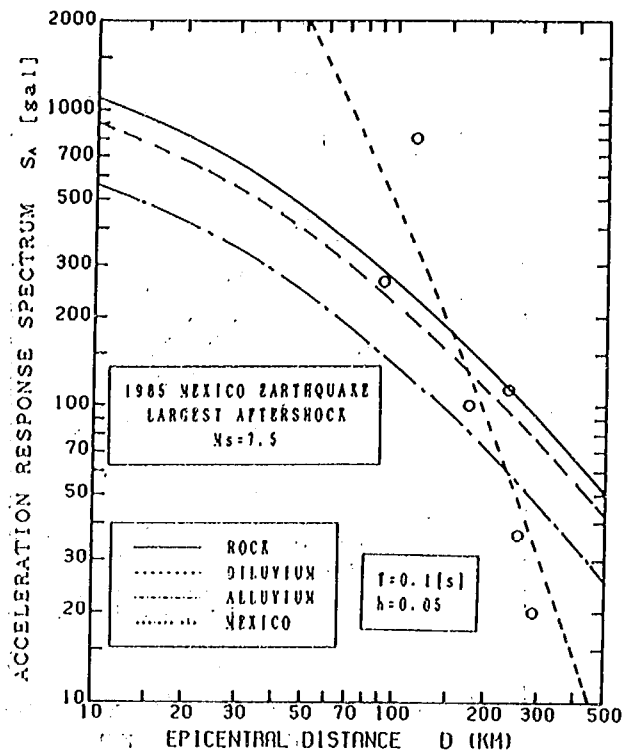
Attenuation of the Maximum Velocity (Largest Aftershock)



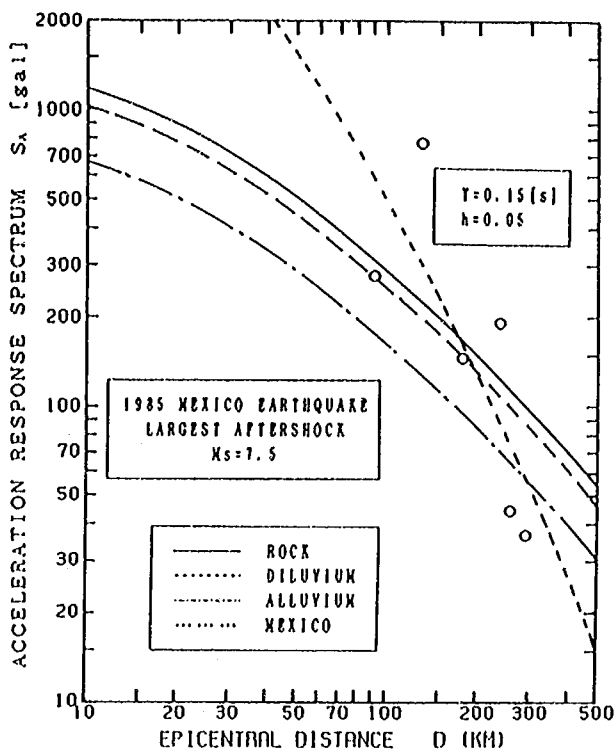
Attenuation of the Maximum Displacement (Largest Aftershock)



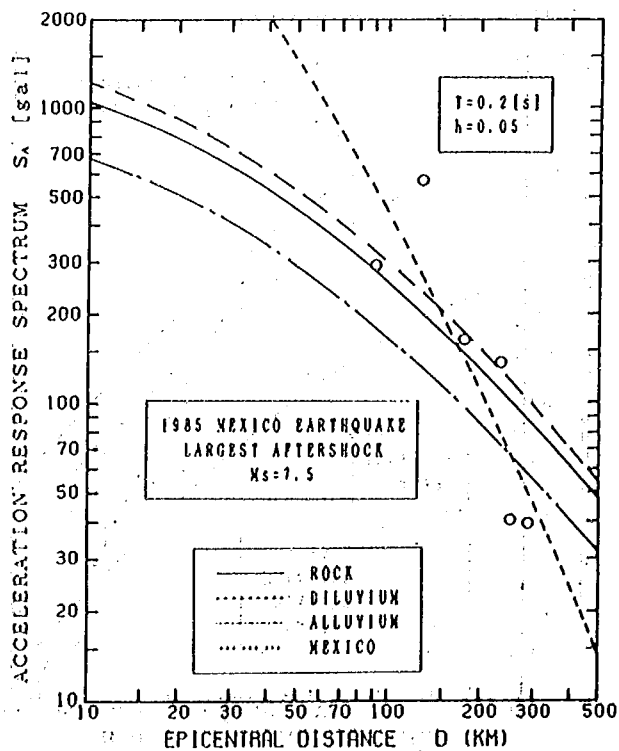
Attenuation of the Acceleration
Response Spectrum
($T=0.1$ [sec], Largest Aftershock)



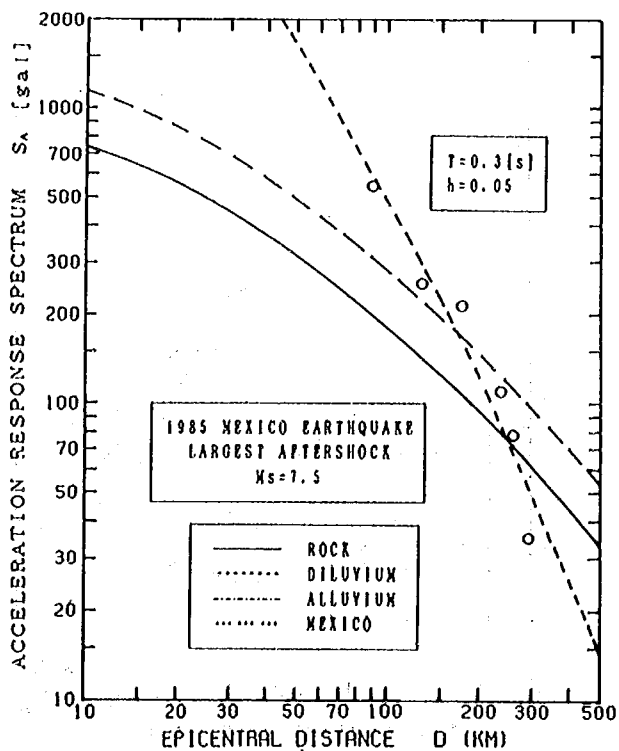
Attenuation of the Acceleration
Response Spectrum
($T=0.15$ [sec], Largest Aftershock)



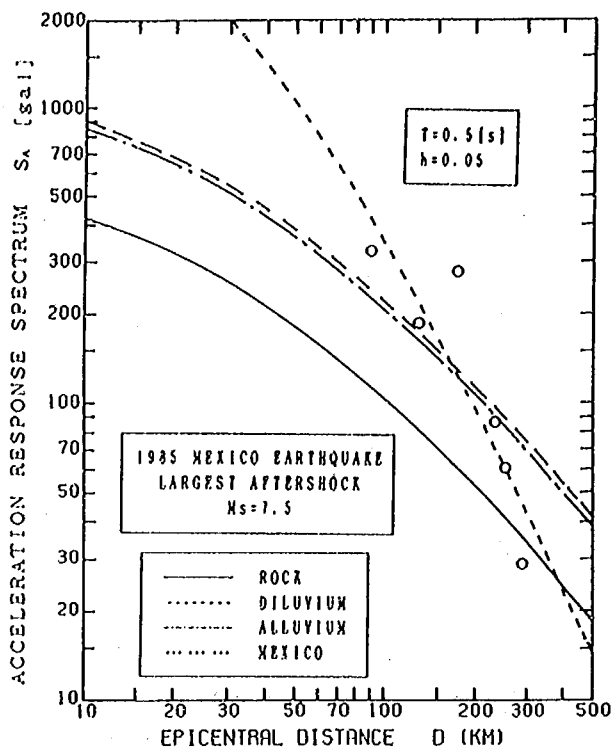
Attenuation of the Acceleration
Response Spectrum
($T=0.2$ [sec], Largest Aftershock)



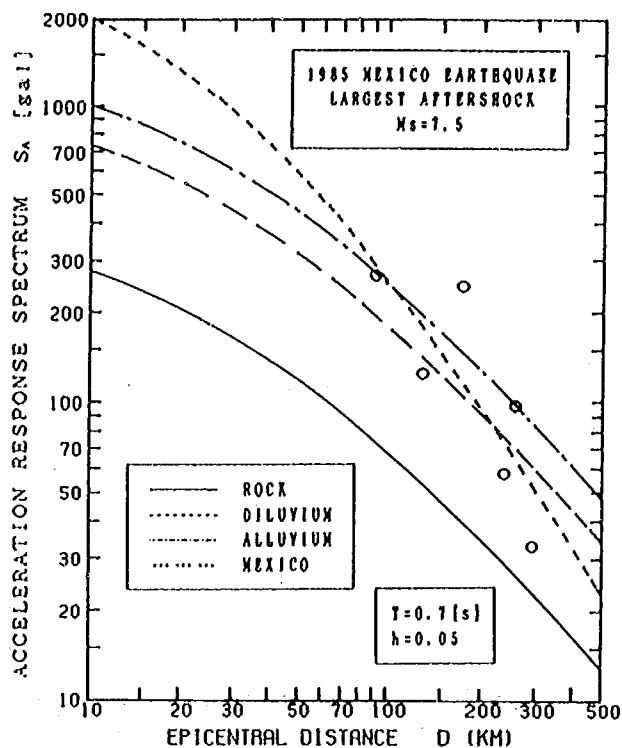
Attenuation of the Acceleration
Response Spectrum
($T=0.3$ [sec], Largest Aftershock)



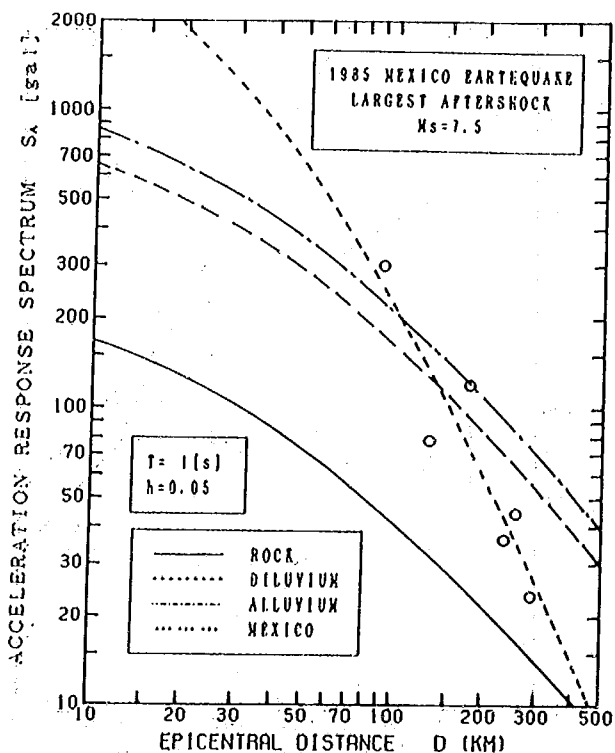
Attenuation of the Acceleration Response Spectrum
($T=0.5$ [sec], Largest Aftershock)



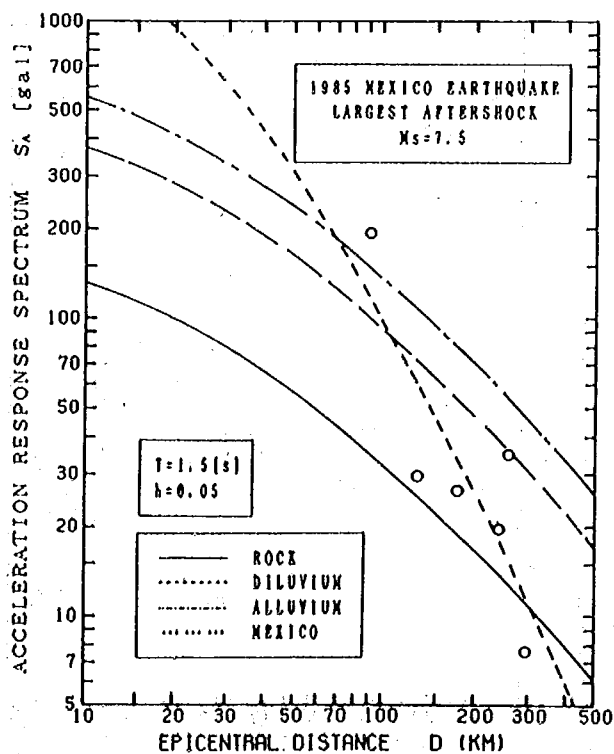
Attenuation of the Acceleration Response Spectrum
($T=0.7$ [sec], Largest Aftershock)



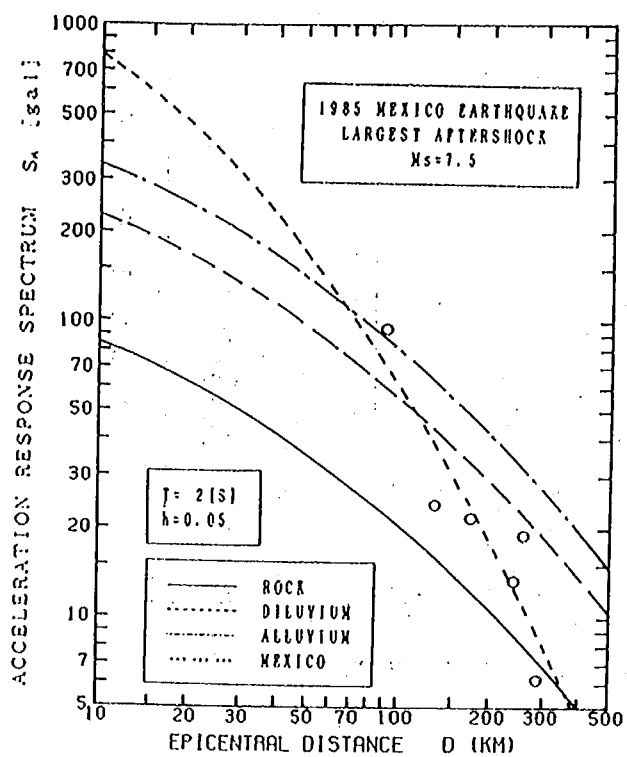
Attenuation of the Acceleration Response Spectrum
($T=1$ [sec], Largest Aftershock)



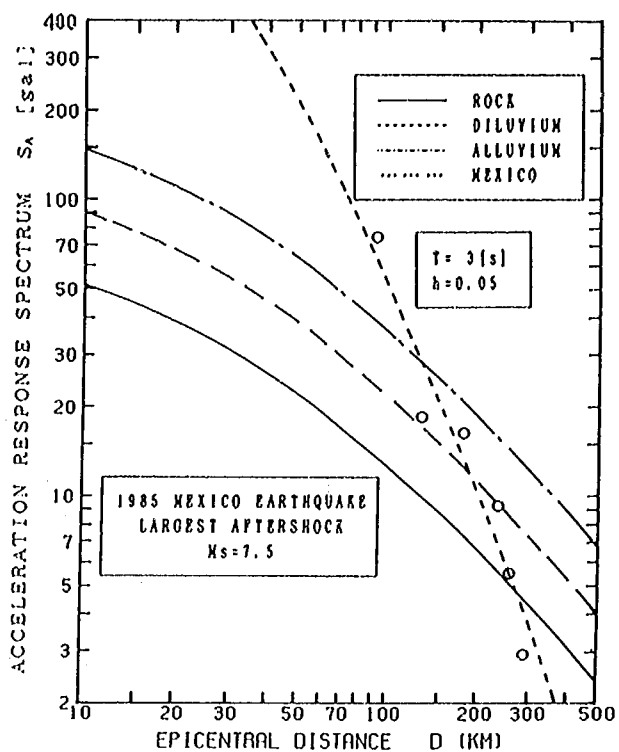
Attenuation of the Acceleration Response Spectrum
($T=1.5$ [sec], Largest Aftershock)



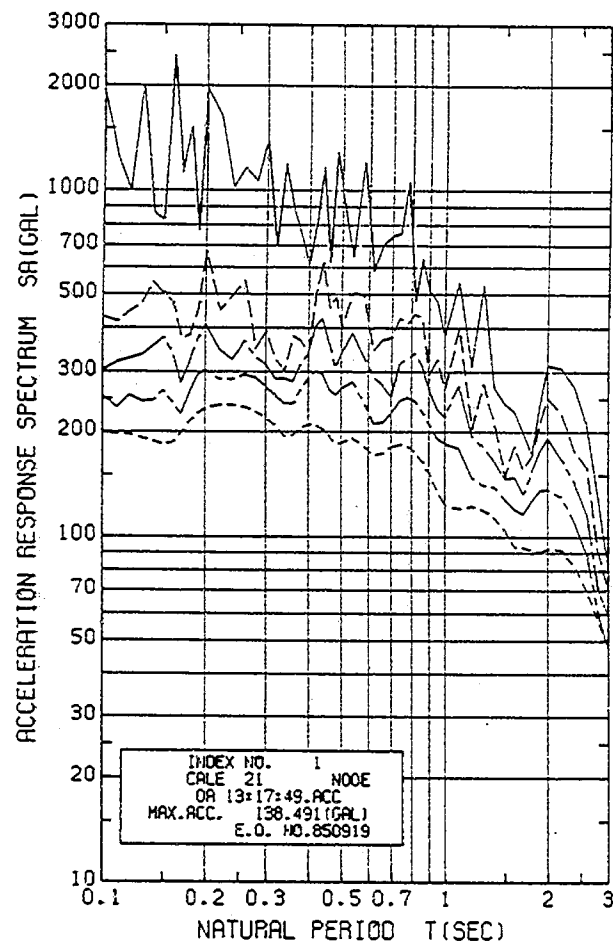
Attenuation of the Acceleration
Response Spectrum
($T=2$ [sec], Largest Aftershock)



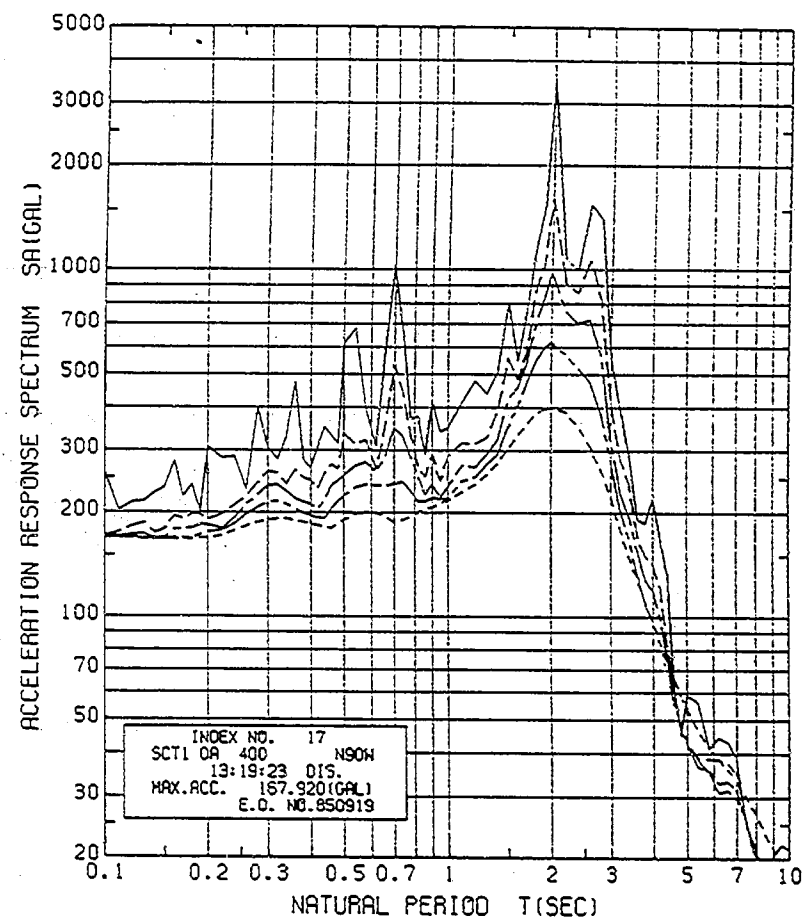
Attenuation of the Acceleration
Response Spectrum
($T=3$ [sec], Largest Aftershock)



Example of Acceleration Response Spectrum
Spectrum (CALE, Guerrero Array, $\Delta=21\text{kHz}$)



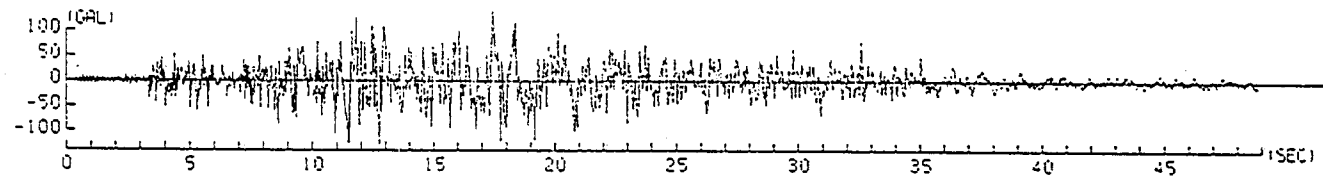
Example of Acceleration Response Spectrum
Spectrum (SCT1, Mexico City, $\Delta=400\text{kHz}$)



Example of Waveforms
(CALE, Guerrero Array, $\Delta=21\text{km}$)

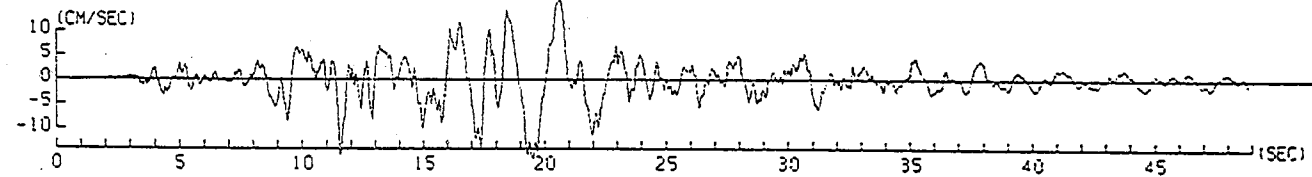
INDEX NO. 1
CALE 21 N90E
OR 13:17:49.ACC
MAX.=138.491(GAL)

E.Q. NO. 850919



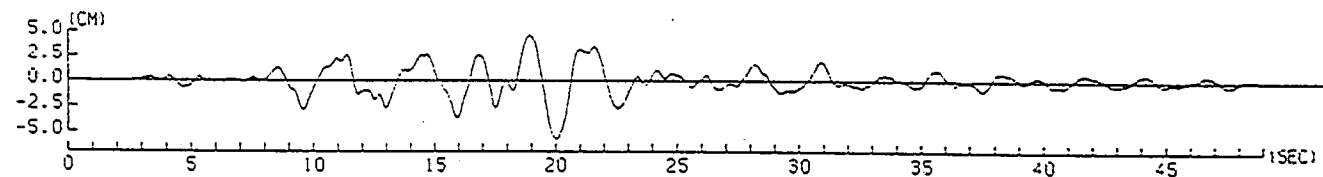
INDEX NO. 2
CALE 21 N90E
OR 13:17:49.VEL
MAX.=16.696(CM/SEC)

E.Q. NO. 850919



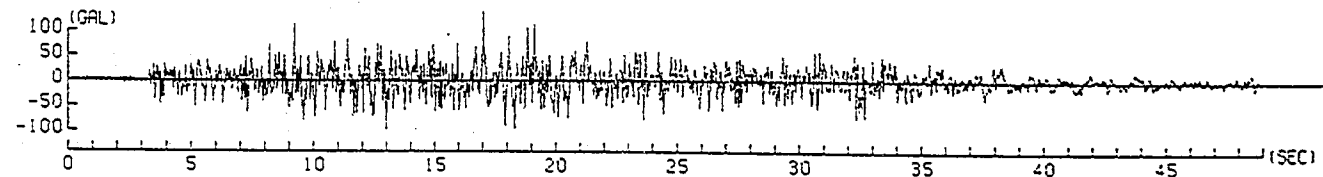
INDEX NO. 3
CALE 21 N90E
OR 13:17:49.DIS
MAX.=5.734(CM)

E.Q. NO. 850919



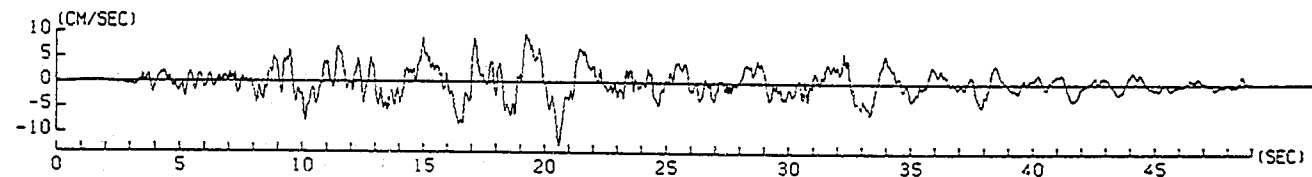
INDEX NO. 4
CALE 21 N90E
OR 13:17:49.ACC
MAX.=137.640(GAL)

E.Q. NO. 850919



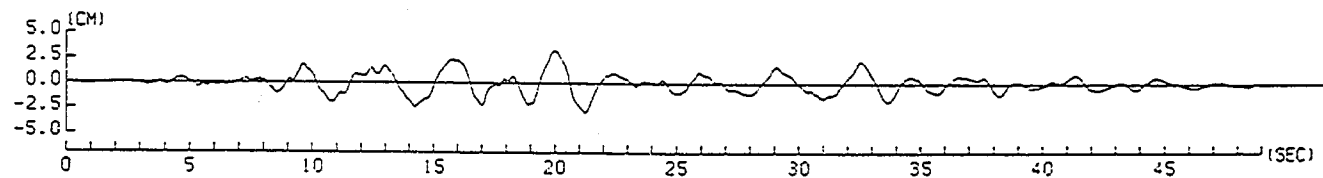
INDEX NO. 5
CALE 21 N90E
OR 13:17:49.VEL
MAX.=12.640(CM/SEC)

E.Q. NO. 850919



INDEX NO. 6
CALE 21 N90E
OR 13:17:49.DIS
MAX.=3.216(CM)

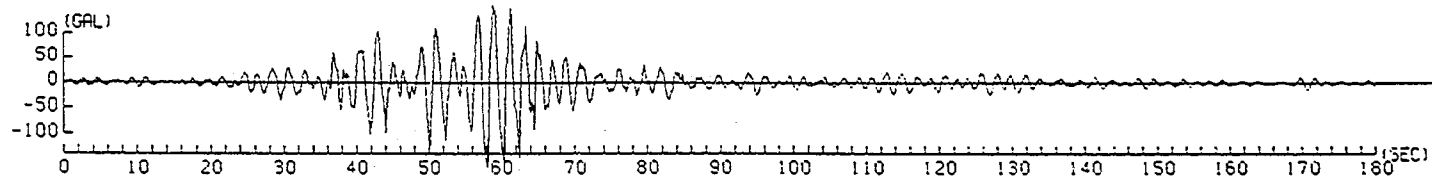
E.Q. NO. 850919



Example of Waveforms
(SCT1, Mexico City, $\Delta=400\text{km}$)

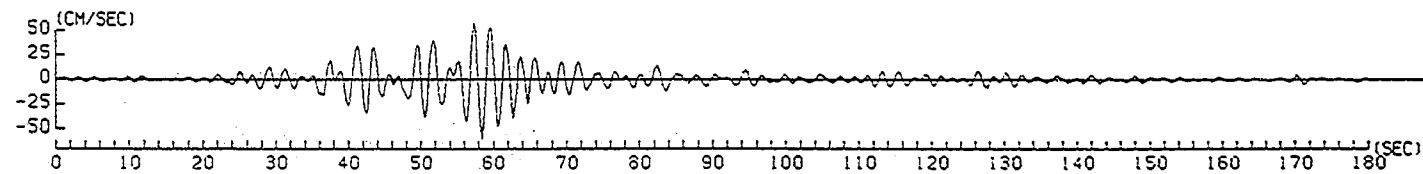
INDEX NO. 49
SCT1 OR 400 N90W
13:19:23 ACC.
MAX.=-167.918(GAL)

E.O. NO. 850919



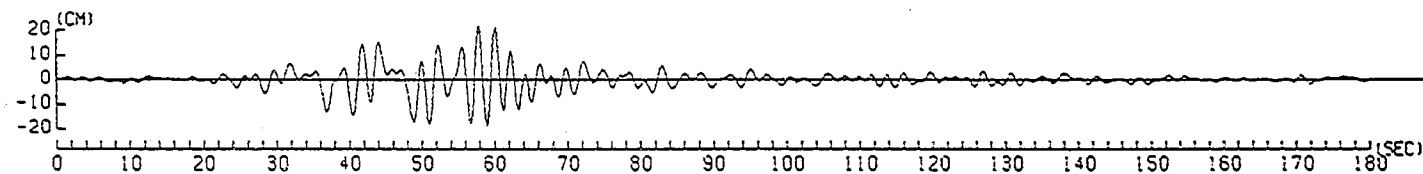
INDEX NO. 50
SCT1 OR 400 N90W
13:19:23 VEL.
MAX.=-60.499(CM/SEC)

E.O. NO. 850919



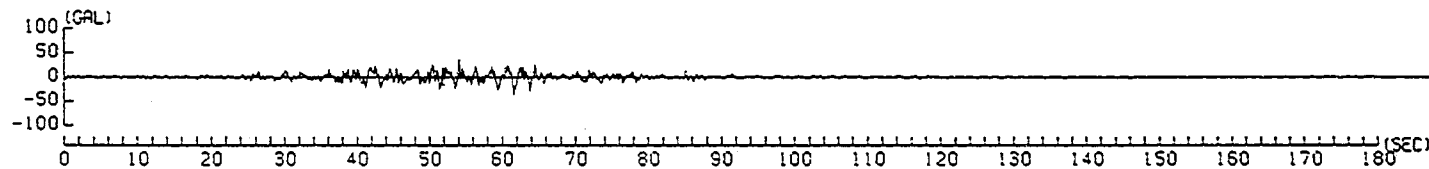
INDEX NO. 51
SCT1 OR 400 N90W
13:19:23 DIS.
MAX.=-21.936(CM)

E.O. NO. 850919



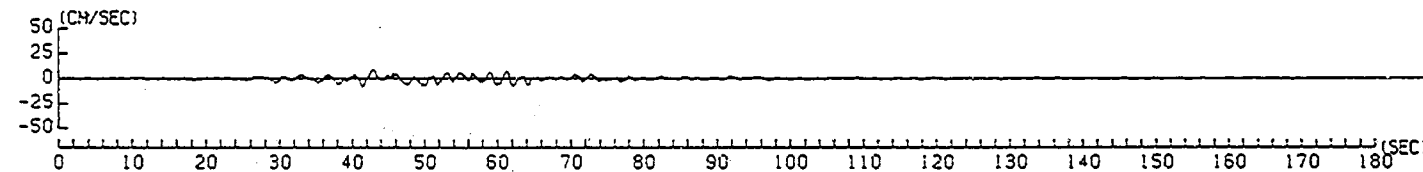
INDEX NO. 52
SCT1 OR 400 VERT
13:19:23 ACC.
MAX.=-36.637(GAL)

E.O. NO. 850919



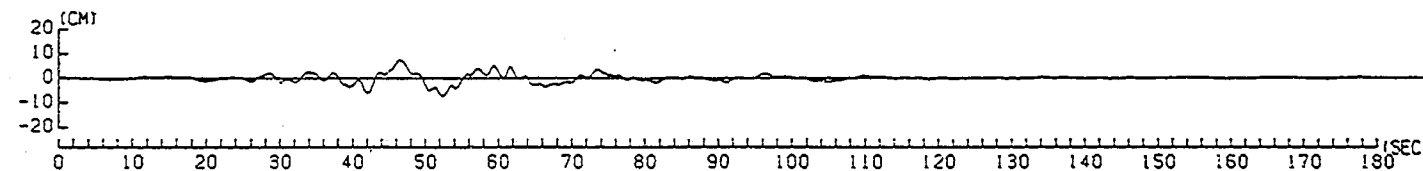
INDEX NO. 53
SCT1 OR 400 VERT
13:19:23 VEL.
MAX.=-8.996(CM/SEC)

E.O. NO. 850919



INDEX NO. 54
SCT1 OR 400 VERT
13:19:23 DIS.
MAX.=-7.578(CM)

E.O. NO. 850919



REPORTS OF VISITING
RESEARCHERS AT GROUND
VIBRATION DIVISION OF PUBLIC
WORKS RESEARCH INSTITUTE

PWRI TECHNICAL MEMORANDUM
NO. 2554
January, 1988
Yasushi Sasaki

Key Words: In-Situ Testing, Liquefaction, Slope Stability, Attenuation of
Ground Motion, Mexico Earthquake of 1985, Earthquake
Engineering

Synopsis :

This report presents the works done by three foreign researchers at the
Ground Vibration Division, Earthquake Disaster Prevention
Department of Public Works Research Institute during the fiscal
year of 1986. Following three subjects are included :

- 1) Study of In-Situ Testing for Evaluation of Liquefaction
Resistance and Occurrence
- 2) Research on the Effects of Topographic Amplification of
Earthquake Shaking on Slope Stability
- 3) Attenuation of Peak Ground Motions from Mexico Earthquakes
of September 19 and 21, 1985

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GROUND VIBRATION DIVISION OF
PUBLIC WORKS RESEARCH INSTITUTE