THE EARTHQUAKE HAZARD
IN
CHRISTCHURCH

A DETAILED EVALUATION

SOILS & FOUNDATIONS LTD

EQC

THE EARTHQUAKE HAZARD IN CHRISTCHURCH

A DETAILED EVALUATION

Prepared by

D.McG. Elder; I.F. McCahon and M.D. Yetton
Soils & Foundations Ltd

with assistance from

University of Canterbury

March 1991

Study funded by

EQC

CONTENTS

SECTION III. EARTHQUAKE HAZARD: CONSEQUENCES FOR CHRISTCHURCH AND POTENTIAL DAMAGE

SECTION IV. CONCLUSIONS OF EARTHQUAKE HAZARD STUDY FOR CHRISTCHURCH

EXECUTIVE SUMMARY

EXECUTIVE SUMMARY

In considering the earthquake hazard in Christchurch it is useful to apply the law of precedence: the past is the best indicator of the future. In the first 80 years of the city's history, four large earthquakes significantly damaged the growing settlement, one seriously. Any one of these four events today would cost the city millions of dollars in direct damage and could result in major disruption to the local economy. The largest of these events was virtually under the city, with an epicentre close to New Brighton. It is nearly 70 years since the last large event (1922, Motunau Earthquake, intensity VII in the city - refer Fig. S1). When can the next similar event of this size be expected? In this report, the first detailed seismic hazard assessment of the city, we have attempted to answer this question by adopting current seismic hazard analysis techniques and applying them specifically to Christchurch.

Seismic hazard analysis involves three components: a Seismicity Model (a model of earthquake occurrence probability in regions close enough to affect the city), an Attenuation Model (approximating energy loss and wave modification as the seismic waves travel to the basement rock under the city) and a Site Response Model (predicting the changes to the earthquake waves as they propagate up through the gravels, sands and silts underlying the city).

The Seismicity Model developed here makes use of the traditional Gutenberg Richter occurrence relationship ($\log N = a - bM$). In the common case when b is close to one, an approximate tenfold reduction in earthquake occurrence occurs with each step up the Richter magnitude scale. Thus by knowing the number of relatively frequent small earthquakes, the average recurrence interval of more infrequent larger events can be predicted. The basic model has been refined for the central and northern South Island by subdividing the area into 10 seismicity zones. In each zone the maximum credible earthquake has been inferred from the length (and, where known, the displacement per event) of the known active faults in the zone.

Probabilistic information is obtained from the number of earthquakes historically recorded in the zone over a given period of time. For magnitudes less than 6 the recorded instrumental data is from 1964 - 1988; for magnitudes greater than 6 and less than 6.5, 1940 - 1988; and magnitudes greater than 6.5, 1840 - 1989. It should be noted that even the period 1840 - 1989 is much shorter than the return period for major earthquakes on many faults near Christchurch and, while being the maximum record available, this time span is still relatively short.

Analysis indicates that potential exists for relatively rare but very large earthquakes (magnitude approximately 8) along the Alpine Fault, which essentially marks the western edge of the Southern Alps. More frequent moderate to large earthquakes (magnitude around 6 - 7.5) can be expected in the Canterbury Plains foothills and

North Canterbury area, and less frequent moderate earthquakes under the Canterbury Plains and Christchurch itself. The Attenuation Model predicts that the damage in the city from these three types of event are likely to be similar. Of the four serious earthquakes in the early city history, three occurred in the foothills and North Canterbury region (the Amuri, Cheviot and Motunau Earthquakes) and one virtually beneath the city (the New Brighton Earthquake).

An important component of this study has been to consider the additional effect at Christchurch of the deep, relatively soft sediment underlying the city (the Site Response Model). This creates major changes in the nature of the earthquake shaking by modifying the ground acceleration, velocity and displacement at any frequency. In some areas of the city the earthquake vibrations are amplified (see Figure 7.22) As a result the overall average hazard for the city increases when compared to areas on bedrock, (for example most of Banks Peninsula) by approximately 0 to 2 intensity units, or by 0 - 1 units when compared to areas on 'average ground' (comprising shallow sediment). Within the city distinct local variation results in particular from gradational changes in the top 30 m of sediment.

On average the calculated probabilities for various intensities of shaking in Christchurch are as follows (see Figure S2 or Figure 7.5a):

Modified Mercalli Intensity	**Approximate Expected Effect**	**Average Return Period**
Intensity VI	Minimal property damage	7 years
Intensity VII	Some property damage Loss of life unlikely	20 years
Intensity VIII	Significant property damage Loss of life possible	55 years
Intensity IX	Extensive property damage Some loss of life	300 years
Intensity X	Catastrophic property damage. Major loss of life.	In excess of 6,000 years

These probabilities indicate that Christchurch has an overall seismic hazard level comparable to Wellington for medium intensity earthquake shaking. However this level of hazard is lower than that in Wellington for very large catastrophic events.

Section III of the report considers surface ground damage which may occur associated with an earthquake. The greatest concern for Christchurch, located near a saturated, sand and silt rich, prograding coastline, is the potential for liquefaction. This phenomenon occurs when the tendency for loose granular materials to compact during earthquake shaking results in a pore water pressure increase, and reduction or total loss in strength. This may cause subsidence, foundation failure and damage to services. Analysis shows that large areas of the city are underlain by sands or silts which, if sufficiently loose, would be highly susceptible to liquefaction. Although insufficient soil testing has been carried out to characterise densities in all areas, extensive investigation has been done in the central city. Some silts and sands in this area are loose and extremely vulnerable to liquefaction.

It should be noted that the historical earthquakes which the city has experienced do not appear to have generated significant liquefaction in the city areas occupied at the time. However, when considering the magnitude and location of these four historical earthquakes, available analysis models indicate they were probably too small to initiate the process. The lack of past evidence does not exclude the possibility of liquefaction in the future.

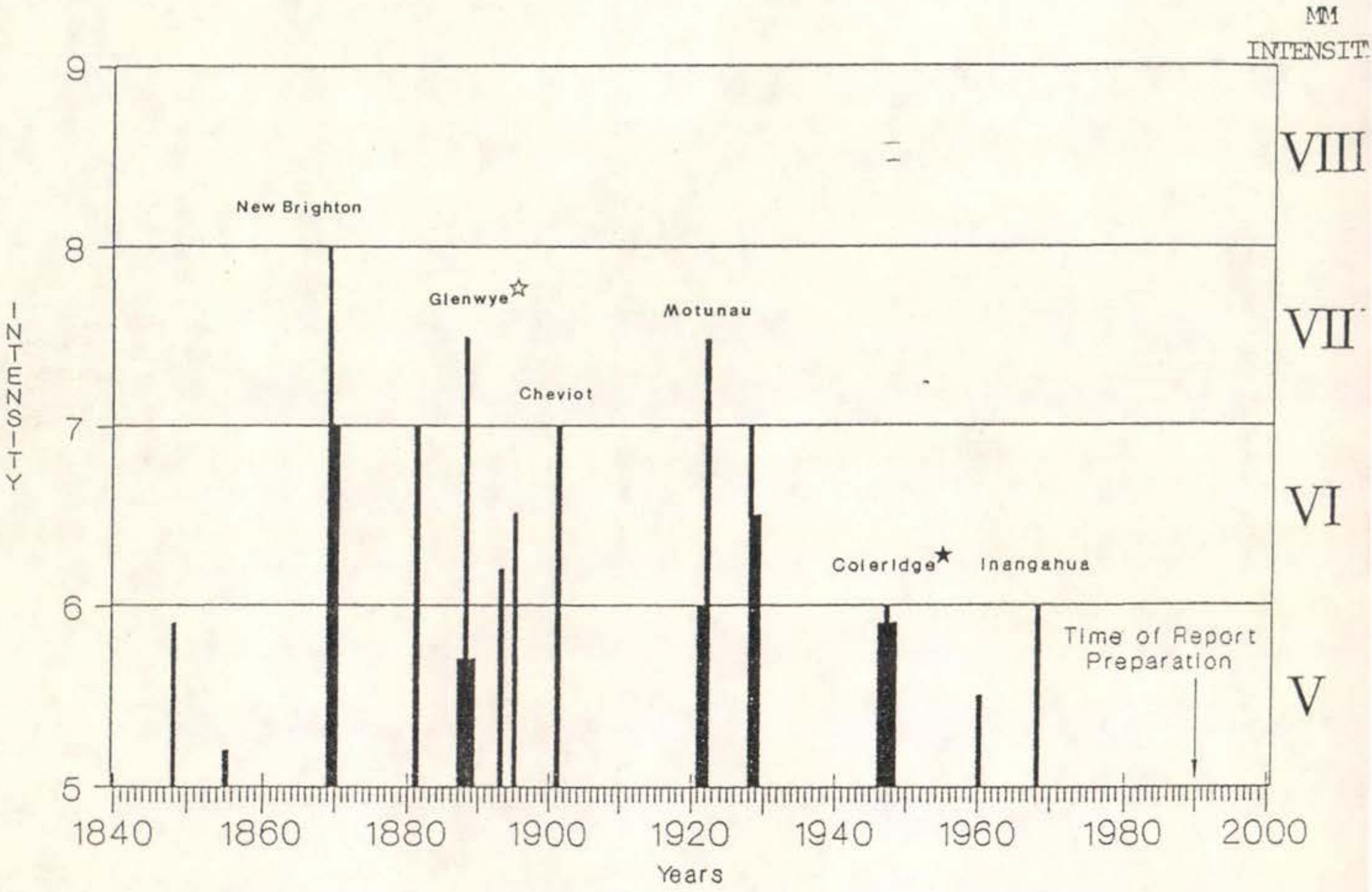
Figure S3 summarises the available information regarding areas of potential liquefaction and areas of expected seismic wave amplification. It is obvious from examination of the figure that a very large part of the eastern city is potentially subject to liquefaction while amplification effects are pronounced in the areas to the north of the central city and in scattered south-western areas.

Consideration of the likely effect of a large earthquake in the hill areas suggests damage by landsliding is likely if a large earthquake coincides with the winter wet period. Wet or saturated conditions only exist each year for a relatively short time (two to four months), but assuming wet conditions exist, the probability of significant damage from soil slope movement in the hill areas is estimated to have a return period of around 300 years. Houses below steep hillslopes in rural catchments are generally most at risk. However liquefaction induced landsliding in alluvial materials along the lower reaches of the Avon and Heathcote rivers, and around the margins of the Estuary, may be a more significant hazard.

Chapter 11 of the report considers briefly the potential damage to the city in terms of the impact on physical structures. One consequence of the effect of the deep alluvium beneath the city is to reduce the structural response at high frequencies and amplify the lower frequency shaking. This will probably subject mid to high rise structures to levels of resonant shaking exceeding those anticipated by present design methods. Residential buildings may be shielded from resonance effects, however the anticipated large amplitude ground motion may cause inertial effects. Damage to contents, heavy furniture and fittings, hot water cylinders and chimneys is likely.

Possibly the most significant physical impact on the city may be damage to water, sewer and power supply services. With the depths of relatively soft alluvium under the city, the strains experienced by pipelines are expected to be high, with corresponding high pipe stresses and pipe joint displacements. If liquefaction occurs the sewerage reticulation system and treatment station could be severely damaged. Liquefaction could also affect main electrical supply nodes. Inertial loads on electrical heavy equipment at substations are likely to be larger than presently assumed for design with a corresponding high incidence of power failure. Subsidence or displacement of the very soft reclaimed land at Lyttelton is likely (little of this land existed at the time of previous large historical earthquakes) and the associated damage to oil tanks could have serious environmental consequences.

We have not attempted an in-depth lifelines study for Christchurch, or included economic or sociological analysis in this report. In addition to the need for this type of work, we recommend further action from the engineering profession including a review of the current seismic loadings code, local seismic design practices and building stock. We suggest site specific studies for the Lyttelton tank farm, Bromley sewerage ponds, pumping stations, substations, hospitals, civil defence facilities, airport and key bridges. Major areas of further research include studies of sand density variations and susceptibility to liquefaction across the city; continued paleoseismic evaluation of adjacent active faults, particularly the Alpine Fault, and further investigation of the deep sediments below the city.



☆ Three Earthquakes in 1888 of I = 5.7,5.7 & 7.0

★ Two Earthquakes in 1948 at I = 5.9 & 6.0

FIGURE S1

Historical Earthquake Record for Christchurch

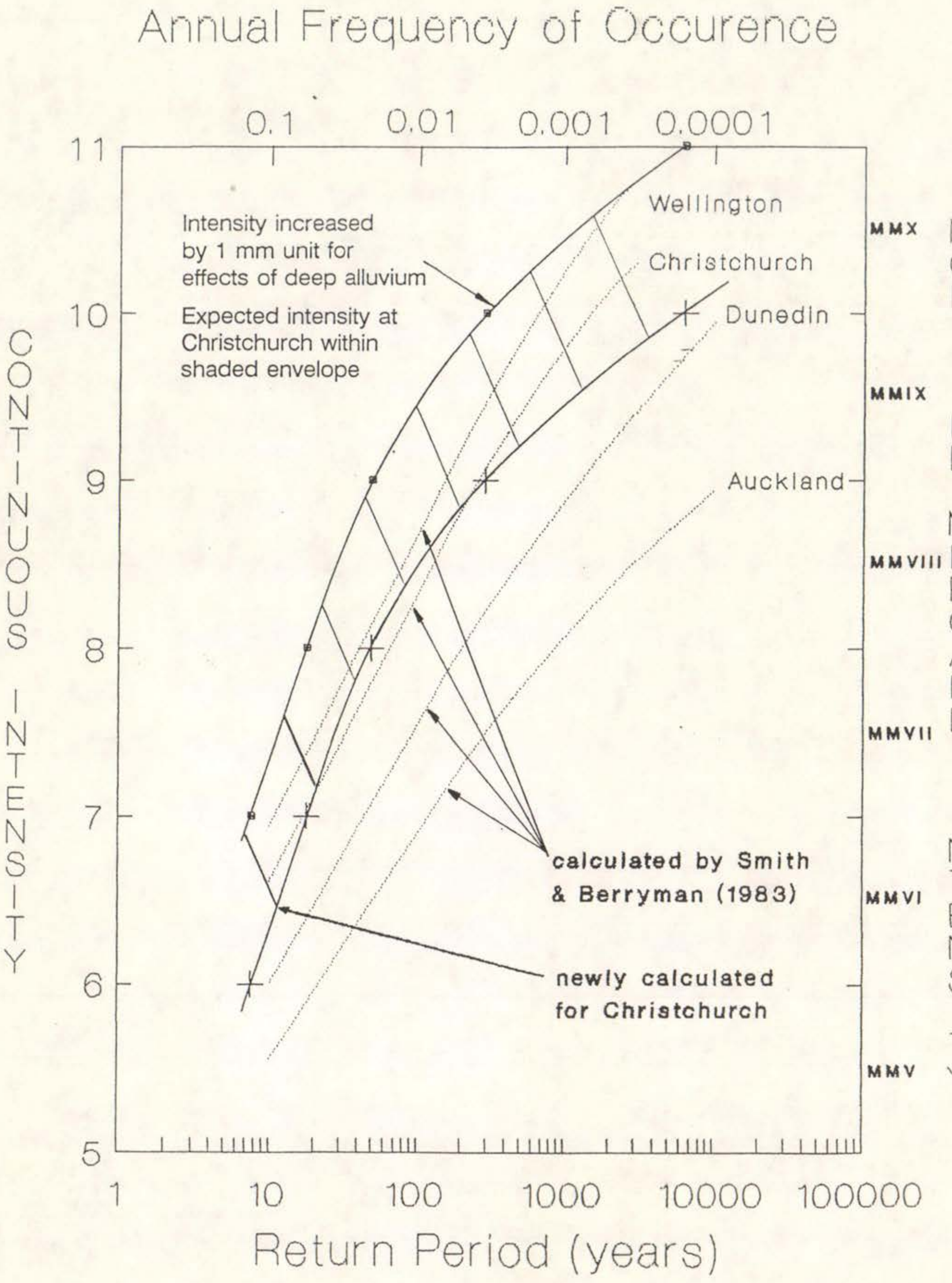
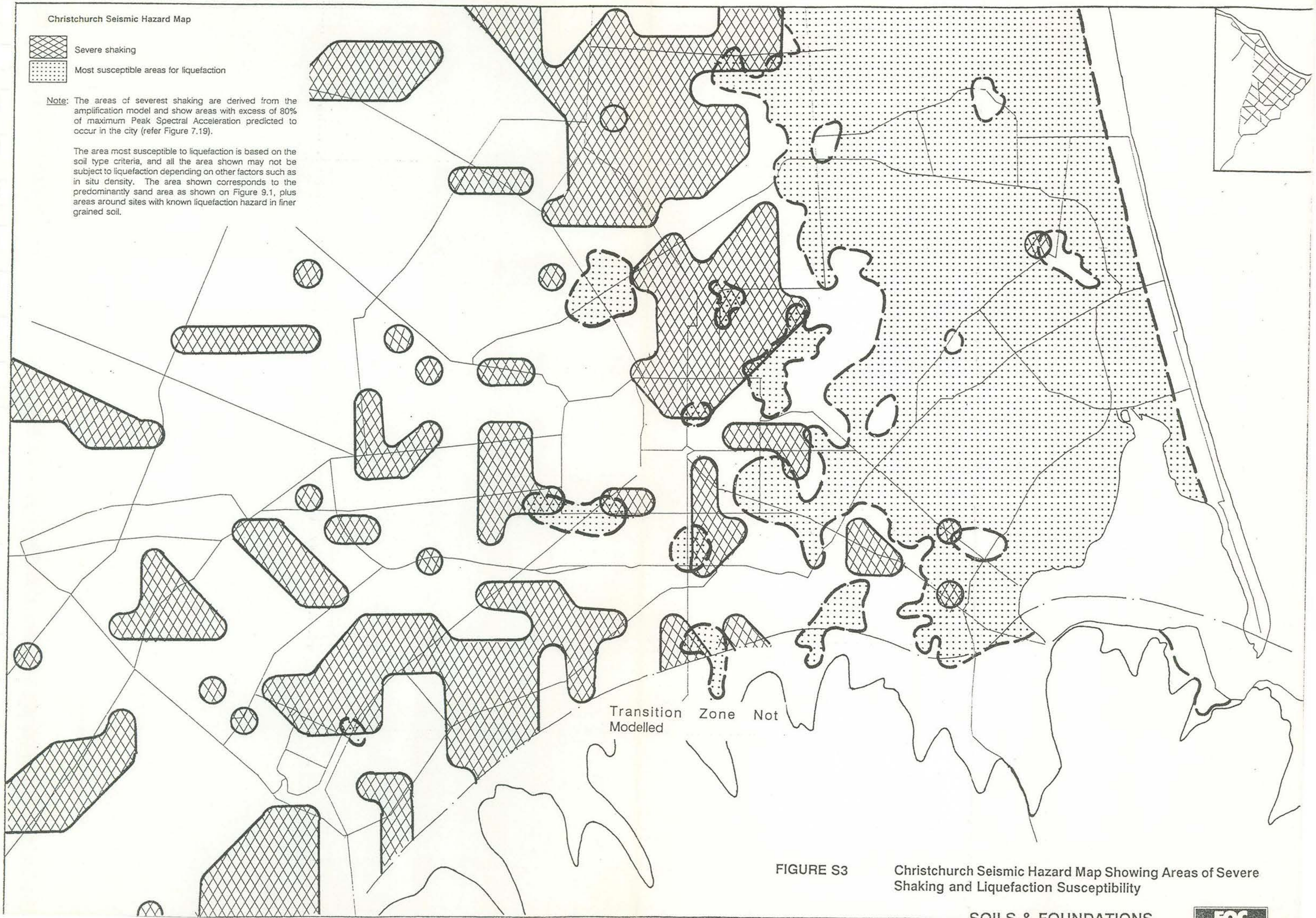


FIGURE S2

Occurrence Frequencies for Seismic Intensities at Christchurch - corrected for ground amplification effects



Note: The areas of severest shaking are derived from the amplification model and show areas with excess of 80% of maximum Peak Spectral Acceleration predicted to occur in the city (refer Figure 7.19).

The area most susceptible to liquefaction is based on the soil type criteria, and all the area shown may not be subject to liquefaction depending on other factors such as in situ density. The area shown corresponds to the predominantly sand area as shown on Figure 9.1, plus areas around sites with known liquefaction hazard in finer grained soil.

FIGURE S3 Christchurch Seismic Hazard Map Showing Areas of Severe Shaking and Liquefaction Susceptibility

SECTION I. EARTHQUAKE HAZARD: CHRISTCHURCH STUDIES

CHAPTER 1: INTRODUCTION

Many destructive earthquakes have occurred in New Zealand since reliable historic recording began about 1840. On average, a major earthquake (magnitude $M \geq 7$) has occurred every six to eight years, although there has been a marked quiescence of large earthquakes during the past 40 years.

The largest earthquake recorded in New Zealand since 1840 was the 1855 Wairarapa earthquake, with probable magnitude[1] $M = 8.0$. This caused a felt intensity in Christchurch, 300 km distant from the epicentre, of about Modified Mercalli Intensity MM V. In contrast, the greatest generally felt intensities reported in Christchurch were probably up to MM VII or VIII. These intensities were caused by the relatively unknown 1869 earthquake when the city was a sparse scattering of small buildings contained within the four avenues. The earthquake was around $M = 5.5 - 6.0$, with an approximate epicentre 10 km from the city centre in the New Brighton area. Other larger earthquakes a little later in the city's history included the 1888 North Canterbury earthquake with $M = 7.0 - 7.3$, centred 104 km from Christchurch near Glynn Wye, and the 1901 Cheviot earthquake with $M = 7.0$, centred near Parnassus, 60 km from Christchurch. Both these later events caused damage in Christchurch which included fallen chimneys and, most notably, the toppling of the upper part of the Cathedral spire (see front cover). Intensities reported indicate local amplification in certain parts of the city.

These observations of large earthquakes affecting Christchurch highlight the three most important factors which influence seismic hazard at a particular location:

- the seismicity, or number of earthquakes in a source region exceeding a given magnitude in a given period of time;
- the attenuation of observed effects with distance from the epicentre of an earthquake;
- the potential amplification effects which may vary with ground conditions at each observed location.

Seismic hazard assessments have been carried out in New Zealand on both a national and local basis. National studies have necessarily been on a coarse scale and taken only nominal account of specific regional factors such as fault seismicity or attenuation pattern variations. Localised research has concentrated on either critical facilities, such as major dam sites, or populous centres with either an historic record of large nearby earthquakes or proximate, active faults. For these reasons considerable work

---

[1] Throughout this study, no attempt has been made to differentiate between the several specific definitions of magnitude which exist. The reason this has not been done is that this study draws on technical information and theory from many sources. Most of these sources do not define the magnitude precisely. Furthermore, magnitudes deduced for many historical earthquakes are estimates only, and to attempt further refinement would imply an unwarranted accuracy.

has been carried out on the seismic hazard in Wellington but little specific assessment done for Christchurch, where no devastating earthquake effects have occurred in recent recorded history and which, until recently, has been thought to have few close active faults.

The realisation, from results of seismic hazard studies carried out for specific sites in Christchurch, that many areas of the city are particularly susceptible to damaging effects under medium to strong bedrock excitation, provided the impetus to carry out the extensive research reported here. The current New Zealand Loadings Code requires structures on very deep, uncemented soils to be subject to "special studies". These should therefore strictly be carried out for all structures in Christchurch, but in practice for all buildings greater than about 3-5 stories in height. It is not clear how frequently this requirement has been adhered to.

In addition, recent geological and geodetic studies are providing new information on fault zones in the north Canterbury region. Some of these faults have the potential to generate large earthquakes and subject Christchurch to effects considerably more severe than any experienced in recent historic times. Other work on the Alpine Fault, relatively quiescent during the past 150 years, suggests the regular occurrence of very large earthquakes, possibly with magnitude $M \geq 8$, on the central section of the fault which passes within 125 km of Christchurch. Best available estimates suggest the mean interval between each of the last four such earthquakes was about 550 years; the most recent was about 550 years ago. The localised effects in parts of Christchurch from such an earthquake could cause intensities up to MM X to occur.

Examination of this recent research highlighted the urgent need for a comprehensive evaluation of the earthquake hazard in Christchurch, taking into account all the information currently available. This report comprises the results of that evaluation.

In some areas, little of the specific information required as input into seismic hazard models is available because no previous work has been carried out. This applies particularly to detailed geologic investigations of some significant faults or zones of high seismicity within 100 km of Christchurch. In these cases best available estimates of likely parameters have been made considering general regional behaviour, historical seismicity records, and following discussions with researchers working in those areas, particularly at the Geology Department of the University of Canterbury. In other areas the forms of mathematical models used in hazard prediction are somewhat subjective. Current and future research in these areas may allow their refinement or modification. In all cases, however, we have either used models previously employed by others for New Zealand studies, or where separate analyses have been developed, the preference for these has been justified by available data. In our evaluations we have attempted neither to be unnecessarily conservative (i.e. alarmist), nor to downgrade predicted effects where there is no reasonable justification for this. Inclusion of probabilistic assessments has allowed natural variability in model techniques or data to be quantified.

Each major section of this report has been critically reviewed by at least one independent expert in the appropriate area. Where a result or conclusion may be controversial, we attempt to highlight and discuss this in the text. As a result, we believe the predictions presented in this report represent the best currently available for the seismic hazard in Christchurch. We expect that results of research already in progress and future work recommended here, may eventually allow our predictions to be refined in some areas.

REPORT OUTLINE

The report is divided into thirteen chapters under four major headings.

Section I introduces the study, describing previous work of relevance to Christchurch (Chapter 2) and briefly summarising the analysis philosophy, predictive methods available, and those adopted (Chapter 3).

In Section II all available information on regional earthquakes, active faults and historical seismicity is described (Chapter 4). This is combined to produce a seismicity model, which can be used in conjunction with attenuation models to predict intensities (Chapter 5) and structural acceleration response spectra (Chapter 6), to determine the effects at Christchurch on a probabilistic basis for various future time periods.

Specific modifications to earthquake effects caused by the variable geologic profile beneath Christchurch are considered in Chapter 7. Compiled soil information from over 15,000 borelogs is presented, and ground surface and structural response variations across the city are described.

Section III considers the practical and engineering consequences of the predicted earthquake effects for Christchurch (Chapter 8), including liquefaction and ground displacement (Chapter 9) and hillslope instability (Chapter 10). Potential damage or disruption to services and structures is discussed briefly in Chapter 11, although detailed considerations are beyond the scope of this study.

Conclusions drawn from the study are described in Section IV, including recommendations for engineering design in Christchurch (Chapter 12), and for other measures which can readily be adopted in future to mitigate the consequences of a severe earthquake (Chapter 13). Recommendations for further work required to refine or validate the results of this study are outlined.

CHAPTER 2: PREVIOUS STUDIES AND CURRENT HAZARD PERCEPTION

A number of previous workers have attempted to quantify earthquake hazard in New Zealand on a national or regional basis.

Early studies were reviewed by Smith (1976); only those particularly relevant to Christchurch are summarised here. Geologic studies, specific hazard prediction techniques, and analyses of consequential effects such as liquefaction, are described in later chapters.

Results of the first comprehensive national studies were reported by Bastings and Hayes (1935) and by Hayes (1936a, 1936b, 1941, 1943) who estimated the frequency of occurrence of felt earthquakes, and of resulting intensities (using the Rossi-Forel scale), on a regional basis and for selected cities and towns. Even at this early stage Hayes (1936b) noted that Christchurch regularly experienced intensities higher than those expected for 'normal' locations. Clark et al (1965) zoned New Zealand for earthquake hazard severity on the basis of geologic considerations. Dickenson & Adams (1967) compiled maps contouring the frequency of earthquake occurrence through the country, and Eiby (1971) refined rates of earthquake activity using both instrumental and historical data.

The first comprehensive study employing both a seismicity model based on historical seismicity records and an attenuation model to predict felt intensities was that reported by Smith (1976, 1978a, 1978b). He estimated return periods for Modified Mercalli intensities in Christchurch of: MM VI, 20 years; MM VII, 50 years; MM VIII, 100 years; MM IX, 250 years. (The probability of a particular intensity being felt once at a particular location in a time interval equal to its return period at that location is 63%). It is often more appropriate to determine the intensity which has a given, low probability of occurrence; Smith estimated the intensity with a 5% probability of occurrence in Christchurch in 50 years to be MM X. He also noted that for 'poor soil' conditions (not specified, but dependent on geologic conditions at a particular site) predicted intensities could be up to one Modified Mercalli unit higher. This would certainly be expected to apply in Christchurch. Walley (1976) carried out a similar study, developing a rigorous statistical model for intensity attenuation with epicentral distance.

Matuschka (1980) and Peek (1980) both developed models to predict the frequency of occurrence of intensities or spectral accelerations throughout New Zealand. Although they used different attenuation and seismicity models the results of the analyses were similar in many respects. Peek estimated the spectral acceleration at a natural period T = 0.2 seconds (close to the peak spectral acceleration) for bedrock at Christchurch, with a return period of 150 years, to be about 0.4 g.

Mulholland (1982) analysed the attenuation model used by Peek and proposed modifications which substantially increased spectral accelerations for periods less than T = 0.8 seconds, based on limited New Zealand accelerogram data. He validated the simplifying assumption suggested by Peek that the spectral shape does not vary significantly with location due to the combined smoothing effects of integrating probabilistic seismicity and attenuation models. Mulholland prepared contour maps of 150 year return period spectral acceleration for T = 0.15 s, and showed that these contours are sensitive to the choice of seismicity model. However the two seismicity models used (Peek, 1980; Smith & Lensen, 1981) both yielded similar 150 year spectral accelerations of about 0.65 g for Christchurch, or over 50% higher than the spectral acceleration suggested by Peek.

Smith & Berryman (1983) extended the earlier studies by Smith by dividing the country into regions of uniform seismicity. However they adjusted the seismicity model, based initially on historical and instrumental seismicity, to allow for geological evidence of earthquake events from observed ground deformation. They employed similar integration techniques to those of Peek (1980), Mulholland (1982) and Matuschka (1980), but instead estimated intensities throughout New Zealand using the intensity attenuation function presented in Smith's earlier work. This revised study predicted increased probabilities of low intensities at Christchurch, but a reduction in the hazard from high intensity events. Return periods for Modified Mercalli intensities in Christchurch were estimated to be: MM VI - 14 years, MM VII - 48 years, MM VIII - 160 years, MM IX - 600 years.

Berrill (1985a) at that time, concluded that the Smith & Berryman seismicity model was the best available, but recommended revision of seismicity parameters in the Alpine Fault region to reflect the increased likelihood of great earthquakes suggested by geologic evidence. This suggestion was incorporated by Matuschka et al. (1985) in presenting results of a seismic hazard analysis for New Zealand carried out by the Seismic Risk Subcommittee of the Standards Association. Some modifications to the attenuation model employed by Peek (1980), and Mulholland (1982), were also included, as were further modifications as discussed by McVerry (1986). Maps showing contours of constant spectral acceleration for period T = 0.2 seconds were presented for "Ground Class 3". However at this natural period, little difference is predicted between acceleration response values for Ground Class 1 (used by Peek and Mulholland), and Class 3. Predicted spectral accelerations for Christchurch at various return periods were:

Return period (years)	50	150	450	1000
Spectral acceleration, a_s/g	0.3	0.45	0.8	1.0

The response spectra calculated from these results using normalised spectral shapes form the basis for the design spectra proposed in the draft revision of the current Design Loadings Code, (NZS 4203: 1984) as discussed by Hutchison et al, 1986.

Together with the predictions of intensities made by Smith & Berryman (1983), these have represented the best estimates of earthquake hazard applicable to Christchurch to date. However when considering the three most important factors in seismic hazard determination (see Chapter 1), only the attenuation model is as applicable to a comprehensive site-specific study at one location as it is in the national study for which it was derived. The remaining two factors - the regional seismicity model and the local amplification due to site-specific ground conditions - need to be addressed in considerably more detail when carrying out a detailed seismic hazard study for Christchurch.

Cowan & Pettinga (in press) review general seismic hazard in Canterbury primarily discussing seismotectonics. A preliminary discussion of the relevance of Christchurch geology to earthquake hazards is reported by Brown et al (in prep.).

No detailed seismic hazard assessment for Christchurch has been carried out prior to this work, although site-specific studies have been undertaken (e.g. Dibble et al, 1980; Davis & Berrill, 1988; Soils & Foundations, 1988). The latter two studies considered amplification effects at a particular location in the city by analysing shear wave propagation through deep alluvial strata, however they employed national (coarse) seismicity models without refinement. These early studies showed that significant frequency-dependent amplification of spectral accelerations can be expected in Christchurch which are not able to be accounted for adequately either by existing spectral generation models, or by the allowance for 'soft soil' effects made in the current N.Z. Loadings Code.

CHAPTER 3: EARTHQUAKE HAZARD ANALYSIS TECHNIQUES

Seismic hazard analysis at any site requires information to be incorporated into models in two separate areas. A seismicity model describes the rate of occurrence of earthquakes of different magnitude in each source region. Such a model is generally based on instrumental records defining the magnitude and epicentre of historic earthquakes. Since these data are limited to the short period during which instrumental records have been maintained, extended felt records for large historical earthquakes are usually included, although the magnitudes and locations of these earthquakes are often poorly defined. Where geologic evidence of fault displacements and recurrence intervals associated with particular earthquake events is available, this provides very useful information for calibration and refinement of the probabilistic model, particularly for large magnitudes. Alternatively such evidence may be used directly in deterministic analyses. For some source regions, where historic seismicity records show an absence of medium sized earthquakes yet geologic evidence suggests regular large earthquakes, the deterministic approach may be more appropriate unless the seismicity model can be adjusted to include these observations. The central section of the Alpine Fault is one of these regions. Geologic information also provides a valuable indication of the maximum likely magnitude for earthquakes in a given region. For large earthquakes, incorporation of an upper magnitude bound has a significant effect in reducing the occurrence probability predicted by a seismicity model.

An attenuation model describes the ground shaking effect produced at a site away from the source of the earthquake, generally as a function of magnitude and epicentral distance. Many forms of model have been proposed, and considerable variability exists in predicted effects, both within models where probabilistic effects are included, and between different models.

Although soil characteristics are theoretically part of the attenuation model, in practice it is necessary to make additional allowance for variable site effects caused by the specific geologic profile at any site. Some attenuation models include these effects directly using simplified groupings of ground characteristics. In this study, a general source-to-site attenuation model is used to predict the bedrock motion beneath overlying alluvium at Christchurch, then a separate deep soil response model is employed to determine the variable effects throughout Christchurch caused by spatial soil inhomogeneity in the deep alluvium.

These separate areas of input into the seismic hazard analysis for Christchurch are summarised in Figure 3.1.

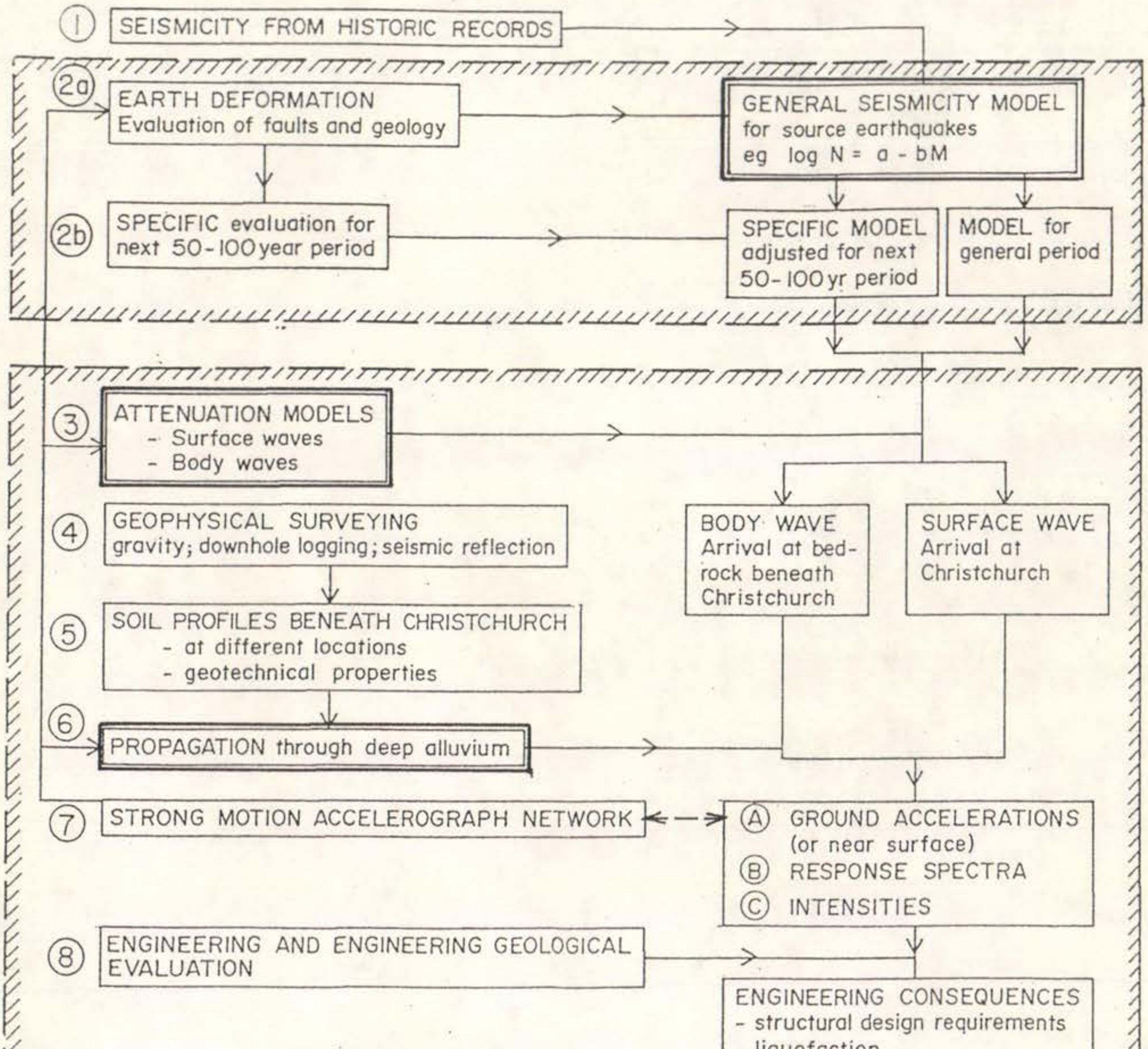


FIGURE 3.1

Seismic Hazard Analysis

SECTION II. THE PREDICTIVE MODEL FOR CHRISTCHURCH

CHAPTER 4: SEISMICITY: POTENTIAL EARTHQUAKES AFFECTING CHRISTCHURCH

4.1 INTRODUCTION

The best available general seismicity model for New Zealand is that described by Smith & Berryman (1983), together with the modification for the Alpine Fault region used by Matuschka et al (1985). However this model is relatively coarse; it uses only seven regions of constant seismicity within 300 km of Christchurch. In this chapter a revised seismicity model is developed, specifically for use in the prediction of the seismic hazard for Christchurch, but having general application to any part of the central South Island.

Section 4.2 considers the seismotectonic geology of the region and identifies known active faults with the potential to generate damaging earthquakes at Christchurch. Previous studies and those currently in progress are reviewed. Particular use is made of preliminary results from concurrent, collaborative research into the seismotectonics of the North Canterbury area by workers in the Department of Geology at the University of Canterbury. Provision of these results is gratefully acknowledged. All available information on significant known active faults is tabulated and used to predict the most likely maximum magnitude (wherever possible) or otherwise the maximum credible earthquake which might be generated in each fault zone.

Large historical earthquakes for the period 1840-1990 and instrumentally recorded earthquake records for more recent periods are evaluated in section 4.3. Three seismicity regions are defined close to Christchurch which have moderate recorded seismicity but, being either offshore or covered by deep alluvium, have no well defined surface fault traces. The evidence for deeper faulting, and potential for generation of medium to large earthquakes, is briefly considered. The seismicity records, in conjunction with geological considerations outlined below clearly suggest appropriate boundaries for seven new seismicity regions, which are defined for central-north Canterbury between the Rangitata River to the south and Waiau River to the north. This refinement avoids the necessary consequence of the coarser zones of Smith & Berryman (1983) that high seismicity near one boundary of a large region is averaged over the whole region.

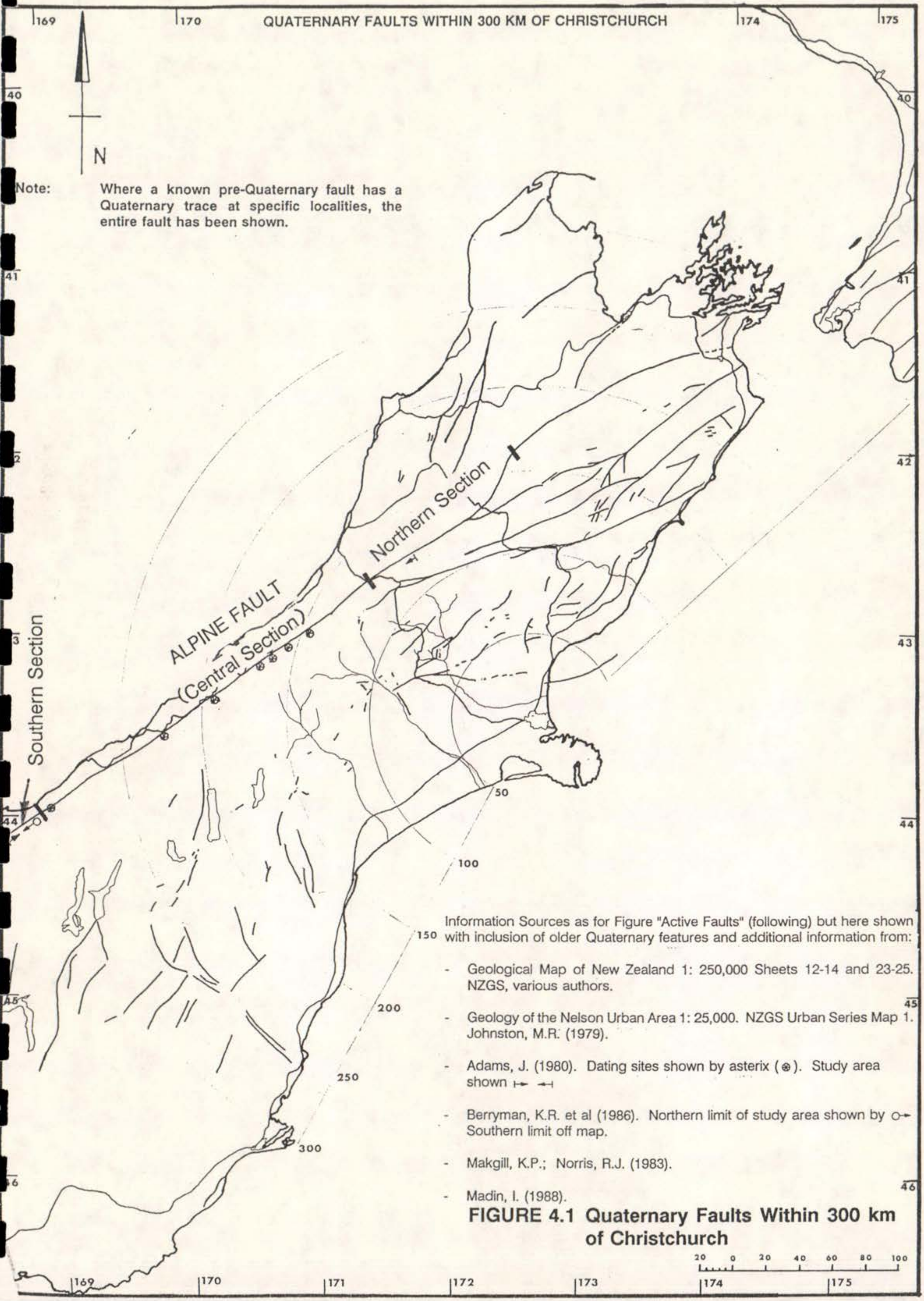
In section 4.4 all geologic and historic seismicity information is compiled to produce a seismicity model for the region. Seismicity occurrence parameters are determined for the seven new regions. Parameters in existing regions are adjusted to account for boundary changes made in this study, or to reflect the detailed geologic evidence, particularly relating to maximum credible earthquake magnitudes able to be generated by particular faults.

4.2 ACTIVE FAULTS

Figure 4.1 shows faults of Quaternary age within 300 km of Christchurch. These are faults known to have moved in the last 1.8 million years. Of greater significance in this study are those faults referred to as "active faults", a term restricted to faults with Late Quaternary traces which are known to have moved in the last 500,000 years, and particularly Class I active faults with movement in the last 5,000 years (Berryman, 1984). Figure 4.2 shows active faults within 150 - 200 km radial distance of Christchurch, the area of most interest in this study. Unfortunately many of the important active faults with potential to cause damage in Christchurch have not yet been studied in sufficient detail to provide paleoseismic information, but some of this work is in progress.

In the absence of specific information, estimates of potential maximum earthquake magnitude used in this study have been based primarily on empirical correlations between surface rupture length or displacement, and magnitude. A number of writers have correlated earthquake magnitude with characteristics of fault rupture length and displacement (e.g. Bonilla & Buchanan,1970; Slemmons, 1977; Mark & Bonilla, 1977; Slemmons, 1982). Unique relationships are unlikely to exist due to the many additional variables such as depth of focus, shape and orientation of rupture surface, variations in regional stress and crustal rigidity, and type of faulting.

The most recent and comprehensive study is that reported by Bonilla et al (1984) who carried out statistical regression modelling for over 100 earthquakes from around the world, including those reported in the earlier studies, subdivided into five different styles of faulting. They deduced that magnitude appeared to be linearly related to log (trace length) and log (displacement). Figures 1c and 2c of their report are reproduced here in Figure 4.3, describing these correlations for faults in their "All" category i.e. strike slip, thrust, normal and reverse faults. This compilation is useful where fault style is in doubt (which is the case for a number of the Canterbury faults). Examination of Figure 4.3 considering the fault classes, indicates that strike slip faults dominate the data set and effectively define the extremes. The correlation ranges drawn are therefore suitable for the majority of faults in the Christchurch study area, since these faults tend to have at least a significant strike slip movement.



Note: Where a known pre-Quaternary fault has a Quaternary trace at specific localities, the entire fault has been shown.

Information Sources as for Figure "Active Faults" (following) but here shown with inclusion of older Quaternary features and additional information from:

- Geological Map of New Zealand 1: 250,000 Sheets 12-14 and 23-25. NZGS, various authors.
- Geology of the Nelson Urban Area 1: 25,000. NZGS Urban Series Map 1. Johnston, M.R. (1979).
- Adams, J. (1980). Dating sites shown by asterix (⊛). Study area shown ↦ ⇤
- Berryman, K.R. et al (1986). Northern limit of study area shown by ○→ Southern limit off map.
- Makgill, K.P.; Norris, R.J. (1983).
- Madin, I. (1988).

FIGURE 4.1 Quaternary Faults Within 300 km of Christchurch

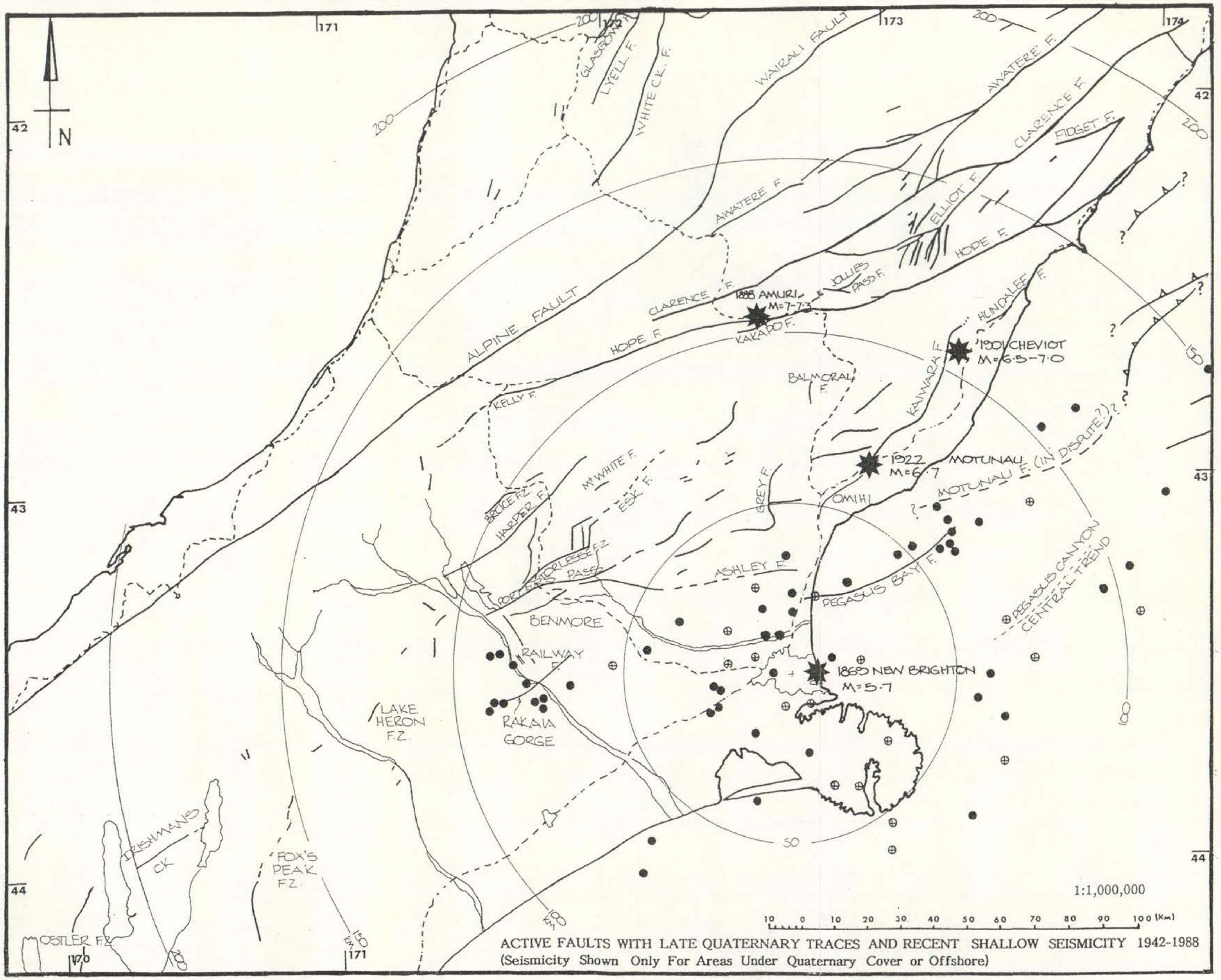
INFORMATION SOURCES FOR LATE QUATERNARY TRACES:

- Geological map of New Zealand 1:250,000, Sheets 15-23
- Quaternary Geology of Sth Island 1:1,000,000 Misc. Series Map 6 NZGS (1973)
- Late Quaternary Tectonic Map of New Zealand 1:2,000,000 NZGS Misc. Series Map 12. Lensen, G.J. (1977)
- Carter, R.M. and Carter, L. (1982)
- Herzer, R.H. and Bradshaw, J.D. (1985)
- Whitehouse, I.E. and Bradshaw, J.D. (1988)
- Beanland, S. (1987)
- Cowan, H.C. (1990)

Note: Where a known older fault has a Late Quaternary trace at a specific locality, the entire fault has been shown.

Information Sources for Seismicity Information (Courtesy of Geophysics Division, DSIR)

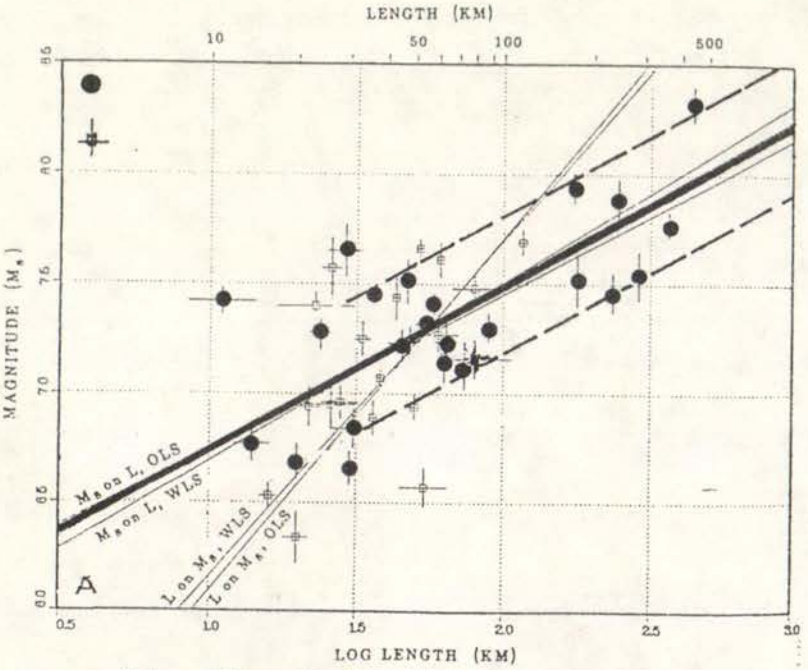
- Crustal seismicity 1942-1964 (shown ⊕)
- Crustal seismicity 1964-1988 (● Magnitudes known accurately)



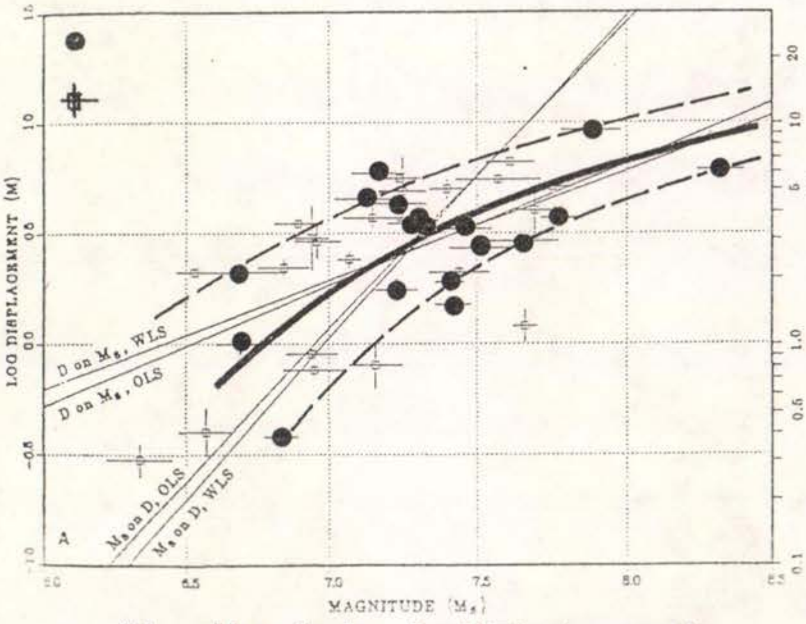
ACTIVE FAULTS WITH LATE QUATERNARY TRACES AND RECENT SHALLOW SEISMICITY 1942-1988
(Seismicity Shown Only For Areas Under Quaternary Cover or Offshore)

FIGURE 4.2

Active Faults Within 200 km of Christchurch



(a) **Magnitude - Rupture Length**



(b) **Magnitude - Fault Displacement**

Key

Strike slip faults
Other fault types
— Bonilla et al linear relationship
— Best fit - this study
– – ±1 standard deviation

FIGURE 4.3

Earthquake Magnitude - Fault Length and Displacement Relationship (Bonilla et al, 1984)

It is evident that the linear relationship described is a reasonable fit to data for magnitude v trace length. For the purposes of this study, the statistical best fit was re-calculated to give more weight to data at magnitudes greater than 7.0 and rupture lengths greater than 30 km, since these earthquakes are of more interest when assessing the maximum magnitude likely to be generated by a given fault. This best fit is shown as a solid line in Figure 4.3a. The range of data for length > 30 km is shown by dotted lines. The range is quite tightly defined and corresponds closely to ± 1 standard deviation from the mean. This range is used to infer ranges in maximum possible earthquake magnitude for the faults considered in this study.

A similar procedure has been followed for the data describing magnitude v log (displacement), however, in this case it is apparent that the linear logarithmic relationship assumed by Bonilla et al is a poorer fit. A curved relationship has been used here instead, with mean and range as shown in Figure 4.3b. In this case the standard deviation is smaller than for the magnitude v log (length) relationship and the range shown represents ± 1.5 standard deviations.

It is relevant and important to note at this stage a number of limitations in paleoseismic estimates based on rupture and displacement information. If the 1931 Napier event (Magnitude M = 7.8) had occurred prior to historical recording, the present paleoseismic techniques would have at least underestimated, if not failed to recognise this event (Hull, 1990). This earthquake resulted in 15 km of surface rupture however only 3 km can now be confidently recognised after 50 years of trace degradation and cultivation.

Similarly the net slip of 4.9 m could only be obtained from historical records because no offset topography detectable now sufficiently defines this movement. If the 15km of reported rupture is used with Figure 4.3a to estimate the earthquake magnitude, the value obtained is 6.9 ± 0.4 which underestimates the event. In this earthquake additional energy appears to have been released in conjunction with growth of a 90 km^2 fold dome. However using the 4.9 m displacement with Figure 4.3b produces a magnitude 7.7 ± 0.4, which is close to the assigned value.

In an example closer to Christchurch, Cowan (1990) has noted the variation in dextral fault displacement caused by the 1888 Amuri earthquake. The measured displacements by Mackay (1890) indicated that dextral offset varied by more than one metre between locations only 4 km apart (i.e 1.5 m and 2.6 m). Cowan demonstrates that the influence of local factors such as fault geometry and kinematics can explain the variations in this example. Adopting the correlation outlined in Figure 4.3 above, such local factors such as this result in Magnitude estimate variations for each location of M = 6.9 ± 0.3 and M = 7.2 ± 0.4.

The absence of detailed geological mapping of active faults is another problem faced in attempting to estimate maximum magnitudes from existing estimates of active trace length. In general the detailed studies which have been carried out, for example those associated with the various hydro electric schemes, have tended to increase the earlier estimates of trace length based on regional scale map work, and in many cases demonstrated the existence of new traces with more subtle expression which had been overlooked in the earlier regional mapping.

It must also be recognised that the identified fault length is only the surface trace of a large rupture plane or surface. A deeper epicentre and rupture surface may cause a shorter surface trace than a shallow earthquake of the same magnitude.

Despite reservations held by the authors about the ability of existing paleoseismic techniques to always accurately reflect the magnitude and frequency of previous large earthquakes, we believe it is evident from the foregoing discussion that in general the limitations in the available paleoseismic information either underestimate or miss earthquake events, rather than the opposite. The extremely short historical seismicity record, considered in relation to the typical recurrence interval for active faults, makes the inclusion of paleoseismic estimates vital in a study such as this.

The main use of these estimates is to provide a realistic upper limit to the probability functions derived for the various seismicity zones. As outlined in more detail in section 4.5, in the majority of cases the activity in these zones has been predicted from the recorded seismicity which is naturally dominated by smaller events. The paleoseismic information can be used to help estimate the occurrence frequency of larger events in conjunction with this recorded seismicity, and to provide realistic maximum magnitude limits for earthquakes of very low occurrence probability.

The most important faults in the study region are discussed below in order of their proximity to Christchurch. Existing paleoseismic information is reviewed where available, and estimates of maximum magnitude are presented based either on this, or on the rupture/displacement relationships in Figure 4.3. No attempt has been made to discuss in detail the structural style or complexities of a given fault unless this has particular influence on the potential earthquake magnitude or recurrence interval.

In discussion of a number of these faults, reference is made to "maximum magnitude estimates" quoted by other writers. The values finally adopted here may differ from those reported previously. In some cases, the methods used by others to derive their estimates are unknown. In other cases, previous writers have used the types of correlations between magnitude and fault length discussed earlier in this section. Generally the correlations used predate those produced by Bonilla et al (1984), and are based on fewer data points or less comprehensive data analysis. Furthermore, they may be based on mean trends for all data over a wide range of earthquake magnitudes. The correlations used in this study are based on the more recent data set of Bonilla et al, and are weighted for the higher magnitudes appropriate when discussing maximum earthquakes for total fault rupture. For this reason, we believe that the maximum magnitudes derived here are the best that can be made using all the currently available information.

Most of the key numerical information is summarised and presented later in Table 4.1. This table also includes information on the seismicity zones adjacent to Christchurch where limited exposure has largely prevented study to date. The reader may wish to refer to this table during the section below.

ACTIVE FAULTS

Pegasus Bay Fault Zone

This fault has been inferred offshore (e.g Carter & Carter, 1982) and Herzer & Bradshaw (1985) suggest the fault comes onshore for a few kilometres near Woodend. This inference is drawn from oil industry reflection profiles and no surface trace has been recognised.

The fault is marked by recent seismicity for approximately 45 - 50 km. No paleoseismic information is available to estimate recurrence interval or the date of the last major event.

Maximum magnitude (for L = 45 - 50 km): M = 7.3 (range 6.9 - 7.6)

Porters Pass Tectonic Zone

This general term has been used to refer to the zone of active seismicity and complex folding and fault deformation bordering the Canterbury Plains (for example Coyle, 1988). It is considered to represent the juvenile stages of a developing strike slip fault system with shear along a number of interrelated structures (Yousif, 1987).

Cowan & Pettinga (1990) warn that the paleoseismic reconstruction methods normally used for a fully developed and closely defined fault may not be applicable to a juvenile developing system such as this, given the discontinuous nature and variable geometry of the individual faults in the zone. While the return periods for short individual strands may be long, collectively the frequency for the whole zone will be higher. Earthquake recurrence intervals and magnitudes, estimated for the zone as a whole adopting current paleoseismic techniques, may therefore be underestimated.

Paleoseismic study is currently under way on the zone. Coyle (1988) suggests at least three significant earthquake events accompanied by ground rupture have occurred in the last 2000 years, however, suitable study sites have not as yet been addressed systematically (Cowan & Pettinga, 1990).

The zone has been considered in Table 4.1 as a total zone, and also as two separate sections. The division into a southern section (Porters Pass Fault extending southwest from approximately Oxford to Lake Coleridge) and a northern section (Ashley Fault) reflects the most obvious surface traces. Local seismotectonic studies have been carried out in the Coleridge area. Studies in this area have not been definitive but a magnitude M ≤ 7.5 and a recurrence interval of 600 - 1500 years have been suggested. The last event is estimated to have been 500 - 4000 years ago. This compares with an inferred maximum magnitude based on trace length of M = 6.8 - 7.6.

Initial reconnaissance in the Loburn area (Cowan, pers. comm., July 1990) suggests that the Ashley Fault has a relatively low activity rate by comparison with the Hope Fault which Cowan has recently studied in detail (the recurrence interval of the Hope Fault is 90 - 170 yrs). There is no recent trace of the Ashley Fault obvious on a late Pleistocene surface which has not yet been dated. The Holocene surface is 7 - 10 m below the Late Quaternary surface and around 2 m above the active flood plain. Magnitude - trace length correlations for this section suggest a maximum event magnitude M = 6.7 - 7.3. However the Porters Pass Tectonic Zone is such a complex network of short fault strands and folds that it is difficult to consider one isolated element.

An un-named, range-bounding fault inferred from stratigraphy along the face of Mt Hutt has a splay fault off the west side which displaces late Otiran deposits in the Rakaia Gorge (Yousif, 1987). This broadly corresponds to the epicentres of recorded seismicity in this area and may represent the south-western extension of the Porters Pass Tectonic Zone.

Published information concerning the zone as a whole is restricted to a slip rate estimate by Berryman & Beanland (1988) of 4 mm/yr and an estimate of a maximum event on the zone M≤ 7.5 (Berryman, pers. comm., 1989). It is not clear what information this is based on. Seismicity in the zone is remarkably active. Reyners (1989) noted that the Porters Pass Tectonic Zone is perhaps the only major fault zone in New Zealand which can be delineated from the seismicity. This may imply the fault zone is creeping, however to date there is no further evidence of this.

Total Zone:
Maximum magnitude (for L = 70 - 90 km): M = 7.4 (range 7.1 - 7.8)

Ashley (Northern) Section:
Maximum magnitude (for L = 20 - 30 km): M = 7.0 (range 6.7 - 7.4)

Coleridge (Southern) Section:
Maximum magnitude (for L = 30 - 60 km): M = 7.2 (range 6.8 - 7.6)

FAULT ZONE				AVAILABLE INFORMATION							INFERRED MAXIMUM MAGNITUDES			USED THIS STUDY		FOR COMPARISON
Seismicity Zone	Zone or Fault	Dist from CH-CH (km)	Max.Trace Length TI (km)	Reference	Last Major Earthquake						Trace Length Mtl	Displace. in last EQ Md	Mean Disp. Md	Maximum Credible Earthquake	Range	RETURN PERIOD (yr) ★
					Date	M	Surface Disp. (m)	Recurrence Int (yrs)	Disp. Rat (mm/yr)	Mean Disp (m)						
CPS	Canterbury Plains Seismicity Zone	0-50	< 80		1869	5-6	-	-	-	-	6.9-7.7	-	-	7.4	7.1-7.7	13,000
BPS	Banks Peninsula Seismicity Zone	0-75	40-50		1965	6.2					6.9-7.5			7.2	6.9-7.5	1,200
PGS	Pegasus Bay Fault	25	45-50		1895	4.5-6					6.9-7.6			7.3	6.9-7.6	1,200
PPT	Porters Pass TZ	50	70-90	-Berryman & Beanland 1988		<=7.5		1000?	4.0	(4)	7.1-7.8		7.0-7.9	7.4	7.0-7.9	2,000
	-Ashley Section	25	20-30	-Cowan, pers. comm. 1990	<500-2000B				[4]		6.7-7.4			7.0	6.7-7.4	270
	-Southern Section	50-75	30-60	-Campbell, pers. comm. & Coyle, 1988	500-4000BP	<=7.5		600-1500	>4	(8-16)	6.8-7.6			7.2	6.8-7.6	560
CBne	Kaiwara	75	45		1901	7.0					6.9-7.6			7.2	6.9-7.6	218
HFs	Hope -SW section	100	25-35	-B&B, 1988 -Cowan, 1989	1888	7.0-7.3	2.0	[90-170] 90-170	10.0 14±3	(1.2-2.2) 1.3-2.4	6.7-7.5	6.6-7.4	6.4-7.5 6.4-7.5	7.1	6.4-7.5	130
Gn	Hope -NE section	125	30-90	-Van Dissen, 1989					30mm/yr		6.8-7.7			7.2	6.8-7.7	For all Gn Seismicity Zones M=7.0 128 yrs M=7.3 280 yrs M=7.5 480 yrs
	Clarence/Elliot	150	< 180	-Kieckhefer 1979				(1100-1400	3.6-4.4	5.0	7.4-8.0		7.2-8.1	7.6	7.2-8.1	
	-SW section	125	30-90						[4]		6.8-7.7			7.3	6.8-7.7	
	-NE section	175	30-90						[4]		6.8-7.7			7.3	6.8-7.7	
	Awatere -SW Section	175 150	< 180 30-90	-B&B, 1988					4.0 [4]		< 7.4-8.0 6.8-7.7			7.3	6.8-7.7	
	-NE Section	200	30-90	-Lensen, 1975	?	?	6.0	(1500)	[4]		6.8-7.7	7.3-8.3		7.5	6.8-8.3	
	Wairau	175	140	-Eiby, 1980 -Lensen, 1968 -Wellman, 1985	1848?	7.1?		<900 500-900 [500-900]	3.8 6.0	4-5 (3.0-5.4)	7.3-7.9		7.0-8.1 6.8-8.2	7.6	6.8-8.3	
CBsw	Fox's Peak	150	20-40	-B&B, 1988					1.0		6.7-7.5			7.1	6.7-7.5	
	Ostler	225	20-50	-Read, 1984	> 3000BP			2800	1.3	3.6	6.7-7.6		6.9-7.7	7.2	6.7-7.7	
H	Alpine Fault	125-400 (200)	400								7.6-8.3			8.0	7.6-8.3	465
	-Central Section	125-200 (150)	270-350	-Adams, 1980	550BP	7.5-8.1	9	550+100	14-20		7.5-8.2	7.8-8.8		8.1	7.5-8.8	584
	-South Section	375	130	-Hull & Berryman, 1986 -Cooper & Norris	> 100BP 260BP	7.4-8	8	425+70	20	7.1-8.9	7.3-7.9	7.7-8.7	7.2-8.7	7.9	7.2-8.7	385

Key	
(Info.)	Implied by other data for same fault segment
[Info.]	Adopted (in absence of other specific data) from a comparable segment

(★) NOTE : Return Period is for maximum credible earthquake, and is predicted by the seismicity model derived in this study. (see section 4.4)

TABLE 4.1 Active Faults and Seismicity Zones - Inferred Maximum Magnitudes

Kaiwara Fault

This fault has not yet been studied in detail. The 1901 Cheviot earthquake had an epicentre close to the southern end, however epicentres were not accurately located at that time and no surface rupture has been correlated with this event. McPherson (1988) suggests that the earlier projection of the fault as a continuous major structure across North Canterbury by Gregg (1964) may be open to doubt, however the maps of both authors suggest a fault length of at least 45 km. This in turn corresponds to a maximum magnitude event M = 6.9 - 7.5 and compares reasonably well with the 1901 event estimated to have been around magnitude M = 7.0.

Maximum magnitude (for L = 40 - 50km): M = 7.2 (range 6.9 - 7.6)

Hope Fault

The Hope Fault extends ENE from the Alpine Fault to Hapuku, north of Kaikoura. The fault is considered here for the purposes of seismicity analysis as two sections (one each side of Hanmer) the south-west section of which incorporates a more east - west aligned component where tension is most pronounced (Cowan, 1989). Cowan (pers. comm., October 1990) recognises three structural segments further subdividing the south-western section, as used in our seismicity analysis, at the Boyle River; while others (e.g. Bull, pers. comm. per Cowan, 1990) argue for a fourth structural segment from the Kahutara River to the Hapuku.

The south-western section of the fault is one of the few in the area which has been studied in detail and for which reliable information is available. The 1888 Amuri earthquake occurred in this area on the fault and had an inferred magnitude M = 7 - 7.3. The surface rupture length identified for this event was about 30 ± 5 km. Cowan (1989) and Cowan & McGlone (in prep.) recognise five earlier events of similar magnitude and consider the return period for these events to be between 90 and 170 years. The fault slip rates appear to vary locally with fault strike but an average rate of 14 mm ± 3 mm/yr for the last 17,000 years is considered realistic (Cowan, 1990).

The eastern section of the fault between Hanmer and Hapuku has not yet been studied in detail but Pettinga (pers. comm.) considers that based on structural considerations it is likely that this section has a more irregular return period with generally larger magnitude events.

SW section:
Maximum magnitude (for L = 25 - 35 km): M = 7.1 (range 6.4 - 7.5)

NE section:
Maximum magnitude (for L = 30 - 90 km): M = 7.2 (range 6.8 - 7.7)

Clarence and Elliot Faults

Only limited information is available on the activity of these faults. They are considered Class I active faults by Kieckhefer (1979) who deduced 65 - 80 m of horizontal movement on the Clarence Fault in the last 18,000 years (an average of 3.6 - 4.4 mm/yr). He considered up to 5 m dextral movement per earthquake event was likely along the Clarence fault which, given the average rate of movement and assuming a uniform slip rate, implies a return period of 1100 - 1400 years. The correlations between surface displacement and magnitude in Figure 4.3 suggest a maximum earthquake magnitude of 7.2 - 8.1 would be associated with a heave of 5 m. No recorded historical earthquakes of this magnitude have been attributed to the Clarence Fault.

Pettinga (pers. comm., October 1990) notes that the Clarence Fault may be a reverse/oblique structure. It does not appear to be a through-going direct transfer structure to the Alpine Fault and in this sense is less important than the Hope or Awatere Faults.

Considering the 180 km total length of the active trace, if it is assumed that this entire length may rupture in one event, a similar maximum magnitude is obtained of M = 7.4 - 8.0. However, given that the available information on the more active Hope Fault suggests rupture in separate sections, it is unrealistic to consider rupture along the entire Clarence Fault as typical. For this reason the rupture length correlations suggested below divide the fault arbitrarily into two equal sections comparable to the Hope Fault rupture lengths and include a considerable range of 30 - 90 km in trace length.

The Elliot Fault is comparatively less active with an average Post Glacial movement of less than 1 mm/yr. No information on return period is available.

SW and NE sections:
Maximum magnitude (for L = 30 - 90km): M = 7.3 (range 6.8 - 7.7)

Awatere Fault

Less information is available for the Awatere Fault than for the Clarence Fault however their movement rates and styles have traditionally been considered to be similar. An average slip rate of 4 mm/yr is suggested by Berryman & Beanland (1988). Lensen (1978a) noted a recent heave of 6 m at the NE end of the fault (Saxton River). He considered this to have been the result of movement during the 1848 Marlborough earthquake (estimated magnitude of 7.1). Eiby (1980) challenges this, suggesting instead that the 1848 event occurred on the Wairau Fault and the 6 m of heave reported by Lensen was associated with some other earlier earthquake.

Adopting the 6 m of recent offset reported by Lensen (1978) and the 4 mm/yr slip rate suggested by Berryman and Beanland (1984) the return period for movement of a similar order is 1500 years.

Awatere Fault, entire fault:
Maximum magnitude (for L ≤ 180 km): M = 7.7 (range 7.4 - 8.0)

Awatere Fault, SW section:
Maximum magnitude (for L = 30 - 90km): M = 7.3 (range 6.8 - 7.7)

Awatere Fault, NE section:
Maximum magnitude (for displacement = 6 m): M = 7.5 (range 6.8 - 8.3)

Wairau Fault

Information available for the Wairau Fault suggests that activity resembles that deduced for the other northern Marlborough faults. Lensen (1968) originally suggested the average heave on the fault was 3.3 - 6.6 m with a return period of 500 - 900 years. Later work by the same author (Lensen, 1976) suggests an average Post Glacial slip rate of 3.8 mm/yr and favours the 900 year return period. Assuming this average slip rate of 3.8 mm/yr reduces the heave for return periods of 500 - 900 years to 1.9 - 3.4 m. However in a private communication (Lensen, 1978b) he estimated the next movement at the Branch River Terraces to be 4 - 5 m which appears to be inconsistent with the 3.8 mm/yr slip rate. Wellman (1985) suggests that the average slip rate is higher at 6 mm/yr, which is also supported by Van Dissen & Yeats (1989), and would result in 3 - 5.4 m of heave per event.

Maximum magnitude (for L ≤ 140 km): M = 7.6 (range 7.3 - 7.9)

(for heave = 4 - 5 m): M = 7.6 (range 7.0 - 8.1)

(for heave = 3.0 - 5.4 m): M = 7.5 (range 6.8 - 8.2)

Fox's Peak Fault

This fault is a predominantly reverse fault extending SSW for at least 40 km near Fairlie. Little work has been done on the fault, and rates and timing of displacement are poorly documented (Beanland ,1987). Traces displace terrace surfaces only a metre or so above current stream floodplains in some locations suggesting recent fault rupture. Berryman estimates a long term slip rate of 1 mm/yr from unpublished data (Berryman & Beanland, 1988).

The nearby Lake Heron Fault may represent the northern continuation of this structure, however the active trace length appears to be considerably shorter. No detailed study has been published to date on this feature.

Maximum magnitude (for L = 20 - 40 km); Ms = 7.1 (6.7 - 7.5)

Alpine Fault

The Alpine Fault is the largest single fault in New Zealand and lies just outside Canterbury bounding the western side of the Southern Alps. It has traditionally been viewed as a transpressive dextral strike slip fault with 450 - 500 km of horizontal displacement, most of this movement having occurred during the last 10 - 15 million years.

The Alpine Fault can be divided into three sections in the South Island with the most clearly defined and linear portion extending for 400 km along the west central South Island (refer Fig. 4.1). This section, extending from Jackson Bay (44 S) to the Taramakau River (42.75 S)), appears to be relatively aseismic and has been referred to as the "Seismic Gap" by many authors following the terminology of Adams (1980).

This has also been referred to as the "Central Section" by Evison (1971) which is adopted here as a preferable term. The original definition extended this section slightly north to the Ahaura River (42.75 S). However the Taramakau River is a more suitable northern limit of the Central Section since it appears to mark the first large transfer of horizontal movement to a significant splay fault (the Hope Fault), and is adopted here.

The area south of the Central Section, where the fault extends south-westward from Jackson Bay and offshore through Milford, is referred to here as the Southern Section. Historical seismicity is more obvious in this area, with larger earthquakes recorded, particularly offshore.

The shorter Northern Section is considered here to extend from the Taramakau River to Lake Rotoiti, where the Alpine Fault appears to continue as the Wairau Fault. This area is a progressive transfer zone with the majority of movement occurring on the Hope Fault and smaller average movements on the Clarence, Awatere and Wairau faults.

Recently the traditional view of the Alpine Fault as a steeply dipping single shear zone has been challenged. Mapping on the Central Section in central Westland between Fox Glacier and the Whataroa River has shown that the fault is not straight but consists of alternately striking, shallow dipping thrust fault segments linked by vertical strike slip faults on the scale of a few kilometres (Cooper & Norris, 1989). The east dipping thrusts tend to be aligned with a more northerly strike than the regional fault trend while the dextral strike slip faults are more easterly. Thus in plan view the result resembles a sawtooth pattern.

A new conceptual model for the Alpine Fault has recently been proposed by Wise et al (in prep.) which considers the complexities of transferring plate movement between two opposing subduction zones. The authors view the fault dip as flattening dramatically eastwards and extending at relatively shallow depths (10 - 15 km) under the Alps and Canterbury. Canterbury and

Marlborough are viewed as a detached surface slab riding up over a ramp structure in the underlying Australian Plate.

Scientific debate is just commencing on this new model and it is too early to draw conclusions for incorporation in a seismic analysis such as this. Both the results of the recent mapping of the fault and the new tectonic interpretations indicate that the interpretive framework in which we attempt to understand and predict the regional seismicity is rapidly changing. There may be implications for regularity of return period and focal locations inherent in these new concepts, but at this stage we must rely on existing paleoseismic investigations of earthquakes which have occurred in the last few thousand years.

The first detailed paleoseismic work on the Alpine Fault was by Adams (1980) who looked at the Central Section. The location of the sites studied is included in Figure 4.1. Adams concluded that despite the absence of recorded earthquakes dextral movement on the fault is episodic and therefore seismic. This conclusion was based on the existence of hyalomylonite, fault scarps and the abrupt change in height of aggradation surfaces at the fault. No evidence of creep was observed.

Adams described matched flights of aggradation terraces in a number of river valleys crossing the fault. He assumed these to have developed in response to regional aggradation episodes following widespread earthquake triggered landslides in the catchments. Radiocarbon dates from wood in the terraces at widely separated localities appear to cluster about dates which he infers to be those of earthquakes generated by the Alpine Fault. In general both assumptions may be reasonably valid, and neither has been significantly contested since. Although there is no way to be sure the earthquake events responsible for the aggradation were definitely on the Alpine Fault rather than some moderate distance away, in terms of general seismicity in the region this is of little importance.

The dates themselves have an error associated with the dating method and presumably further uncertainties relating to the age of the tree fragments prior to the aggradational event. However Adams suggests dates for past events (in years before present) at around 550, 1000, 1550 and 2200 years BP. He suggests probable magnitudes of 7.5 - 8.1 resulting from an average heave per event of 9 m. No evidence was encountered for an event more recent than 550 BP. Unfortunately no more paleoseismic investigation has been carried out in this Central Section since this work of Adams was published in 1980. Given the critical nature of this fault and the proximity of the Central Section to Christchurch such work is an obvious priority.

The Southern Section has been investigated in greater detail. Hull & Berryman (1986) investigated the fault in the vicinity of Lake McKerrow, Fiordland. They obtained reliable indications of a heave of 8 m for at least one seismic event near Hokuri Creek and estimate an average slip rate of around 20 mm/yr. Adopting the 8 m heave they determined a return period of 350 - 500 years, and used the empirical formula of Slemmons (1982) to estimate a magnitude M = 7.4 - 8.0. No accurate information was gained on the date of the last event but the historical record indicates this was more than 100 years ago.

Berryman et al (1986) studied at a 60 km segment of the Alpine Fault in the Southern Section from Jackson Bay south to Lake McKerrow. They note consistent vertical offsets of 1.5 m which they attribute to a single earthquake event. Based on a plunge angle on some slickensides exposed at one locality on the fault, trigonometric calculation suggests a heave of 8.5 m. Although this reasonably matches their earlier estimates, a heave obtained in this way is potentially subject to significant error. Slip rates were calculated in the Cascade valley from the offset of landforms. Unfortunately the exact ages of many of these features are in dispute but Berryman et al deduce from this limited evidence a relatively high average slip rate of 28 - 38 mm/yr. This slip rate is then used in conjunction with the derived heave of 8.5 m to determine a return period of 220 - 300 years. It should be noted that adoption of a more conservative average slip rate of around 20 mm/yr would result in an estimated return period of 425 years. The age of the last fault movement is not well constrained but is concluded to be less than 800 yrs B.P.

The most recent paleoseismic work on the Alpine Fault has again been on the Southern Section immediately adjacent to Milford Sound. Cooper & Norris (1989) infer two scarp degradation events associated with earthquakes on the fault from cataclasite derived sand and gravel in a local swamp. Dating suggests earthquake events at > 180 - 280 yr B.P. and > 1920 - 2040 yr B.P. Intermediate age events may also have occurred and been unrecorded at this particular site. Large beech trees growing on the fault scarp have had their tops shaken out by earthquake vibration (this phenomena was also observed in the 1990 Lake Tennyson earthquake). Large trees unaffected by shaking have ages less than 265 ± 32 years and the authors deduce from this combined evidence that the last movement occurred approximately 260 years ago.

No other paleoseismic work has been carried out to date on the Alpine Fault. In summary the evidence suggests large earthquakes definitely occur on the fault with return period estimates ranging from 220 years to 550 years. Many researchers agree on a likely magnitude range M = 7.4 - 8.1, but no historical earthquakes of this magnitude have been recorded. Some conflict exists in the dates estimated for specific paleoseismic events, particularly the most recent. This may be the result of errors in either dating or inference. However another possibility may be that the fault did not rupture along the full length in the 260 year event noted in the Southern Section. Adams' dates of around 550 years for the Central Section come from the middle of the Central Section in the area around Franz Josef.

It is possible a significant earthquake could occur centred in the Southern Section of the Alpine Fault which would not necessarily affect the Central Section. Adopting the correlation in Figure 4.3, a maximum heave as high as

7 m centred on Milford might correspond to a total rupture length of 360 km with heave reduced to zero by Franz Josef. If the epicentre was 100 km further south than Milford a much greater rupture length and associated heave could be accommodated without leaving a trace in the Franz Josef area. If this is the case, and significant lengths of the Alpine Fault are effectively out of phase, the realistic assessment of return period for the entire fault zone becomes much more difficult. In effect some intermediary areas of the fault which are potentially affected by two adjacent sections would have an enhanced return period. However the overall maximum magnitude for the entire fault would be reduced.

It is interesting to note that in a recent follow-up to his earlier work, Adams (1990) has speculated that the observed displacements on the surface trace of the Alpine Fault may be a consequence of great earthquakes (up to M = 8.7!) on the "anti-Alpine Fault". This feature is postulated to be a zone up to 50 km deep <u>beneath</u> the surficial Alpine Fault, with oblique (dextral) thrusting on a down-going plane as opposed to the up-going plane of the Alpine Fault. Adams' hypothesis is supported by earthquake hypocentres located by Reyners (1989). A 'great' earthquake on the anti-Alpine Fault would severely strain the Alpine Fault above and cause it to creep, or fail in a slightly smaller earthquake (M = 7.5?). To date there has been no published discussion of this new attempt to explain the anomalous data around the Alpine Fault region, and at present it remains speculation. However at Christchurch the effects of a great earthquake on the anti-Alpine Fault would be as severe as those if an earthquake of the same magnitude had occurred on the Alpine Fault.

Given the current state of limited knowledge the Alpine Fault must be considered a likely source of a large earthquake, possibly with a magnitude as high as, or exceeding, M = 8. If the work of Adams is accepted (despite some informal scepticism there has been no written rebuttal of his conclusions in the 10 years since he published his work) the Central Section of the fault is the area most likely to experience this. If the clustering of Adams' dates around the 550 year return period is considered as a series of normal distributions about the most probable time for a given event, then the probability of an earthquake occurring in the near future, for example the next 100 years, appears to be very high.

Central Section:		
Maximum magnitude:	(for L = 270 - 350 km):	M = 7.8 (range 7.5 - 8.2)
	(for heave - 9 m):	M = 8.3 (range 7.8 - 8.8)
Adopted here:		M = 8.1 (range 7.5 - 8.8)
South Section:		
Maximum magnitude:	(for L ≤ 130):	M = 7.6 (range 7.3 - 7.9)
	(for heave = 8 m):	M = 8.2 (range 7.6 - 8.7)
	(displacement rate = 20 mm/yr):	M = 7.9 (range 7.2 - 8.7)
Adopted here:		M = 7.9 (range 7.2 - 8.7)

Other Active Faults

Table 4.1 does not include information on a number of smaller active faults in inland Canterbury (e.g. Harper Fault, Bruce Fault, Mt White Fault etc). Other active faults not discussed outside the province include the White Creek, Lyell & Inangahua Faults (Berryman, 1980); 88 Fault & Bishopdale Faults (Johnston, 1979); Irishmans Creek Fault (Fox, 1987); Ostler Fault (Read,1984) and the faults in Otago (e.g. Madin, 1988). In most cases little is known of the smaller Canterbury traces and the faults outside the province are neither close enough to Christchurch, nor large enough for any revised characteristics resulting from detailed analysis, to significantly affect the regional seismicity model and seismic hazard for Christchurch.

SEISMICITY ZONES ADJACENT TO CHRISTCHURCH

A large number of epicentres for the shallow (crustal) seismicity recorded in Canterbury since 1942 are located in areas away from known active faults under thick Quaternary cover or ocean. Figure 4.2, presented earlier, showing the known active fault traces, also shows the epicentres of these relatively small events. The close proximity of much of the seismicity to Christchurch means the zones responsible are particularly important. Figure 4.4 shows the recorded shallow seismicity 1964 - 1988 for the entire Canterbury region. Following the larger scale procedure of Smith & Berryman (1983), we have defined seismicity zones on this figure to which we refer below and in later sections. This represents a significant outcome of this study. These zones have been constructed considering both the geology and the recorded seismicity. The boundaries in areas away from known faults are somewhat arbitrary at this stage, and may be refined following further research.

The three new zones proposed in this study which are nearest Christchurch, and contain active seismicity not accounted for previously in separate zones or attributed to known, active faults, are discussed here.

Canterbury Plains Seismicity Zone (CPS)

This term is introduced in this study to refer to the wide zone of earthquake epicentres recorded instrumentally between 1942 and 1988 under the Quaternary cover of the Canterbury Plains. These epicentres are shown in conjunction with the known active faults in Figure 4.2 and the 1964 - 1988 seismicity data shown again in relation to the adopted seismicity zones in Figure 4.4.

Currently very little is known regarding the source of this active seismicity. It is clear the epicentres show some clustering with a suggestion of linearity, particularly for the northern most points, along 050 - 060 degrees, a common structural orientation in the older rock materials along the foothills. It is tempting to consider at least the northern most epicentres through the zone to be a south westward continuation of the Pegasus Bay Fault. Given the width of the epicentres the zone could represent a low angle single fault or a more complex series of sub-parallel features. Much more detailed subsurface and seismological investigation is required before making informed comment.

The most damaging historical Christchurch earthquake to date, the 1869 New Brighton earthquake, is assigned an epicentre location by Dibble et al (1980) which lies just within this zone. The fault or faults responsible for the recorded seismicity may continue south-west for a total length approaching 100 km and account for the cluster of epicentres around the Rakaia area. However, in terms of a "rupture length" estimate in the absence of any real data, a reduced maximum figure of 80 km has been arbitrarily adopted.

Maximum magnitude (for L = 80 km): M = 7.4 (range 7.1 - 7.7)

Banks Peninsula Seismicity Zone

This term is introduced in the study to refer to the more diffuse area of epicentres recorded beneath and offshore from Banks Peninsula (refer again Figure 4.2 and Figure 4.4). Division between the seismicity zones to the south east of Christchurch is inferred and has been drawn to fit the scatter of epicentres rather than for any particular geological difference. Figure 4.4 shows the distribution of epicentres within the zone. The largest recorded earthquakes occurred in 1870 and 1921. Both were estimated to be around magnitude M = 4.5 - 6.0 .

Even less is known (or can be reasonably inferred) concerning the source of seismicity in this zone than for the adjacent Canterbury Plains Zone. No active fault traces are known on Banks Peninsula; in fact the Upper Tertiary volcanic and Quaternary materials in this area are remarkably free of faulting and folding (see for example Sewell et al, 1988). This contrasts markedly with the volcanics of similar age at Otago Peninsula mapped by Benson (1968). Recent suggestions that low angle sub-horizontal faulting may be active in the Canterbury area at crustal depth (Wise et al, in prep.) may account for the absence of exposed vertical faulting despite observed seismicity.

Based on the recorded epicentres and in the absence of other information, a "rupture length" of 40 - 50 km has been arbitrarily adopted for maximum magnitude estimates, to avoid unjustified influence of this zone in the seismicity model.

Maximum magnitude (for L = 40 - 50 km): M = 7.2 (range 6.9 - 7.5)

Pegasus Bay Seismicity Zone

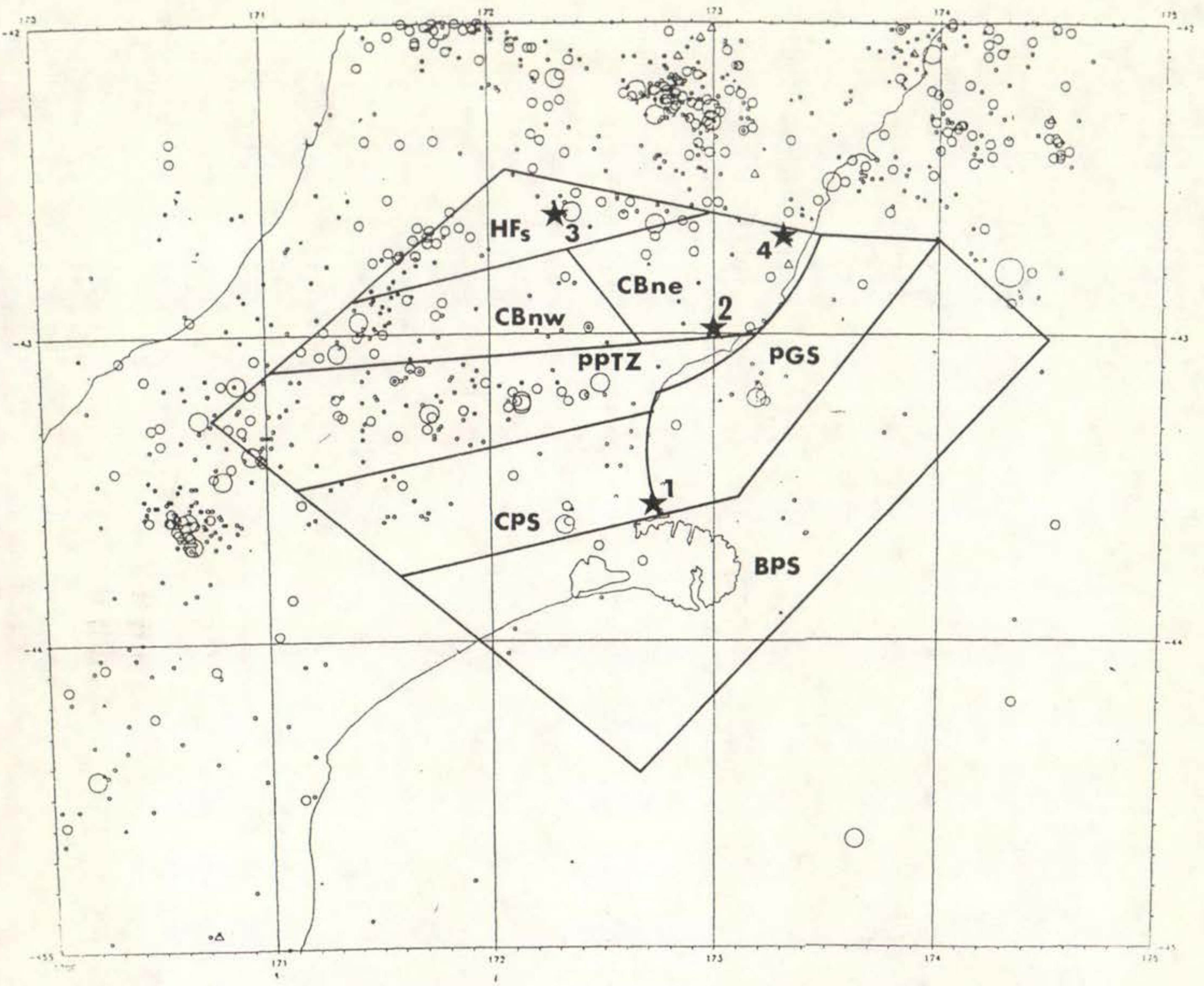
Although at least one active fault, the Pegasus Bay Fault, is known to occur in this zone, and reasonably accounts for the bulk of the observed seismicity, epicentres occur a considerable distance further south in the area adjacent to the 1895 earthquake. This earthquake of M = 4.5 - 6.0 resulted in (inferred) intensity MM VI in Christchurch and is the largest recorded to date in the zone. The Pegasus Bay Fault has been used to define the maximum magnitude in this zone.

Summary Diagram

All the information presented in previous sections on faults or seismicity zone is presented in Figure 4.5 and Table 4.1. This figure shows the maximum magnitude of earthquake able to be generated for each zone or fault, and the epicentral distance to Christchurch. Error bars have been used to indicate the range of magnitude uncertainty and a solid circle shown for the adopted mean value. Distances are average radial distances from the centre of Christchurch to the middle of each feature. It is significant that proximate epicentres may be considerably closer to the outskirts of the city (by up to 10 km) than indicated by this figure.

Those faults with magnitudes close to the upper bound are the most critical faults in terms of energy likely to reach Christchurch. These faults include the Pegasus Bay Fault, the Porters Pass Tectonic Zone (particularly the Ashley section), and the Central Section of the Alpine Fault. In addition the Canterbury Plains & Banks Peninsula Seismicity Zones, particularly the former which is very close with relatively high levels of seismicity, are capable of generating levels of energy at Christchurch comparable to the better defined faults noted above.

Analysis of the effects of seismicity in these zones on the seismic hazard at Christchurch follows in later sections, where frequency and probability are considered in conjunction with this basic magnitude/distance plot.



Epicentre	Magnitude Range
∘	M3 - M4
○	M4 - M5
○	M5 - M6
◯	> M6
△	Undifferentiated: depth >33km

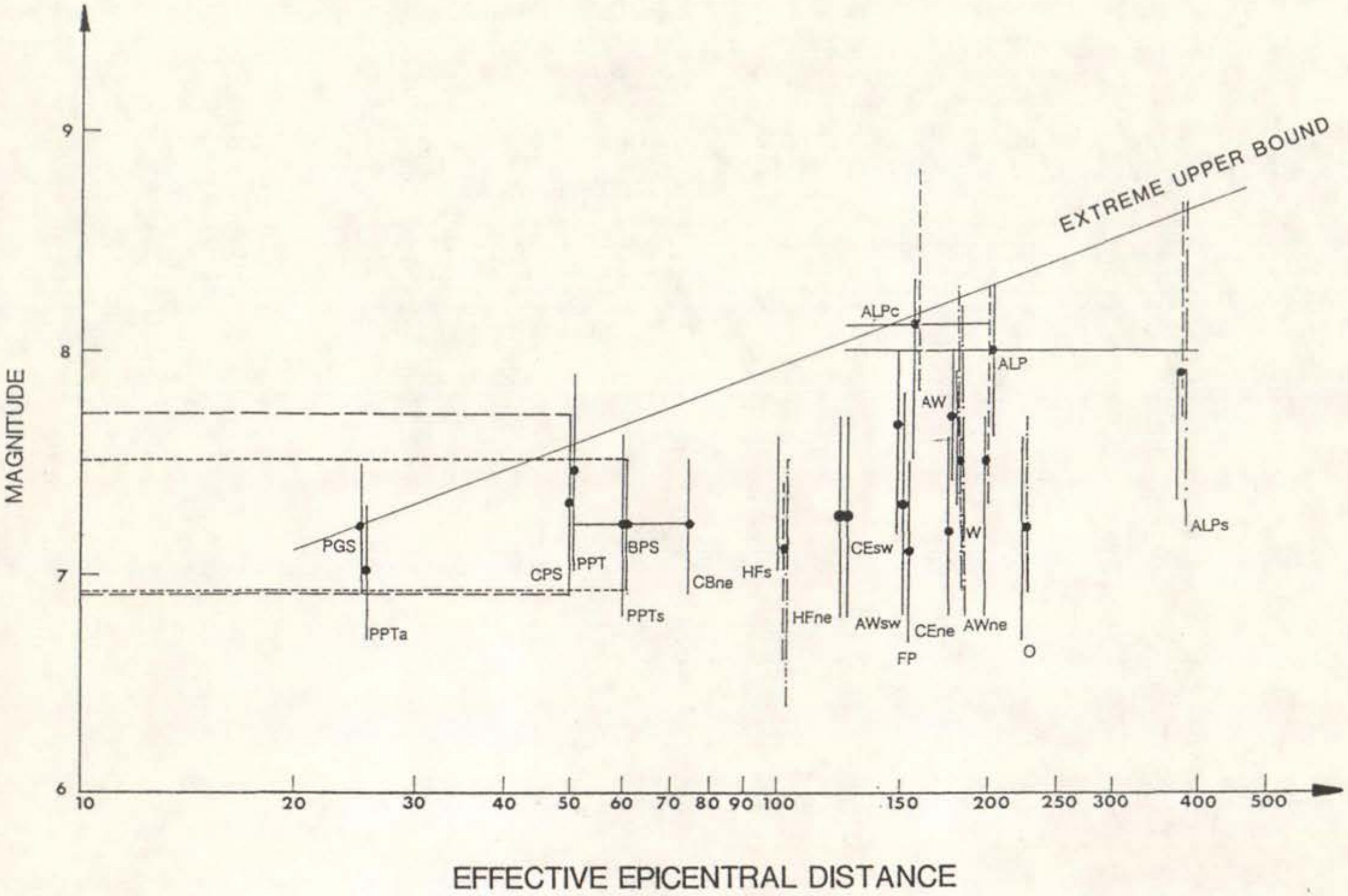
Largest Historical Earthquakes ★

1. 1869 New Brighton, M = 5.7
2. 1922 Motunau, M = 6.7
3. 1888 Amuri, M = 7.0 - 7.3
4. 1901 Cheviot, M = 6.5 - 7.0

Sources: various - see Table 4.2

FIGURE 4.4

Canterbury Region Shallow Seismicity (1964 - 1988)
(Plot Courtesy Geophysics Division, DSIR)



Key	
CPS	Cant. Plains Seismic
BPS	Banks Pen. Seismic
PGS	Pegasus Seismic
PPT	Porters Pass TZ
PPTa	- Ashley section
PPTs	- Southern section
CBne	Kaiwara
HF	Hope
HFs	- SW section
HFne	- NE section
CE	Clarence/Eliot
CEsw	- SW section
CEne	- NE section
AW	Awatere
AWsw	- SW section
AWne	- NE section
W	Wairau
FP	Fox's Peak
O	Ostler
ALP	Alpine Fault
ALPc	- Central section
ALPs	- South section

FIGURE 4.5

Earthquake Magnitude - Effective Epicentral Distance from Christchurch

4.3 HISTORICAL EARTHQUAKES

Table 4.2 lists the historical earthquakes which probably caused an inferred intensity >MM V in Christchurch. The bulk of the information comes from the Seismological Observatory, Geophysics Division (DSIR) and the inferred intensities are those predicted by the attenuation model of Smith (1976). However many of the larger earthquakes have been the subject of historical research by Dibble et al (1980), as part of the LPG facilities seismic risk assessment, and intensities are those actually felt as deduced from historical reports.

In many cases, particularly for the earlier earthquakes, the epicentral locations favoured by Dibble et al are substantially different to the Observatory information. Dr Euan Smith (Seismological Observatory) notes that in so far as Dibble et al have made a special study of the historical earthquake epicentres, their results should be preferred (facsimile correspondence, 21 August 1990). Figure 4.6 shows the epicentral locations in relation to the seismicity zones introduced in the previous section.

Similarly the intensities of Dibble et al are felt intensities based on actual reports of damage, albeit a limited number, and thus reflect any amplification and geographic variation which may have occurred.

In general there has been a marked quiescence in significant earthquakes affecting Christchurch since 1929. Figure 4.7 shows the historical record in histogram form. In the period between 1888 and 1929 a total of fourteen events exceeded MM V in contrast to only five since that time. None of the recent events have exceeded MM VI. Deriving return periods from this short record of relatively crude intensity estimates gives the following:

		Average Return Period (1880 - 1940)	Period Lapsed Since Last Event
≥MM VII	4 in 150 yrs	37.5 yrs	68 yrs
≥MM VI	10 in 150 yrs	15.0 yrs	62 yrs
≥MM V	19 in 150 yrs	7.9 yrs	22 yrs

Given such a short record, the return periods above are very sensitive to fluctuations in activity. For example if a similar exercise had been carried out in 1930 the results would have been as follows:

	Average Return Period (1840 - 1930)	Compared to Return Period Above (1840 - 1990)
≥MM VII	22.5 yrs	37.5 yrs
≥MM VI	9 yrs	15.0 yrs
≥MM V	6.4 yrs	7.9 yrs

However, regardless of the short sample time, it is clear that the 68 year lapse since the last event exceeding MM VII in relation to a return period of 37.5 yrs suggests either that the period 1840 to 1930 was unusually active or that the shorter period since this time has been unusually quiet.

Considered below in order of severity are the four most important historical earthquakes and their known effects. Epicentres for these earthquakes are shown on Figures 4.2 and 4.4, in addition to Figure 4.6.

1869 June 5th, New Brighton Earthquake

Dibble et al (1980) consider this early event in the city's history to have caused the most destructive effects in Christchurch. Their intensity assessment is MM VII - VIII and the data is consistent with a relatively small magnitude $M = 5.75$ earthquake located 10 km from the city centre.

This location is substantially closer to Christchurch than the 60 km distant epicentre located off Akaroa listed in the seismological Observatory files, and based on calculations by Hogben (1891), which are considered flawed by Dibble et al. A much closer location is supported by the extreme attenuation effects i.e. an intensity MM VII -VIII in Christchurch reducing to MM VI in Lyttelton. The earthquake was not felt in Selwyn or Oxford (31 km and 46 km respectively from Christchurch city centre). Some local amplification is also implied by the reported damage.

Few of the houses already built in Christchurch at that time escaped damage of some sort. There was general demolition of chimneys, glass and furniture breakage and considerable damage to masonry (cracking and dislodgement). One side of a brick house near Manchester Street was entirely shaken down. The area north of the Avon and east of Papanui Road suffered the most damage. This earthquake is not widely known amongst Christchurch residents or historians because it predated construction of a number of the major stone buildings (e.g. Cathedral, Museum, Arts Centre etc.) and Christchurch was still essentially contained within the four avenues.

We have referred to the June 5th, 1869, earthquake here as the "New Brighton" earthquake in view of the epicentral location suggested by Dibble et al. There was very little development at New Brighton itself at the time of this earthquake. No liquefaction was reported but unrecognised evidence may have existed. Alternatively the short duration typical of a small magnitude event may not have generated liquefaction.

1922 December 25th, Motunau Earthquake

Dibble et al consider the extent of damage to buildings in this earthquake was second only to the 1869 New Brighton event. The epicentre was approximately

YEAR	DATE	LATITUDE OF EPICENTRE	LONGITUDE OF EPICENTRE	MAGNITUDE	DISTANCE FROM CH-CH (Km)	OBSERVATORY PREDICTED INTENSITY AT CH-CH	FELT INTENSITY AT CH-CH (MM)	COMMON NAME	INFORMATION SOURCE
1460		41.4	174.8	>7.5	293	5.3			Observatory
1848	OCT 15	41.5	173.8	7.1	241	5.9		Malborough	Observatory
1855	JAN 23	41.5	175.2	8.0	305	5.2		SW Wairapa	Observatory
1869	JUN 05	43.5	172.7	5.75	10	6.0	VII-VIII	New Brighton	Dibble & Ansell 1980
1870	AUG 31	43.5	173	4-6.5	50	6.0	VI-VII		Dibble & Ansell 1980
1881	DEC 05	43.2	171.8	6.25	80	6.8	VI-VII		Dibble & Ansell 1980
1888	AUG 31	42.6	172.3	7.0-7.3	104	8.0	VII	Amuri	Cowan, 1989
1888	OCT 23	42.5	172.5	7.0	222	5.7			Observatory
1888	DEC 27	43	172	4.5-6	77	5.7			Observatory
1893	FEB 11	41.5	173	7.2	224	6.2			Observatory
1895	AUG 05	43.5	172.9	5-6	25	7.1	VI		Dibble & Ansell 1980
1901	NOV 15	42.7	173.3	6.5-7	105	8.1	VII	Cheviot	Dibble & Ansell 1980
1921	NOV 05	44	173	4.5-6	62	6.0			Observatory
1922	DEC 25	43	173	6.75	62	6.6	VII	Motunau	Dibble & Ansell 1980
1929	MAR 09	42.8	171.8	6.9	109	7.2	VI-VII	Arthurs Pass	Dibble & Ansell 1980
1929	JUN 16	41.7	172.2	6.7-6.9	203	6.7		Murchison	Observatory
1946	JUN26	43.2	171.7	6.2	86	5.9		Coleridge	Eiby, 1990
1948	MAY 22	42.5	173	6.4	115	5.9			Observatory
1948	MAY 22	42.8	173.1	6.2	80	6.0			Observatory
1960	FEB 21	42.3	173.1	6.4	144	5.5			Observatory
1968	MAY 24	41.8	172	6.7	195	5.3	VI	Inangahua	Dibble & Ansell 1980

TABLE 4.2

Historical Earthquakes Resulting in MM Intensity >5 in Christchurch

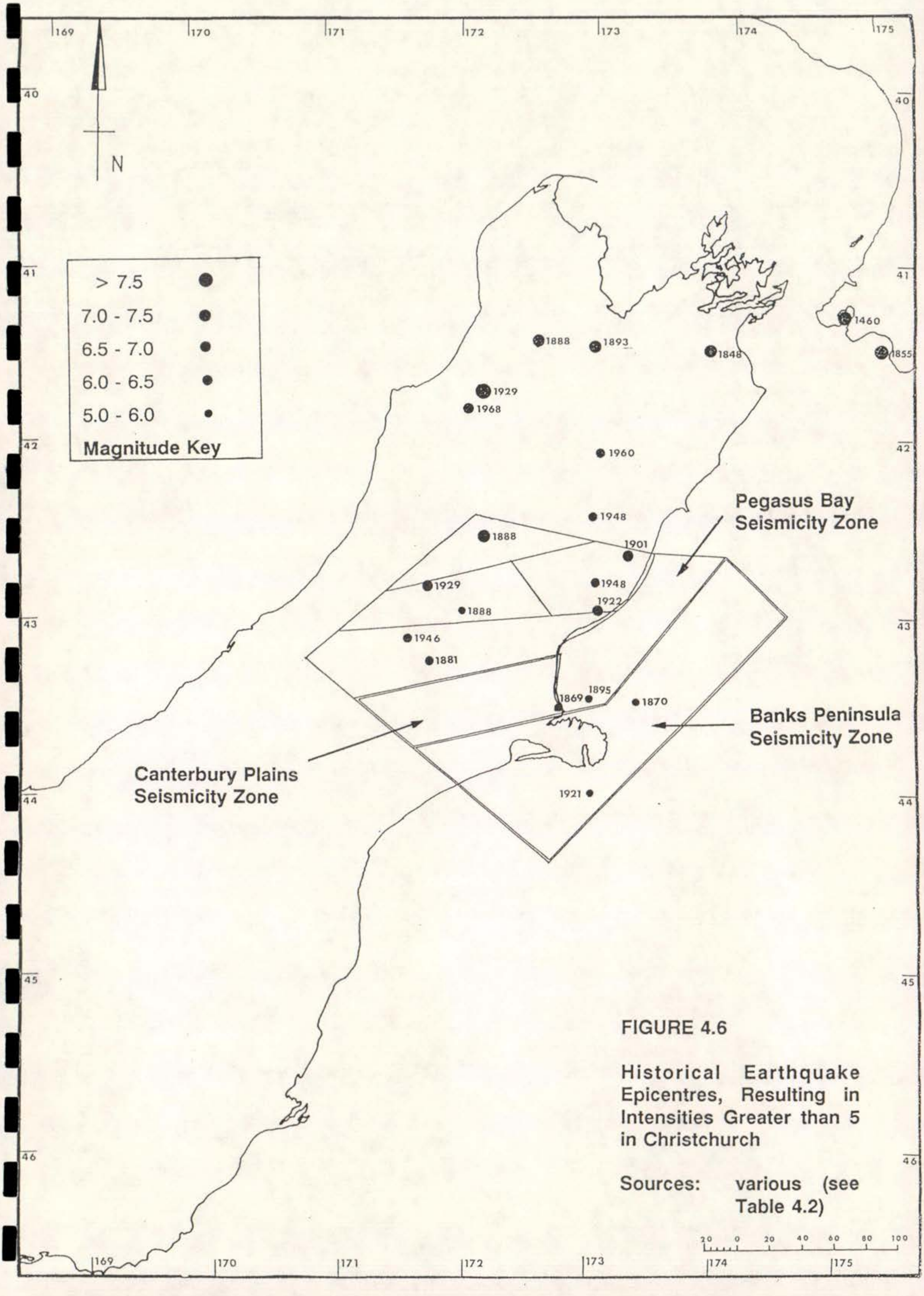
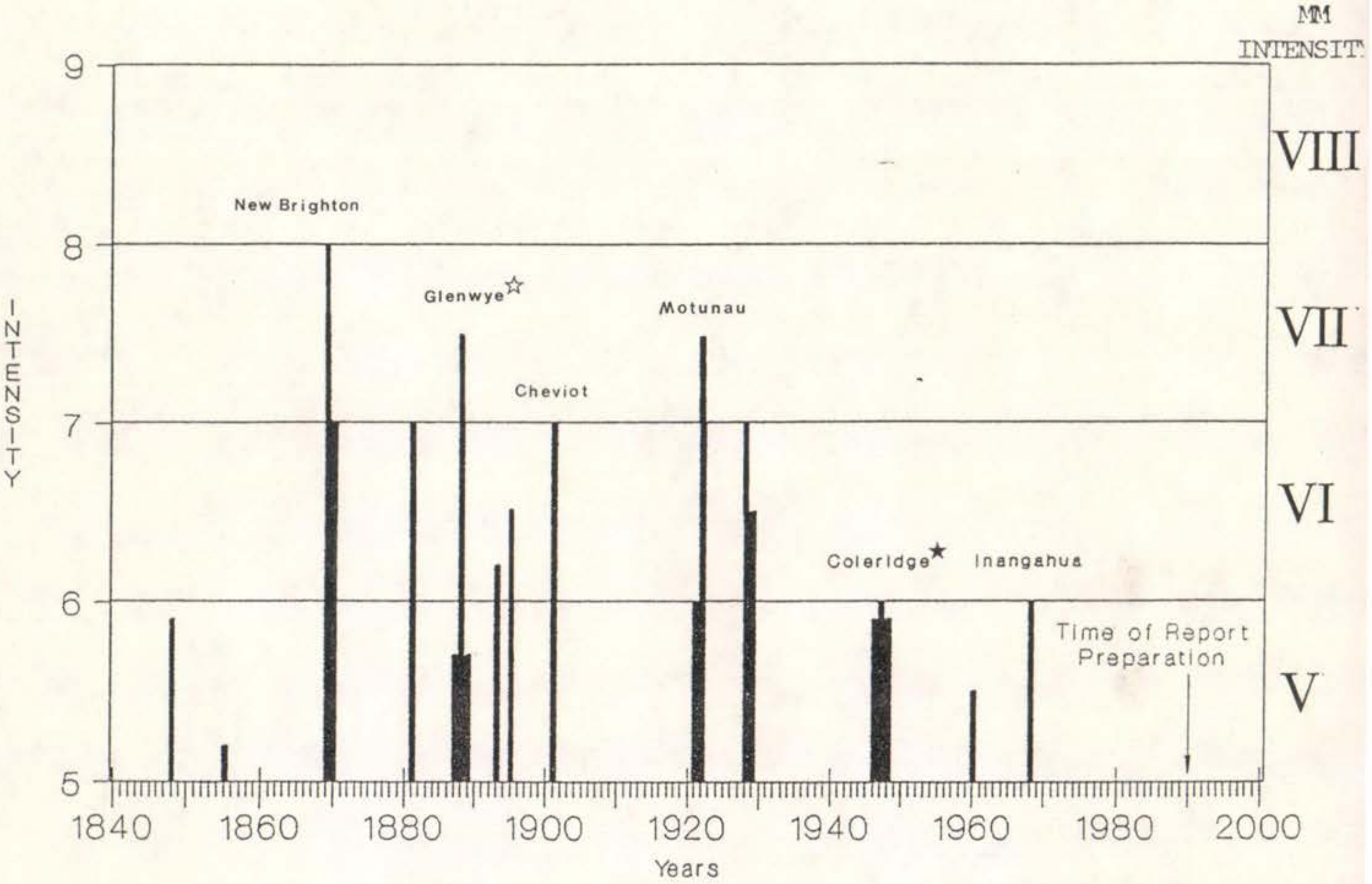


FIGURE 4.6

Historical Earthquake Epicentres, Resulting in Intensities Greater than 5 in Christchurch

Sources: various (see Table 4.2)



☆ Three Earthquakes In 1888 of I = 5.7,5.7 & 7.0

★ Two Earthquakes In 1948 at I = 5.9 & 6.0

FIGURE 4.7

Historical Earthquake Record for Christchurch

70 km away near Motunau at a location 30 km south of the earlier 1901 Cheviot earthquake, but in the same general area. We adopt here the term "Motunau" for this event. The magnitude has been estimated as M = 6.75.

In Christchurch there was general demolition of chimneys, fall and cracking of masonry and pavements littered with debris. Liquefaction was reported 30 km from the city centre at Waikuku Beach and further away at Leithfield Beach.

1888 September 1st, Amuri (Glynn Wye) Earthquake

This earthquake is relatively well known in Christchurch as a result of damage to the upper 8 m of the Anglican Cathedral spire (refer report cover photograph). Dibble et al assign a felt intensity of MM VII in Christchurch in contrast to the inferred intensity of MM VIII by the Observatory.

Cowan (1989, in press) supports this lower estimate by Dibble et al, estimating felt intensities from reports of MM VI - VII and concludes there was local amplification in Christchurch. Chimneys were damaged and slates and brickwork affected.

The epicentre was approximately 100 km away and associated with rupture of the Hope Fault at Glynn Wye (McKay, 1890). The magnitude is estimated by Cowan 1990 to have been M = 7.0 - 7.3.

1901 November 15, Cheviot Earthquake

This earthquake resulted in intensities in Christchurch up to MM VII, similar to the Amuri (Glynn Wye) event i.e. MM VII. The Anglican Cathedral spire was once again damaged and there are reports of cracked masonry and chimney damage. Dibble et al suggest approximately half an intensity unit average amplification occurred under Christchurch, but it is not clear what this is referenced to since the Canterbury Plains around Christchurch are also on deep alluvium and the Port Hills (rock) are more distant from the epicentre.

Although Observatory records suggest other earthquakes reached comparable intensities in Christchurch, for example the 1895 August 5th event with an epicentre 25 km from the city centre in Pegasus Bay, detailed studies suggest this was probably not the case (Dibble et al assign intensity MM VI). More work could usefully be done, both collating existing research notes and extending this research, and we suggest this is an obvious area which requires further study.

4.4 SEISMICITY MODEL FOR NORTHERN SOUTH ISLAND

The seismicity model used here is based on the traditional model describing the rate of occurrence of earthquakes of different magnitude (Gutenberg & Richter, 1954)

$$\log N = a - bM \tag{4.1}$$

where N is the number of earthquakes having magnitudes M or greater and parameters a and b vary among regions but are assumed constant throughout each region. The specific form used by Smith & Berryman (1983) is obtained by integrating the general model and constraining the magnitude to be below a maximum value, M_{max}:

$$N = N_o[10^{b(Mo - M)} - 10^{b(Mo - Mmax)}] \tag{4.2}$$

N_o is the number of earthquakes of magnitude M_o or greater, where M_o may be, but is not necessarily, a lower detection threshold. By defining parameter a_4 to be the annual number of earthquakes of magnitude $M \geq 4$ in an area 1000 km^2, equation 4.2 becomes:

$$N = a_4\ [10^{b(4 - M)} - 10^{b(4 - Mmax)}] \tag{4.3}$$

In order to define the seismicity in each region it is therefore necessary to determine three parameters; a_4, b, M_{max}, which may be determined from records of historic seismicity and geologic considerations. It is useful first to review previous work carried out to determine these parameters.

Statistical studies for large regions containing many fault zones have confirmed that the logarithmic Gutenberg/Richter seismicity model is valid for these regions. In these regions, the "b" value is constant and generally close to $b = 1.0$. Until the mid 1970's it was commonly assumed that the 'constant b-value' model was equally appropriate to individual faults or small, tightly constrained fault zones. However studies of active late Quaternary faults has indicated that this assumption is not valid except for lower magnitude earthquakes.

A marked mismatch has been shown between occurrence frequencies projected from lower magnitude seismicity data to larger magnitude earthquakes, and geologically derived recurrence intervals (Schwartz & Coppersmith, 1984). The logarithmic relationship is non-linear, and in all cases reported represents an increase in probability of higher magnitude earthquakes over that obtained by extrapolation from low magnitude seismicity data.

Youngs & Coppersmith (1985) compared this behaviour in the Alaskan subduction zone, the Mexican subduction zone, and crustal fault zones in Turkey, Sweden, Greece, Japan, and the U.S.A. In all these cases the data sets appear sufficient to clearly define non-linear relationships (e.g. Singh, et al, 1981; Schwartz et al, 1981; Lahr & Stephens, 1982; Wesnousky et al, 1983). The relationship is generally non-linear for the upper 1.5 magnitude units. This represents the offset between the constant b-value prediction from lower magnitudes, and the approximate magnitude of larger earthquakes in each fault zone.

The implication is that for individual fault zones, estimates of large earthquake frequency based on extrapolation of small magnitude seismicity data may underpredict the larger magnitude event by up to 1.5 magnitude units. Without good geological evidence for large historic or prehistoric earthquakes it is difficult to incorporate this effect into a hazard model.

It is necessary to assume a low b-value in the moderate earthquake range to reconcile geologic and seismicity data (Youngs & Coppersmith, 1985). Schwartz & Coppersmith (1986) suggest b-values decrease from about b = 1.0 at low magnitudes to b = 0.2 - 0.4 at higher magnitudes.

Smith & Berryman (1983) used New Zealand values for b in the relatively narrow range from 0.95 to 1.2, based on instrumental observations from 1965 to 1983 for each of their 15 regions, although they do not describe precisely how each value was determined. These values are consistent with the relatively coarse regions employed, rather than the conclusions for specific faults described above, since most of their regions not only incorporate numerous faults, but also large areas of lower seismicity. However it might be expected that lower b values will be found for some of the tightly constrained, smaller regions used in this study.

Parameter M_{max} is commonly taken as the magnitude corresponding to rupture of the entire length of the fault zone if occurrence rate equation 4.1 is used, truncated at M_{max}. However the form of equation 4.2 dictates a zero occurrence frequency at M_{max}. It is therefore necessary to use a slightly higher value than that estimated from the fault length if a small but finite probability is admitted that the entire fault length may rupture in a given earthquake. This does not significantly affect the hazard calculations for Christchurch, and is consistent with other seismic hazard analysis models. The maximum magnitude estimates for total fault rupture used in the seismicity model are those discussed in section 4.2 and calculated in Table 4.1.

The seismicity model of Smith & Berryman (1983) is based on instrumental data for the period 1965-82 for earthquakes with $M \geq 4$, instrumental data from 1942-82 for $M \geq 5$ and combined instrumental data and historic records from 1840-1982 for $M \geq 6.5$. In this study the following earthquake data has been used, with occurrence frequencies calculated at each magnitude step:

$M \geq 3$ $M \geq 4$ $M \geq 5$	Instrumental data recorded 1964-88 See Figure 4.4 (Geophysics Division, DSIR, 1989)
$M > 6$	Earthquake records 1940-88
$M > 6.5$ $M > 7$	Earthquake records 1840-1989

Use was made of a list of earthquakes in the central-north South Island provided from the computer catalogue held by the Seismological Observatory. Particular acknowledgement is made of the discussions and advice provided by Dr Euan Smith at the Observatory. Several important facts have been considered when fitting seismicity models to the data. Some of these are discussed by Smith (1982).

- Early earthquakes (mainly pre 1940) may have large uncertainties in assigned magnitude and epicentre. Prior to 1900 the record is dependent solely on interpretation of felt information.
- The period of recording in New Zealand is relatively short, and very short for instrumental records especially of smaller earthquakes.
- The accuracy and sensitivity of the instrumental network may cause a deficiency in the record of earthquakes with magnitude $3 \leq M < 4$ at the lower end of this range.
- There has been a relative quiescence of larger earthquakes in New Zealand since 1930-1940, following a more active period when a number of large earthquakes occurred.

By extending the range of earthquake magnitudes considered (while taking into account the factors above) and by including $M \geq 6$ as an additional category for analysis, it has been found in this study that it is possible to identify and make reasonable allowance for most of these factors in developing the seismicity model. As argued by Smith & Berryman (1983), the attenuation function will assist in smoothing local anomalies when effects at Christchurch are assessed.

It is uncertain whether the quiescence in larger earthquakes since about 1930 is matched by a quiescence in small earthquakes. Only long-term records reanalysed regularly (every 20 years) will be able to confirm or refute this.

Details of the seismicity model employed are listed in Table 4.3, for the regions discussed earlier and shown in Figures 4.4, 4.6 and 4.8. The approximate error in the number of earthquakes assigned to each region, in each magnitude range and time period, may be assessed deterministically for very low recorded occurrences (1 or 2 per time period) as ± 1 earthquake. For example, if earthquakes occurred immediately before and after the time period considered,

ZONE		DESCRIPTION	AREA (1000's sq km)	Smith & Berryman				M >= 3 (1964-1988)		M >= 4 (1964-1988)		M >= 5 (1964-1988)		M >= 6 (1848-1949)		M >= 6.5 (1848-1989)		M >= 7 (1848-1989)		USE		
New	Old Smith & Berryman			Mmax	b	a4		N3	N3/At	N4	N4/At	N5	N5/At	N6	N6/At	N6.5	N6.5/At	N7	N7/At	Mmax	b	a4
Gn	<G	Marlborough	26.60	8.5	1.10	0.600		----		----		----		5	0.00130	1	0.00026	1	0.00026	8.5	1.1	0.6000
H	H	Alpine Fault	24.37	7.5 8.0	1.05 1.05	0.135 0.550		----		----		----		1	0.00029	----		----		7.0 8.5	1.05 (M<6.5) 0.05 (6.5<M<8.5)	0.1350 0.0025
CBsw	<J	SW Canterbury	25.13	8.0	1.10	0.110		----		----		----		----		----		----		8.0	1.0	0.1100
CBse	<K	SE Canterbury	9.50	8.0	1.10	0.030		----		----		----		5	0.00140	1	0.00280	----		8.0	1.1	0.0300
CBnw	<I	NW Canterbury	3.04	8.5	1.10	0.400		46	0.60500	16	0.21100	5	0.06600	2	0.00460	1	0.00230	----		7.5	0.6	0.3000
CBne	<G	NE Canterbury	2.60	8.5	1.10	0.600		17	0.26200	9	0.13800	3	0.04600	3	0.00810	2	0.00540	----		8.0	0.5	0.1500
HFs	<G,I	Hope - south/central	2.50	8.5	1.10	0.400-0.600		37	0.59200	14	0.22400	3	0.04800	1	0.00280	1	0.00280	1	0.00280	7.8	0.5	0.1900
PPT	<I,J	Porters Pass T.Z	5.08	8.5	1.10	0.110-0.400		94	0.74000	28	0.22000	6	0.04700	2	0.00280	----		----		7.5	0.8	0.3000
CPS	<J,K	Canterbury Plains Seismic	4.07	8.5	1.10	0.030-0.110		26	0.25600	6	0.05900	1	0.01000	----		----		----		7.5	0.8	0.0600
PGS	<G	Pegasus Seismic	4.04	8.5	1.10	0.600		16	0.15800	6	0.05900	1	0.01000	----		----		----		7.5	0.6	0.0800
BPS	<G,K	Banks Peninsula Seismic	13.18	8.5	1.10	0.030-0.600		14	0.04200	4	0.01200	1/142yr	0.00110	----		----		----		7.5	0.6	0.0150
		North Canterbury Total	34.51					250	0.29000	83	0.09600	19	0.02200	8	0.00160	4	0.00082	1	0.00020	8.0	0.8	0.1000
(S&B G,H,I,J,K)		Central Sth Island Total	126.90					----			0.12500		0.01500	19	0.00105	6	0.00033	2	0.00011	8.5	0.9	0.1200
(S & B)		NZ Total	502.80			0.430		----			0.24800		0.02200	----			0.00057	----		8.5	1.05	0.2500

TABLE 4.3

Parameters for Use in Seismicity Model

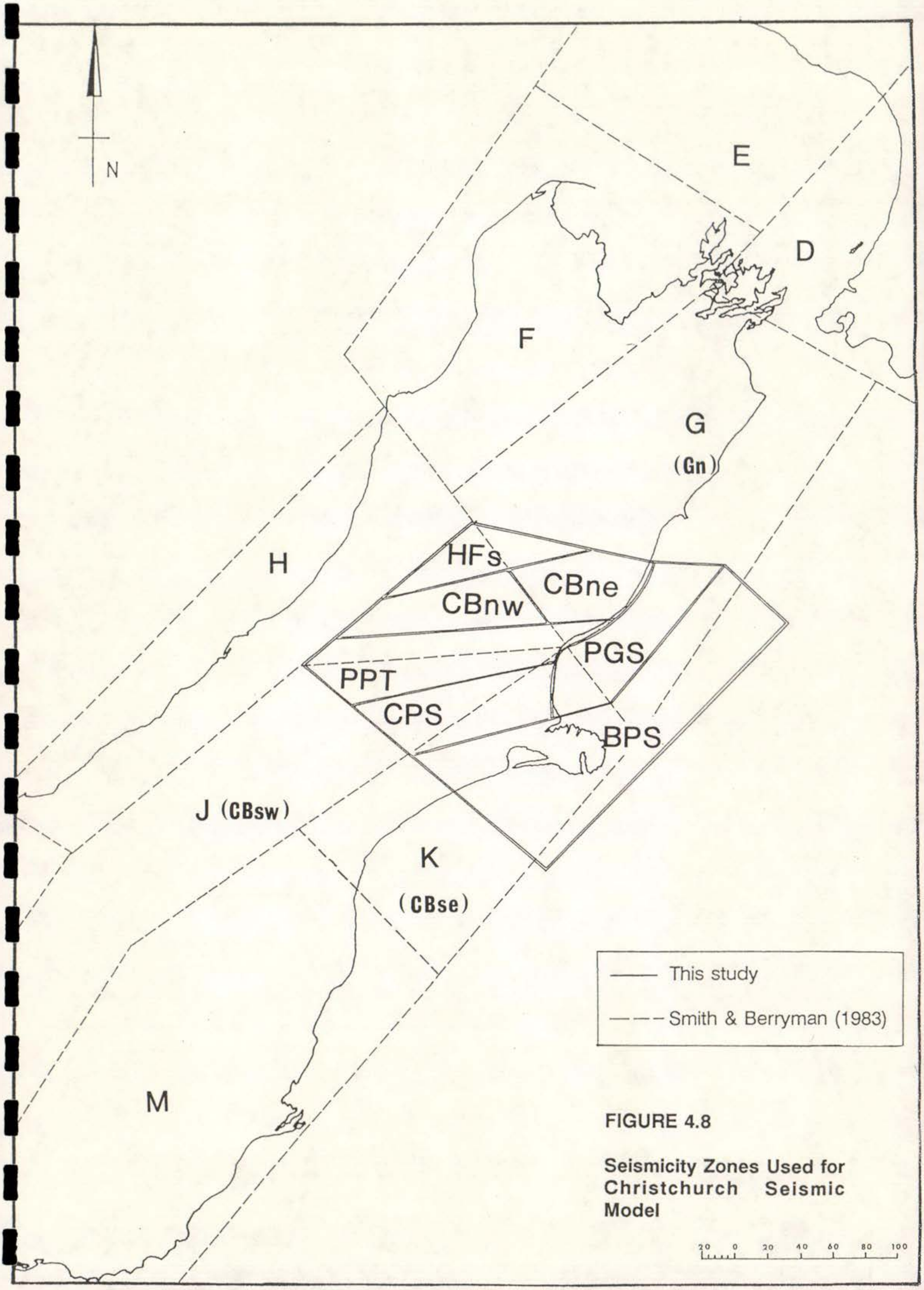


FIGURE 4.8

Seismicity Zones Used for Christchurch Seismic Model then the mean recurrence interval would correspond to just one additional earthquake in that period. Likewise if an earthquake occurred just inside each end of the time period, the mean occurrence frequency would correspond to one fewer earthquake. Where only one earthquake of given magnitude is recorded, it is possible that this is a single event with a very long recurrence interval, so that the actual frequency in that time period is between 0 and 2.

However it should be noted that some moderate earthquakes ($6 \leq M \leq 6.5$) distant from Christchurch may have occurred but not been recorded early in the history of the city. The values for $M \geq 6$ may therefore be low.

Occurrence frequency data are shown in Figure 4.9, for the new zones defined in Figure 4.8. Seismicity data for the seven zones CBnw, CBne, HFs, PPT, CPS, PGS and BPS are shown. Data for all earthquakes in the seven new regions are totalled and shown as "All North Canty".

- Actual magnitudes of earthquakes plotted in Figure 4.9 may be up to one magnitude unit higher for $M<6$ or up to 0.5 units higher for $M>6$.
- Due to the smaller regions, few or no earthquakes are included in some new regions for $M \geq 6$. However this also occurs in the less seismic of the original regions of Smith & Berryman. Where no earthquake occurs in the given time period the actual frequency is therefore 0 - 1.0.
- When all seismicity data for New Zealand are combined the best fit is given approximately by $b = 1.05$.
- Although Smith & Berryman used b values from 0.95 to 1.2, the best fit to their data for all regions in the centra/northern South Island, i.e. their regions G, H, I, J, K, is given by $b = 0.9$.
- When all seismicity data for the seven new, smaller zones proposed for North Canterbury are combined, the best fit for $3<M<7$ is given by $b = 0.8$.
- For each individual new zone in North Canterbury, the best fit is given by $b = 0.5$ to 0.8. The two zones with the smallest b-value ($b = 0.5$) are the most tightly defined and both include faults which are reasonably well defined geologically and have experienced several large historic earthquakes (CBne: Kaiwara Fault, three earthquakes with $M>6$ and HFs: Hope Fault south, good geologic evidence for regular earthquakes $M = 7.0 - 7.3$ at intervals 90 - 170 years).
- These low b-values are consistent with the evidence discussed earlier that for tightly constrained fault zones average b-values are likely to decrease from about 1.0 to 0.2 - 0.4 at high magnitudes.

- Evidence for the (logarithmic) non-linearity discussed earlier is also apparent, even given the limited length of records available and error bounds to be applied to data in Figure 4.9.

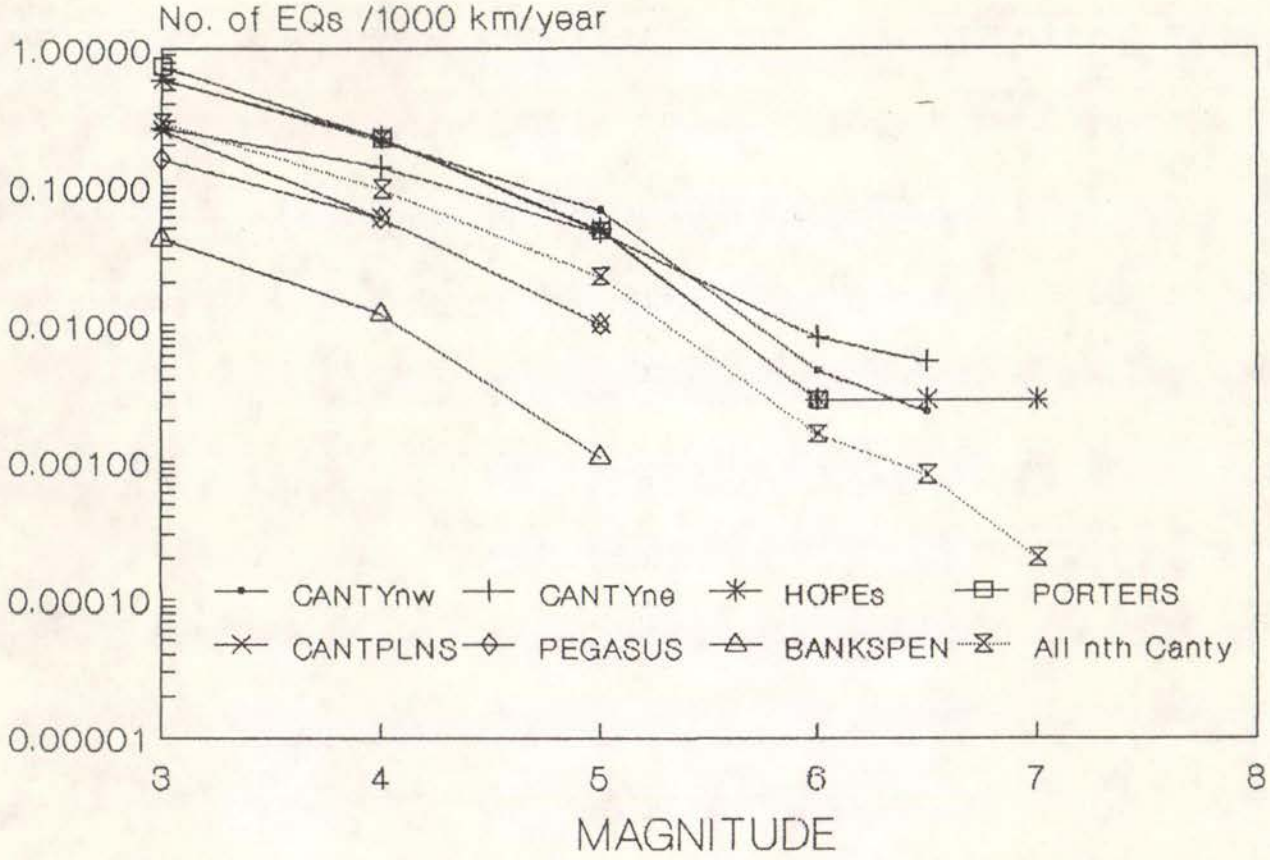
  For example, zone HFs experienced no earthquakes between magnitudes 6 and 7 in 142 years, yet based on the conclusions of Cowan & McGlowan (in prep.) every 90 - 170 years experiences an earthquake with $M = 7.0 - 7.3$. The connected line between the data points at $M = 5$ and $M = 7$ for this zone on Figure 4.9 with $b = 0.5$ is therefore an 'average fit' to low and high magnitude data; the actual data would be better represented by a higher b value for $M<5$ and a much lower b-value for $6<M<7.5$.

- Although non-linear data fits could be sought, there are insufficient data to do this adequately for each zone. Instead, in this study, the 'average b-value' fit has been used. In zones where no large earthquakes have occurred in historic times and where no geological evidence has been obtained for large earthquakes, this method is biased towards the lower magnitude seismicity data. It is very possible that in these cases the probability of large magnitude events ($M>6$) is severely underestimated. This may be so particularly for zones CPS, PGS and BPS, and these zones are the three closest to Christchurch City. However until further research has been carried out to identify deeper faulting in these zones, we consider it is not practical to allow for these potential larger events in a logical manner. This aspect clearly requires additional future consideration.

A brief discussion of each region, and the parameters used in the seismicity model (Table 4.3) is appropriate.

Regions Predominately as used by Smith & Berryman:

Region F:	Nelson. As defined by Smith & Berryman. Same parameters.
Region Gn:	Marlborough. Northern part of Region G of Smith & Berryman. Assigned the same seismicity parameters. Sufficiently distant from Christchurch for this to be satisfactory.
Region H:	Alpine Fault. Region H of Smith & Berryman, who proposed parameter values $b = 1.05$, $M_{max} = 8.5$, $a_4 = 0.20$. Recognising that this gave an extremely low occurrence frequency for a large magnitude earthquake, predicting $M \geq 8$ every 4,600 years, Matuschka et al (1985), following the suggestion of Berrill (1985), proposed revised values:



Seismicity Data for Smaller Regions Defined in this Study

FIGURE 4.9

Earthquake Occurrence Frequency

$M \leq 7.5$:	$b = 1.05$, $M_{max} = 7.5$, $a_4 = 0.135$
$7.5 \leq M \leq 8.0$:	$b = 1.05$, $M_{max} = 8.0$, $a_4 = 0.55$

However this approach has two deficiencies. The first is that it creates an excessive discontinuity at M = 7.5, where the occurrence frequency changes from 0 to 1 every 500 years. The second is that it makes insufficient allowance for a larger earthquake, predicting M≥7.9 every 4,300 years and no earthquakes with M≥8. This is contrary to the geologic evidence reported by Adams (1980) discussed earlier, and not since refuted, that major earthquakes occur at about 550 year intervals on the Alpine Fault with magnitudes possibly M>8. The alternative approach adopted in this study is to match observed data at M<6.5, including N = 0 (±1) for M>6.5 during 1848-1989. This is achieved using the same a and b values proposed by Smith & Berryman/ Matuschka et al, but applying $M_{max} = 7$ which reduces the predicted occurrence of medium size earthquakes up to M = 6.5. (This relationship predicts N = 0.8 earthquakes with M>6.5 since 1848.) A second relationship is used for M>6.5 which matches the relationship for lower magnitude earthquakes at M = 6.5 and predicts a recurrence interval about 500 years for M>8. This bilinear form is very similar to those discussed earlier (Schwartz & Coppersmith, 1986).

Region CBsw: South-west Canterbury. Southern part of Smith & Berryman region J. Assigned similar seismicity parameters. Truncated 175 km from Christchurch for later analyses. Relatively low seismicity or importance for Christchurch.

Region CBse: South-east Canterbury. Southern part of Smith & Berryman region K. Assigned the same seismicity parameters. Low seismicity and importance compared to all other regions.

New Zones

Region CBnw: North-west Canterbury. Small zone (3040 km^2). No known major faults but probably included 1929 Arthurs Pass earthquake (M = 6.7 - 7.0) epicentre. Model predicts M≥7 every 140 years. No other large earthquakes (M>6) in record periods used. This suggests non-linear seismicity is possible.

Region CBne:	North-east Canterbury. Very small zone (2600 km^2). Seismically active area, recently quieter, including Kaiwara Fault and probable epicentre of 1901 Cheviot earthquake, $M = 7$. Model predicts $M \geq 7$ every 120 years. Also epicentre of 1948 $M = 6.4$ event and the 1922 Motunau earthquake. Terminated at the coast.
Region HFs:	Hope Fault, south section. Very small zone (2500 km^2) around the fault. Seismically very active area including 1888 Amuri earthquake ($M = 7.0$ - 7.3) with epicentre near Glynn Wye. Model predicts $M = 7.1$ every 130 years to be consistent with conclusions of Cowan (1990) that recurrence interval is 90-170 years. Strong evidence of non-linear seismicity relationship.
Region PPT:	Porters Pass Tectonic Zone. Larger zone (5080 km^2). Discussed in detail earlier. No historical earthquakes with $M>6.5$. Model predicts $M \geq 7$ every 270 years.
Region CPS:	Canterbury Plains Seismicity Zone. Zone area 4070 km^2. Discussed in detail earlier. No recorded earthquakes with $M \geq 6$ but active seismicity. Model predicts $M \geq 7$ every 1700 years. Large earthquakes may be underpredicted.
Region PGS:	Pegasus Bay Seismicity Zone. Zone area 4040 km^2. Includes offshore Pegasus Bay Fault trace. No recorded earthquakes with $M \geq 6$ but active seismicity. Model predicts $M \geq 7$ every 400 years. Large earthquakes may be underpredicted.
Region BPS:	Banks Peninsula Seismicity Zone. Large zone (13,200 km^2) extending from south of Rakaia offshore to area seawards of Kaikoura. Has experienced several recorded earthquakes with $M \geq 5$. Model predicts $M \geq 7$ every 600 years.

To check the overall validity of the seismicity model, a comparison is carried out of the predicted and actual recorded numbers of earthquakes in the seven new regions:

	$M \geq 5$ (1964-88)	$M \geq 6.8$ (1848-1989)
Actual Number:	19	4
Number Predicted:	18 - 24	3 - 8

The error in the actual number of earthquakes is ±1 event as discussed earlier. Magnitude $M \geq 6.8$ is picked for comparison purposes since the four earthquakes recorded in the period 1848 - 1989 with magnitudes greater than 6.5 all probably had magnitude 6.8 or higher. The ranges in the numbers predicted are the results of summing errors ±1 event in any zones where predicted occurrence frequencies are significant.

When considering the difficulties with fitting a linear logarithmic seismicity model to zones where the relationships are probably non-linear at high magnitudes, the match of the model and observed data is good.

The seven newly created zones have all been assigned maximum magnitudes $M_{max} = 7.5 - 8.0$, creating very low occurrence frequencies for $M > 7.0$. Smith & Berryman used $M_{max} = 8.5$ in these areas. The lower M_{max} values used in this study are considered more realistic.

4.5 SEISMICITY: SUMMARY

A seismicity model has been developed for the central and northern South Island which takes into account the available geologic, tectonic and seismicity evidence. The model is generally based on that of Smith & Berryman (1983). To improve the accuracy of hazard evaluation seven new, small seismicity zones have been employed in north and offshore Canterbury. Instrumental records for smaller earthquakes provide a reliable guide to general trends expected in the seismicity model; geologic evidence has been significant in defining occurrence frequencies and maximum magnitudes for large earthquakes. The only major departures from the general trends of the Smith & Berryman model are a decrease in the value of seismicity parameter b in some smaller zones where required to fit seismicity data, to 0.5 - 0.8, and a significant adjustment in parameters for the Alpine Fault region to reflect available geologic evidence for large earthquakes. Maximum magnitudes have also been reduced in regions where very large earthquakes are considered unlikely. The model parameters are consistent with those found worldwide in detailed studies of fault zone seismicity.

Considerable further work is required to better assess the earthquake potential of several important regions close to Christchurch. Some geologic work has begun in the Porters Pass Tectonic region and in conjunction with detailed monitoring of a microseismicity network has begun to record seismicity in the Canterbury Plains/Pegasus/Banks Peninsula seismic areas. Since the results of these studies could have major significance for the seismic hazard in Christchurch, it is important that they, and related studies, be given considerable priority in the near future. The other critical area requiring further paleoseismic research is the central section of the Alpine Fault. Despite the importance of this area to Christchurch, no further examination of paleoseismicity has been carried out since the work of Adams reported in 1980.

CHAPTER 5: INTENSITY PREDICTION

5.1 INTRODUCTION

The seismicity model developed in this study has been described in Chapter 4. In order to assess ground shaking effects at Christchurch it is necessary to use an attenuation model, describing the effect observed at any distance from the epicentre of an earthquake of some magnitude M. In this chapter, the attenuation model used to calculate intensities is described.

It has been suggested (e.g. Evernden & Thomson, 1985) that intensity is the only commonly used measure of ground shaking that correlates directly with damage to ordinary structures and can be accurately predicted for a postulated earthquake. Although this ignores recent advances in methods for assessing ground motion or structural response directly, intensity remains the simplest measure of an earthquake's effect at a site. Smith (1978a) confirms that intensity is "the single best parameter for measuring damage."

The Modified Mercalli scale is used in New Zealand to measure intensity, and has been adapted for New Zealand conditions by Eiby (1966). Details of the scale are shown in Appendix A. Analysis in this chapter is based in part on published isoseismal (constant intensity) maps prepared for New Zealand earthquakes by staff at the Seismological Observatory, Wellington. Unpublished maps for some earthquakes prior to 1955 were also provided by Dr Euan Smith of the Observatory.

5.2 MODEL FOR PREDICTION OF INTENSITIES IN THE SOUTH ISLAND

The intensity attenuation and prediction model presented in this study is a refinement of that developed by Smith (1978a, b) for 'average ground' and used by Smith & Berryman (1983). Conclusions from these earlier studies were discussed in Chapter 2.

Isoseismals

Isoseismals, or curves connecting locations of constant felt intensity from a given earthquake, are generally elliptical (e.g. Figure 7.4, Inangahua earthquake 1968). The equation of an elliptical isoseismal can be expressed in polar co-ordinates r, θ as shown in Figure 5.1 as:

$$r_e^2/r^2 = \frac{1}{e^2} \sin^2 (\phi - \theta) + \cos^2 (\phi - \theta) \qquad (5.1)$$

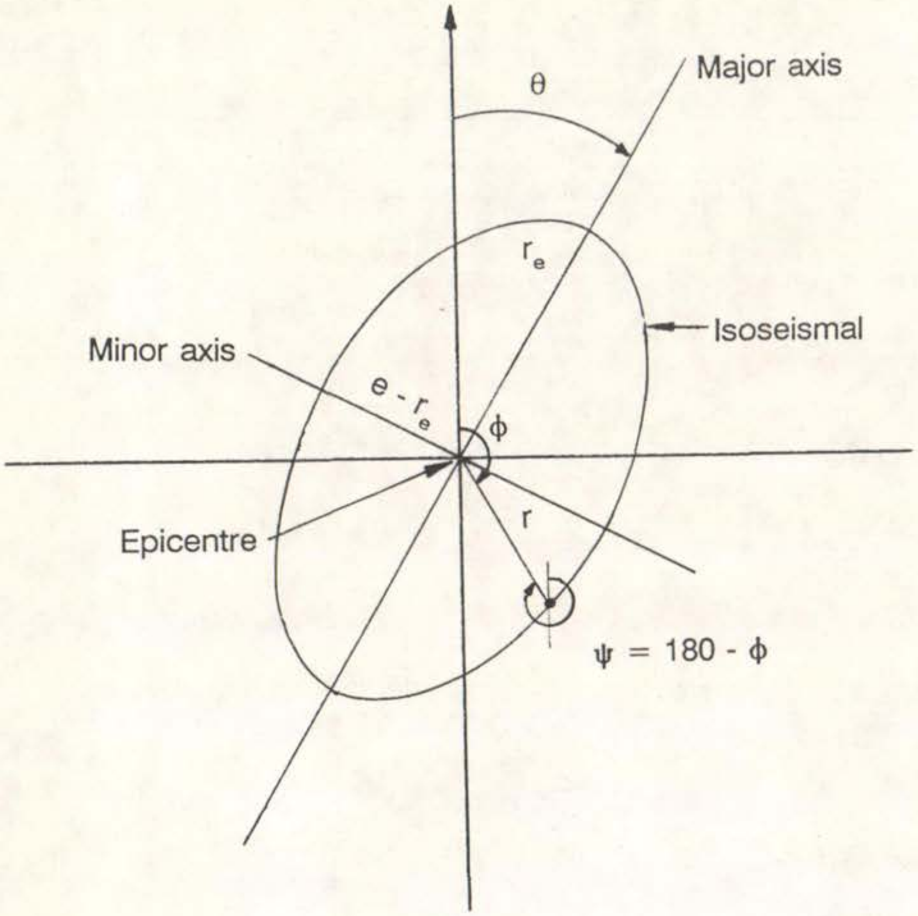
where
r = distance to any point on the isoseismal
r_e = effective epicentral distance along major axis
e = eccentricity ($e.r_e$ = minor axis distance)
ϕ = orientation of point east of north about centre
θ = orientation of major isoseismal axis east of north

Smith showed that the eccentricity of isoseismals varied with epicentral location through New Zealand, and produced a map showing contours of parameter e. This is used in this study and is reproduced, with smoothed contours, in Figure 5.2. Smith also considered the orientation of the major axes of isoseismals recorded in New Zealand. He concluded that most were aligned approximately N40°E, although for some this was the minor axis and others were close to circular. The latter two cases can be accounted for with a variable eccentricity, e.

In this study the major axis orientation was examined in greater detail. It is apparent that for most historical earthquakes for which isoseismal plots are available, the orientation of the major axis is close to that of the general trend of faulting in the epicentral area. This conclusion is not surprising; in fact it might be expected since seismic waves are more easily propagated through intact rock mass (parallel to faults) than perpendicular to the direction of faulting. The N40°E assumption represents a valid averaging for New Zealand, where the mean trend of fault orientation is between N30°E and N50°E.

In Figure 5.3 the following information is shown:

- mean orientation of significant faults (small numbers, uncircled)
- orientation of isoseismal axis in NE quadrant. In some cases, the orientation of axes northward is different to that southward from the



Isoseismal equation: $$\frac{r_e^2}{r^2} = \frac{1}{e^2} \sin^2 (\phi-\theta) + \cos^2 (\phi-\theta)$$

r = epicentral distance to site
r_e = effective epicentral distance
e = eccentricity

FIGURE 5.1

Definition of Elliptical Isoseismal

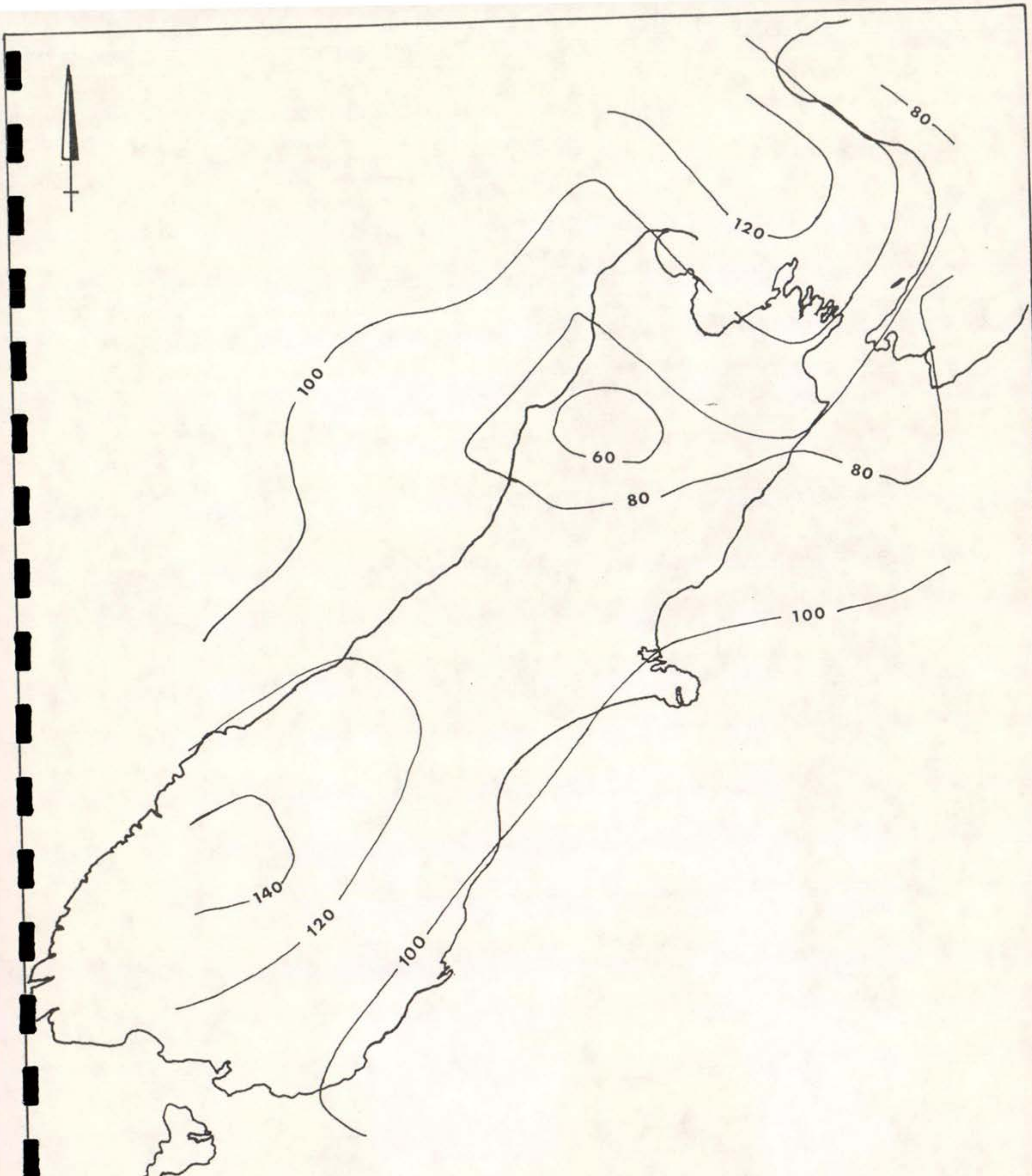


FIGURE 5.2

Mean Values for the Ratio of the Axes of Elliptical Isoseismals

For Shallow Earthquakes Multiplied by 100%
From Smith, (1978), with Smoothed Contours

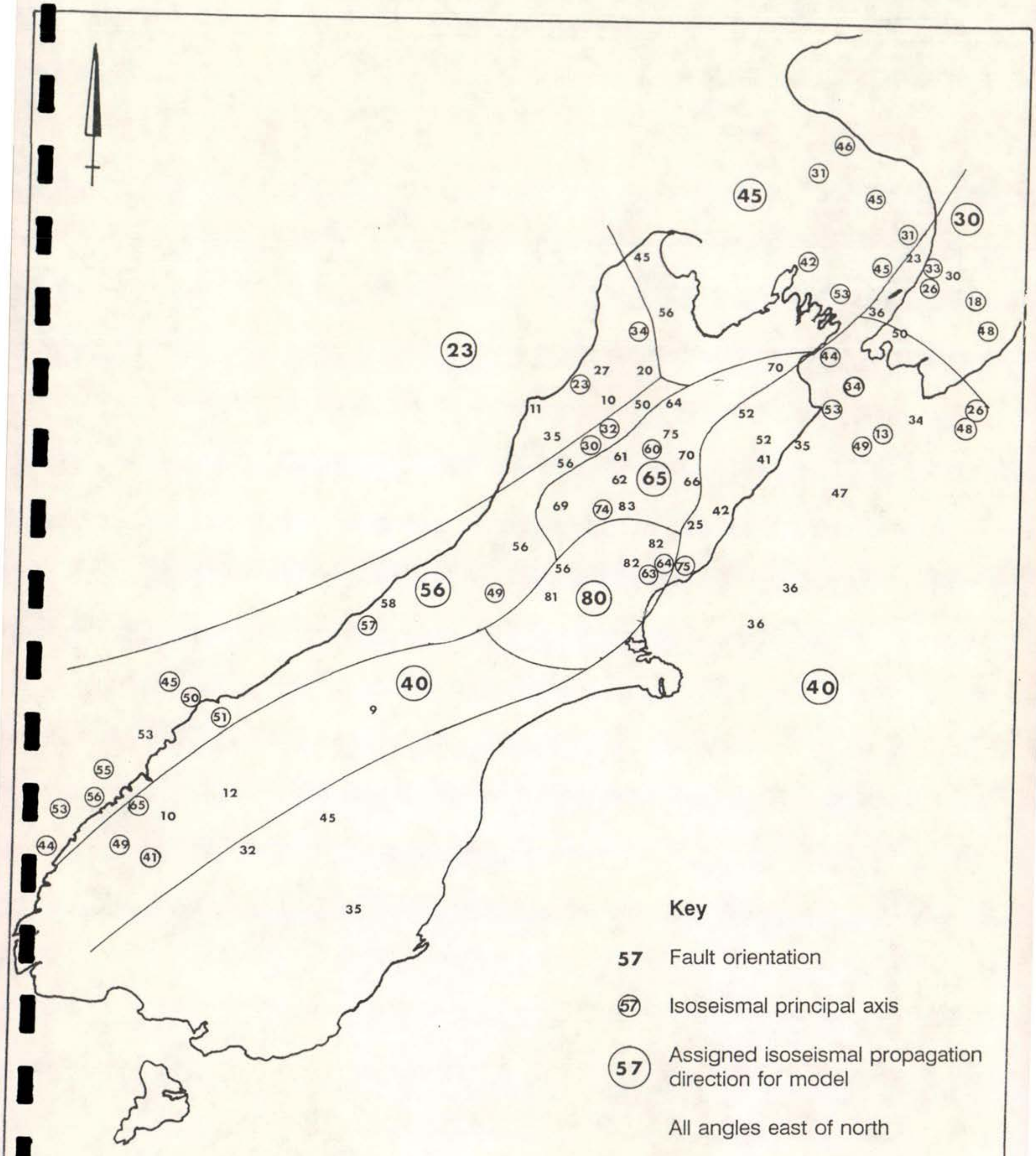
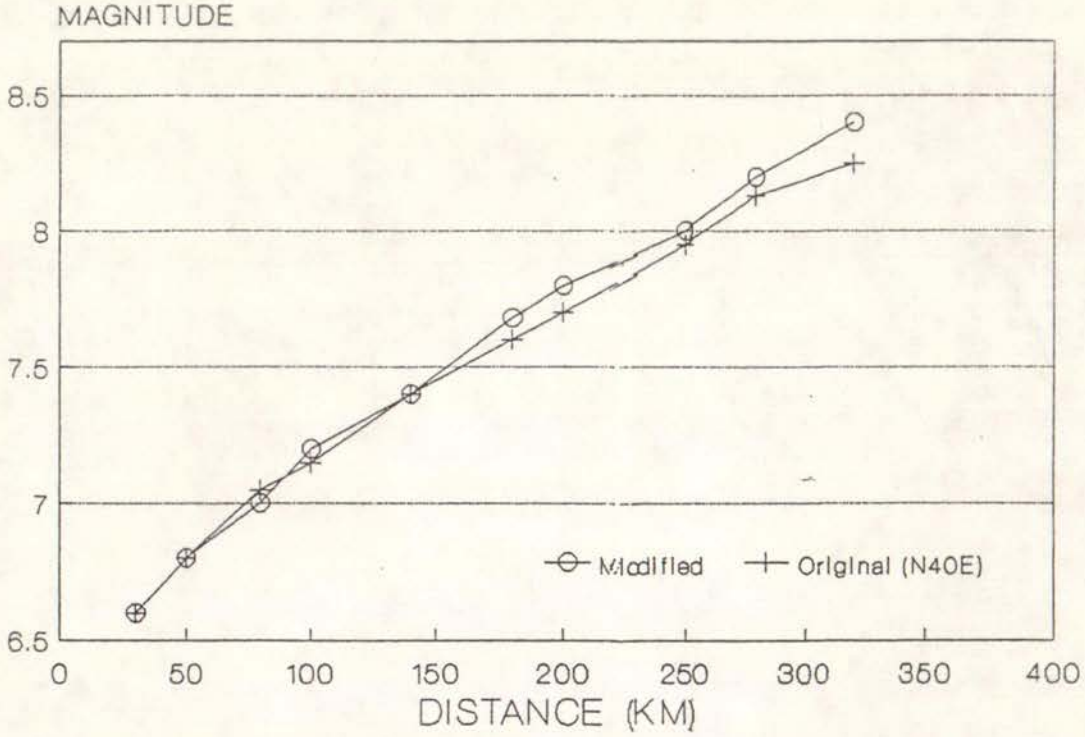
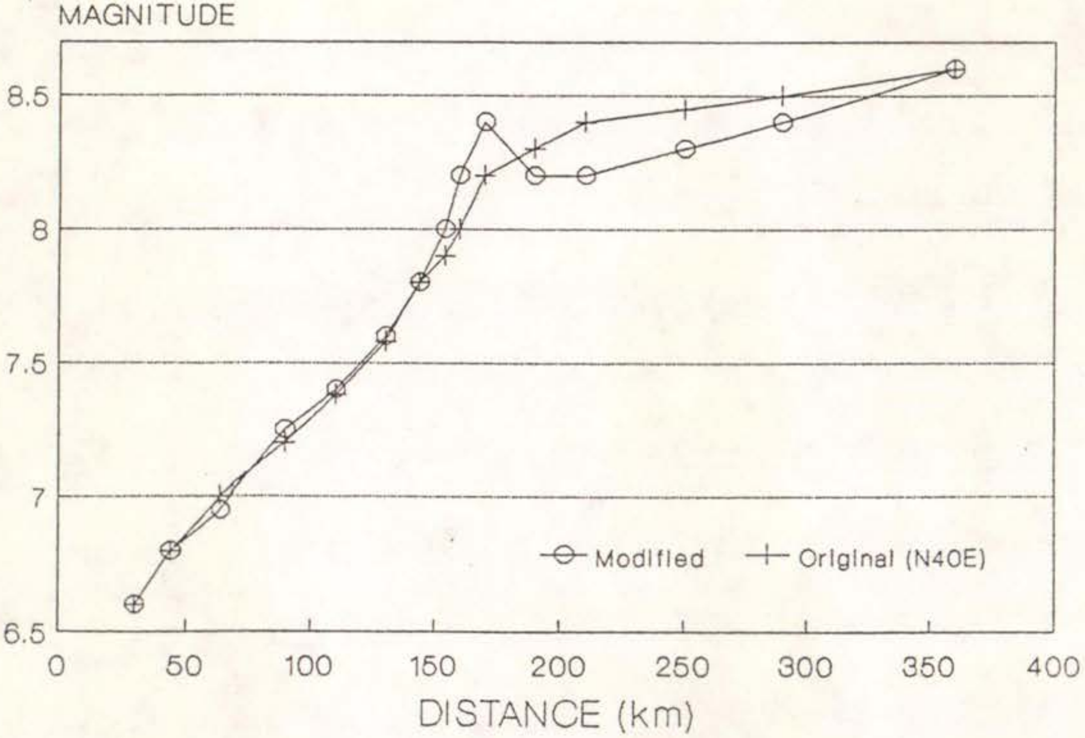


FIGURE 5.3

Principal Axis Orientation for Intensity Isoseismals



(a) **On Bearing 265° from Christchurch Towards Mt Cook**



(b) **On Bearing 350° from Christchurch Towards Lewis Pass**

FIGURE 5.4 Earthquake Magnitudes at Different Distances Required to Produce Intensity I = 8.0 at Christchurch

epicentre. This is expected from the analysis above. In these cases two values are plotted on Figure 5.3 (small numbers, circled).

The correlation between these two orientations is very good in most areas, with several exceptions including the relatively aseismic central South Island. This suggests division of southern New Zealand into the regions shown. In most cases the boundaries conform to recognisable tectonic boundaries and this has assisted their construction.

Equation 5.1 can be used to calculate isoseismal shapes for any earthquake epicentre in southern New Zealand by determining values of e and θ for that epicentre using Figures 5.2 and 5.3. The effect of this refinement to incorporate variable isoseismal axis orientation is illustrated in Figures 5.4(a) and (b). In these plots, the earthquake magnitudes required at different distances to produce 'continuous' intensity I = 8.0 at Christchurch is shown and compared to the simple N40°E model. Modified Mercalli intensities are obtained by truncating 'continuous' intensities. Two different directions are selected; N265°E, towards the South section of the Alpine Fault, and N350°E, through the North Canterbury area towards Nelson.

In the more southern direction (Figure 5.4a) there is no effect up to 150 km because the isoseismal orientation in these regions is 40°, the same as assumed by Smith. At greater distances the modification reduces the effect of a given earthquake at Christchurch. This is because the eccentricity in this region causes the major isoseismal axis to be in the SE quadrant (with $e<1$) and the modification orients this axis further away from Christchurch (E56°S rather than E40°S). Very large earthquakes on the southern section of the Alpine Fault will be less significant for Christchurch than previously supposed, although the change is small. To the north (Figure 5.4b) there is little difference between the two models at likely earthquake distances and magnitudes producing intensity 8.0 at Christchurch. This is primarily because the eccentricity was suggested by Smith to be close to $e = 1$ in North Canterbury. However these figures are also applicable to smaller intensities at Christchurch, and examination of the higher part of the graph is relevant. At distances from 150 - 180 km, the modified model requires a larger earthquake to produce the same intensity. However beyond 180 km, in the Nelson region, isoseismals are aligned much closer to North - South and showed marked eccentricity. A given effect at Christchurch can actually be caused by a smaller, more distant earthquake than predicted by Smith's model.

The general conclusion to be drawn from results of this analysis is that the effect for hazard analysis at Christchurch is relatively small except for very large, distant earthquakes. However the modification will have a far more significant effect in other areas and should be incorporated into future intensity prediction models for the South Island.

Intensity Attenuation with Distance

If the continuous intensity (which can be truncated to the Modified Mercalli intensity) is used, the intensity on each isoseismal may be obtained from an attenuation relationship describing the attenuation along the major axis. For calculation of intensities in Christchurch, only shallow earthquakes, with focal depths less than 40 km, need be considered. The justification for this is twofold. First, very few deep focus earthquakes have been recorded within 150 km of Christchurch; the majority of deep New Zealand earthquakes occur either in Fiordland or in the subduction zone beneath the North Island. Second, earthquake hazard is largely associated with shallow earthquakes since deeper earthquakes generate much lower intensities than comparable shallow events. Smith & Berryman (1983) report that in their study inclusion of deep activity had no effect on the frequency of occurrence of intensities MM VIII or higher.

Smith (1978a) developed three attenuation relationships for intensity with epicentral distance along the major isoseismal axis corresponding to three regional classifications for shallow earthquakes in New Zealand. Almost all earthquakes significantly affecting Christchurch will either originate in Smith's region B, or will propagate to Christchurch predominately through region B. In this study only Smith's type B attenuation has therefore been considered. Smith proposed an intensity attenuation relationship with magnitude and epicentral distance of the form:

$$I = C(r_e).M + D(r_e) \tag{5.2}$$

where $C(r_e)$ and $D(r_e)$ were tabulated as discrete functions of r_e. It was not the intention in this study to attempt to refine the general attenuation form developed by Smith. Construction of isoseismals is highly subjective, and any variation in the attenuation correlations would also be largely subjective. However it is useful for computational purposes to be able to express the attenuation relationship as a continuous function of the magnitude and distance. It was found here that this was well represented by a form proposed by Evernden et al (1973), but extended in this study for New Zealand to be variable with magnitude:

$$I = I_o - k \log (r_e + d) \tag{5.3}$$

where I_o, k, d are constant for a given magnitude, and

$$I_o = 0.6319 M^2 + 9.661 M - 60.15 \tag{5.4a}$$
$$k = 5.586 M - 26.87 \tag{5.4b}$$
$$d = 143 M - 768 \tag{5.4c}$$

all for $M \geq 5.5$.

Although this approach is purely empirical, it is logical to define the constants in equation 5.3 as functions of magnitude since this will allow the form of the attenuation equation to reflect the type and size of earthquake. The high quality of this fit to the curves proposed by Smith is shown in Figure 5.5. The maximum error is about one tenth of an intensity unit. In Figure 5.6 major isoseismal axes data are compared to the model for a number of South Island earthquakes. The correlation is generally good.

For M<5.5 the parameters in equation 5.3 become unstable and should not be used. Due to the sensitivity of the empirical fit to small variations in magnitude it is also necessary to calculate I_o, k and d using the four significant digits indicated, although of course this does not imply such precision in the calculated intensities. Eliminating r_e between equations 5.1 and 5.3 gives the equation of any isoseismal of given intensity, I as a function of location and earthquake magnitude

$$r = \frac{10^{(Io-I)/k} - d}{\left[\frac{1}{e^2} \sin^2 (\phi-\theta) + \cos^2 (\phi-\theta) \right]^{\frac{1}{2}}} \qquad (5.5)$$

where I_o, k, d are given by equations 5.4 which are implicit in the earthquake magnitude, M. In order to estimate the probability of occurrence of a given intensity at Christchurch it is necessary to calculate, for each source region, the probability of an earthquake occurring with sufficient magnitude to cause that intensity at Christchurch. This can be done by solving equation 5.2 iteratively for I, but in practice it is simpler to invert the equation to define "isosources", or curves connecting locations of constant magnitude which produce intensity I at Christchurch. This is done by using the transforming angle $\phi = 180 - \psi$, as shown in Figure 5.2, to give

$$r' = \frac{10^{(Io-I)/k} - d}{\left[\frac{1}{e^2} \quad \sin^2 (\psi-180-\theta) = \cos^2 (\psi-180-\theta) \right]^{\frac{1}{2}}} \qquad (5.6)$$

where I_o, k, d are given by equations 5.4 and ϕ is the orientation of each respective potential epicentre about the location of given felt intensity (e.g. Christchurch). To determine the probability of intensity I_{Chch} occurring at Christchurch it is only necessary to determine magnitude M (causing I_{Chch}) in each of a series of source regions, then sum the probabilities of occurrence for each regional magnitude. The accuracy of this method is determined by the accuracy of the seismicity model for each region, of the attenuation relationship described above, and the fineness of the source regions used in calculation.

These 'isosources' are plotted in Figures 5.7(a) to (e) for intensities I = 6, 7, 8, 9, 10 at Christchurch. The shapes of all isosources are similar between figures and as a good rule-of-thumb, an increase of 0.5 magnitude units at any source location causes an increase in intensity of one MM unit at Christchurch. Consideration of these figures, together with the discussion of seismicity in Chapter 4, indicates that Christchurch can expect to feel widespread shaking intensities up to MM VIII at regular intervals, since most of the seismic regions discussed appear capable of generating earthquakes of the magnitudes required. This is based on "average ground" and makes no allowance for local amplification which may occur on deep alluvial soils. Smith & Berryman (1983) calculated a return period of 600 years for MM IX at Christchurch; this estimate is likely to be too long and is reviewed in the next section. Intensity MM IX may be generated by large, infrequent earthquakes. Intensity X ('continuous' intensity $I \geq 10$) would only occur in a large earthquake ($M \geq 7.5$) very close to Christchurch, or a major earthquake ($M \geq 8$) probably on the Alpine Fault.

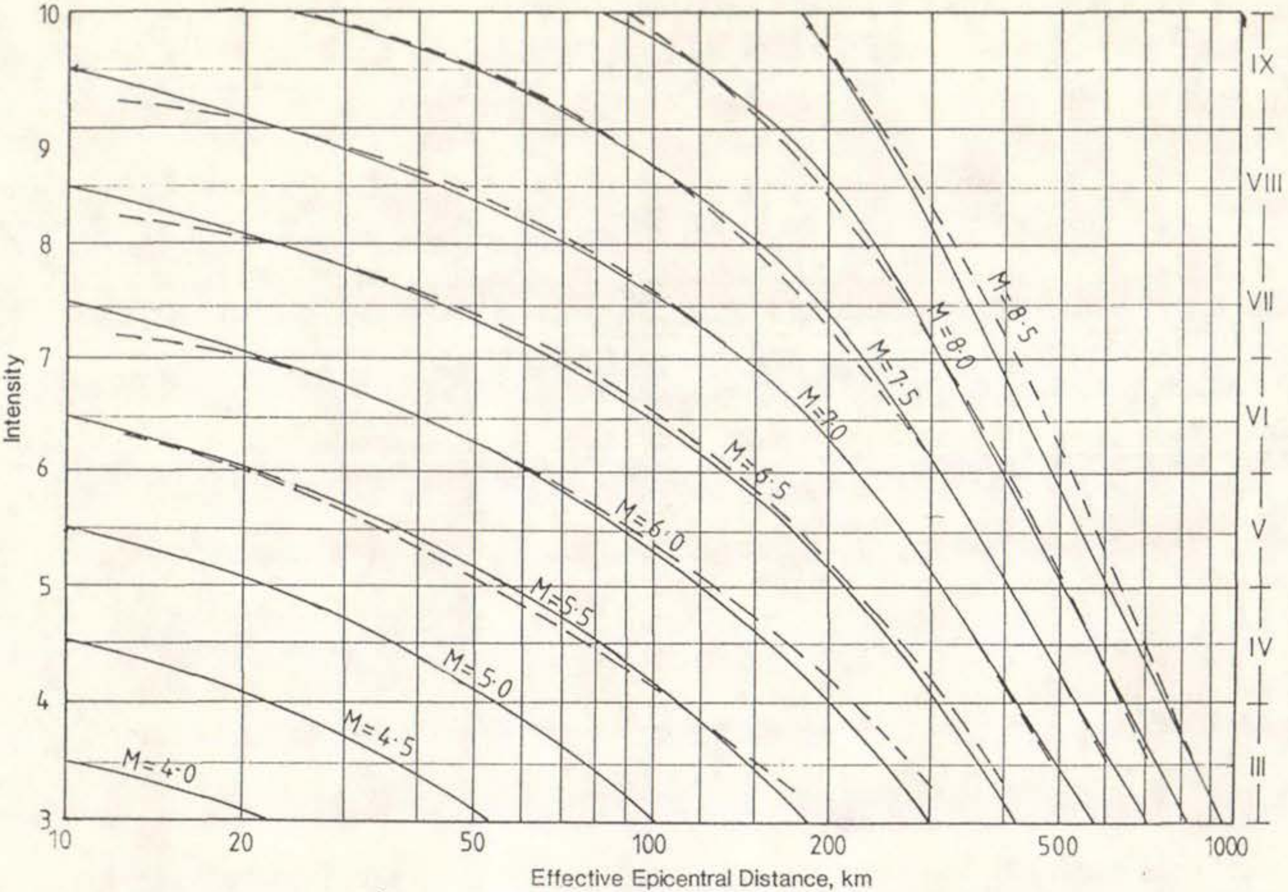


FIGURE 5.5

Attenuation Relationship for Intensity with Epicentral Distance Along Major Isoseismal Axis

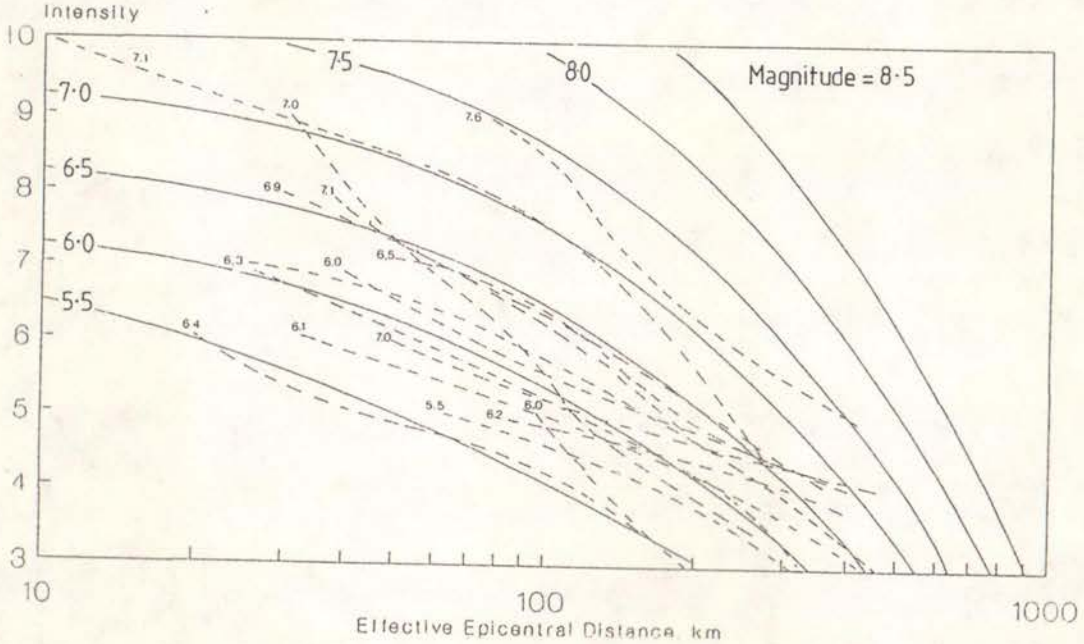


FIGURE 5.6

Attenuation Data Along Major Isoseismal Axis for South Island Earthquakes

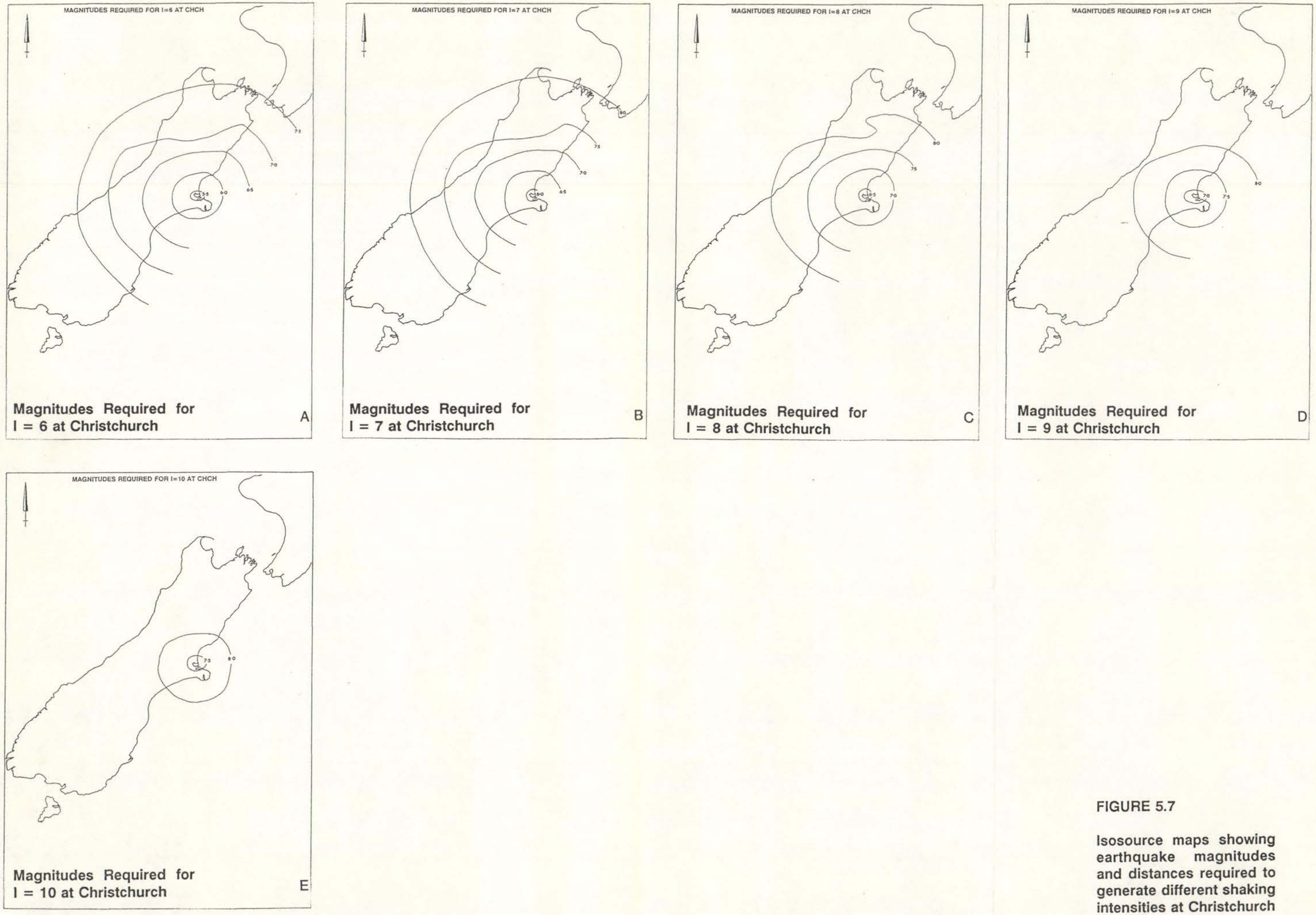


FIGURE 5.7

Isosource maps showing earthquake magnitudes and distances required to generate different shaking intensities at Christchurch

5.3 INTENSITIES AT CHRISTCHURCH: PROBABILITY AND RECURRENCE

In order to predict the earthquake hazard at Christchurch using intensities, it is necessary to determine the probability of different intensities occurring within defined time periods. A common method of describing general exceedance probabilities is to state the return period for each intensity level. If N_1 is the mean number of occurrences each year equalling or exceeding the stated intensity I (generally $N_1 << 1$ for significant intensities), then the probability of exceedance in any time period, t, is:

$$p(i \geq I) = 1 - \exp(-N_1 t) \qquad (5.7)$$

The return period, τ, is the inverse of the annual exceedance probability, p_1

$$\tau = p_1^{-1} = [1 - \exp(-N_1)]^{-1} \qquad (5.8)$$

The probability of exceedance $i \geq I$ in τ years is 63%, while if $N_1 t$ is very small then the probability, p is approximately $N_1 t$.

Using the seismicity model from Chapter 4, for each region k

$$N_k = A_{4k} [10^{b(4-M)} - 10^{b(4-Mmax)}] \qquad (5.9)$$

which gives the number of earthquakes per 1000 km^2 per year with magnitude $\geq$M, together with the intensity attenuation model from equation 5.6 of this chapter, the probabilities of different intensities are calculated as follows:

- Divide each seismicity region into subregions of sufficiently small area that a mean distance may be used from Christchurch to a central node, j, in the subregion. For each node, the area (A_j), attenuation parameters (θ_j, e_j), distance and orientation from Christchurch (r_j, ψ), and seismicity parameters (a_k, b_k, $M_{max\ k}$) are determined.
- For each intensity value I selected at Christchurch the earthquake magnitude M_j at node j which will cause intensity I is calculated from equation 5.6.
- The annual frequency of occurrence N_j of earthquakes with magnitude M_j or greater is calculated from equation 5.9 for each node (per 1000 km area).

The total number of earthquakes annually, $N_1(I)$, which cause intensity i>I is

$$N_1(I) = \Sigma_j (N_j A_j)$$

The probability of intensity I being exceeded in any one year is

$$p_1\ (i \geq I) = 1 - \exp\ (-\Sigma_j\ (N_j A_j))$$

with return period $\tau(I) = p_1(I)^{-1}$. The probability of exceeding intensity I in any period t is

$$p(i \geq I,t) = 1 - \exp\ (-t \times \Sigma_j\ (N_j A_j))$$

Detailed results of these calculations are presented in Table 5.1 and exceedance probabilities are summarised in Table 5.2, where the relative contributions to the overall probability from each seismicity region are also shown.

Return periods calculated in this study using the new seismicity model and the slightly revised attenuation model are in many cases shorter than those reported previously for Christchurch from a general national study by Smith & Berryman (1983) as shown in Figure 5.8. This is primarily due to the detailed analysis of tectonics and historical seismicity in smaller zones close to Christchurch which upgrades the probability of damaging earthquakes occurring within 150 km of the city. For intensities MM VII to MM VIII the likely probabilities approach those assessed by Smith & Berryman for Wellington, although Wellington will more often experience intensities MM IX or greater.

It is interesting to note the relative contributions to the overall probability made by different seismicity regions. For medium intensities $6 \leq 1 \leq 8$, probabilities are contributed almost equally by regions BPS (Banks Peninsula), PPT (Porters Pass), CBnw (NW Canterbury), CBne (NE Canterbury/Kaiwara) and Hfs (Hope Fault south). Smaller but significant contributions arise from regions CPS (Canterbury Plains), PGS (Pegasus Bay), Gn (Marlborough) and H (Alpine Fault). The contributions of other regions CBse, CBsw (South Canterbury east and west), F (Nelson), D (Wellington) and M (East Otago) are negligible. For intensities $1 > 8$ probabilities are contributed almost equally, and most significantly, by a major Alpine Fault earthquake (region H) or by a medium to large earthquake in the north-east Canterbury region, CBne. Also significant are regions CPS and BPS, for intensity 9 and Gn, increasing with increasing intensity.

It is common to consider the effect of an earthquake with a given, low probability of exceedance in a time period comparable to the design life of typical structures. From Table 5.2 the (continuous) intensity with 15% probability of exceedance in 50 years is about $I = 9$, although the stepped nature of the Modified Mercalli Scale means the probability of intensity MM IX occurring in 50 years is about 6%.

Finally it should be noted that the attenuation model used assumes 'average' ground conditions at a given site. It is likely that intensities may vary by ±1 unit or more at specific sites, according to geologic conditions. This is discussed further in Chapter 7.

NODE	ZONE	AREA	M(I=6)	M(I=7)	M(I=8)	M(I=9)	M(I=10)	Mmax	b	a4	N(I=6)	N(I=7)	N(I=8)	N(I=9)	N(I=10)
1	CPS	0.74	5.6	6.0	6.5	7.1	7.4	7.5	0.70	0.050	0.00267	0.00134	0.00053	0.00012	0.00002
4		1.13	5.8	6.2	6.6	7.1	7.6	7.5	0.70	0.050	0.00317	0.00143	0.00065	0.00018	
13		1.02	6.0	6.4	6.9	7.4	7.8	7.5	0.70	0.050	0.00185	0.00089	0.00030	0.00003	
21		0.93	6.1	6.6	7.1	7.6	8.0	7.5	0.70	0.050	0.00141	0.00054	0.00015		
31		0.25	6.3	6.8	7.3	7.7	8.0	7.5	0.70	0.050	0.00026	0.00009	0.00002		
2	PGS	0.30	5.7	6.0	6.5	7.1	7.4	7.5	0.70	0.040	0.00073	0.00044	0.00017	0.00004	0.00001
6		0.99	5.8	6.2	6.7	7.2	7.6	7.5	0.70	0.040	0.00204	0.00100	0.00037	0.00009	
11		0.86	6.0	6.3	6.9	7.4	7.8	7.5	0.70	0.040	0.00125	0.00073	0.00020	0.00002	
17		0.76	6.2	6.7	7.1	7.6	7.8	7.5	0.70	0.040	0.00077	0.00028	0.00010		
27		0.90	6.3	6.8	7.3	7.8	8.0	7.5	0.70	0.040	0.00076	0.00027	0.00005		
X		0.22	6.5	7.0	7.4	7.9	8.0	7.5	0.70	0.040	0.00013	0.00004	0.00001		
3	BPS	0.96	5.7	6.0	6.5	7.1	7.4	7.5	0.70	0.030	0.00175	0.00104	0.00041	0.00009	0.00002
7		2.87	5.8	6.2	6.7	7.2	7.7	7.5	0.70	0.030	0.00443	0.00218	0.00080	0.00019	
12		4.96	6.0	6.4	6.9	7.4	7.8	7.5	0.70	0.030	0.00539	0.00258	0.00086	0.00009	
18		1.41	6.2	6.6	7.1	7.6	8.0	7.5	0.70	0.030	0.00107	0.00049	0.00014		
28		1.27	6.4	6.9	7.3	7.8	8.0	7.5	0.70	0.030	0.00066	0.00022	0.00005		
35		1.32	6.5	7.0	7.4	7.9	8.0	7.5	0.70	0.030	0.00056	0.00017	0.00002		
19		0.40	6.2	6.8	7.1	7.6	7.9	7.5	0.70	0.030	0.00030	0.00009	0.00004		
5	PPT	0.82	5.8	6.2	6.7	7.2	7.6	7.5	0.74	0.170	0.00612	0.00292	0.00104	0.00024	
8		1.37	6.0	6.4	6.9	7.4	7.8	7.5	0.74	0.170	0.00710	0.00330	0.00106	0.00011	
14		1.02	6.2	6.6	7.1	7.6	8.0	7.5	0.74	0.170	0.00365	0.00162	0.00044		
22		1.08	6.3	6.8	7.3	7.7	8.0	7.5	0.74	0.170	0.00316	0.00108	0.00019		
32		0.80	6.5	6.9	7.4	7.9	8.0	7.5	0.74	0.170	0.00156	0.00062	0.00006		
9	CBnw	0.61	6.1	6.6	7.0	7.5	7.9	7.5	0.60	0.150	0.00428	0.00178	0.00072		
15		1.66	6.2	6.7	7.2	7.6	8.0	7.5	0.60	0.150	0.00992	0.00399	0.00101		
23		0.78	6.4	6.8	7.3	7.8	8.0	7.5	0.60	0.150	0.00330	0.00151	0.00029		
10	CBne	0.50	6.1	6.6	7.0	7.4	7.9	8.0	0.60	0.150	0.00382	0.00177	0.00089	0.00039	0.00004
16		1.68	6.2	6.7	7.1	7.7	8.0	8.0	0.60	0.150	0.01103	0.00503	0.00247	0.00051	
26		0.43	6.3	6.8	7.2	7.7	8.0	8.0	0.60	0.150	0.00240	0.00108	0.00051	0.00013	
24	HFs	2.50	6.5	7.0	7.4	7.9	8.0	7.8	0.60	0.310	0.02044	0.00822	0.00300		
25	Gn	1.46	6.5	7.0	7.4	7.8	8.2	8.5	1.10	0.600	0.00155	0.00043	0.00015	0.00005	0.00001
34		4.07	6.6	7.1	7.6	8.0	8.4	8.5	1.10	0.600	0.00334	0.00092	0.00024	0.00007	0.00001
40		8.91	6.8	7.3	7.8	8.2	8.5	8.5	1.10	0.600	0.00439	0.00119	0.00029	0.00007	
45		7.70	7.1	7.6	8.0	8.4	8.5	8.5	1.10	0.600	0.00175	0.00045	0.00013	0.00001	
49		4.46	7.3	7.7	8.2	8.5	8.5	8.5	1.10	0.600	0.00060	0.00020	0.00003		
20	CBse	1.97	6.2	6.6	7.1	7.5	7.9	8.0	1.10	0.030	0.00022	0.00008	0.00002	0.00001	0.00000
29		2.55	6.3	6.8	7.2	7.7	8.0	8.0	1.10	0.030	0.00022	0.00006	0.00002	0.00000	
36		2.77	6.5	7.0	7.4	7.9	8.0	8.0	1.10	0.030	0.00014	0.00004	0.00001	0.00000	
41		2.21	6.6	7.1	7.6	8.0	8.0	8.0	1.10	0.030	0.00009	0.00002	0.00000		
30	CBsw	0.90	6.3	6.8	7.3	7.7	8.0	8.0	1.00	0.110	0.00049	0.00015	0.00004	0.00001	
37		1.88	6.5	6.9	7.4	7.9	8.0	8.0	1.00	0.110	0.00063	0.00024	0.00006	0.00001	
42		4.50	6.7	7.2	7.6	8.0	8.0	8.0	1.00	0.110	0.00094	0.00026	0.00007		
46		3.75	7.0	7.5	7.9	8.0	8.0	8.0	1.00	0.110	0.00037	0.00009	0.00001		
50		3.55	7.3	7.7	8.0	8.0	8.0	8.0	1.00	0.110	0.00016	0.00004			
51		3.35	7.5	8.0	8.0	8.0	8.0	8.0	1.00	0.110	0.00008				
33	H	2.71	6.6	7.1	7.6	8.0	8.4	8.5	0.05	0.003	0.00098	0.00070	0.00044	0.00024	0.00005
38		9.88	6.8	7.3	7.8	8.2	8.5	8.5	0.05	0.003	0.00318	0.00218	0.00123	0.00052	
43		6.01	6.9	7.5	7.9	8.3	8.7	8.5	0.05	0.003	0.00181	0.00109	0.00064	0.00021	
47		3.85	7.3	7.7	8.2	8.6	8.8	8.5	0.05	0.003	0.00085	0.00055	0.00020		
52		1.92	7.5	7.9	8.4	8.7	9.0	8.5	0.05	0.003	0.00035	0.00020	0.00003		
39	F	2.40	7.4	7.8	8.4	8.5	8.5	8.5	1.13	0.700	0.00023	0.00007	0.00000		
44		7.28	7.3	7.8	8.3	8.5	8.5	8.5	1.13	0.700	0.00091	0.00022	0.00004		
48		10.00	7.4	7.9	8.3	8.5	8.5	8.5	1.13	0.700	0.00095	0.00022	0.00004		
53		9.06	7.6	8.1	8.5	8.5	8.5	8.5	1.13	0.700	0.00049	0.00011	0.00000		
54		2.71	7.7	8.2	8.5	8.5	8.5	8.5	1.13	0.700	0.00011	0.00002			
55	D	7.50	7.6	8.1	8.5	8.5	8.5	8.5	1.13	0.850	0.00049	0.00011	0.00000		
56	M	3.61	6.8	7.2	7.7	8.0	8.0	8.0	1.10	0.080	0.00023	0.00008	0.00001		
57		6.90	7.0	7.5	7.9	8.0	8.0	8.0	1.10	0.080	0.00025	0.00006	0.00001		
58		7.50	7.3	7.8	8.0	8.0	8.0	8.0	1.10	0.080	0.00012	0.00002			
59		7.40	7.6	8.0	8.0	8.0	8.0	8.0	1.10	0.080	0.00004				
										TOTAL	0.131949	0.056521	0.020304	0.003424	0.000159

TABLE 5.1 Probabilities of Occurrence of Different Intensities Detailed Results

TABLE 5.2 Probabilities of Occurrence of Different Intensities at Bedrock below Christchurch and Relative Contributions from each Seismicity Region

Percentage Contribution to Total Probability from Each Seismicity Zone

SEISMICITY	INTENSITY AT CHRISTCHURCH				
REGION	6	7	8	9	10
CPS	7.1	7.6	8.1	9.7	14.4
PGS	4.3	4.9	4.4	4.3	4.7
BPS	10.7	12.0	11.4	10.9	11.2
PPT	16.4	16.9	13.8	10.2	
CBnw	13.3	12.9	10.0		
CBne	13.1	13.9	19.1	30.1	27.8
HFs	15.5	14.5	14.8		
Gn	8.8	5.7	4.2	5.9	12.0
CBse	0.5	0.4	0.3	0.3	0.4
CBsw	2.0	1.4	0.9	0.4	
H	5.4	8.4	12.6	28.2	29.4
F	2.0	1.1	0.4		
D	0.4	0.2			
M	0.5	0.3	0.1		
Totals	100.0	100.0	100.0	100.0	100.0

Annual Exceedance N_1	0.132	0.0565	0.0203	0.00342	0.00016
Return Period (yrs)	7.6	18	50	292	6300
Probability (%) of Exceedance in:					
50 yrs	99.9	94.5	64.1	15.8	0.8
150 yrs	100	100	95.4	40.2	2.4
450 yrs	100	100	100	78.6	6.9
1000 yrs	100	100	100	96.8	14.7
Modified Mercalli Intensity	MM VI	MM VII	MM VIII	MM IX	MM X
Return Period	10	30	100	800	10,000

Note:

1. No modification has been made for site-specific intensity amplification effects.
2. Intensities calculated at top are 'continuous' intensities.
3. Modified Mercalli intensities are obtained by truncation, so that (e.g.) MM VII corresponds to 7.0 ≤ 8.0.

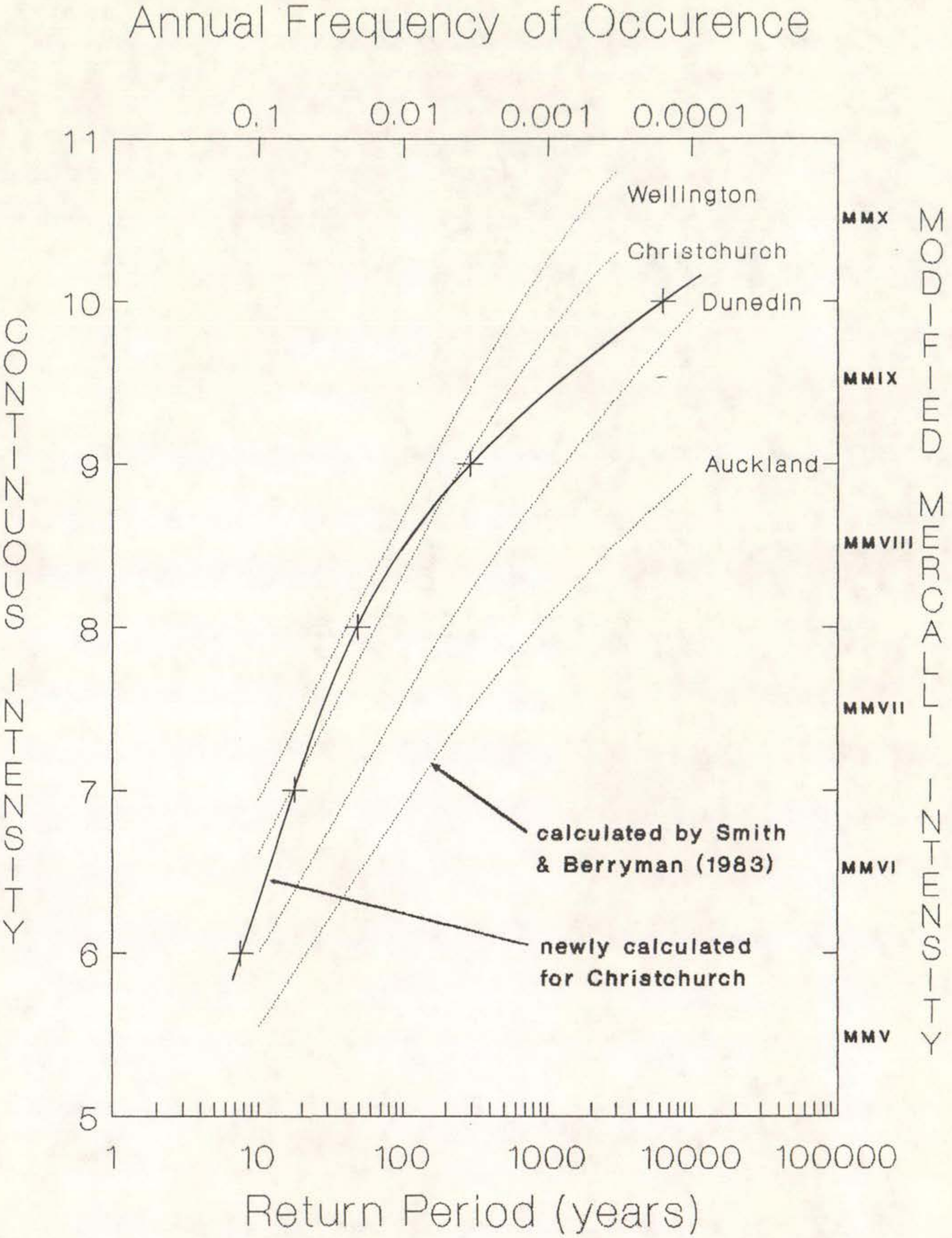


FIGURE 5.8

Occurrence Frequencies Recalculated for Seismic Intensities at Christchurch - no correction for ground amplification effects

5.4 SUMMARY

The intensity attenuation model developed by Smith (1978a or b) has been refined to allow for variable directions of energy propagation which are consistent with tectonic features in the South Island. A functional relationship has been developed which avoids the need for discretisation of the intensity-magnitude-distance correlation and simplifies analysis. This has been used with the seismicity model proposed in Chapter 4 to estimate exceedance probabilities for different intensities at Christchurch, for 'average' ground conditions only. Site-specific intensity amplification or reduction is considered in Chapter 7.

For medium intensities of shaking in Christchurch ($6 \leq I \leq 8$) the hazard is contributed almost equally by the seismicity zones nearest Christchurch i.e. Porters Pass (PPT), NW and NE Canterbury (CBnw and CBne), Hope Fault South (HFs) and the Banks Peninsula and Canterbury Plains seismicity zones (BPS and CPS). For higher intensities the Alpine Fault and NE Canterbury dominate with significant hazard from the Banks Peninsula and Canterbury Plains seismicity zones. Marlborough also becomes more important for very high intensities. Of these regions, only Hope Fault South and the Alpine Fault have been investigated from the seismotectonic viewpoint in detail, while work has commenced on the Porters Pass Tectonic Zone. There is a clear need for urgent further study of the seismically active regions nearer Christchurch which in many cases are either offshore or covered by deep Quaternary sediments.

Although values of the seismicity parameter 'b' have been calculated which are low by comparison with other New Zealand studies, this has not resulted in increased frequency of very high intensities (refer I≥MM 10 in Figure 5.8). The reduced frequency when compared with other studies is due to the compensating effect of the lower maximum magnitudes Mmax used in this study, and to the high relative importance of moderate earthquakes close to Christchurch.

CHAPTER 6: RESPONSE SPECTRA PREDICTION

6.1 INTRODUCTION

Although intensity correlates reasonably well with earthquake damage, it is a difficult parameter to incorporate into engineering seismic analysis and design. Instead an estimate of actual ground motion or forces generated on a structure may be required. Simple analysis of earth structures and slope stability often employs a single parameter, related to the peak ground acceleration. Detailed structural analysis may consider predicted time history of ground acceleration, velocity or displacement. However the most widespread general methods for structural design, including the current N.Z. Loadings Code NZS 4203: 1984 and its draft revision, incorporate a pseudo-static horizontal seismic force. This is derived from the structural response acceleration for the fundamental mode natural period of the structure. A structural response spectrum, defining response accelerations, velocities or displacements for all natural periods of typical structures, is required to allow this design approach to be used.

A number of methods have been proposed for construction of response spectra. Early approaches simply used scaled versions of spectra calculated from available strong motion records. More recent approaches, including that on which the current N.Z. Loadings Code is based, rely on predictive models which take account of the three major factors affecting the response spectrum ordinates; earthquake magnitude, epicentral distance and ground conditions at the site studied.

In this chapter a standard model of this type is used to predict acceleration response spectra for bedrock conditions at Christchurch. Various modifications proposed for New Zealand conditions are considered. Exceedance probabilities are estimated by using the spectral acceleration attenuation model together with the seismicity model presented in Chapter 4.

6.2 MODEL FOR PREDICTION OF RESPONSE SPECTRA

The basic spectral acceleration attenuation model used in this study is that described by Katayama (1982), based on records from over 100 Japanese earthquakes. Details of the model and its applicability to New Zealand conditions have been debated extensively elsewhere (Peek, 1980; Peek et al, 1980; Mulholland, 1982; Berrill, 1985a, b; Matuschka et al, 1985; McVerry, 1986). It is not necessary to review most of these reports here. However a number of modifications to the original model have been proposed and some were incorporated into the analysis which produced response spectra envelopes in the current New Zealand Loadings Code, NZS 4203: 1984. The effects and validity of these modifications require consideration.

The Katayama model predicts response accelerations, a_s, at 5% of critical damping for natural structure periods T = 0.05 - 4 seconds using a multiplicative form

$$a_s = f_m . f_r . f_{ac} \qquad (6.1)$$

where	$f_m(T)$	magnitude factor, for M = 4.5 - 7.9
	$f_r(T)$	distance factor, for r = 6 - 405 km
	f_{gc} (T)	ground condition factor

Katayama determined and tabulated values of f_m, f_r, f_{gc} in discrete ranges of magnitude and distance and for four ground conditions. A scatter in predicted vs observed data occurs which is not just due to the limitations in the form of the model, but which is caused by variations in earthquake type, in ground conditions for wave propagation from the source to the site, and by other natural, semi-random factors. This is discussed by Berrill (1985b). The resulting scatter in the attenuation is well represented by a log normal distribution of the calculated spectral acceleration about the predicted value, i.e. the parameter log (a_s) is normally distributed about the predicted value, which forms the mean of the distribution. Mitchell (1981) showed for Katayama's original data that the standard deviation σ_{10} of this normal distribution is a function of period, varying in the approximate range σ_{10} = 0.288 - 0.325. One logarithmic standard deviation above the mean therefore corresponds to a factor of 1.9 - 2.1 times the predicted spectral acceleration.

When carrying out a probabilistic analysis to estimate spectral accelerations, the larger number of smaller earthquakes predicted by a general seismicity model causes a substantial increase in the probability of any given spectral acceleration being exceeded. This probabilistic enhancement effect has been discussed in most of the work referred to above, particularly by Berrill (1985b) and McVerry (1986). Various spectral acceleration enhancement factors, B_z, have been calculated using different data sets.

Mulholland (1982) considered the distortion to accelerogram records used by Katayama, as caused by the particular form of accelerograph in widespread use in Japan, and recommended that predicted spectral accelerations be increased by a factor ranging from 1.66 at T = 0.1 second to 1.0 at T = 0.8 seconds.

McVerry (1986) argued that New Zealand records show a greater rate of attenuation with distance than is apparent in Japan. He presented modified, continuously defined distance attenuation factors (f_r). These were used in the analyses on which the current loading code is based.

Although there may be other reasons for making these modifications to Katayama's original model, we believe the justification is not apparent on the basis of presented or available data. The data set for New Zealand earthquakes is relatively small, and records are available for very few of the many combinations of magnitude, distance and ground conditions presented by Katayama. This applies particularly for large earthquakes and for very short epicentral distances - i.e. two extremes likely to define the trend of the attenuation relationship. There is no N.Z. earthquake for which a series of response spectra can be calculated over a wide range of distances.

To circumvent this problem, McVerry considered all recorded acceleration data, including scratch plate records measuring peak ground acceleration, for the 1968 Inangahua earthquake. When plotted on a log (acceleration) v log (distance) scale, a number of the points fell close to a straight line with slope equal to -1.1. This slope was adopted for the peak spectral acceleration, assumed to occur at $T = 0.25$s, and other values were adjusted accordingly using predicted but smoothed values at $r = 35$ km. However, the following points should be considered:

- The magnitude of the Inangahua earthquake has recently been revised from $M = 7.1$ to $M = 6.7$ so that it lies at the extreme end of Katayama magnitude range 6.1 - 6.7. Predictions from this range or from the range above may not match observed values particularly well.

- Peak predicted spectral accelerations occur at $T = 0.2$s rather than $T = 0.25$s as assumed by McVerry, at most distances.

- Isoseismals for the Inangahua earthquake (see Figure 7.4) are highly elliptical indicating a strong preferred direction of propagation, or conversely far more rapid attenuation in the transverse direction than would be expected for most New Zealand earthquakes. Many recording stations for the earthquake were, however, in transverse directions.

- Although many peak acceleration data fell close to the line constructed by McVerry, a number of data were extremely poorly fitted by the line, and indicated far higher accelerations than the line would predict.

- No allowance was made for possible variation in ground conditions at different recording sites.

In Figure 6.1 the peak spectral accelerations predicted by the original Katayama model, without modification, are shown for Ground Conditions 1 and 4 together with recorded peak ground acceleration data. Ranges of ±1 standard deviation for the predicted values are shown for $\sigma_{10} = 0.30$, the mean of values suggested by McVerry. The best fit trend proposed by McVerry is also shown; although it fits some data well, other data are very poorly matched. It is evident that almost all Inangahua data lie within the Katayama model predicted range, despite the fact that peak spectral accelerations are being compared to peak ground accelerations. It is common to assume the peak ground acceleration to be about 0.35 to 0.45 x peak spectral acceleration. If this correction were applied to the predicted spectral accelerations in Figure 6.1 then almost all observed data would plot above the mean predicted curve. If anything, the Katayama model therefore underpredicts spectral accelerations.

The basis for these modifications was examined in the course of this study. On the basis of the discussion above, the validity of the downward correction to Katayama's original attenuation model with distance must be seriously questioned. The attenuation data for the Inangahua earthquake are adequately predicted by the unmodified Katayama model and much of the scatter may be due to propagation directivity effects. Until further analysis of New Zealand strong motion accelerograms is available for a wide range of magnitude-distance-ground condition values, and particularly for one or more large single events, there is little justification for modifying the original model proposed by Katayama. However the combination of New Zealand and Japanese records discussed by Berrill (1985b) in order to assess better values of the log normal standard deviation σ_{10} is useful and appropriate.

Although upward modification to spectral accelerations at periods less than 0.8 second as recommended by Mulholland may be justified on the grounds of maintaining consistency among predictions using different types of accelerograph, there is enough overall doubt about reasons for scatter in the attenuation model to suggest that this modification could also be deferred until strong supporting data are available.

In this study, the original attenuation model of Katayama (1982) was therefore used without modification. Further justification of this is that the spectral values calculated from this method will be changed significantly and directly by the analysis of the effects of deep alluvium beneath Christchurch, described in the next chapter. The precise shape of the 'bedrock' spectrum is therefore less important than it would be at other locations. Probabilistic enhancement was included corresponding approximately to the log normal standard deviation values suggested by Berrill (1985) and McVerry (1986). This causes predicted spectral accelerations to increase by a factor close to 2 for probabilities calculated using the mean attenuation model, and a seismicity model of the form described in Chapter 4.

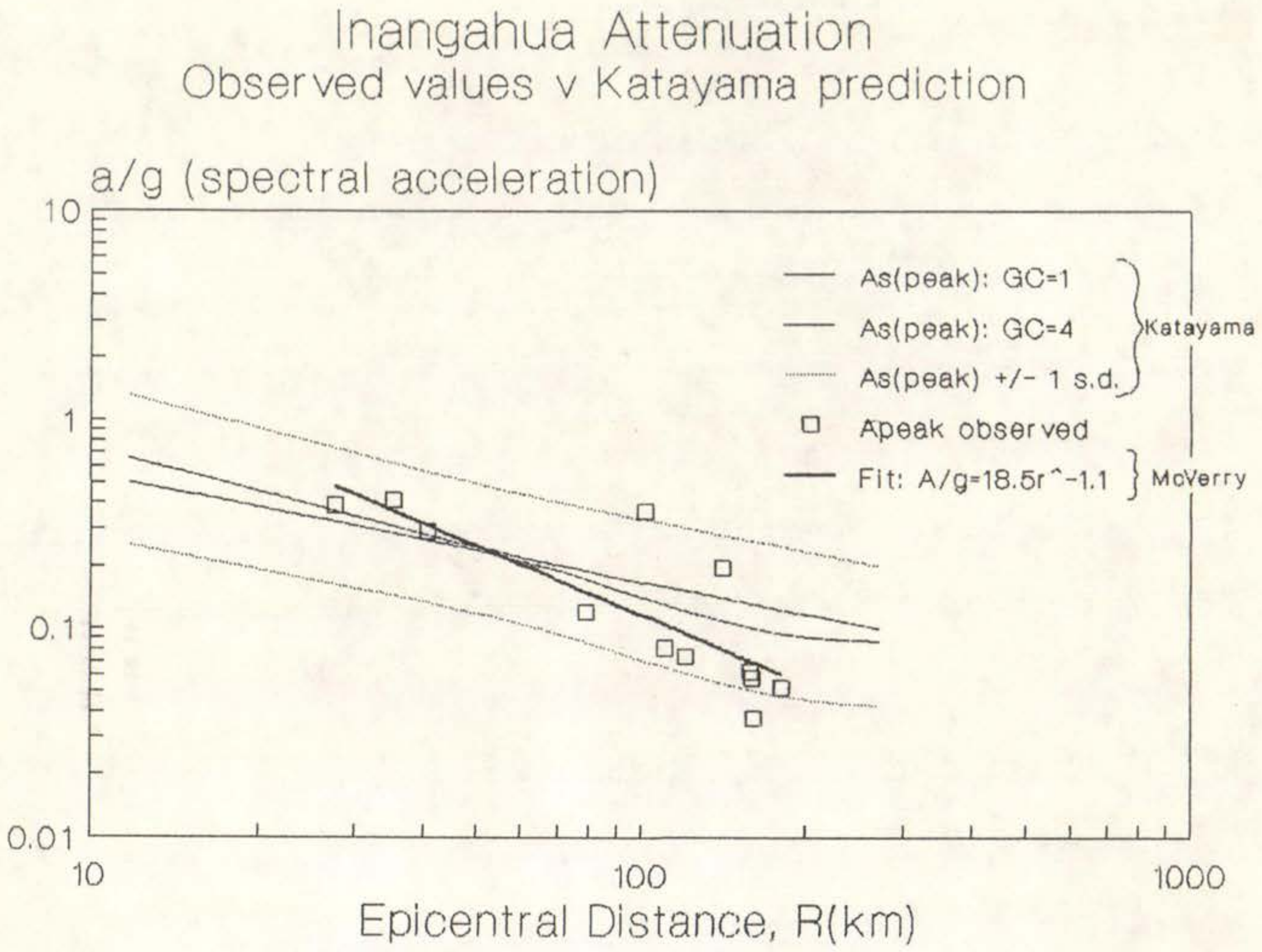


FIGURE 6.1

Peak Spectral Accelerations for Inangahua Earthquake Showing Katayama Predictions and Observed Values

6.3 BEDROCK RESPONSE SPECTRA AT CHRISTCHURCH

Matuschka et al (1985), in deriving general response spectra for use in constructing an envelope for design, used the modified version of the Katayama model proposed by McVerry (1986) for Ground Condition type 3, which corresponds most closely to 'average' conditions throughout New Zealand. For long period response, spectral accelerations were increased to those corresponding to constant spectral velocity, recognising that these values are often underestimated.

However in this study effects of variations in ground conditions were taken into account directly, as discussed in the next chapter. The original Katayama model without modification was used to construct bedrock spectra, used as input into the deep alluvium propagation model.

To calculate appropriate bedrock acceleration spectra, the following steps were employed:

- Katayama magnitude and distance factors f_m and f_r were plotted at the mean of data for each magnitude/distance range as stated by Katayama. Curves were extrapolated to allow prediction of effects from very large earthquakes. At present there is no way of testing the validity of this technique and further verification is required.
- Fault or seismic region earthquake magnitudes and distances from Christchurch discussed in Chapter 4 were reviewed and acceleration response spectra constructed using equation 6.1 and the values of f_m, f_r discussed above.
- Spectra were calculated using Ground Condition 1, corresponding most closely to rock.

It has been recognised that a relatively uniform spectral shape can be assumed throughout New Zealand, independent of location or return period. Given this, and that as input into a deep alluvium propagation model the precise spectral shape is relatively unimportant, a limited selection of spectra corresponding to particular earthquakes were considered in order to determine this shape. Analysis of Figure 4.5 in Chapter 4 shows that typical earthquakes affecting Christchurch may be represented by three examples:

1.	$M = 7.0$, $r = 25$ km	e.g. Earthquakes in regions: Porters Pass, Ashley section; Canterbury Plains seismic; Pegasus seismic.
2.	$M = 7.3$, $r = 50$ km	e.g. Earthquakes in regions: Porters Pass Tectonic; Banks Peninsula seismic.
3.	$M = 8.1$, $r = 150$ km	e.g. Earthquakes on Alpine Fault.

The calculated response spectra at bedrock for Christchurch for each of these three earthquakes are shown in Figure 6.2. The three spectral shapes are very similar, confirming the postulated constant spectral shape discussed above. This shape is used as input to the deep alluvium propagation model in Chapter 7. Adjustments to the magnitudes of individual spectra for specific exceedance probabilities are discussed below in Section 6.4.

To complete this deterministic approach, the extreme bedrock response spectrum at Christchurch is calculated and also shown in Figure 6.2. This is the spectrum resulting from maximum magnitude earthquakes defined by an "upper bound" curve shown in Figure 4.5. Example earthquakes are $M = 7.2$ at $r = 25$ km, $M = 7.6$ at $r = 45$ km, and $M = 8.1$ at $r = 140$ km. The response spectrum is calculated using the magnitude/attenuation method described above. However all ordinates are increased to be one standard deviation above the mean predicted value. The appropriate standard deviation for the Katayama model is used. This causes spectral accelerations for this deterministic prediction to increase by a factor of approximately 1.8. The extreme predicted peak spectral acceleration at bedrock beneath Christchurch is about 0.75 g, occurring at period about $T = 0.2$ seconds.

6.4 BEDROCK RESPONSE SPECTRA: PROBABILITIES AND RECURRENCE

Probabilities of different spectral response accelerations occurring at Christchurch were determined using a similar analysis to that described in Chapter 5 for intensities. The same set of subregions and nodes was used, together with the seismicity model described in Chapter 4. Only peak spectral accelerations at $T = 0.2$s were considered since other values scale directly from the peak value as a result of the assumption of uniform spectral shape. Smoothed, continuous functions were fitted to the Katayama magnitude and distance factors, described approximately by

$$f_m\ (T = 0.2) = 1.5\ (9\text{-}M)^{-1.5} \qquad \text{for } M \leq 8.3 \qquad (6.2)$$

$$\text{and} \quad f_r\ (T = 0.2) = 27\ r^{-0.6} \qquad \text{for } r \leq 300 \text{ km} \qquad (6.3)$$

These models provided very good fits to the Katayama factors for $M \leq 8.1$ and $r \leq 150$ km, and reasonable fits within the limits described in equations 6.2 and 6.3, which constrain most of the earthquakes likely to significantly affect probability calculations for Christchurch.

For spectral accelerations $a_s/g = 0.15, 0.3, 0.45, 0.75$ the magnitude M_j at node j required to produce a_s/g in Christchurch was calculated, then the annual frequency N_j of occurrence of M_j determined from the seismicity model. As described in Chapter 5 the total number N_1 of earthquakes annually causing

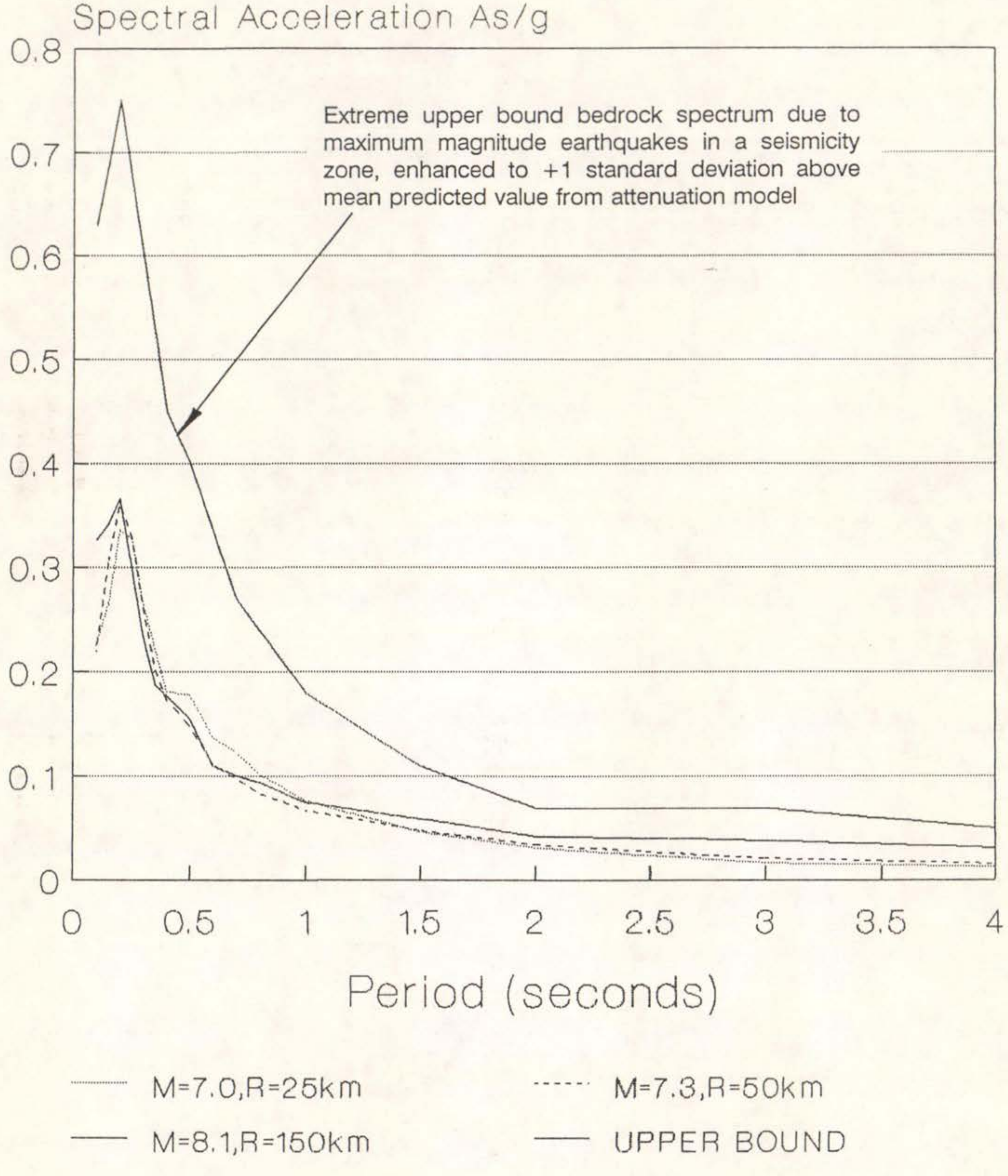


FIGURE 6.2

Response Spectra at Bedrock for Christchurch

Calculated Spectra From Three Upper Bound Earthquakes

CHRISTCHURCH SEISMIC HAZARD STUDY - RESPONSE SPECTRA PROBABILITIES

NODE	ZONE	AREA	Asp/g	Asp/g	Asp/g	Asp/g	Asp/g	Mmax	b	a4	N for As/g values of; (includes probabilistic enhancement)					DISTANCE
			0.1	C.2	0.3	0.4	0.5				0.15	0.3	0.45	0.6	0.75	rj
1	CPS	0.74	3.62	5.61	6.41	6.87	7.16	7.5	0.70	0.050	0.06801	0.00262	0.00062	0.00023	0.00010	20
4		1.13	4.82	6.37	6.99	7.34	7.57	7.5	0.70	0.050	0.01494	0.00105	0.00026	0.00006		37.5
13		1.02	5.59	6.85	7.36	7.65	7.83	7.5	0.70	0.050	0.00376	0.00033	0.00005			62.5
21		0.93	6.02	7.12	7.57	7.82	7.98	7.5	0.70	0.050	0.00162	0.00014				87.5
31		0.25	6.31	7.30	7.70	7.93	8.00	7.5	0.70	0.050	0.00026	0.00002				112.5
2	PGS	0.30	3.62	5.61	6.41	6.87	7.16	7.5	0.70	0.040	0.02207	0.00085	0.00020	0.00008	0.00003	20
6		0.99	4.82	6.37	6.99	7.34	7.57	7.5	0.70	0.040	0.01049	0.00074	0.00018	0.00004		37.5
11		0.86	5.59	6.85	7.36	7.65	7.83	7.5	0.70	0.040	0.00254	0.00023	0.00003			62.5
17		0.76	6.02	7.12	7.57	7.82	7.98	7.5	0.70	0.040	0.00107	0.00009				87.5
27		0.90	6.31	7.30	7.70	7.93	8.00	7.5	0.70	0.040	0.00075	0.00005				112.5
X		0.22	6.51	7.43	7.80	8.00	8.00	7.5	0.70	0.040	0.00012	0.00000				137.5
3	BPS	0.96	3.62	5.61	6.41	6.87	7.16	7.5	0.70	0.030	0.05280	0.00203	0.00048	0.00018	0.00007	20
7		2.87	4.82	6.37	6.99	7.34	7.57	7.5	0.70	0.030	0.02277	0.00160	0.00039	0.00009		37.5
12		4.96	5.59	6.85	7.36	7.65	7.83	7.5	0.70	0.030	0.01092	0.00097	0.00013			62.5
18		1.41	6.02	7.12	7.57	7.82	7.98	7.5	0.70	0.030	0.00148	0.00013				87.5
28		1.27	6.31	7.30	7.70	7.93	8.00	7.5	0.70	0.030	0.00079	0.00005				112.5
35		1.32	6.51	7.43	7.80	8.00	8.00	7.5	0.70	0.030	0.00055	0.00002				137.5
19		0.40	6.02	7.12	7.57	7.82	7.98	7.5	0.70	0.030	0.00042	0.00004				87.5
5	PPT	0.82	4.82	6.37	6.99	7.34	7.57	7.5	0.74	0.170	0.03419	0.00211	0.00050	0.00011		37.5
8		1.37	5.59	6.85	7.36	7.65	7.83	7.5	0.74	0.170	0.01486	0.00120	0.00016			62.5
14		1.02	6.02	7.12	7.57	7.82	7.98	7.5	0.74	0.170	0.00511	0.00040				87.5
22		1.08	6.31	7.30	7.70	7.93	8.00	7.5	0.74	0.170	0.00313	0.00019				112.5
32		0.80	6.51	7.43	7.80	8.00	8.00	7.5	0.74	0.170	0.00152	0.00004				137.5
9	CBNW	0.61	5.59	6.85	7.36	7.65	7.83	7.5	0.60	0.150	0.00939	0.00105	0.00015			62.5
15		1.66	6.02	7.12	7.57	7.82	7.98	7.5	0.60	0.150	0.01328	0.00135				87.5
23		0.78	6.31	7.30	7.70	7.93	8.00	7.5	0.60	0.150	0.00389	0.00029				112.5
10	CBNE	0.50	5.59	6.85	7.36	7.65	7.83	8.0	0.60	0.150	0.00803	0.00116	0.00042	0.00019	0.00008	62.5
16		1.68	6.02	7.12	7.57	7.82	7.98	8.0	0.60	0.150	0.01442	0.00236	0.00082	0.00029	0.00003	87.5
26		0.43	6.31	7.30	7.70	7.93	8.00	8.0	0.60	0.150	0.00238	0.00041	0.00013	0.00003		112.5
24	HFS	2.50	6.31	7.30	7.70	7.93	8.00	7.8	0.60	0.310	0.02801	0.00402	0.00057			112.5
25	GN	1.46	6.31	7.30	7.70	7.93	8.08	8.5	1.10	0.600	0.00254	0.00019	0.00006	0.00003	0.00002	112.5
34		4.07	6.51	7.43	7.80	8.01	8.15	8.5	1.10	0.600	0.00417	0.00038	0.00013	0.00007	0.00004	137.5
40		8.91	6.74	7.58	7.91	8.10	8.23	8.5	1.10	0.600	0.00509	0.00056	0.00020	0.00010	0.00006	175
45		7.70	6.96	7.71	8.02	8.19	8.30	8.5	1.10	0.600	0.00252	0.00033	0.00012	0.00006	0.00003	225
49		4.46	7.12	7.81	8.09	8.25	8.36	8.5	1.10	0.600	0.00097	0.00014	0.00005	0.00003	0.00001	275
20	CBSE	1.97	6.02	7.12	7.57	7.82	7.98	8.0	1.10	0.030	0.00035	0.00002	0.00000	0.00000	0.00000	87.5
29		2.55	6.31	7.30	7.70	7.93	8.00	8.0	1.10	0.030	0.00022	0.00001	0.00000	0.00000		112.5
36		2.77	6.51	7.43	7.80	8.00	8.00	8.0	1.10	0.030	0.00014	0.00001	0.00000			137.5
41		2.21	6.66	7.53	7.88	8.00	8.00	8.0	1.10	0.030	0.00008	0.00001	0.00000			160
30	CBSW	0.90	6.31	7.30	7.70	7.93	8.00	8.0	1.00	0.110	0.00048	0.00004	0.00001	0.00000		112.5
37		1.88	6.51	7.43	7.80	8.00	8.00	8.0	1.00	0.110	0.00061	0.00006	0.00001			137.5
42		4.50	6.74	7.58	7.91	8.00	8.00	8.0	1.00	0.110	0.00085	0.00008	0.00001			175
46		3.75	6.96	7.71	8.02	8.00	8.00	8.0	1.00	0.110	0.00041	0.00004				225
50		3.55	7.12	7.81	8.09	8.00	8.00	8.0	1.00	0.110	0.00026	0.00002				275
51		3.35	7.24	7.89	8.15	8.00	8.00	8.0	1.00	0.110	0.00018	0.00001				325
33	H	2.71	6.51	7.43	7.80	8.01	8.15	8.5	0.05	0.003	0.00104	0.00053	0.00034	0.00023	0.00017	137.5
38		9.88	6.74	7.58	7.91	8.10	8.23	8.5	0.05	0.003	0.00330	0.00165	0.00103	0.00069	0.00047	175
43		6.01	6.96	7.71	8.02	8.19	8.30	8.5	0.05	0.003	0.00174	0.00085	0.00051	0.00033	0.00021	225
47		3.85	7.12	7.81	8.09	8.25	8.36	8.5	0.05	0.003	0.00099	0.00047	0.00027	0.00017	0.00010	275
52		1.92	7.24	7.89	8.15	8.30	8.40	8.5	0.05	0.003	0.00045	0.00021	0.00012	0.00007	0.00003	325
39	F	2.40	6.74	7.58	7.91	8.10	8.23	8.5	1.13	0.700	0.00133	0.00014	0.00005	0.00002	0.00001	175
44		7.28	6.96	7.71	8.02	8.19	8.30	8.5	1.13	0.700	0.00227	0.00028	0.00010	0.00005	0.00003	225
48		10.00	7.12	7.81	8.09	8.25	8.36	8.5	1.13	0.700	0.00205	0.00029	0.00011	0.00005	0.00003	275
53		9.06	7.24	7.89	8.15	8.30	8.40	8.5	1.13	0.700	0.00134	0.00020	0.00008	0.00004	0.00002	325
54		2.71	7.34	7.95	8.20	8.34	8.43	8.5	1.13	0.700	0.00031	0.00005	0.00002	0.00001	0.00000	375
55	D	7.50	7.24	7.89	8.15	8.30	8.40	8.5	1.13	0.850	0.00135	0.00020	0.00008	0.00004	0.00002	325
56	M	3.61	6.77	7.59	7.93	8.00	8.00	8.0	1.10	0.080	0.00025	0.00002	0.00000			180
57		6.90	6.96	7.71	8.00	8.00	8.00	8.0	1.10	0.080	0.00029	0.00002				225
58		7.50	7.12	7.81	8.00	8.00	8.00	8.0	1.10	0.080	0.00020	0.00001				275
59		7.40	7.24	7.89	8.00	8.00	8.00	8.0	1.10	0.080	0.00014	0.00001				325
										TOTAL	0.389490	0.032340	0.008404	0.003265	0.001540	

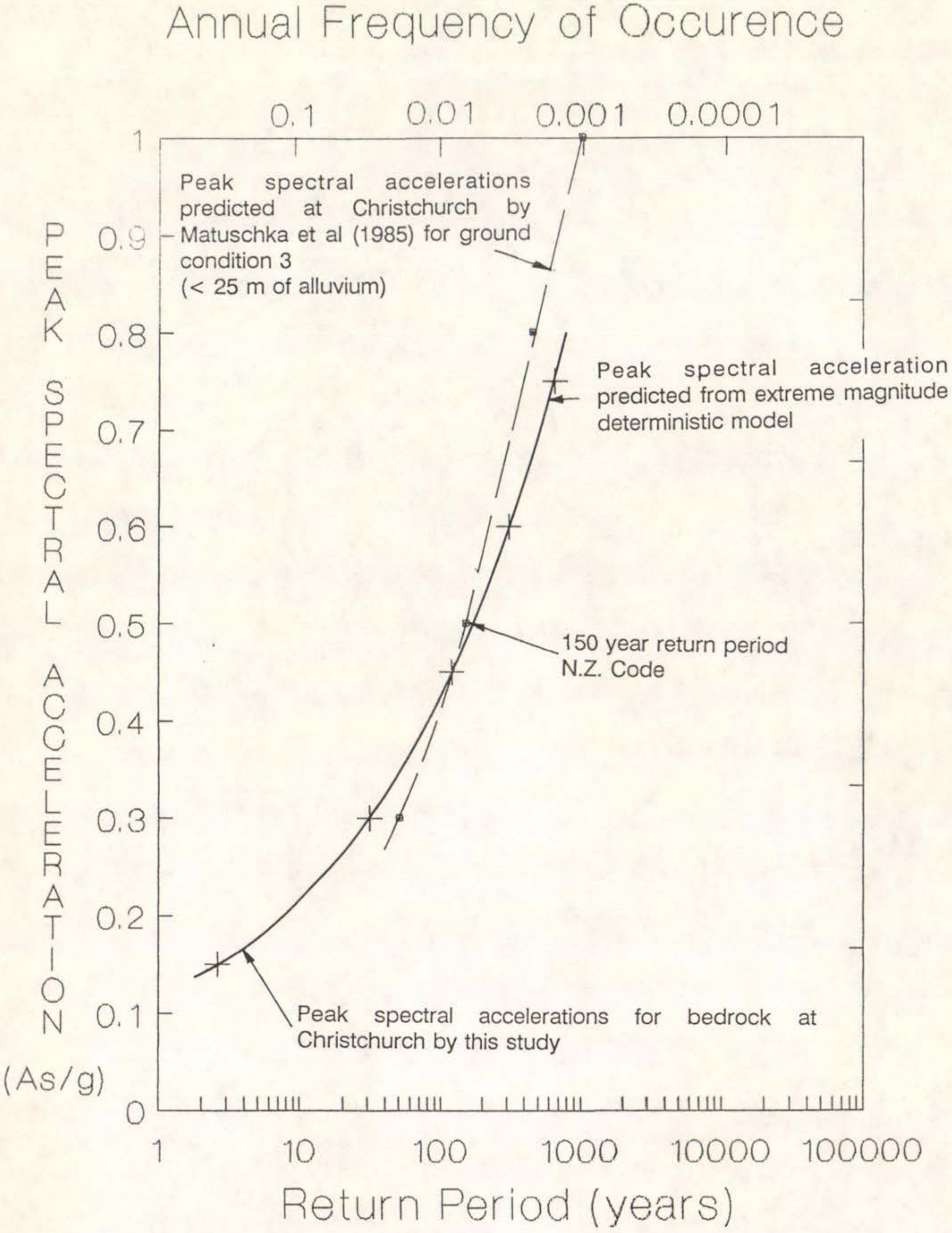
TABLE 6.1 Probabilities of Occurrence of Different Spectral Accelerations Detailed Results

TABLE 6.2 Probabilities of Occurrence of Different Spectral Accelerations at Christchurch and Relative Contributions from each Seismicity Region

Percentage Contribution to Total Probability from Each Seismicity Zone

SEISMICITY	A_s/g AT CHRISTCHURCH				
REGION	0.15	0.3	0.45	0.6	0.75
CPS	22.8	12.8	11.0	8.9	6.2
PGS	9.5	6.0	4.9	3.5	2.0
BPS	23.0	14.9	12.0	8.3	4.8
PPT	15.1	12.2	7.8	3.4	
CBnw	6.8	8.3	1.8		
CBne	6.4	12.2	16.3	15.2	6.7
HFs	7.2	12.4	6.8		
Gn	3.9	5.0	6.9	8.9	10.7
CBse	0.2	0.2	0.1	0.1	
CBsw	0.7	0.8	0.4	0.1	
H	1.9	11.4	26.9	45.2	62.9
F	1.9	3.0	4.2	5.3	5.7
D	0.4	0.6	0.9	1.1	1.0
M	0.2	0.2			
Totals	100.0	100.0	100.0	100.0	100.0

Annual Exceedance	0.389	0.0323	0.00840	0.00327	0.00154
Return Period (yrs)	2.6	31	120	306	650
Probability of Exceedance in:					
50 yrs	100	81	34	15	7
150 yrs	100	99	72	39	21
450 yrs	100	100	98	77	50
1000 yrs	100	100	100	96	79



Peak spectral accelerations predicted at Christchurch by Matuschka et al (1985) for ground condition 3 (< 25 m of alluvium)

Peak spectral acceleration predicted from extreme magnitude deterministic model

150 year return period N.Z. Code

Peak spectral accelerations for bedrock at Christchurch by this study

FIGURE 6.3 Annual Occurrence Frequencies and Return Periods for Bedrock Peak Spectral Acceleration spectral acceleration >a_s/g was obtained by summing the frequencies for each node. Detailed calculation results are presented in Table 6.1 and exceedance probabilities are summarised in Table 6.2, including relative contributions from each seismicity region. Effects of 'probabilistic enhancement' as discussed in section 6.2 are included in the above calculations. The enhancement factor is calculated, as discussed earlier, and described by Berrill, 1985. A mean b value, b = 0.6 in critical zones gives an enhancement factor, Bz = 1.6 at the T = 0.2 sec period.

The pattern shown for spectral acceleration probabilities in Table 6.2 is somewhat different from that for intensities in Table 5.2. For low spectral accelerations at a_s/g = 0.15, exceedance probabilities are contributed significantly by the local regions CPS (Canterbury Plains) and BPS (Banks Peninsula), slightly less by PPT (Porters Pass) and lesser again by PGS (Pegasus Bay), CBnw, CBne (north-east and north-west Canterbury), and HFs (Hope South).

For medium spectral accelerations, a_s = 0.45 g, the most significant contribution is from the Alpine Fault, with reasonable contributions from CPS, BPS and CBne regions, and lesser contributions from PPT, HFs and Gn.

For large spectral accelerations, by far the most important contribution to the exceedance probabilities comes from the Alpine Fault, with much smaller contributions from CPS, BPS, CBne, Gn and F (Nelson). This pattern demonstrates that low spectral acceleration probabilities are dominated by smaller earthquakes in the local regions, and the high spectral acceleration probabilities by large, more distant earthquakes, particularly on the Alpine Fault.

Return periods for different peak spectral accelerations are shown graphically in Figure 6.3.

6.5 SUMMARY

The attenuation model described by Katayama (1982) has been used without modification in this study, as a consequence of examination of the factors which have led to previously proposed alterations. This model has been combined with the seismicity model from Chapter 4 to predict response spectra for bedrock at Christchurch with various probabilities of exceedance. Consideration of maximum earthquakes has led to application of a reasonable 'upper bound' response spectrum.

Most of the potential hazard corresponding to high spectral accelerations at Christchurch is derived from a major earthquake on the Alpine Fault, but some hazard is contributed by large distant earthquakes in the North Canterbury - Marlborough area, and by very infrequent but close earthquakes in the Canterbury Plains and Banks Peninsula seismic regions.

Response spectra derived in this chapter are used as input to derive modified spectra specific to Christchurch geologic conditions in Chapter 7.

CHAPTER 7: PREDICTED INFLUENCE OF CHRISTCHURCH GEOLOGY

7.1 INTRODUCTION

Damage to structures at a given epicentral distance during a particular earthquake has been observed to vary considerably with ground conditions at each location (e.g. Tinsley & Fumal, 1985). Ground shaking is usually greatest on geologically recent, soft or loose sedimentary deposits. Notable examples include the Mexico City earthquake of 1985, and the Loma Prieta (San Francisco) earthquake of 1989. The deep quaternary sediment deposits beneath Christchurch are a prime example of a soil profile with very high potential for magnification of earthquake effects.

The Mexico City earthquake, magnitude M = 8.1, caused about 4,000 deaths, or 1 in 2000 of the city's population, and destroyed about 1,000 buildings (Esteva, 1988). Although the epicentral distance to the city was 400 km, deep soft lake sediments beneath the central area of the city caused amplification of long period motions. The peak ground accelerations measured in this area were 0.19 g, greater than those recorded near the epicentre. These ground accelerations, while only moderate, had a 2-second period, resulting in very large actual displacements. At least 20 cycles of this swaying motion were recorded and resulted in very strong resonant response for structures 6 to 15 stories high.

Although the alluvial soils beneath most of Christchurch are generally stiffer than the soft clays beneath the critically affected areas of Mexico City, the correlation remains relevant. The soils beneath Christchurch are considerably deeper than those beneath Mexico City, enhancing the potential for amplification of incident seismic waves due to impedance mismatches and constructive interference at soil strata boundaries. In some areas of Christchurch geotechnical investigations have revealed very soft peat or organic silts at depths of up to 25 m deep. In addition the distances from Christchurch to major active faults are less than one-third the 400 km epicentral distance for the Mexico City Earthquake. In the central South Island, the effects of an earthquake with magnitude M = 8.1 at 400 km at equivalent to M = 7.5 at 100 - 150 km, or M = 7.0 at 50 - 100 km. Most fault zones described in Chapter 4 are easily capable of generating earthquakes which would exceed these conditions.

The current New Zealand Loadings Code, NZS 4203: 1984 and its draft revision provide separate response spectra for structural design on 'flexible subsoil' sites as shown in Figure 7.1. The general effect is to extend the natural period range within which any particular seismic design force applies.

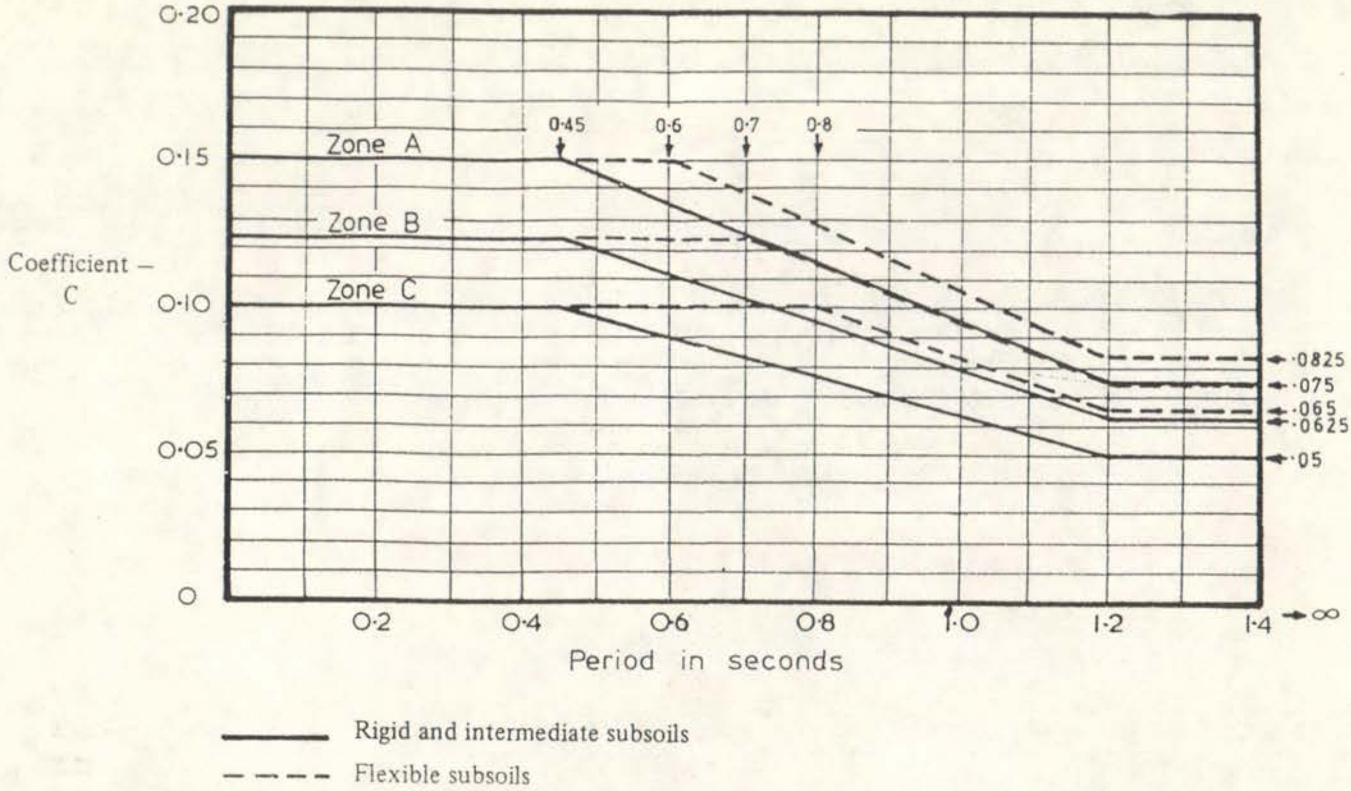
The code states that *"a building shall be determined to be on 'flexible subsoil' if there are uncemented soils exceeding one of the following depths...15 m of cohesionless sands or gravels..."*. In Christchurch, where sedimentary deposits generally exceed 500 - 1000 m depth, this is satisfied at all sites away from the hills. The resulting design requirement (Zone B) is that structures be designed for the peak seismic force at all natural periods up to 0.7 seconds, decreasing to the minimum seismic force level for natural periods above 1.2 seconds (Figure 7.1). These periods correspond to typical building heights three to five stories and five to seven stories respectively. However the code also states that:

> *"For long period structures on very deep uncemented soils this provision might not be adequate and special studies should be made."*

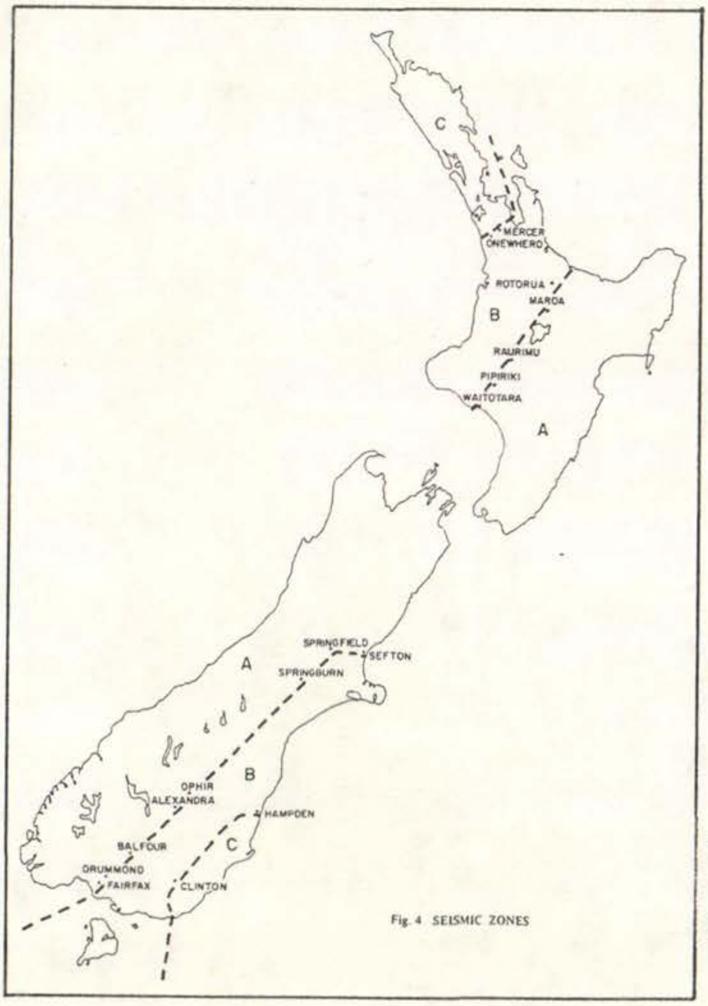
Although 'long period' is not defined, 0.7 seconds (the period above which the maximum seismic force need not be applied) is logically an appropriate definition. It follows that any structure in Christchurch with natural period above 0.7 seconds (e.g. buildings higher than three to five stories) or which could degrade to enter this range during an earthquake, should be subject to a special study. This has been carried out for one site in Christchurch by Soils & Foundations Ltd (1988) and Davis & Berrill (1988). Design response spectra differing considerably from those determined from the New Zealand Loadings Code were computed.

The analysis described in this chapter describes the type of study which is required and presents general results for the entire Christchurch area on a grid zoning basis. These results might be used in lieu of specific analyses. However the relatively coarse scale necessarily employed here, and the inevitable smoothing of data, means that great care should be taken when attempting to interpolate results for specific sites.

In section 2 evidence for amplification of felt intensities at Christchurch during previous earthquakes is presented. The geologic profile beneath the city is developed in section 3. Section 4 presents the deep soil response model and predicted site effects are described in section 5.



(a) Response Spectra - Basic Seismic Coefficient, Fig. 3, NZS 4203: 1984 - Code of Practice for General Structural Design and Design Loadings for Buildings



(b) Seismic Zones - Fig. 4,NZS 4203: 1984

FIGURE 7.1

Code Response Spectra

7.2 EVIDENCE OF SITE-SPECIFIC INTENSITY INCREASES AT CHRISTCHURCH

The Canterbury Plains comprise relatively loose, cohesionless alluvial soil deposits which are at least 500 - 1000 m deep in many places, including beneath Christchurch. During a number of historical earthquakes, higher intensities have been recorded in Christchurch than at other locations equidistant from the epicentre but with minimal soil cover over bedrock. Three examples are shown in Figures 7.2, 7.3 and 7.4.

Figure 7.2 shows isoseismals for an earthquake of magnitude M = 6.3 on 25 December 1922, in North Canterbury about 60 km from Christchurch. Isoseismals are those constructed by Brown (pers. comm. 1990). Although the Seismological Observatory prepares standardised isoseismal plots for most significant New Zealand earthquakes, none is available for this earthquake. The pattern is somewhat confused near Christchurch, but shows that much higher intensities (up to MM VIII) were felt on the Canterbury Plains near the city than were recorded further inland (MM V to MM VI at similar epicentral distances), even though Christchurch is perpendicular to the apparent major axis of energy propagation.

Isoseismals produced by the Seismological Observatory for a lower crustal earthquake of magnitude M - 6.4 on February 21 1960, with epicentre calculated to be at the head of the Awatere Valley about 140 km from Christchurch, are shown in Figure 7.3. The greater focal depth may explain the displacement of the macrocentre (about which isoseismals are concentric) away from the calculated epicentre, although the direction is the opposite of that usually observed for deep New Zealand earthquakes and discussed by Smith (1978). On the northern Canterbury Plains near Christchurch intensities are up to one intensity unit higher than those inland.

Relevant, but less direct, evidence is available from the May 23 1968 Inangahua earthquake of magnitude M = 6.7 (revised). Isoseismals published by the Seismological Observatory have been widely reproduced and used elsewhere, and are shown in Figure 7.4. They show a very marked eccentricity which has been described as characteristic of earthquakes in this area of New Zealand (Smith, 1976). However closer examination of intensities reported for specific locations in Figure 7.4 suggests that higher intensities may have been felt on alluvial soils of coastal plains, including around Christchurch, than in central mountainous areas. More of these reporting locations where soft ground is likely are oriented from the epicentre in directions close to the major isoseismal axis (as constructed) than to the minor axis. Some of the apparent eccentricity may therefore be due to variability in site effects, rather than being solely due to actual variation in directivity of energy propagation.

Since construction of isoseismals does not generally allow site-specific effects to be differentiated on a detailed basis, isoseismal plots for any earthquake represent an averaging of reported intensities over soft and firm ground. Other effects, such as geometric focusing, will also be important, so that considerable spatial scatter in reported intensities is inevitable. Populated areas are commonly concentrated on flat ground, which in the South Island is often alluvial and consequently isoseismals are likely to reflect the larger number of records from these areas. It is possible that the characteristic elliptical shapes of isoseismals observed in New Zealand for historical earthquakes and discussed, for example, by Smith (1976) are partially a result of these site-specific effects. It would be a valuable contribution to earthquake hazard analysis in New Zealand to investigate this hypothesis in detail for a number of large historical earthquakes where localised recorded intensities can be compared to probable geological conditions.

A more direct assessment of the effect on felt intensities of the deep alluvium beneath Christchurch is made in Figure 7.5. Intensities felt in Christchurch, and at unspecified locations on Banks Peninsula, are compared for fifteen historical earthquakes where published data are available. For three of these, the mean intensity reported in Christchurch was up to one MM unit lower than that felt on Banks Peninsula. For six earthquakes, the mean intensities were the same, while for six the mean intensity in Banks Peninsula was up to three MM units lower than that reported for Christchurch.

Scatter in the comparison is expected due to the different reporting locations likely for different earthquakes, and the variable epicentral distances. However it is also likely that at many locations on Banks Peninsula some intensity amplification could occur due to geometric focusing of incident waves, or due to soft soil deposits particularly in valleys, where population centres are found. Lower intensities may not have been felt or reported for true bedrock sites in other areas. In addition to the fifteen earthquakes analysed above there are a further four earthquakes for which intensities above MM II are reported for Christchurch, but no intensities recorded for Banks Peninsula. This is probably equivalent to at least one MM unit increase in Christchurch.

Although the correlation initially appears neither consistent nor strong, when these other factors are considered there appears to be a definite trend that intensities felt in Christchurch are, on average, higher than those experienced during the same earthquake on adjacent bedrock sites on Banks Peninsula. This is consistent with the findings of Dibble et al, 1980 who concluded from evaluation of available intensity data in the Christchurch area that intensities in Christchurch were, on average, 0.9 to 1.6 MM units higher than at Lyttelton and Akaroa respectively. Further research is required if this correlation is to be more fully substantiated. For the purposes of this study it appears reasonable to assume that intensities in Christchurch are, on average, 0 - 2 MM units higher than those on Banks Peninsula. In some places intensities may be more than 2 MM units higher than elsewhere.

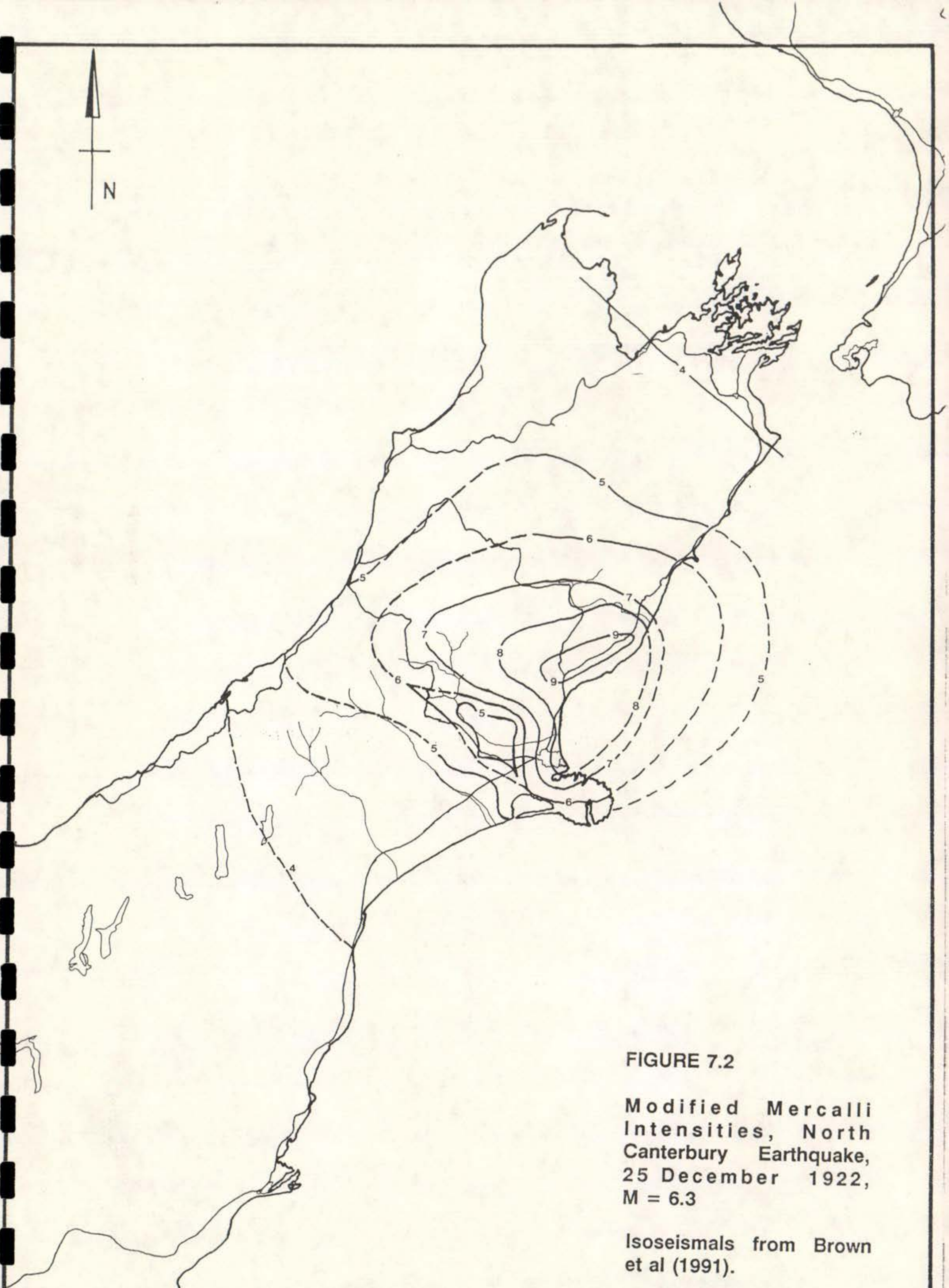


FIGURE 7.2

Modified Mercalli Intensities, North Canterbury Earthquake, 25 December 1922, M = 6.3

Isoseismals from Brown et al (1991).

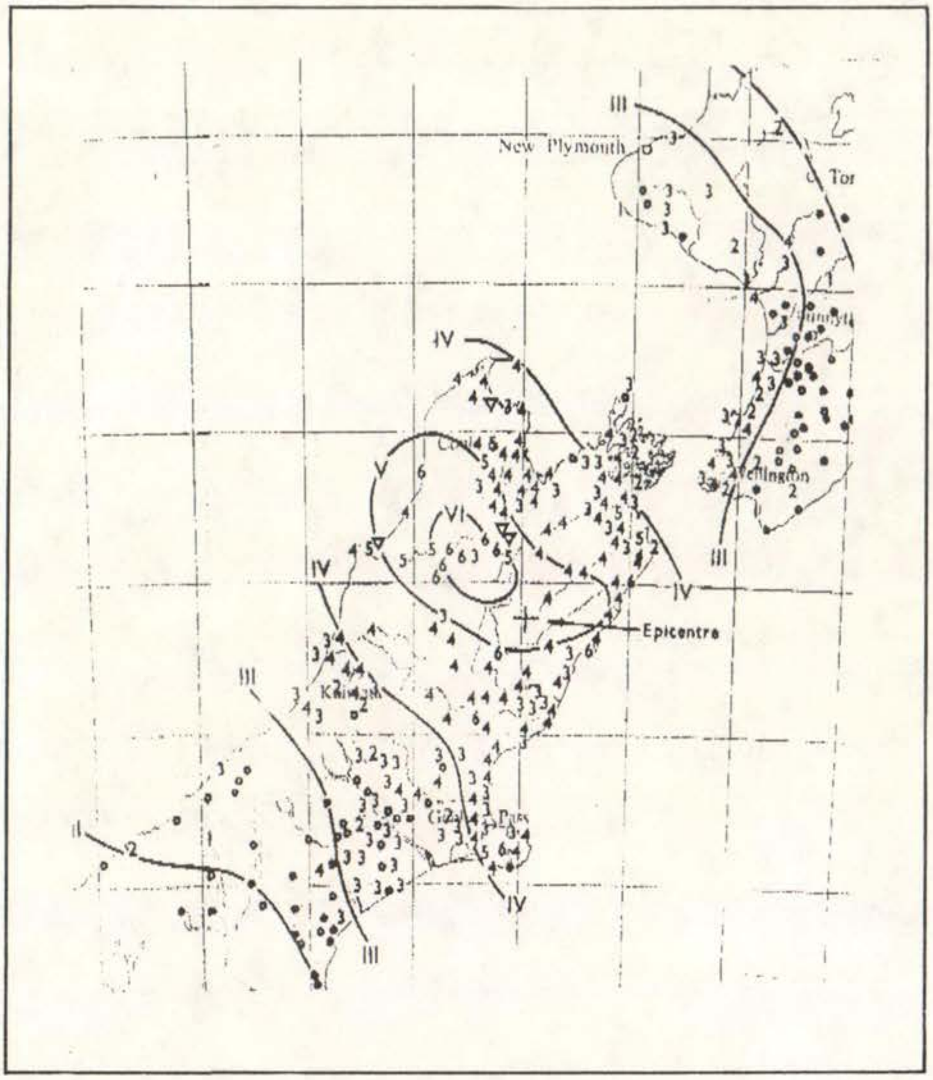


FIGURE 7.3

Modified Mercalli Intensities, Awatere Valley Earthquake, 21 February 1960, M = 6.4

Isoseismals from Seismological Observatory

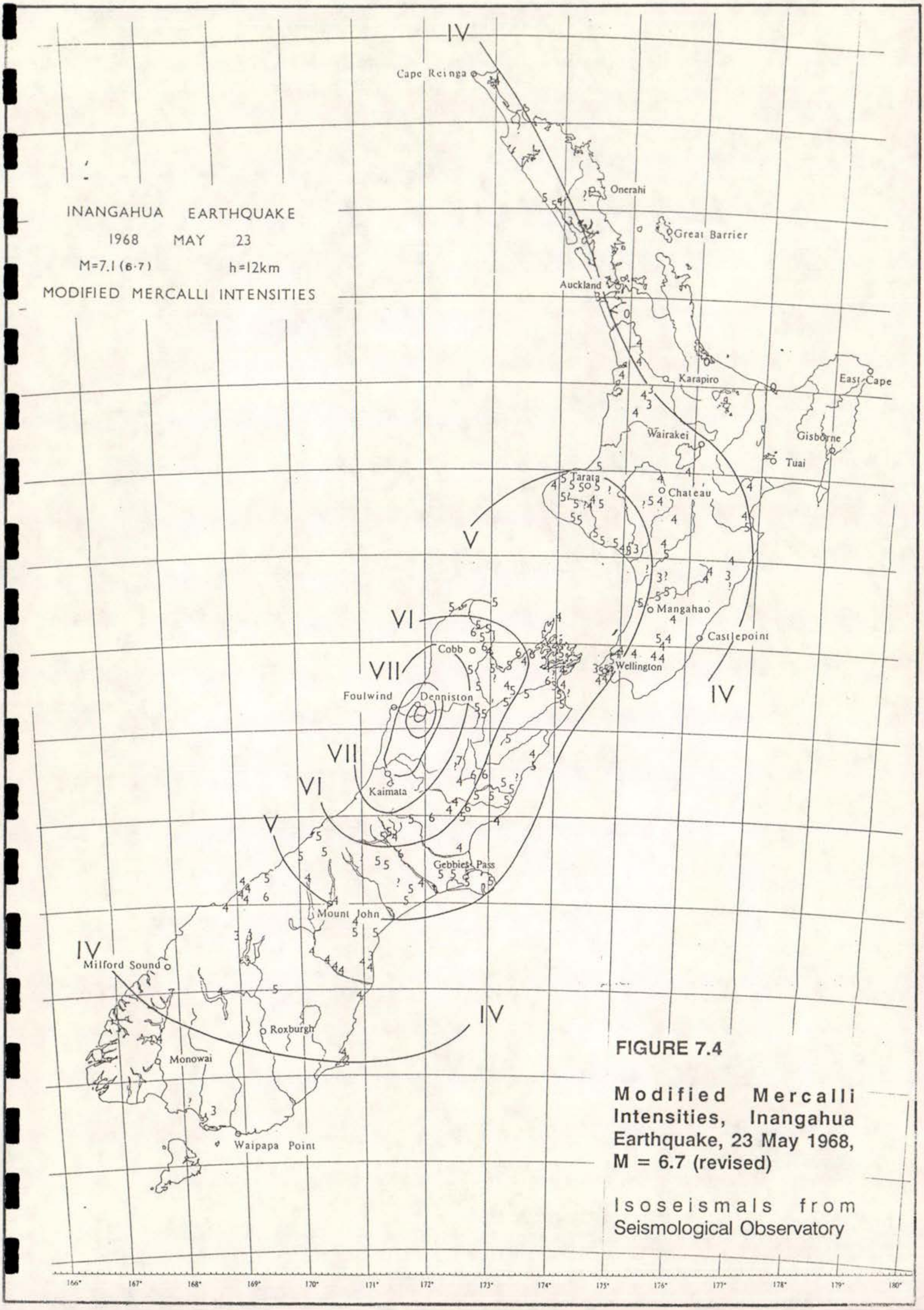


FIGURE 7.4

Modified Mercalli Intensities, Inangahua Earthquake, 23 May 1968, M = 6.7 (revised)

Isoseismals from Seismological Observatory

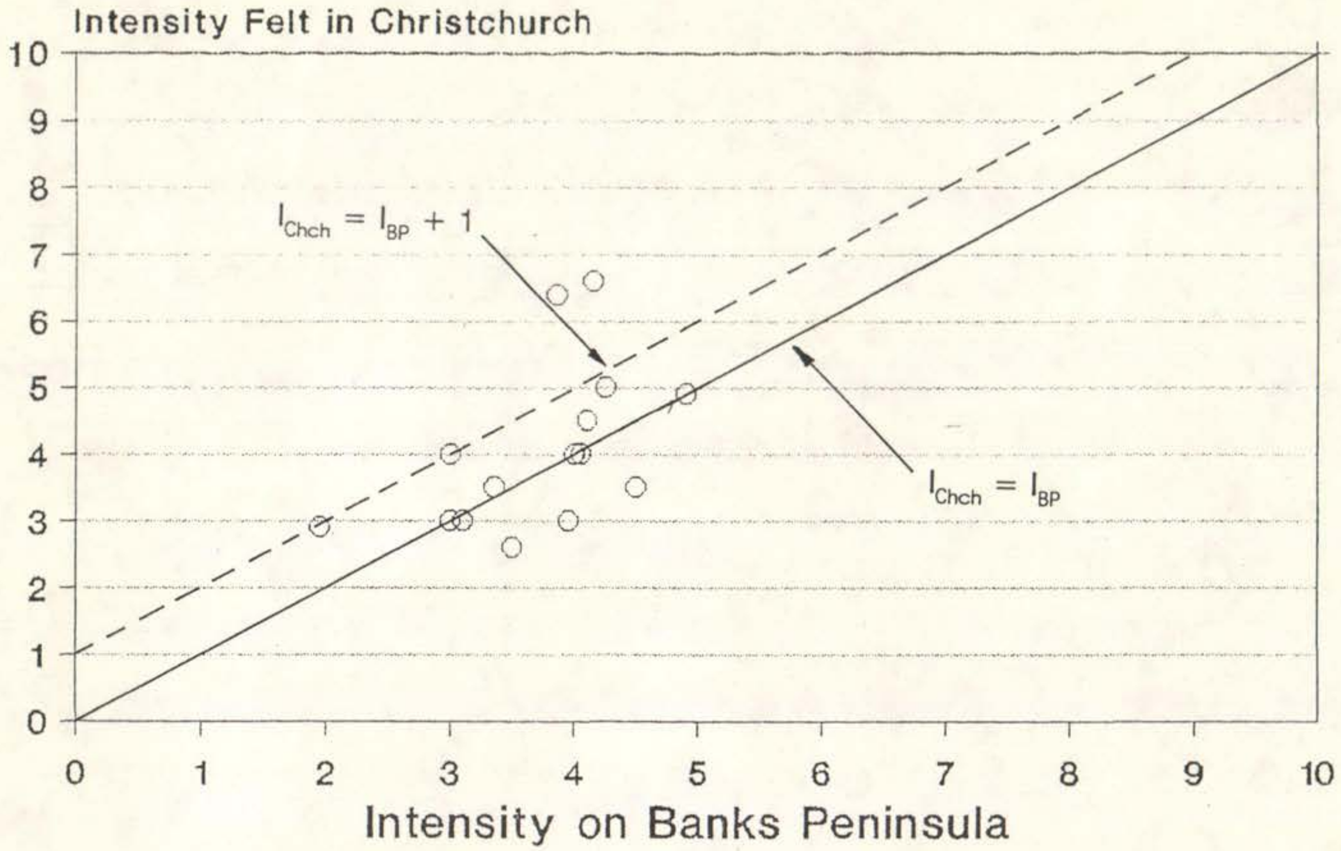


FIGURE 7.5

Comparison of Felt Intensities in Christchurch and on Banks Peninsula

7.3 PREDICTED INTENSITIES AT CHRISTCHURCH

In Chapter 5 a model was developed to predict intensities at Christchurch with associated occurrence probabilities, assuming "average ground" conditions equivalent to elsewhere in New Zealand. It has been shown in section 7.2 that intensities in Christchurch are generally 0 - 2.0 MM units higher than on Banks Peninsula.

Although it is likely that many Banks Peninsula sites are better than 'average ground', previous discussion suggests that at least some sites will not be on bedrock. It may be concluded that intensities in Christchurch will be at least 0 - 1 MM unit higher than on "average ground" at equivalent epicentral distances elsewhere in New Zealand.

The occurrence frequencies of different intensities predicted as a result for Christchurch are obtained directly from Figure 5.8 by increasing the intensity ordinates of the curve for Christchurch by 0 - 1 MM units. This is shown in Figure 7.5a, from which it is apparent that Christchurch is likely to experience occurrences of shaking, similar to, or possibly more frequent, than Wellington for all except very high intensities of MM X. The average return periods for different intensities of shaking on the areas of Christchurch away from the Port Hills are about seven years for MM VI, 20 years for MM VII, 55 years for MM VIII and 300 years for MM IX, allowing for about 0.5 MM unit increase in intensity due to the deep alluvium.

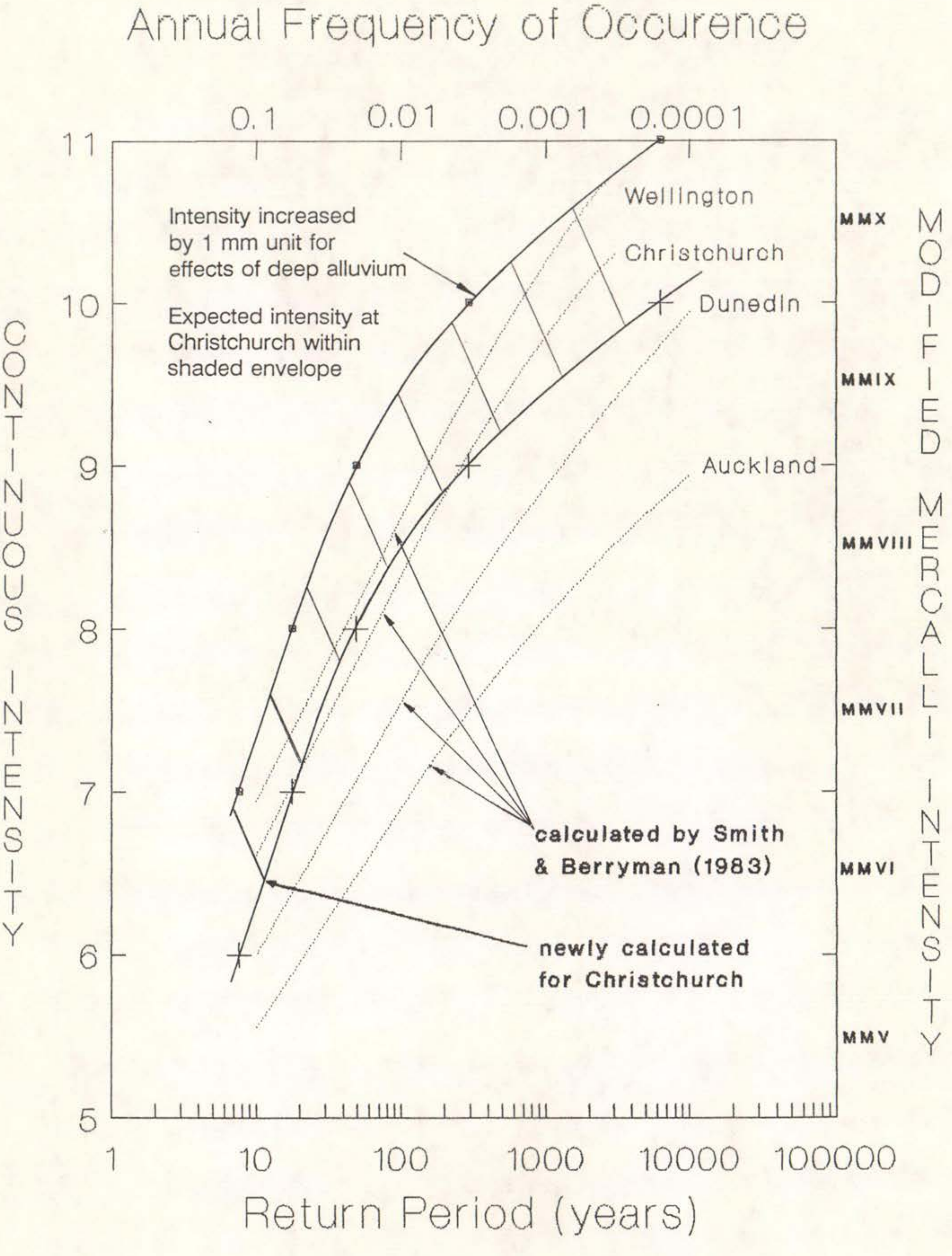


FIGURE 7.5a

Occurrence Frequencies for Seismic Intensities at Christchurch - corrected for ground amplification effects

7.4 GEOLOGIC SOIL PROFILES BENEATH CHRISTCHURCH

In order to carry out detailed modelling of the variation in site-specific seismic response throughout Christchurch it is necessary first to construct a detailed three dimensional model of the geologic profile beneath the city. To be useful for predicting earthquake hazard for design and hazard mitigation purposes it is necessary that a spatial accuracy of soil information better than 100 - 200 metres in the horizontal direction and 5 - 10 metres in the vertical direction be obtained in the upper 30 metres of soil, while differentiating among, at least, peat, clay, silt, sand and gravel. The urban area of Christchurch covers approximately 300 km^2; the resulting number of nodes required to achieve the resolution sought in the upper 30 metres alone is therefore of the order of 100,000.

Three-dimensional modelling of this complexity has not been attempted previously in Christchurch. Nor, to our knowledge, has it been attempted anywhere else for seismic hazard prediction purposes. The most comprehensive deep geological compilation for the city to date is that by the North Canterbury Catchment (Talbot et al, 1986). However the primary objective of the study was *"to collate and review existing information on the groundwater resource beneath metropolitan Christchurch and, as far as it is possible at (that) stage, assess the abstractive 'safe yield' of the resource"*.

The report, and more recent work by geologists at the now Canterbury Regional Council, compiled about 4,000 borelogs, mainly from water wells up to 100 metres deep. Unfortunately most of these borelogs either do not record near surface soils, or simply use generic descriptions such as "0 - 30 metres, sand and clay" (i.e. not water bearing for well purposes) without further refinement, or classification. Strata are simply determined as 'aquifer' or 'aquitard'. However these borelogs are of considerable use at depths below about 30 metres, where high quality boreholes from foundation drilling are not available, and where the change in seismic wave velocity between different soil types is less important for modelling purposes.

A full description of the geological or hydrogeological conclusions of Talbot et al is not attempted here but may be found in chapter four of their report. A detailed description of the geology of Christchurch will be shortly be available (Brown et al, in prep.). The general geologic profile beneath the city is best described by the simplified profile shown in Figure 7.6, reproduced from Figure 4.11 of Talbot et al (1986).

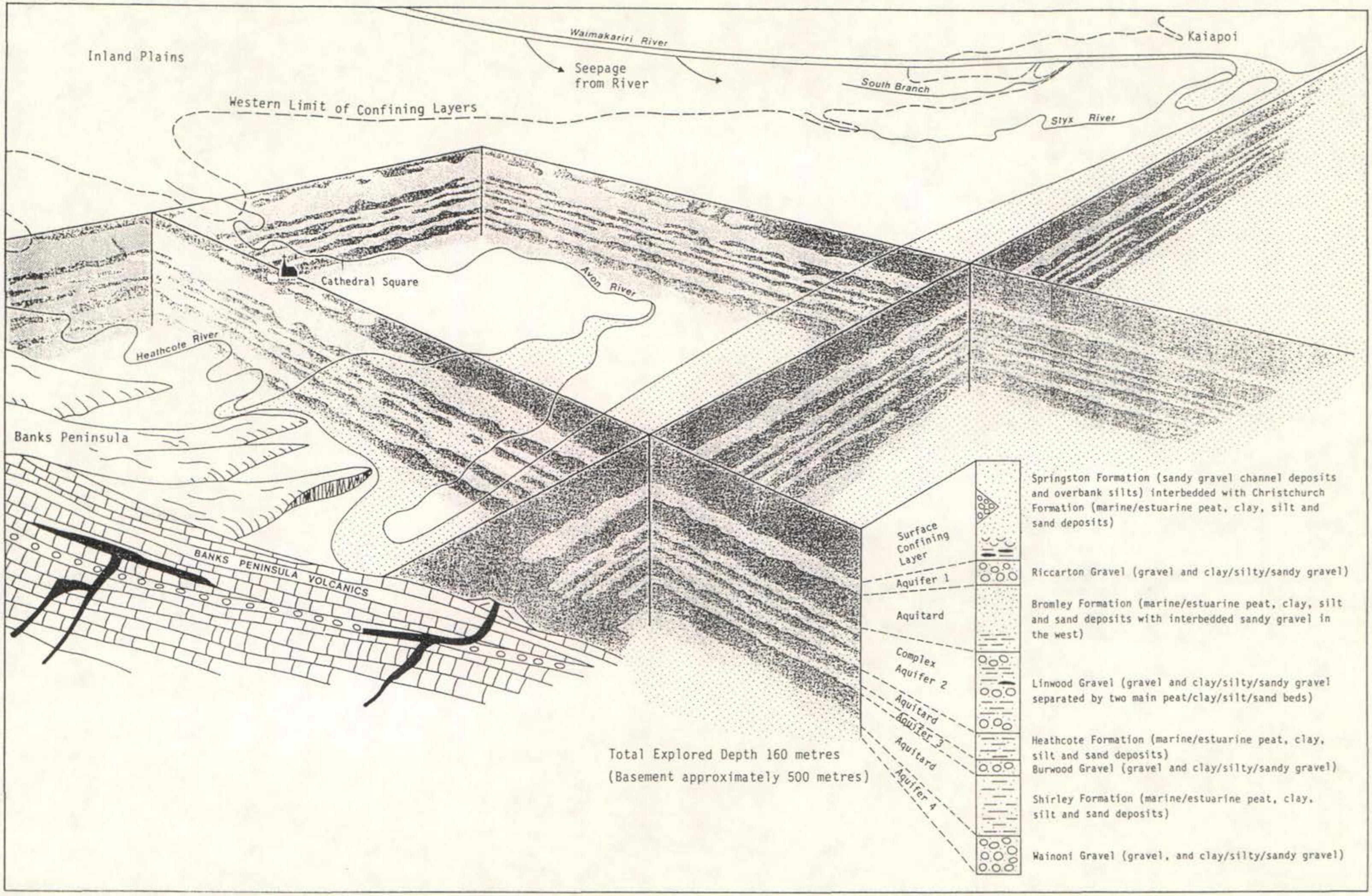


FIGURE 7.6 Diagramatic Geology Beneath Christchurch
(From Fig. 4.11, Talbot et al, 1986)

There are three additional sources of deep (>100 metres) subsurface information beneath Christchurch. The first is a borehole drilled at Bexley, in eastern Christchurch, to 450 metres depth. This hole was drilled for the Canterbury Regional Council to investigate deeper aquifers, but as part of this study we obtained all returned soil samples for accurate geotechnical logging. Unfortunately undisturbed soil sampling, using Shelby tube samplers in cohesive strata, was not possible within the logistical confines of the drilling operation. The borelog for this hole is shown in Appendix B.

The other useful sources of deep information are the results of seismic refraction surveying carried out and reported by Dibble et al, 1980 at Woolston, in south-east Christchurch near the foot of the Port Hills, and seismic reflection surveying reported by Kirkaldie & Thomas (1963), also in south-east Christchurch. From these results it may be inferred that surface alluvium in this area overlies volcanics at variable depths and of variable thickness away from the hills. Tertiary sediments beneath the alluvial quaternary cover overlie greywacke basement at about 800 metres depth.

The most comprehensive existing geotechnical database is that compiled from investigations carried out by Soils & Foundations Ltd and comprising about 10,000 borelogs from about 3,000 sites in Christchurch. These boreholes ranged from 2 - 30 metres deep and were all logged using standard geotechnical conventions. Many included additional in situ or laboratory soil testing to allow determination of advanced soil parameters.

A number of other repositories of site investigation records were traced and borelog information was assimilated into our master database. These records were obtained primarily from the sources listed below. The co-operation of the holders or owners in permitting us to obtain these records is gratefully acknowledged.

Christchurch Drainage Board
Canterbury Regional Council
P.J. Alley borelog records
G.L. Evans
Waimairi County Council
Christchurch City Council
Works Corp
Heathcote County Council
Paparua County Council

The final database compiled from all the sources of information above contains over 20,000 soil records for about 15,000 sites in the Christchurch area. This is thought to represent at least 95% of all available ground information records for Christchurch, and is sufficiently dense in most areas of the city to provide the spatial resolution required. Although many borelogs, particularly those from older, non-geotechnical sources, contain poor soil descriptions by geotechnical standards, when used in conjunction with nearby borelogs to determine continuity of soil strata almost all are useful in a regional seismic hazard analysis.

Due to the enormous number of data collected, and the variable quality of soil records from different sources, a single computerised database containing all borelogs was discovered not to be practical for the purposes of this study. In order to maximise accessibility to the soils information for the purposes of the hazard study, while allowing rapid reinterpretation of soil descriptions on borelogs where conventional geotechnical classification systems were not used, the database records were utilised in the following way:

1. All borelogs were filed according to data source (e.g. Soils & Foundations Ltd).

2. An index database was prepared for each in a standard format.

3. A city street map was prepared at a scale of 1:25,000.

4. All borelog locations were plotted on this map.

5. Detailed city street maps were prepared at a scale of 1:5,000.

6. All borelog locations were replotted on these maps, using a different symbol for each database source, and labelling each location with a database reference.

7. Five reproductions of the 1:25,000 map were prepared, and assigned to represent continuous layers beneath the city at the following depths beneath the ground surface:

   0 - 2 metres
   2 - 5 metres
   5 - 10 metres (refer Figs 7.7 - 7.11)
   10 - 20 metres
   20 - 30 metres

8. The representative soil type in each of the above depth ranges was assessed for each borelog, and plotted on the appropriate map. Colour coding was used to represent each of the following soil type groupings, which were assigned on the basis of our experience in Christchurch of their likely frequency of occurrence and behaviourial properties, particularly under seismic conditions:

   A - Gravel. Sandy Gravel. Gravelly Sand.

   B1 - Sand; uniform medium to coarse (beach or dune sand).

   B2 - Sand; well graded, or uniform fine. Silty sand.

B3/C1	-	Interbedded fine Sand/Silty Sand/Sandy Silt (Silty Sand grading dominant).
C2	-	Sandy Silt. Silt. Clayey Silt.
C3	-	Organic Sandy Silt. Organic Silt.
C4/D1	-	Interbedded Sandy Silt/Silt/Organic Silt/Peat (Sandy Silt or Silt dominant).
D2	-	Highly Organic Silt. Silty Peat. Peat.
E	-	Fill.

9. A 500 metre grid was overlain on each map and the average or representative soil type in each grid square assigned as type A, B, C, D, E using the broader designations described above. These maps are reproduced in this report as Figures 7.7 to 7.11, with the general soil groupings shown as C - predominantly gravel, M - predominantly sand, F - predominantly fine grained, P - peat, and X - fill.

10. Using nodes centred on each grid square, a computerised subsurface database of mean soil types in a given area at given depth was constructed for Christchurch.

Classification of soil types below 30 metres depth was carried out slightly differently. Most soil records at these depths are water well logs obtained from the Canterbury Regional Council and have already been collated by Talbot et al (1974), who constructed simplified hydrogeological cross-sections through the city along two east-west lines, and one north-south line. These simplified sections (Figures 5.2 - 5.4 of their report), designating strata simply as 'aquifer' or 'aquitard', are reproduced here as Figures 7.12 to 7.14. Figure 7.15 shows a simplified typical east-west section with near surface soils added.

Careful analysis of these cross-sections reveals that at depths greater than about 25 - 30 metres the soil profiles may be considered simply as a series of four interconnected gravelly aquifers (1, 2a, 2b, 3, 4), separated by aquitards of peat, silt and sand. The depths and thicknesses of these aquifers vary in the east-west direction, but are relatively constant in the north-south direction.

Amplification of seismic waves occurs due to interference of incident and reflected waves in layered soils, as a function of the strata thicknesses and of the shear wave velocity differences between strata. At depths below about 30 metres the differences in shear wave velocity are considerably less important than they are near the ground surface and soil strata types can therefore be simplified considerably for seismic wave propagation analysis.

For analysis in this study, the ground below 30 metres depth is divided into three typical profiles with depth, which can be identified beneath the west, the centre and the east of the city respectively. These profiles are described below:

(all depths in metres)

	West Zone	Central Zone	East Zone
Aquifer 1	30 - 45	30 - 40	30 - 45
Aquitard	45 - 50	40 - 55	45 - 60
Aquifer 2a	50 - 65	55 - 65	60 - 70
Aquitard	65 - 70	65 - 75	70 - 85
Aquifer 2b	70 - 85	75 - 85	85 - 95
Aquitard	85 - 95	85 - 100	90 - 115
Aquifer 3	95 - 110	100 - 110	115 - 120
Aquitard	110 - 115	110 - 125	120 - 135
Aquifer 4	115 - 140	125 - 140	135 - 150
Aquitard	140 - 160	140 - 160	150 - 160

Below 160 metres depth borelog information is too sparse to differentiate profiles between different areas. One representative profile is assumed for the entire area, extending from 160 metres to basement rock, based primarily on the borelog from one deep borehole at Bexley, the results of geophysical surveying discussed earlier, and other work on the Canterbury Plains but more distant from Christchurch, in the Ashburton River area (Atkins & Hicks, 1977):

Lyttelton Volcanics	500 - 700 metres
Tertiary Sandstones	700 - 1000/1500 metres
Cretaceous Volcanics/Greywacke Basement	>1000/1500 metres

In summary, the subsurface database used in this study consists of detailed soil type information to 30 metres depth, simplified to information in five layers with a 500 metres horizontal grid. Three profiles describe the deeper soils to 160 metres depth beneath different parts of the city, and one assumed profile is used from 160 metres to basement rock.

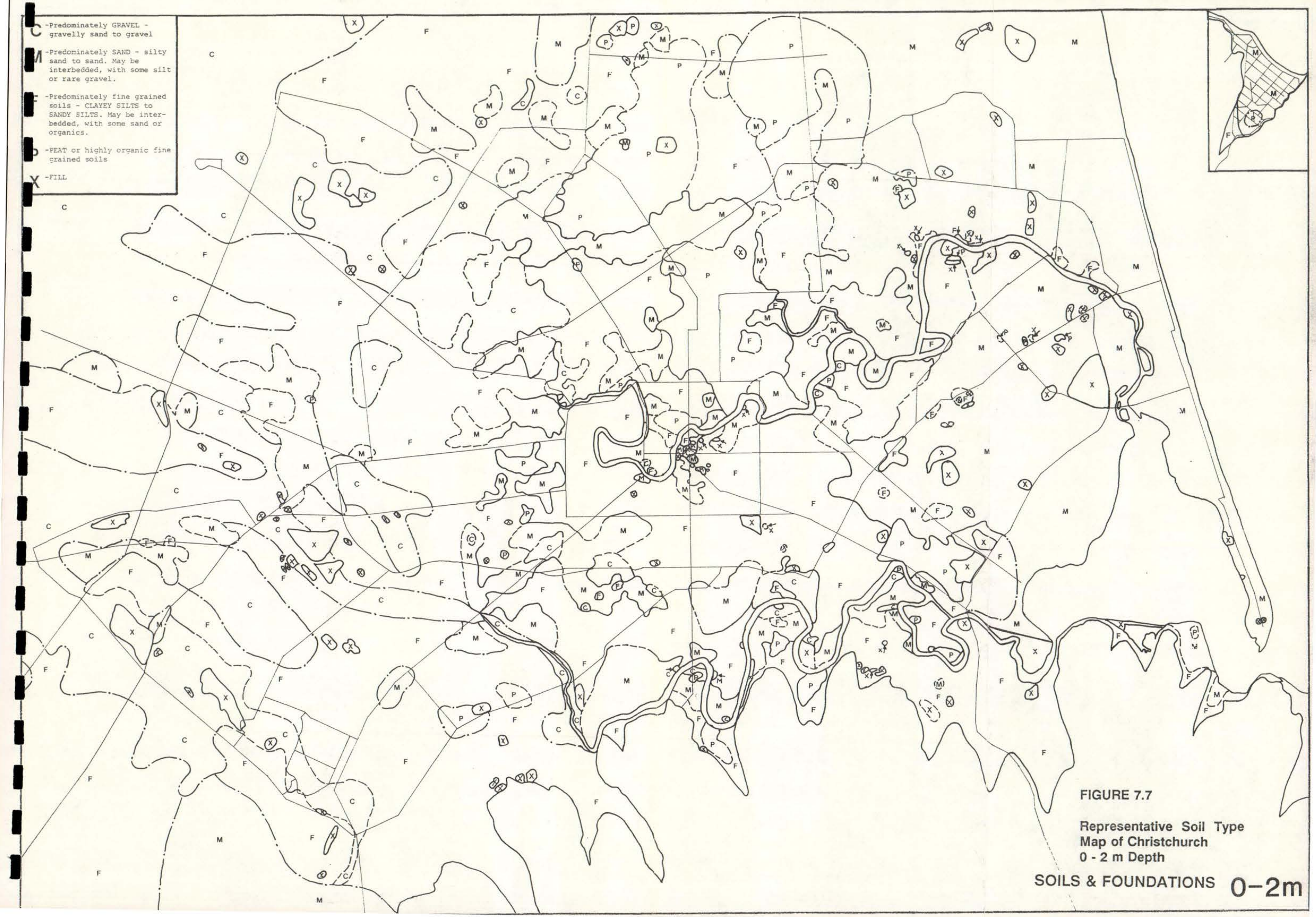


FIGURE 7.7

Representative Soil Type Map of Christchurch 0 - 2 m Depth

SOILS & FOUNDATIONS **0–2m**

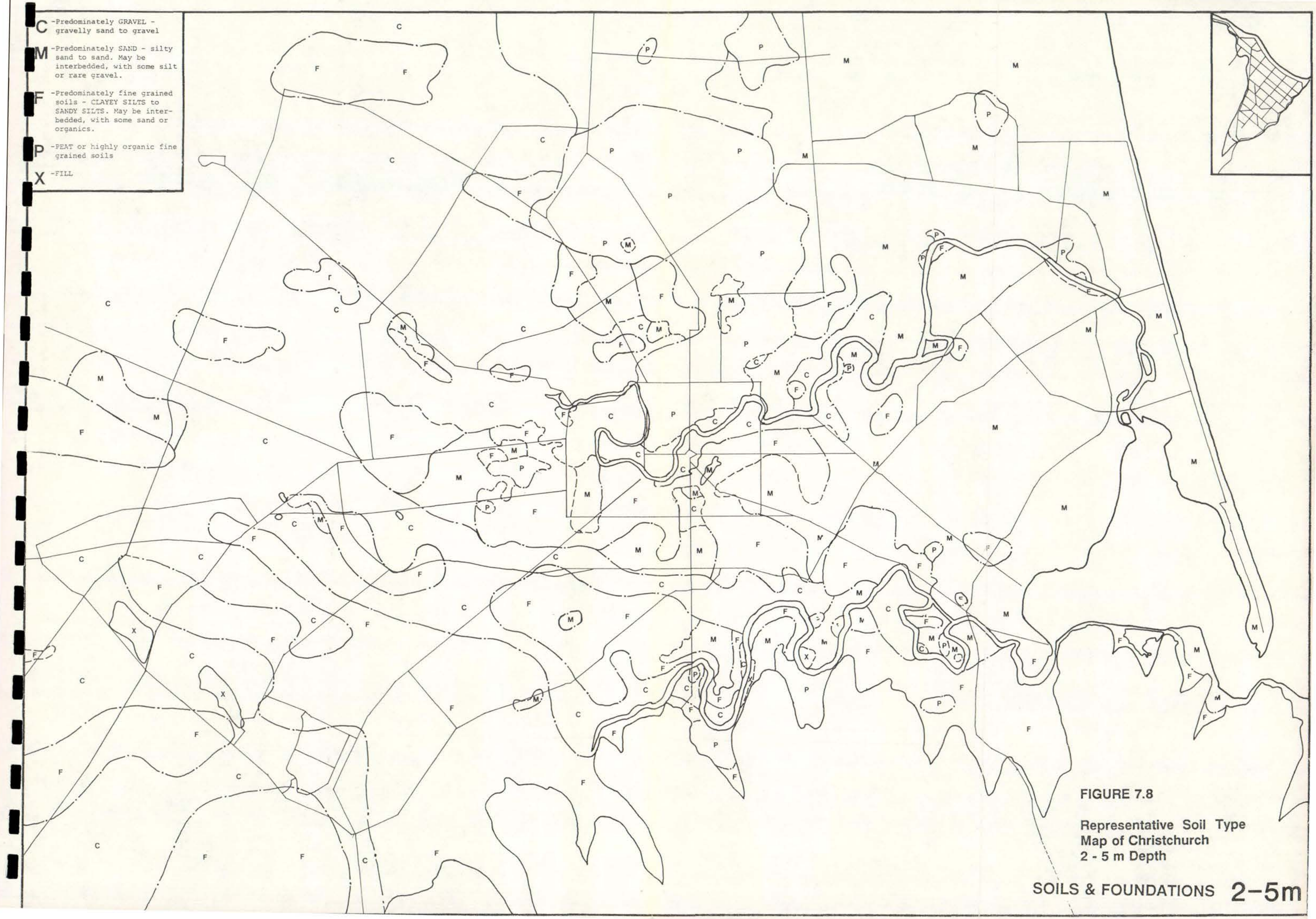


FIGURE 7.8

Representative Soil Type Map of Christchurch 2 - 5 m Depth

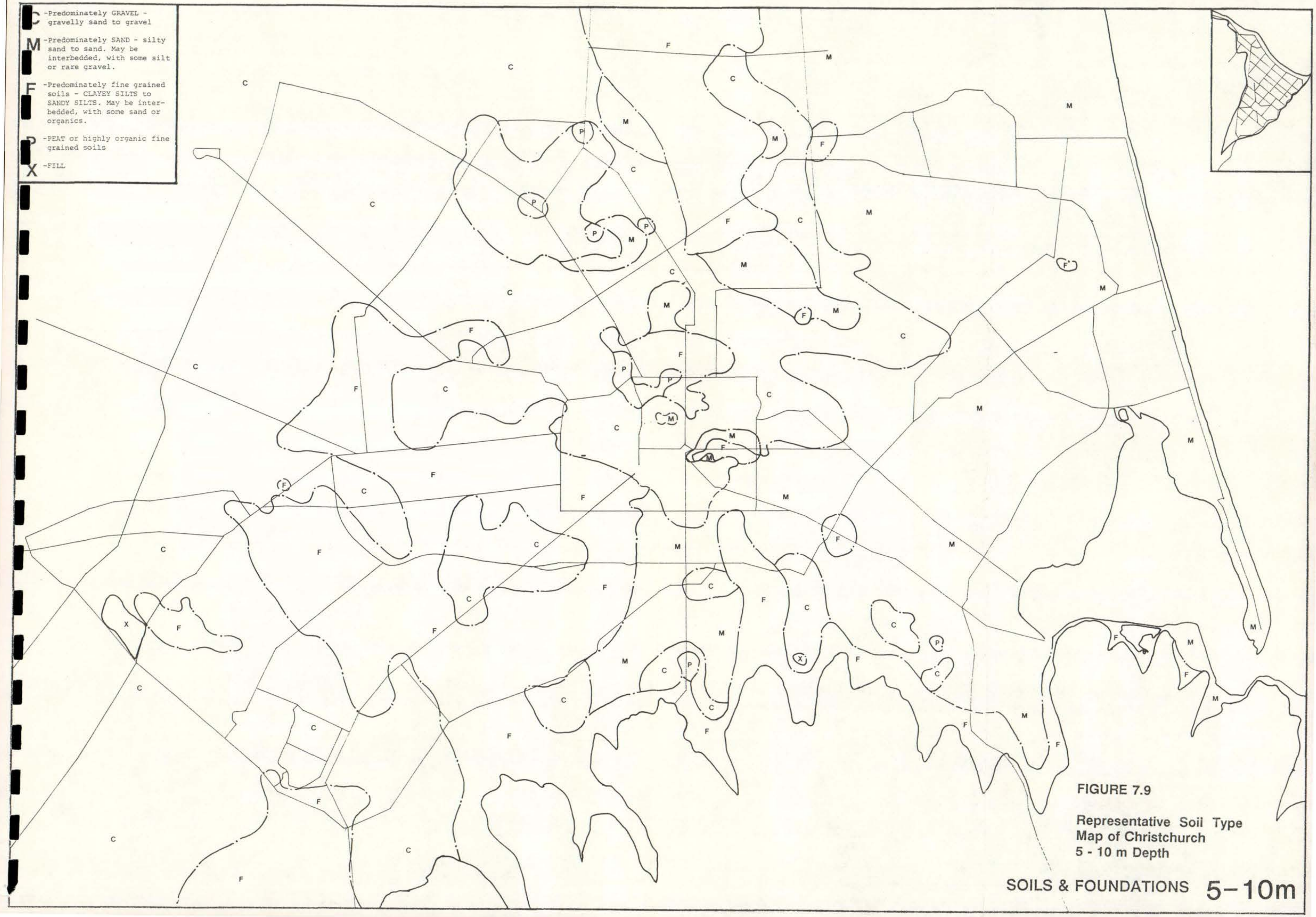


FIGURE 7.9

Representative Soil Type Map of Christchurch 5 - 10 m Depth

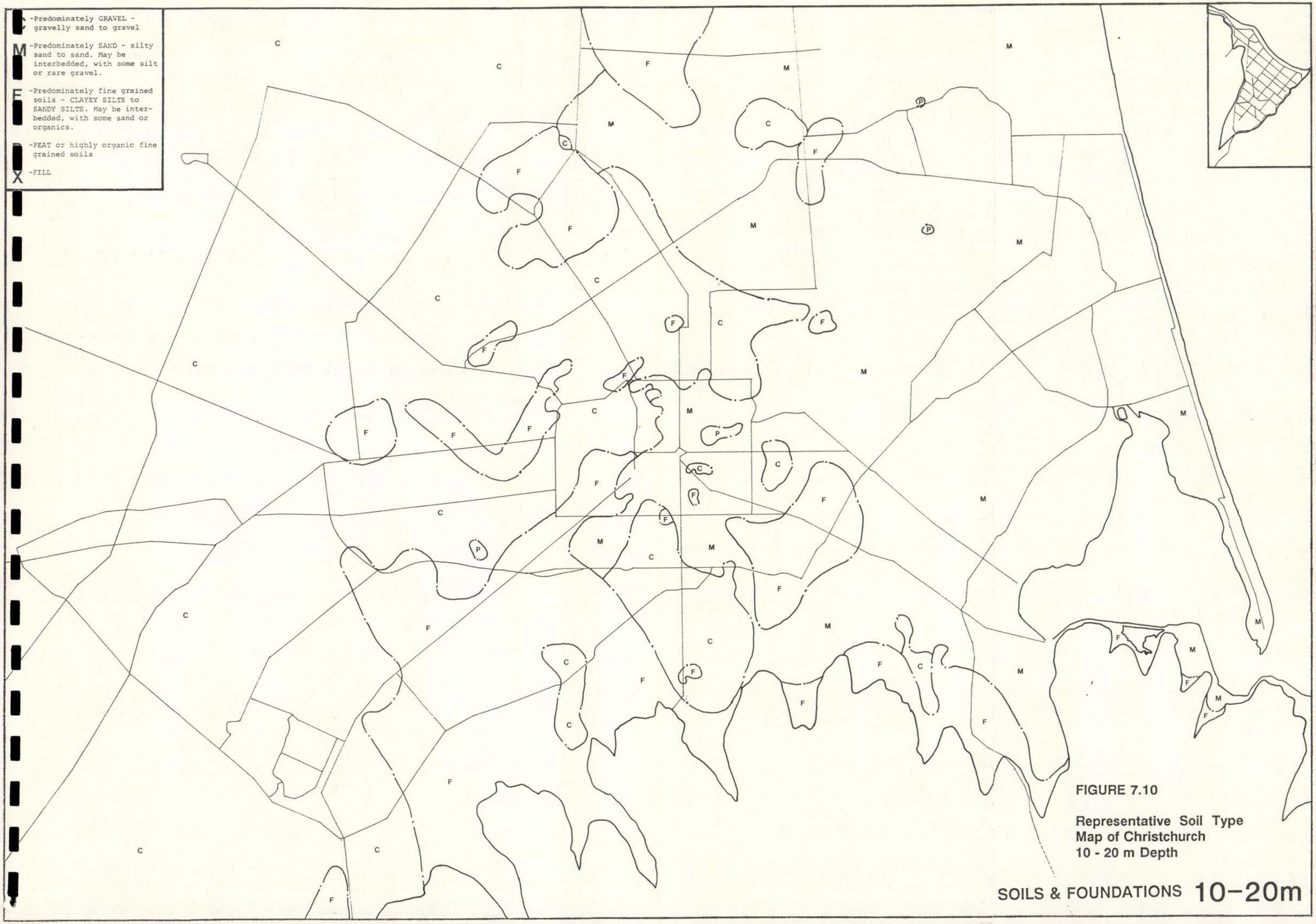


FIGURE 7.10

Representative Soil Type Map of Christchurch 10 - 20 m Depth

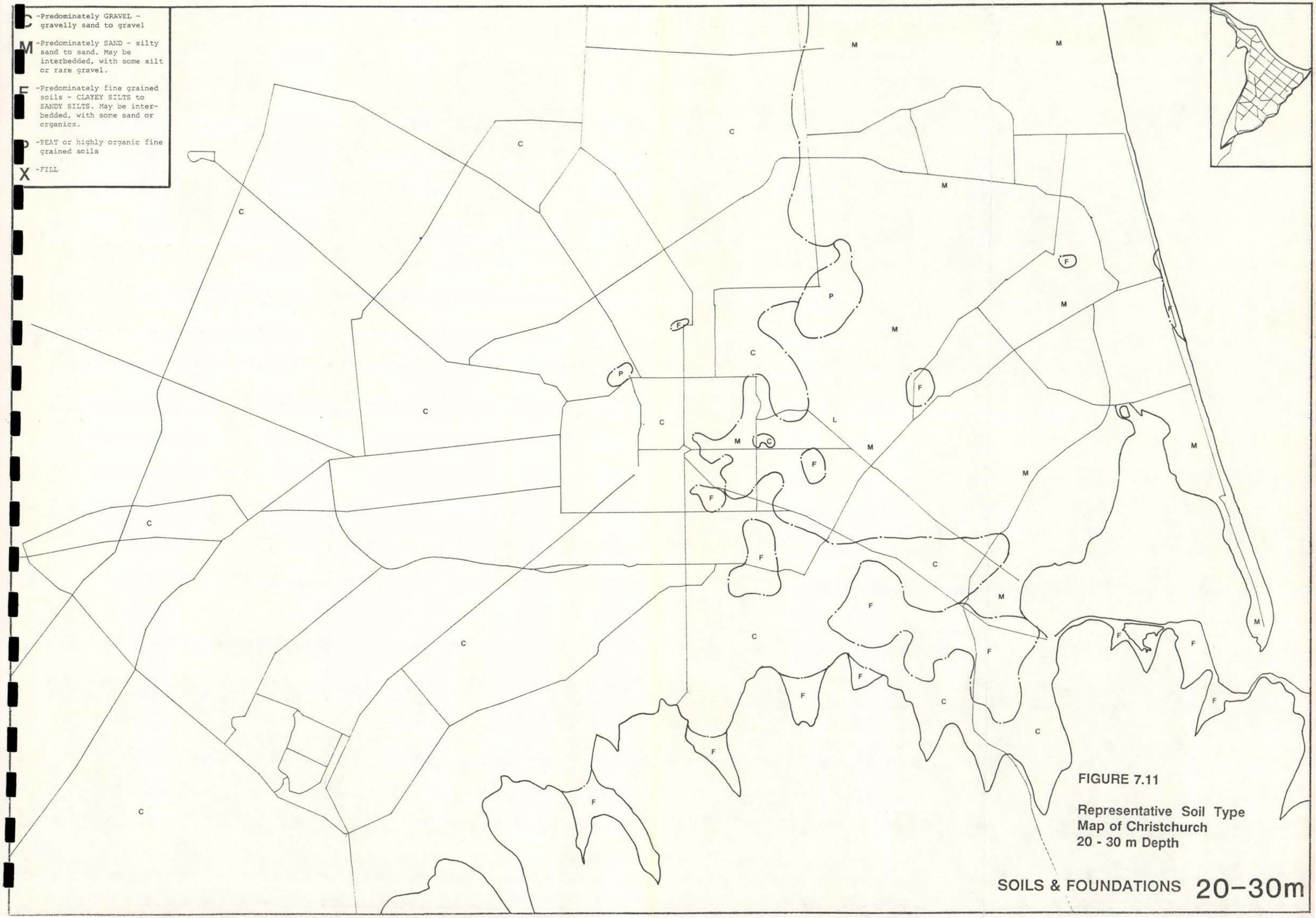


FIGURE 7.11

Representative Soil Type Map of Christchurch 20 - 30 m Depth

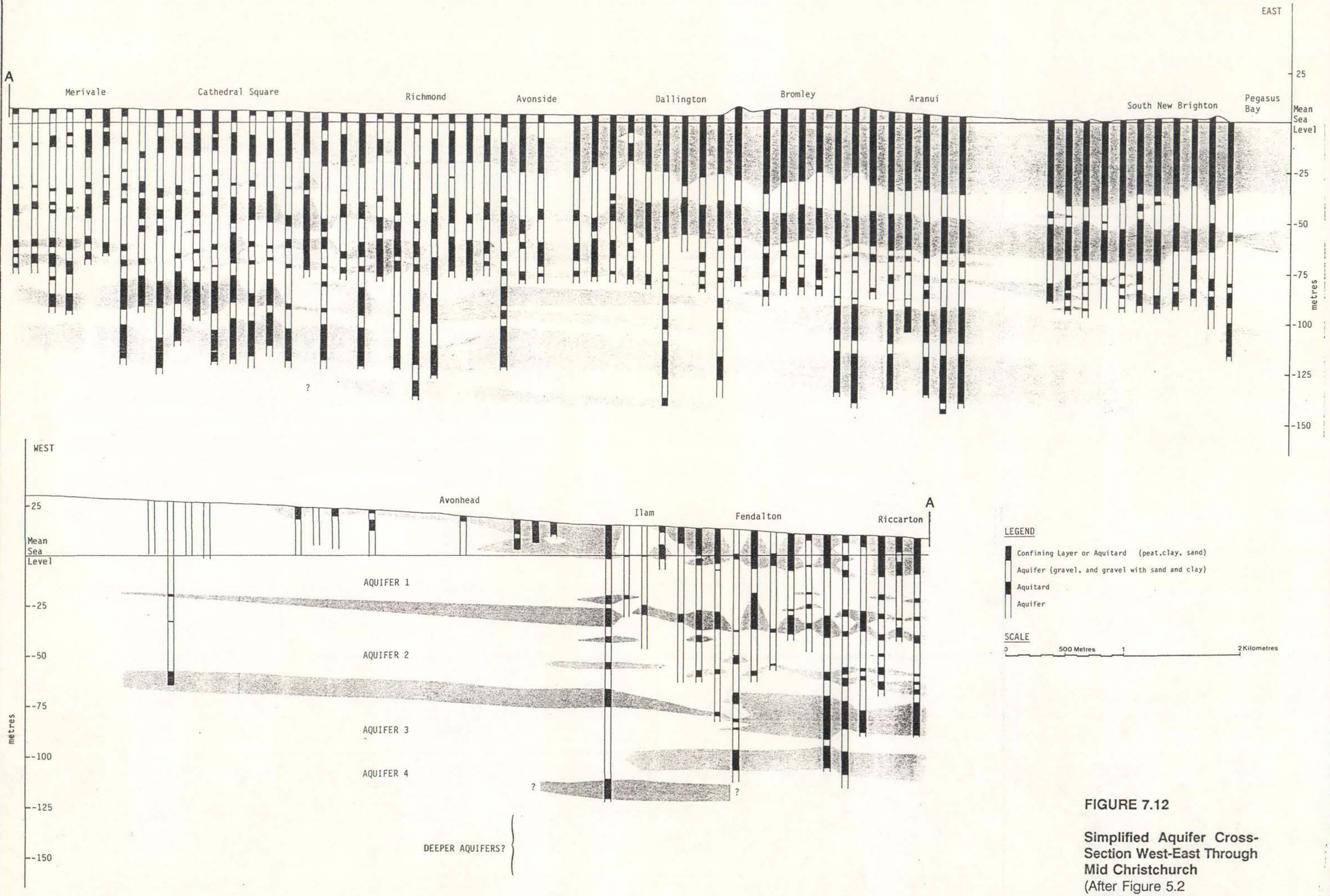


FIGURE 7.12

Simplified Aquifer Cross-Section West-East Through Mid Christchurch
(After Figure 5.2
Talbot et al, 1986)

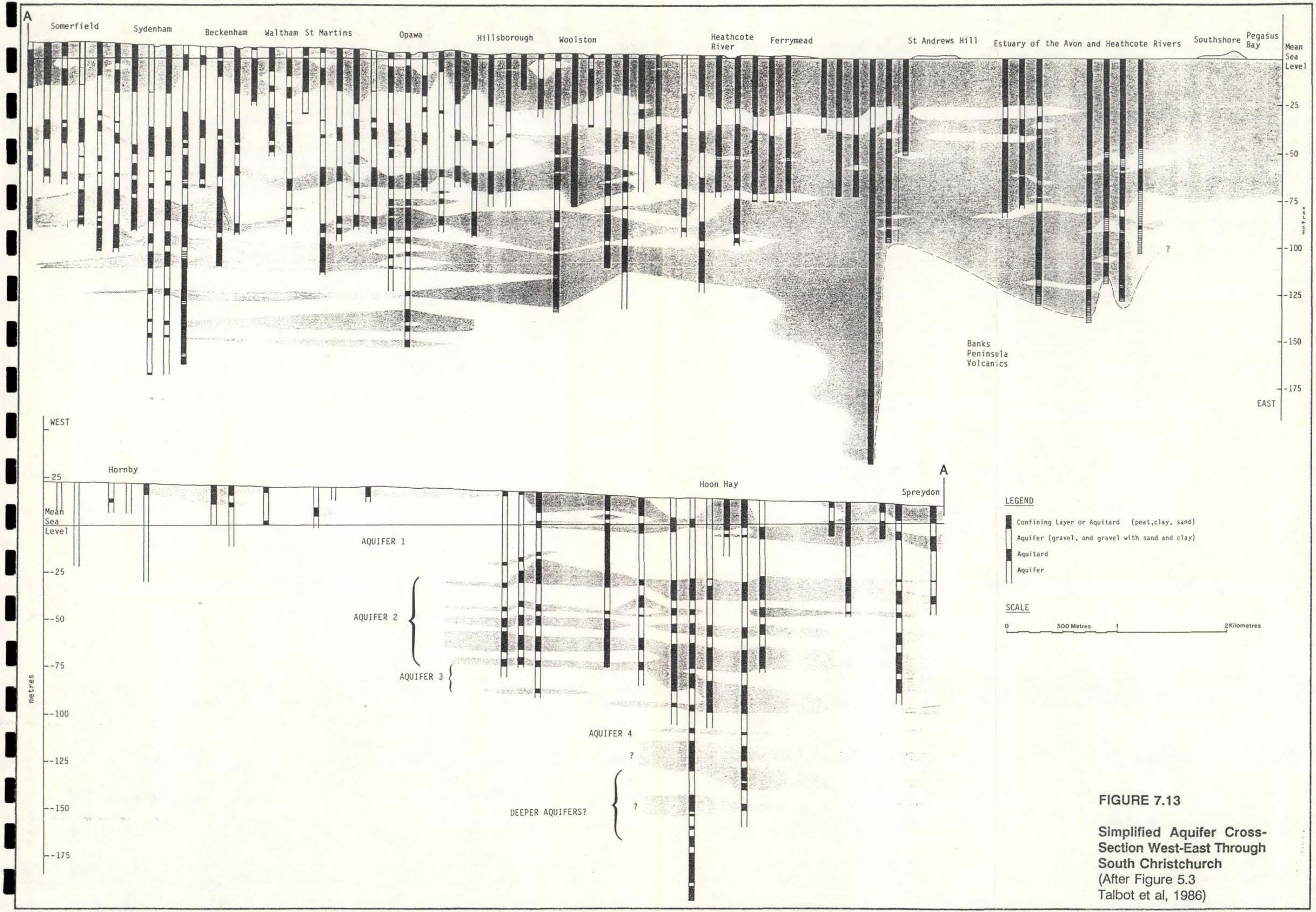


FIGURE 7.13

Simplified Aquifer Cross-Section West-East Through South Christchurch
(After Figure 5.3
Talbot et al, 1986)

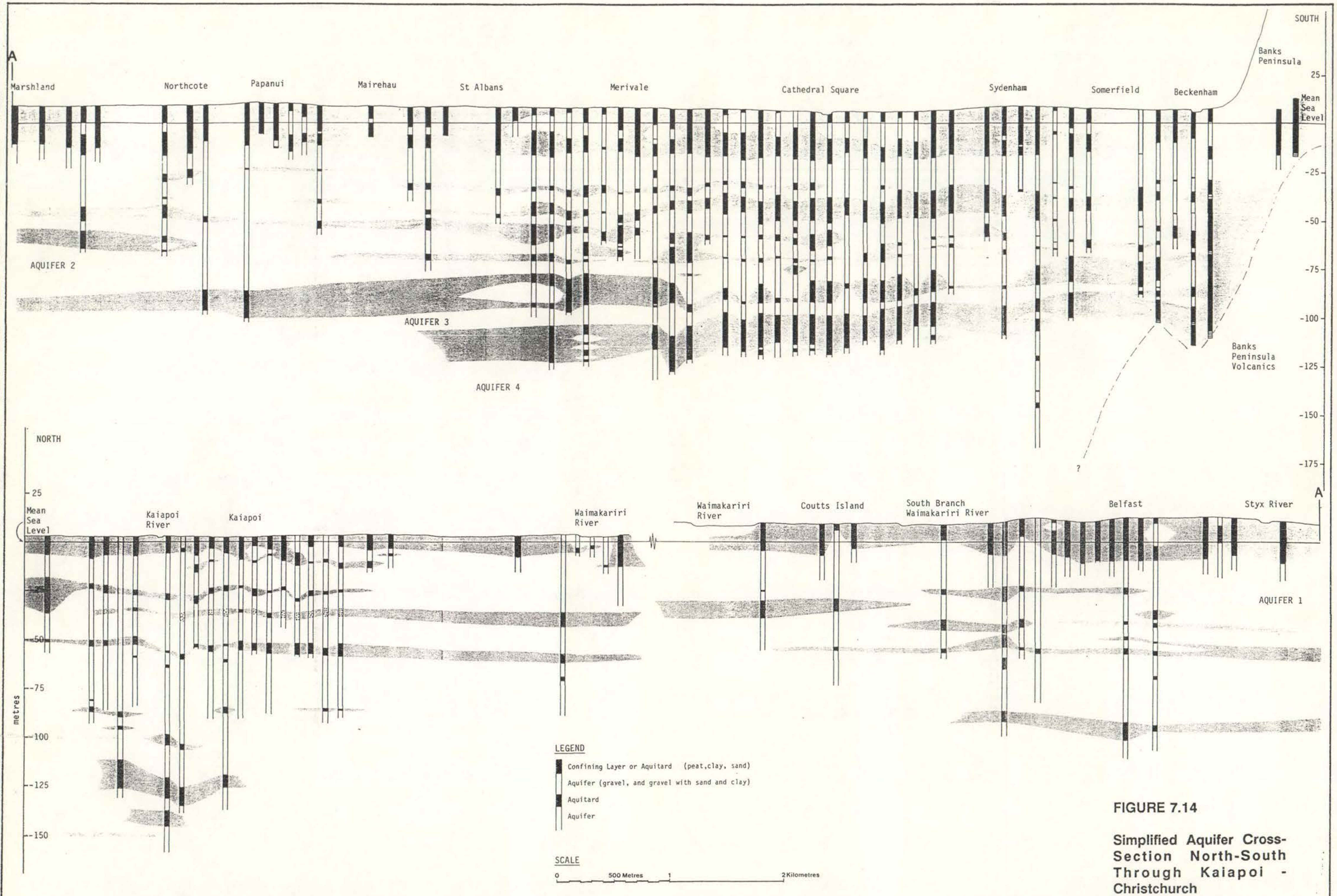


FIGURE 7.14

Simplified Aquifer Cross-Section North-South Through Kaiapoi - Christchurch
(after Figure 5.4
Talbot et al, 1986)

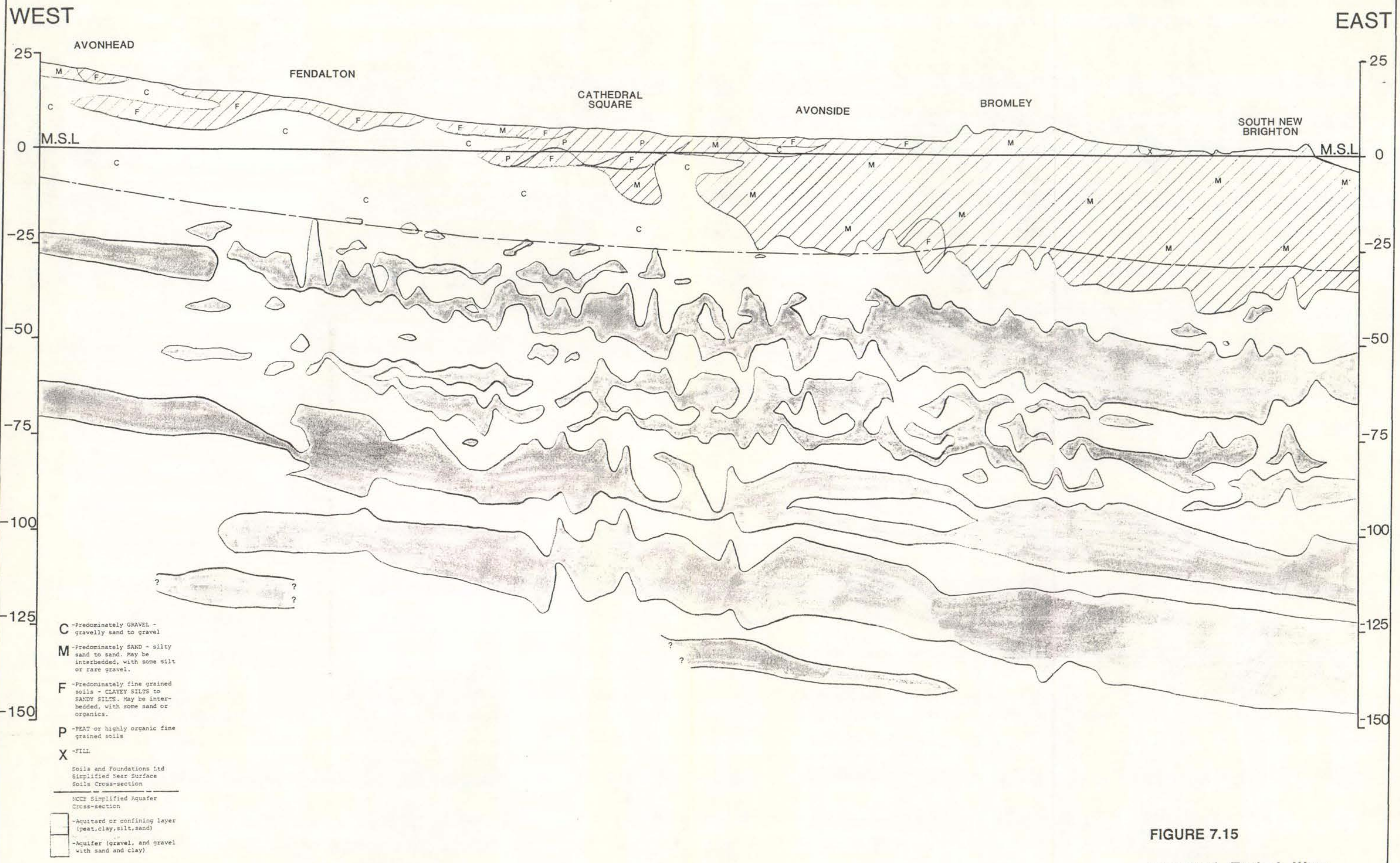


FIGURE 7.15

Simplified Typical West-East Section Through Central Christchurch with Near Surface Soil Types

7.5 DEEP SOIL SEISMIC WAVE PROPAGATION MODEL

In order to consider the modifications to response caused by deep soil geological profiles at a particular site, it is necessary generally to consider four separate effects. Part of the following discussion is reproduced from the report of Davis & Berrill (1988).

- Amplification of motion due to geometric focusing of seismic waves by non-planar basement geology.
- Amplification of motion due to the impedance mismatch between harder rock basement and overlying soft rock or soil.
- Resonance characteristics caused by constructive and destructive interference between incident and reflected waves in layered soils.
- Attenuation caused by non-linear dissipative response of soft soils.

Geometric focusing of seismic energy in soils generally occurs where non-horizontal layering of soil or rock causes non-uniform refraction at layer interfaces. It is also common in hill areas where the geometry of rock profiles may direct and concentrate seismic waves. However, away from the hills in Christchurch, layering is usually continuous over distances of tens to hundreds of metres in the horizontal direction but less than ten metres in the vertical direction. Focusing is therefore insignificant when compared to the other effects described above, and is not analysed in this study.

Amplification of earthquake motion due to the bedrock-soil impedance mismatch is a universal feature of site response. Refraction of seismic waves will generally result in incident waves approaching the rock-soil interface in a nearly vertical propagation direction. Most site response studies therefore consider only vertically propagating SH waves (horizontally polarised shear waves) as these are the most destructive waves, and vertical propagation gives the worst possible case.

The extensive interlayering of peats, silts, sands and gravels beneath Christchurch will clearly cause significant resonance effects.

Whereas the first three site effects may result in amplification of seismic waves, the fourth effect results in attenuation due to energy dissipation caused by hysteretic damping. As waves propagate through the soil layers, stress reversals and hysteresis remove energy, especially from high frequency components. The high frequency, short wave length components suffer more stress reversals and hence more energy is dissipated. The overall result is a decrease in the high frequency response spectrum.

The three site effects considered in this study are all embodied in a transfer function which, for a given frequency of motion, represents the ratio of the ground surface response to the basement rock motion. If the transfer function is known for all frequencies of interest, then the product of the rock acceleration response spectrum with the transfer function gives the ground surface response spectrum. It is assumed that this is independent of the actual magnitude of accelerations. To calculate the transfer function it is assumed that the basement rock and overlying soil layers are linear viscoelastic, and that all motions are vertically propagating SH waves.

Laboratory tests on soil samples reveal that non-linear strain softening will generally be present at the strains induced by earthquake motions. It is not directly possible to carry out the transfer function calculation for non-linear materials, but an equivalent linear analysis may be used instead. Assuming linear behaviour, the strains developed within each soil layer may be calculated. The peak strain magnitude may then be used to suggest a reduced shear modulus, consistent with typical data from laboratory tests. A new calculation is then performed with the reduced moduli, and a new set of peak strains are found. This procedure is performed iteratively until a consistent set of moduli and peak strain are determined. Relationships between modulus and shear strain outlined by Seed & Idriss (1970) have been used in the calculations described in this study. Damping coefficients have also been based on the Seed & Idriss model.

The specific method of calculation follows that presented by Haskell (1960). Hysteretic damping is incorporated directly. This procedure used is that developed for computer analysis by Dr R.O. Davis at the University of Canterbury. The use of this computer program, and assistance given by Dr Davis, is gratefully acknowledged.

The analysis requires the following input data:

- Modelled layers numbered from 1 at deepest layer, immediately overlying bedrock;
- Thickness (metres) of each layer;
- Density (t/m^3) of each layer;
- Shear wave velocity v_s (m/s) in each layer; or
- Shear modulus G_s (MPa) in each layer;
- Hysteretic damping characteristic of each layer;
- Density and shear wave velocity or shear modulus in bedrock.

As discussed in section 7.4, little reliable information is available on soil properties for Christchurch at depths below about 30 metres.

Density

Results of gravity surveys carried out beneath the central Canterbury Plains and reported by Atkins & Hicks (1979) suggest the following mean values of soil and rock densities in the plains area:

Quaternary gravels (700 metres deep) - dry	1.77 t/m^3
Undifferentiated Tertiary sandstone/ Quaternary gravels (1800 metres deep)	2.17 - 2.42 t/m^3
Mesozoic basement (>1800 metres deep)	2.67 t/m^3 assumed

No direct measurement of soil density has been carried out for any location at depth in Christchurch, other than measurement of moisture contents, and occasionally saturated densities, on samples of silt recovered in Shelby tubes during foundation investigations. Some indirect assessment of soil densities may be made by considering results of Standard Penetration Tests carried out during foundation investigations. These results may be used to estimate relative densities of granular soils. Assuming maximum and minimum saturated densities for these soils based on results of compaction testing for typical soil types, reasonable estimates of likely densities in situ may be made.

Using these methods, together with our experience of the range and typical properties of different soil types in Christchurch, a mean density profile range with depth has been assumed for this study.

Shear Wave Velocity

A number of methods are available for assessing the shear wave (S-wave) velocities in different soil strata. The geophysical study by Atkins & Hicks (1979) beneath the central plains reported mean compression wave (P-wave) velocities from 0 - 700 metres depth of v_p = 1000 - 2900 m/s. Corresponding S-wave velocities will be about v_s = 500 - 1500 m/s, if these P-wave velocities are assumed to be in saturated soils. However a large part of the 700 metres depth may be unsaturated. Dibble et al (1980) estimates the following S-wave velocities at Woolston:

Alluvium, 0 - 10 metres depth	200 m/s
Alluvium, 10 - 160 metres depth	600 m/s
Volcanics, below 50 metres depth	1800 m/s
Tertiary sediments, above 840 metres depth	-
Greywacke basement, below 840 metres depth	3250 m/s

Various authors have suggested correlations between S-wave velocity and other soil test parameters. Martin (1988, after Lew et al, 1981) described typical profiles of S-wave velocity with depth for "soft natural soils", such as Holocene flood plain deposits, and for "firm natural soils", such as Pleistocene and Holocene high density silty and gravelly sands, for the Los Angeles basin region. Values ranged from 100 m/s for soft soils and 250 m/s for firm soils at the ground surface to 500 m/s and 1000 m/s for soft and firm soils respectively at 30 metres depth. Martin suggested that these values were appropriate for use in seismic wave propagation analyses, such as that carried out here.

Fumal & Tinsley (1985) proposed different correlations for different soil types with the corrected Standard Penetration Test resistance at depths from 0 - 30 metres. Although considerable scatter is evident in their data, this is common in any correlations attempted with SPT results since the test has many uncontrolled factors. An apparent relationship for all soil types, based on their data, may be approximately represented for mean values by:

SPT-N (corrected blows/300 mm):	0	10	20	30	50	75
S-wave velocity (m/s):	140	200	260	320	430	600

These results, adjusting SPT-N values for depths from 0 - 30 metres, are similar in this depth range to those of Lew et al (1981) described above. This suggests that the validity of such correlations for use in seismic wave propagation analyses suggested by Martin (1988) may be more general than the specific regions for which the correlations were derived.

Gibbs & Roth (1989) presented a profile of S-wave velocity with depth from 0 - 200 m in Pleistocene and Pliocene sands and gravels overlying Miocene sandstone, mudstone and claystone at Parkfield, California. This compares well to the range of values which may be calculated by extrapolating the correlations of Lew et al, or Fumal & Tinsley, to depths greater than the 30 metres for which they were derived.

Laboratory studies have been reported where the shear modulus, G_s, has been determined as a function of void ratio for coarse grained soils or as a function of the undrained shear strength for cohesive soils (e.g. Seed & Idriss, 1970; discussion by Hughes, 1987). These correlations, although perhaps more accurate, are generally of less direct use since neither the void ratio nor the undrained strength is as widely known as the SPT resistance. However analysis of S-wave velocities obtained using the two different approaches for typical soil types suggests that similar results would be obtained.

In this study, the following S-wave profile range with depth was adopted following consideration of all the results described above:

S-wave velocity (m/s)

Depth (m)	Very soft/loose (N = 0-5)	Medium dense/stiff (N = 15-20)	Very dense/hard (N >40-50)
(0 - 2)	75	120	250
10	150	250	400
20	-	350	600
50	-	500	800
100	-	700	1100
200	-	1000	1500
500	-	1500	2200

Soil Property Profile

The geologic soil profile beneath Christchurch was discussed in section 7.4 where the model used in this study was presented. By combining this soil model with the profiles of density and shear wave velocity compiled in this section the model described in Table 7.1, with ranges of values for use in the seismic wave propagation analysis, is produced. At any location in Christchurch the appropriate values for use in the propagation analysis are determined as follows:

0 - 30 metres	Individual soil types from five soil maps
30 - 160 metres	Soil profile according to area (west, central or east)
> 160 metres	Single soil profile used

Using this method to construct soil property profiles with depth, the seismic wave propagation analysis can be performed for any location in the city.

TABLE 7.1

LAYER	DEPTH	THICKNESS	SOIL/ROCK	DENSITY (t/m^3)	V_s (m/s)
	0				
1		2	QUARTERNARY	1.6-1.9	60-150
	2		SEDIMENTS		
2		3		1.7-1.9	80-180
	5		Detailed		
3		5	information	1.8-2.0	125-250
	10		from layered		
4		5	soil maps	1.8-2.1	150-300
	15				
5		5		1.8-2.1	180-350
	20				
6		15		1.8-2.2	300-400
	35		QUARTERNARY		
7		15	SEDIMENTS	1.9-2.2	400-500
	50				
8		20	General	2.0-2.2	500-600
	70		soil profiles		
9		30	only according	2.0-2.2	600-700
	100		to area		
10		20		2.0-2.2	630-800
	120				
11		20		2.0-2.2	660-850
	140				
12		40		2.1-2.3	700-900
	180		QUARTERNARY		
13		40	SEDIMENTS	2.1-2.3	800-1000
	220				
14		60	No information	2.1-2.3	950-1150
	280		available		
15		60	except from	2.1-2.3	1100-1300
	340		deep Bexley		
16		60	borehole	2.1-2.3	1200-1400
	400		to 430 m		
17		60	Use	2.3	1300-1500
	460		single		
18		40	profile	2.3	1400-1600
	500				
19		200	LYTTELTON	2.5	1700
			VOLCANICS		
	700				
20		300/800	TERTIARY	2.6	1900
	1000 or		SANDSTONE		
	1500		CRETACEOUS		
21			VOLCANICS/	2.6	3000 or
			GREYWACKE		3250

7.6 SITE EFFECTS PREDICTED BY MODEL

In this section the upper bound bedrock response spectrum calculated in Chapter 6 (see Figure 6.2) is used as input into the deep soil propagation model described in the previous section. Response spectra at the ground surface are calculated for each nodal point on the Christchurch 500 metre grid. A simple method for construction of approximate response spectra based solely on the soil profile at any location is described.

Before carrying out detailed modelling of response spectrum modification by the deep alluvium for each grid node in the city, the effects of some assumptions made in the modelling were evaluated. Parameter studies considered the following factors:

- Depth to greywacke basement
- S-wave velocity in basement
- Depth to volcanics
- S-wave velocity in volcanics
- Effect of small changes in S-wave velocity in any layer
- Effect of small change in thickness of any layer
- Depth of most critical layers affecting analysis results
- Effect of soft/hard soil hysteresis damping in different layers

It was found that results were insensitive to the depth to basement rock, or the incident S-wave velocity in the basement rock. The same held for the volcanic stratum. Analyses were only sensitive to the choice of hysteresis damping and degradation rate for soil layers within 20 metres of the ground surface, where soil types and properties are well defined by the information available. Changes in layer thickness or soil properties were most critical for soil above about 50 metres depth; below about 150 metres depth the effect of these changes was almost insignificant.

Analysis of these preliminary results allows the total number of soil profile analyses required to be substantially reduced. Different soil profiles which produce similar surface response spectra are identified and grouped for common analysis. At depths from 0 to 10 metres it is necessary to distinguish between only four soil types: peaty soils; fine grained cohesive soils (primarily silt sizes); medium grained soils (primarily sand size) and coarse grained soils (gravel sizes). From 10 - 30 metres depth peaty and fine grained soils give very similar results and can be analysed as being the same. Below 30 metres depth it is necessary to distinguish only between fine grained, cohesive soils, and coarse grained soils (sands or gravels). The correspondence of these divisions with the Aquifer/Aquitard classification available from deep soil cross-sections is fortunate and convenient.

In total, the modification to the response spectrum caused by the deep, stratified alluvial soils beneath Christchurch was modelled for about 180 different soil profiles from the ground surface to the greywacke basement. The upper bound bedrock spectrum shown on Figure 6.2 is arbitrarily used as input; it was shown in Chapter 6 that other bedrock spectra for different recurrence probabilities may be scaled directly. The numerical results of this analysis therefore do not have particular meaning in themselves, except that spectra represent extreme maxima for Christchurch. The calculation of the spectral shapes is of most importance, together with the amplification factor from the selected input bedrock spectrum.

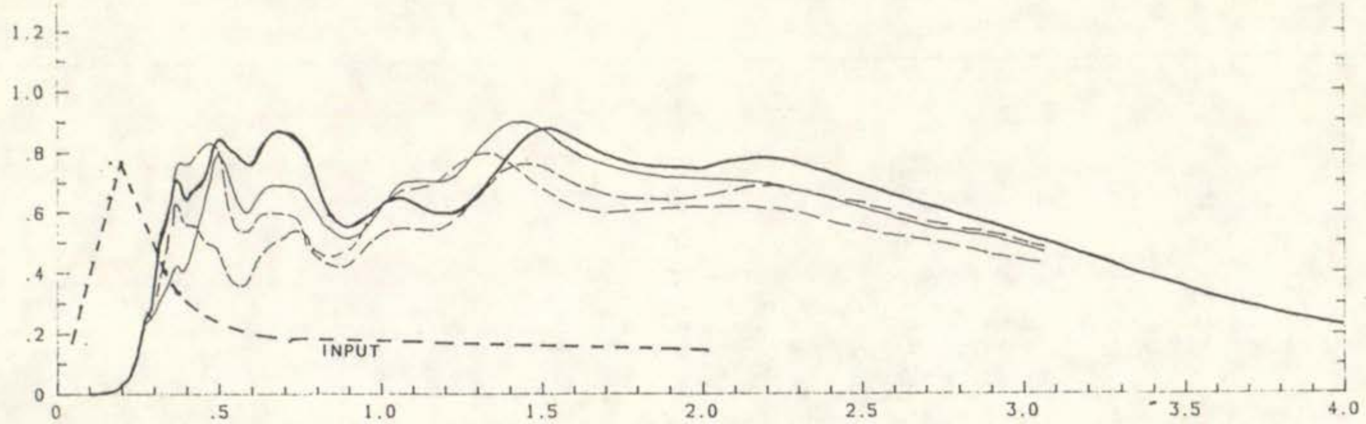
The results of the parameter study are used to classify all other profiles not modelled directly into one of the modelled categories. The result is that, for every node on the 500 metre grid described in section 7.4, the response spectra at the surface resulting from any design earthquake can be estimated.

In order to simplify results further for presentation and use, and recognising the limitations and assumptions inherent in producing the response spectra at the ground surface, all response spectra are further analysed and grouped into one of three categories according to the shape of the spectrum. Examples of actual response spectra calculated in each of these three categories are shown in Figure 7.16, for an input bedrock spectrum with a peak spectral acceleration of 0.75 g at a period of about 0.2 seconds. The modification in spectral shape is extremely pronounced.

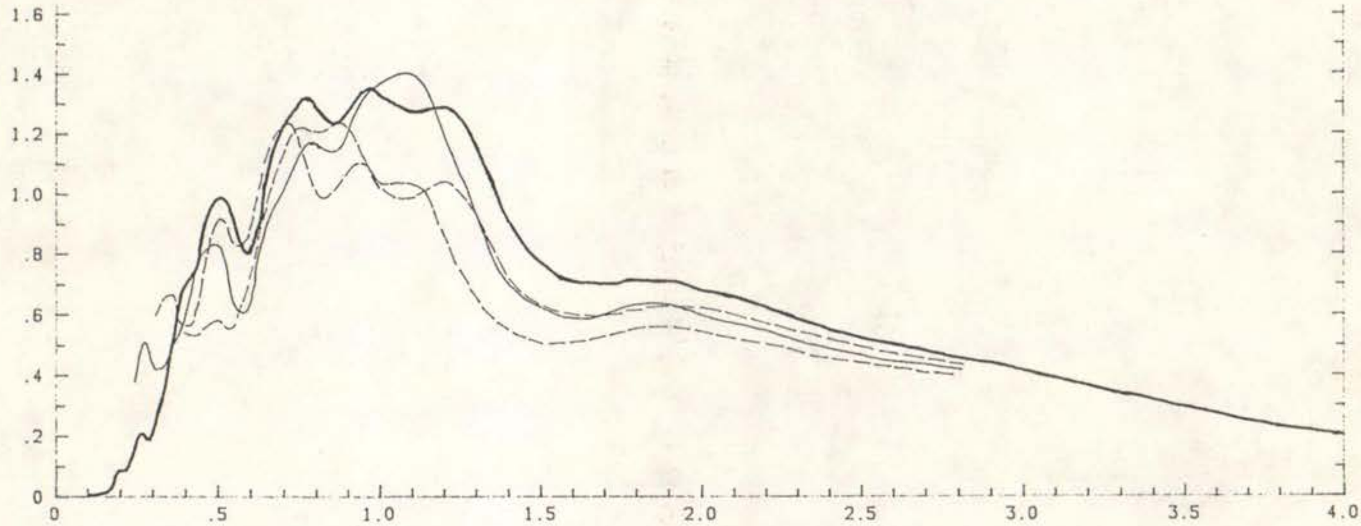
Spectral shape type A occurs where soils from 10 - 20 metres depth are predominately fine grained. For this spectral shape, shown in Figure 7.16(a), the peak acceleration has increased considerably from the maximum value of 0.75 g at bedrock. Equally important, however, is the very long range of period over which this peak now occurs, from 0.4 seconds to 1.5 seconds. For this type of spectral response shape there is very little decrease in spectral acceleration at longer periods.

Spectral type B, shown in Figure 7.16(b), occurs where soil from 10 - 20 metres is coarse-grained but soil from 5 - 10 metres depth is fine-grained. The peak acceleration increase is also pronounced, but occurs over a shorter period range, from about 0.7 - 1.2 seconds. The spectral acceleration decreases faster than for type A at higher periods.

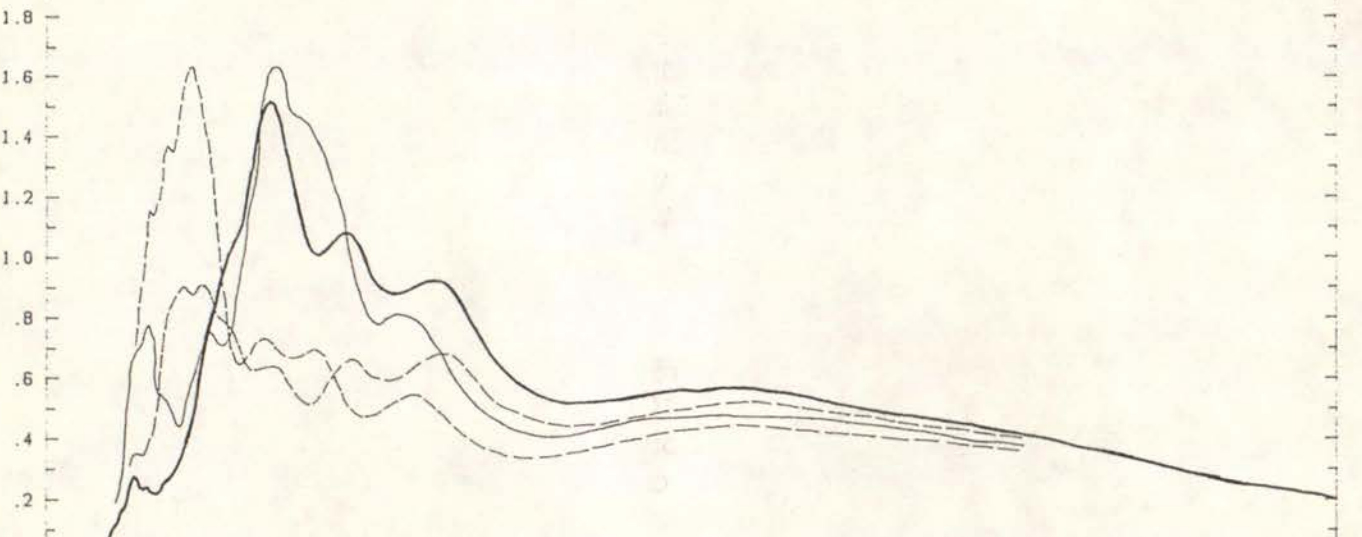
Spectral type C (Figure 7.16(c)) occurs where both layers 10 - 20 metres and 5 - 10 metres depth are coarse grained. A shorter, but often very much higher, peak spectral acceleration occurs between 0.4 and 0.7 seconds. However the post-peak drop is more rapid than for either of the other two cases.



(a) **Spectral Shape A - Fine Grained Soils, 10 - 20 m Depth**



(b) **Spectral Shape B - Fine Grained Soils, 5 - 10 m Depth**
- Coarse Grained Soils, 10 - 20 m Depth



(c) **Spectral Shape C - Coarse Grained Soils, 5 - 20 m Depth**

FIGURE 7.16

Response Spectra at the Ground Surface in Christchurch

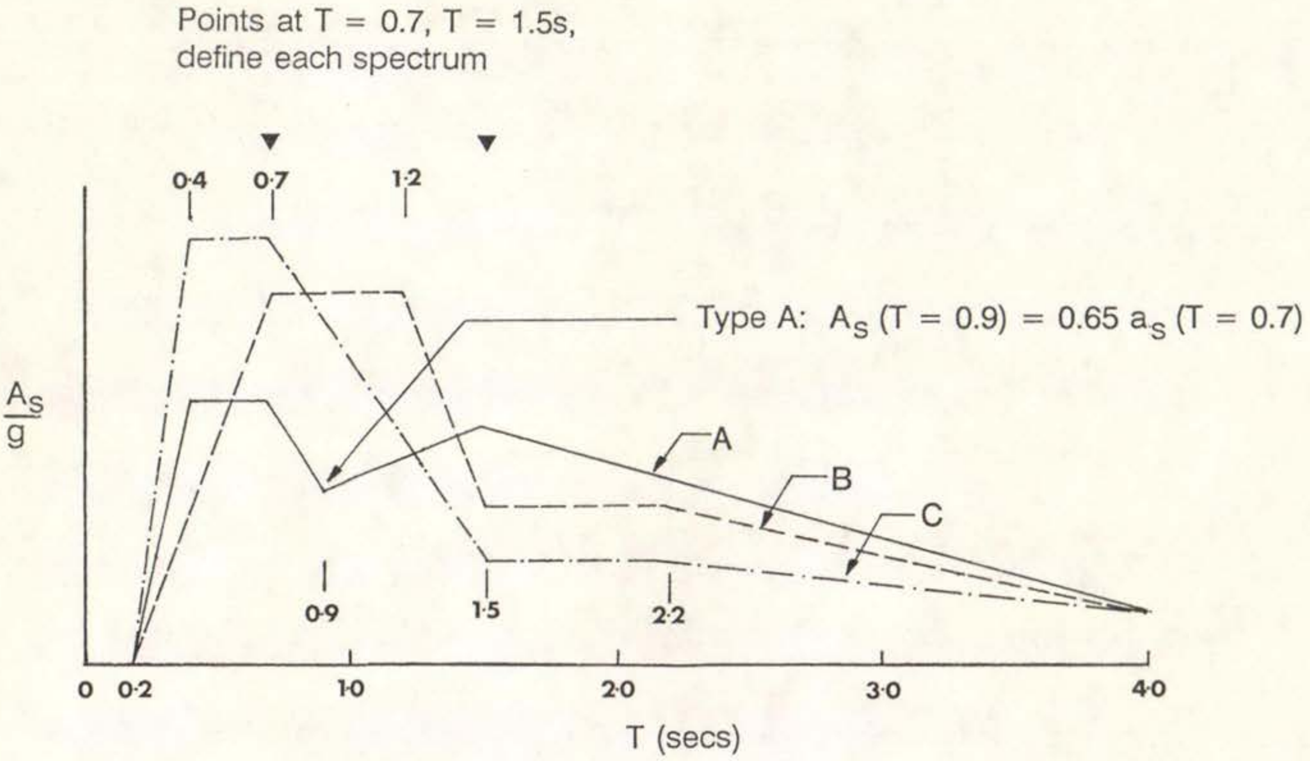


FIGURE 7.17

Response Spectra Envelopes at Christchurch Ground Surface

The areal extent of the three spectral types is shown in Figure 7.18. A zone 500 m wide along the base of the Port Hills and incorporating the alluvial filled valleys has been left unclassified as the depth to bedrock is likely to be less than 150 m, and focusing affects from underlying bedrock geometry make the deep soil response model as used for the rest of the area inappropriate. Shaking may be greater or less than for the general Christchurch area depending on the interaction of a number of factors. For one site that has been studied in this zone in Woolston, it is expected that the underlying bedrock configuration will give some seismic protection, and the resonant period of the surface sediments will be reduced relative to Christchurch generally (Dibble et al, 1980). The area on the hills will have a response close to the bedrock spectrum.

Idealisations of the three distinct spectral shapes, A, B and C are shown in Figure 7.17. Two points are common to all spectra, i.e. the two extreme values of $a_s/g = 0$ at period $T = 0.2$ seconds and $a_s/g = 0.2$ at $T = 4$ seconds respectively.

In order to determine the values defining the shape of the spectrum at any point (any structural response period) multiple regression analyses were performed among the spectral values and the various soil layer type characteristics at each location. It was found that the spectral magnitudes could be defined by two spectral accelerations for each spectrum; the peak spectral acceleration and a second, post peak value. These can conveniently be denoted a_s/g ($T = 0.7s$) and a_s/g ($T = 1.5s$), as can be seen from Figure 7.17. Each of these two spectral accelerations may be determined with sufficient accuracy simply as the product of six factors, of which five are uniquely determined by the soil type in one of the five modelled layers from 0 - 30 metres depth. The sixth multiplicative factor is determined by the area of the city (west, central or east) describing the soil profile type below 30 metres depth.

Using this simplified method, the peak and post peak spectral accelerations were calculated for each node on the 500 m grid for the upper bound bedrock spectrum defined by Figure 6.2. The values are shown graphically in Figures 7.19 (for $T = 0.7s$) and 7.20 ($T = 1.5s$). Peak spectral values for other probabilities of occurrence can be obtained by scaling the spectra by the ratio of the peak bedrock spectral acceleration at Christchurch for that probability (from Table 6.2) to the peak bedrock acceleration used above, $a_{peak} = 0.75$ g.

The value of the peak spectral acceleration is increased by a maximum factor of about 1.9 in the worst areas in Christchurch (Colombo Street north of Bealey Avenue, north end of Cranford Street, Parkhouse Road in Sockburn and south side of Hornby) with an average increase of 20% over the whole of the city.

Of equal significance is the change in period of the spectral accelerations. The acceleration response spectrum for the worst case and average case have been calculated for shaking of 150 year return periods, as shown on Figure 7.21a and b. Also shown is the equivalent bedrock response spectrum for this return period (Figure 7.21c) demonstrating the amplification effects both in peak and period of the deep alluvium. Included on Figure 7.21 is the basic seismic coefficient for Zone B, flexible subsoils from NZS 4203: 1984.

Return periods for different peak spectral accelerations are shown on Figure 7.22. The lower line for bedrock is taken directly from Figure 6.2, with the other lines showing the significant increase in occurrence for a particular peak spectral acceleration due to the amplification effects of the deep alluvium.

Shaking intensities will also be affected by the deep alluvium. An increase in intensity by 0 - 2 MM units is possible, i.e. felt intensity MM VI at bedrock may be increased to MM VII - VIII in Christchurch. However the change in period of the peak ground spectral acceleration may have a mitigating effect. As the Modified Mercalli scale is based in part on the scale of the resulting damage, the increase in intensity may not be reflected by a similar increase in damage.

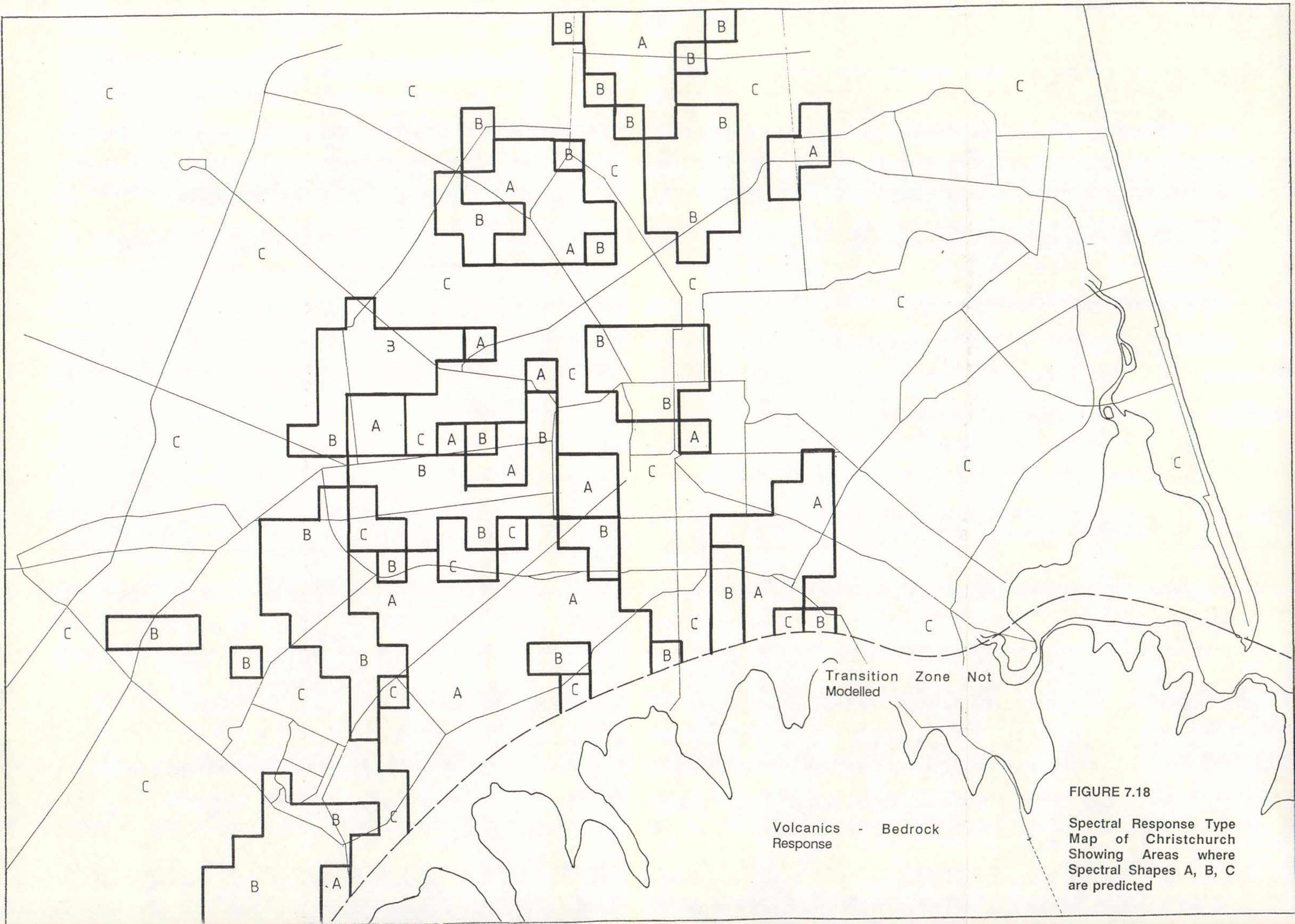


FIGURE 7.18

Spectral Response Type Map of Christchurch Showing Areas where Spectral Shapes A, B, C are predicted

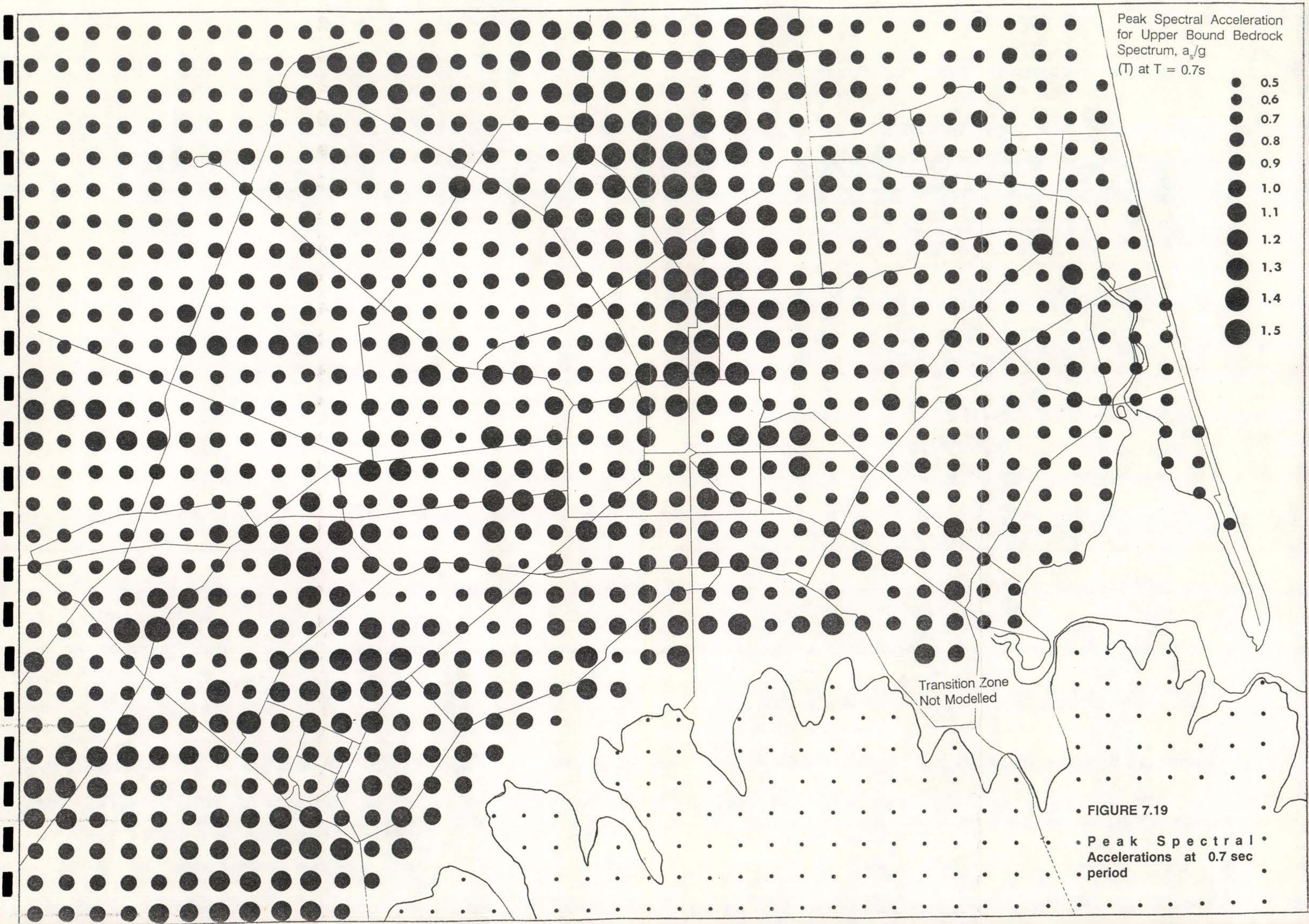


FIGURE 7.19

Peak Spectral Accelerations at 0.7 sec period

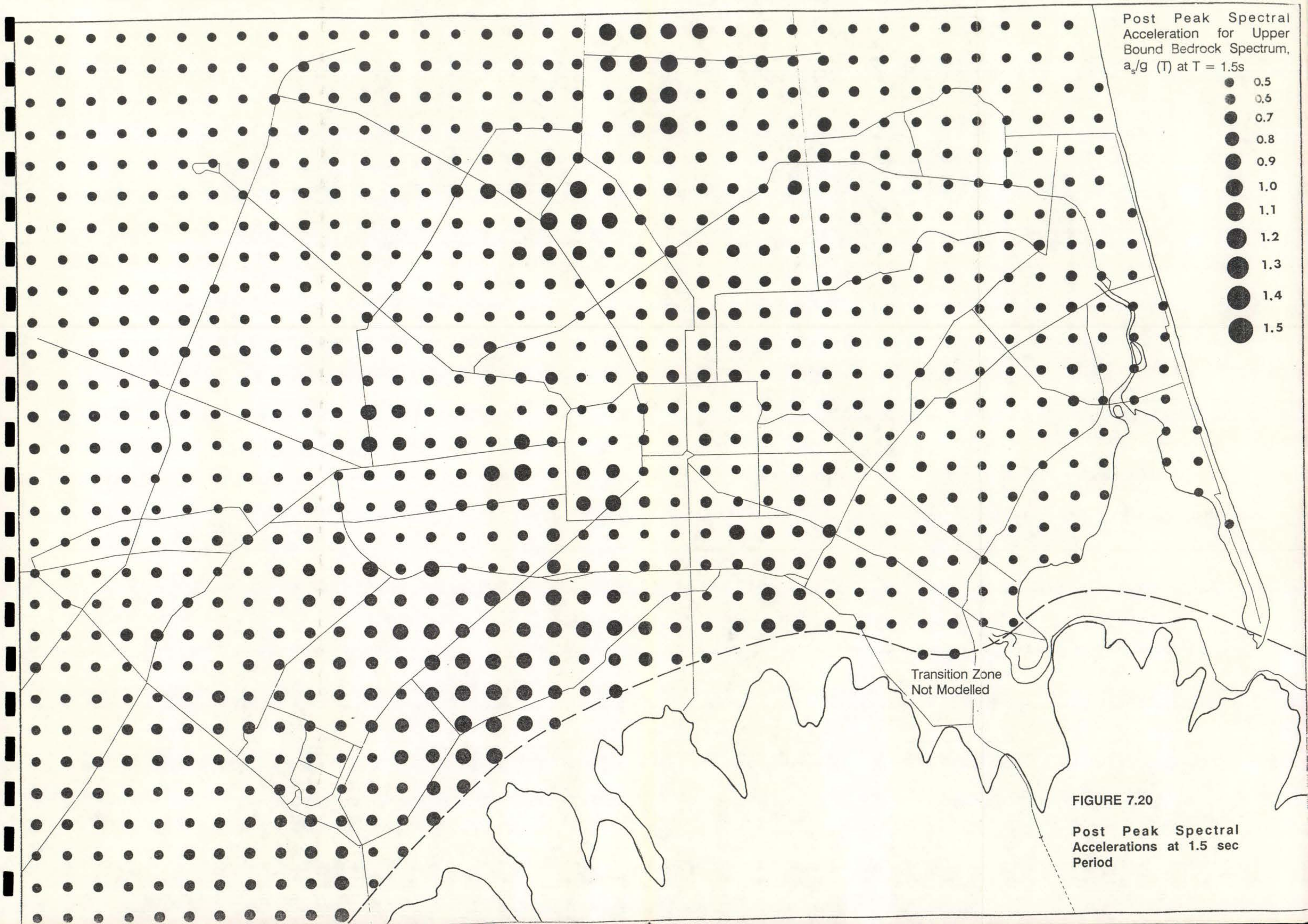
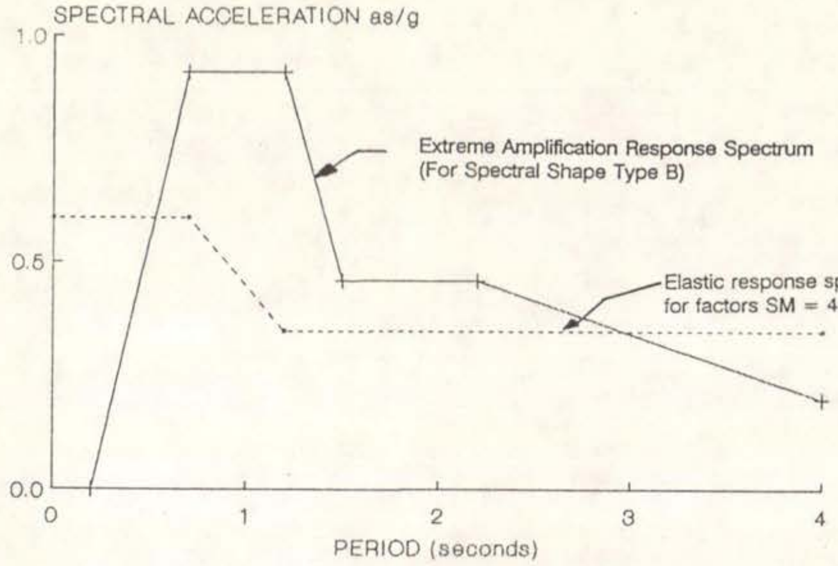
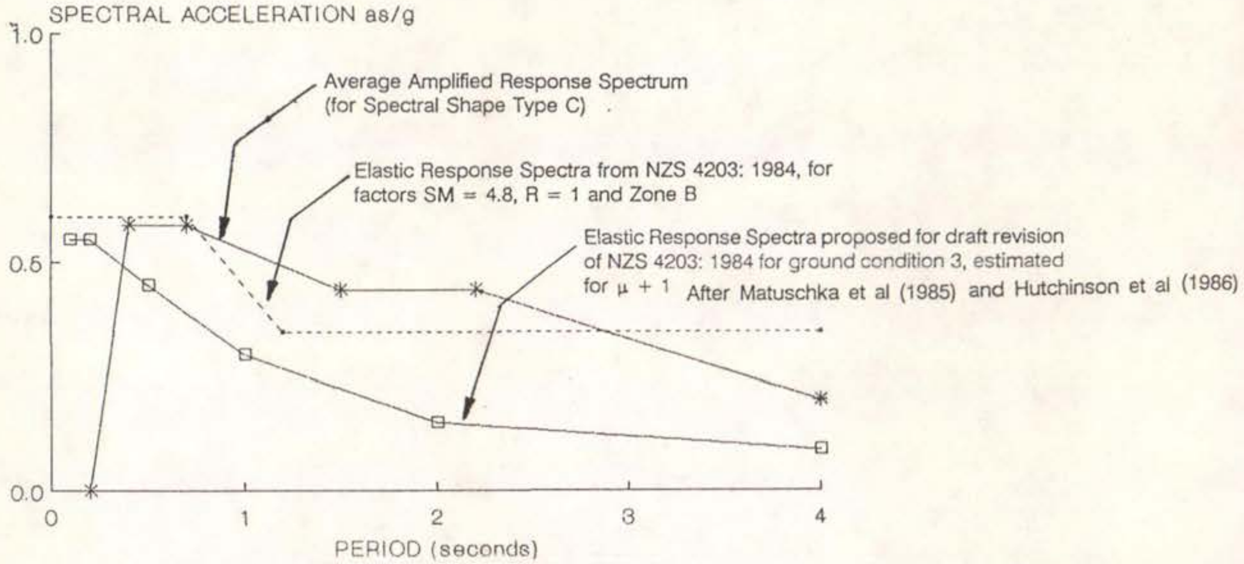


FIGURE 7.20

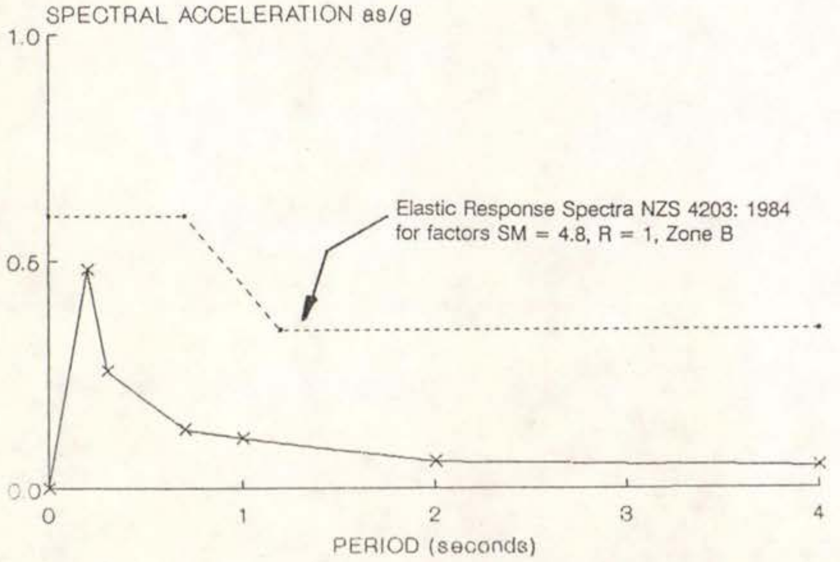
Post Peak Spectral Accelerations at 1.5 sec Period



(a) Worst Case Amplification



(b) Average Amplification



(c) Bedrock Response Spectra

FIGURE 7.21 **Response Spectra for Christchurch for 150 year return period, showing Amplification and comparison with Code**

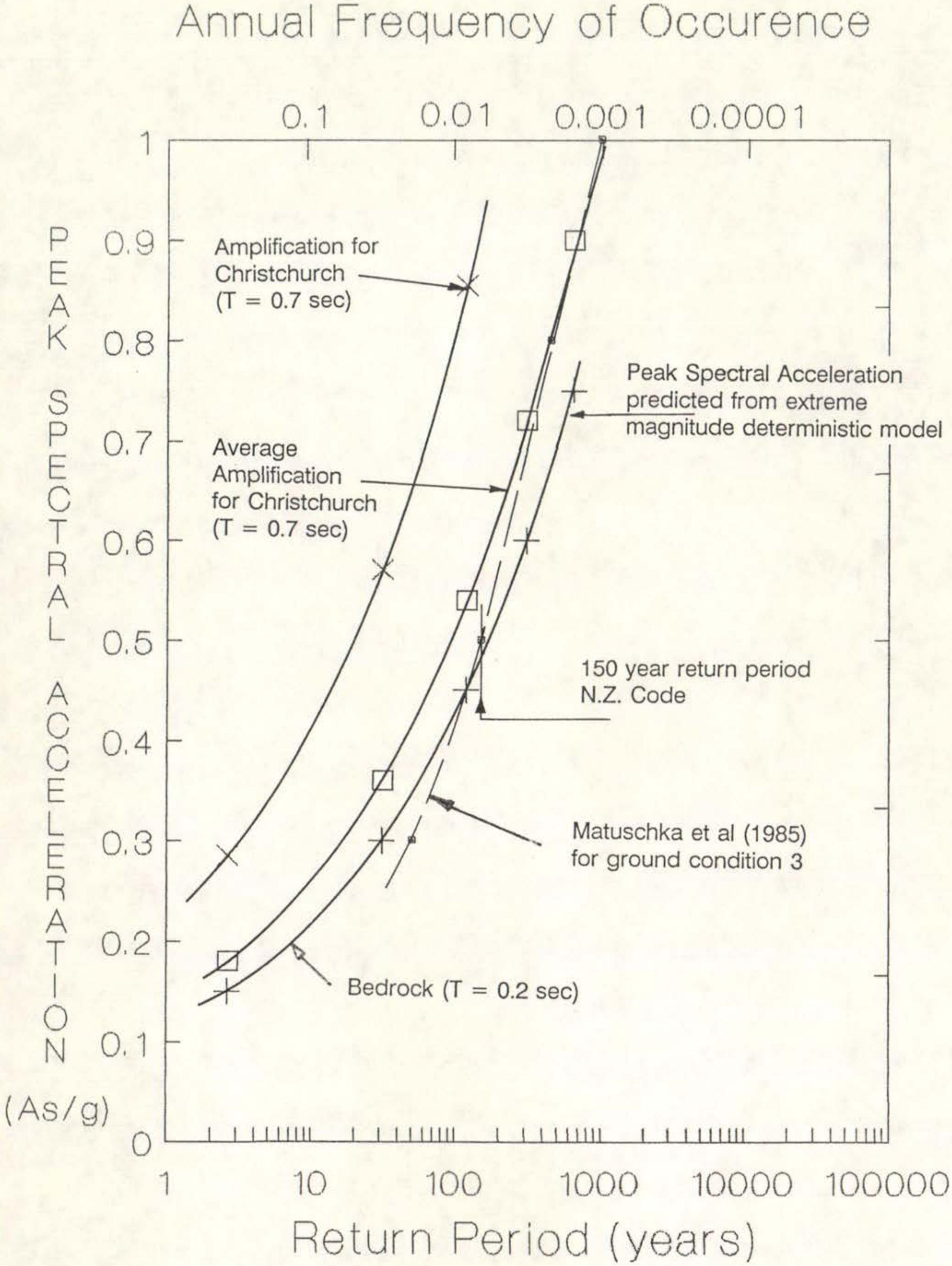


FIGURE 7.22 Annual Occurrence Frequencies and Return Periods for Peak Spectral Accelerations Amplified by Deep Alluvium for Christchurch

7.7 SUMMARY

The effect on structural response spectra at Christchurch of propagation of seismic waves through deep alluvium and soft rock beneath Christchurch has been considered in this chapter.

The geological profile beneath the city has been described and the lack of good quality data on soil properties below about 20 metres depth highlighted. The various sources of subsurface information used in this study have been discussed. The input used for the deep soil propagation model is based on a combination of known information and soil properties at shallow depths, likely geologic cross-sections at greater depths, and geophysical survey results extending beyond any borehole depths.

The recent drilling of a deep well at Bexley to about 450 metres depth has provided a useful calibration on assumed geologic sections, but a further deep borehole in central Christchurch, where the volcanics are likely to be deeper, would be of great value. If such a hole should be drilled in the future, geotechnical and seismic testing should form an integral part of the programme if adequate funding can be obtained.

A method has been developed and used in this study to predict the approximate acceleration response spectrum at any location in Christchurch if the soil profile to 30 metres depth at that location is known. This method has been used to generate maps of the city for two critical values of the a_s/g spectrum at response periods $T = 0.7$ seconds and $T = 1.5$ seconds.

The results of the seismic wave propagation analysis show that significant modification occurs to the response spectra between the bedrock and the ground surface. These modifications are far greater than those which would be expected if the current New Zealand Loadings Code were used for prediction of these effects. The effects are generally threefold:

- A total removal of very short period accelerations from the response spectrum by hysteretic damping through the deep, relatively soft soil.
- A general increase in the peak spectral accelerations at the ground surface from those at bedrock (although in some cases reductions are predicted by the model).
- A shift in the spectral values towards the longer period region of the spectrum.

SECTION III. EARTHQUAKE HAZARD: CONSEQUENCES FOR CHRISTCHURCH AND POTENTIAL DAMAGE

CHAPTER 8: Potential Consequences and Hazards

The preceding chapters have described the seismicity model for the northern South Island (Chapter 4), assessed the probabilities for different shaking intensities for "average" ground conditions (Chapter 5), modelled a bedrock response spectra (Chapter 6) and analysed the influence of the deep alluvial deposits beneath Christchurch on the intensities and ground motion (Chapter 7).

The foregoing analysis has shown that Christchurch is subject to a higher level of seismic hazard than has been generally accepted previously. This has arisen both from the more detailed study of the regional seismicity and geological evidence of faulting, and from consideration of the amplification effects of the deep alluvium. It is apparent from the intensity predictions that Christchurch should at very least be included in Zone A of the N.Z. Loadings Code, rather than Zone B as at present, and that the code design response spectra is not appropriate to the deep alluvial soils of Christchurch.

Christchurch must expect shaking of up to MM VIII at reasonably frequent intervals (with predicted return period 100 years for bedrock conditions and 55 years allowing for amplification effects in the deep alluvium), and severe shaking of greater intensity less frequently. The degree of shaking will not be constant throughout Christchurch, and damage is likely to vary between areas, and different types of structure. The amplification effects determined in Chapter 7 indicate that stiff structures with short natural periods (such as houses) will not be subject to resonance effects on the flat where the response spectra has virtually no acceleration component below 0.2 sec period, but houses will be affected on the hills. Structures in the five to seven storey range are likely to be affected much more by resonance, particularly in the northern central city, St Albans, Addington and Sockburn areas than are equivalent structures at New Brighton or the airport (refer Figure 7.19; relative response accelerations at 0.7 sec period).

Taller buildings 10 to 20 stories high will be more severely shaken if they are located in Papanui, Redwood, Spreydon or Waltham than in the north-western or eastern suburbs (refer Figure 7.20, relative response accelerations at 1.5 sec period). This effect has been most recently indicated by the Hanmer earthquake of February 1989. Shaking from this event was noticeable throughout Christchurch at ground level, but generated little alarm. Being a Saturday, few people were in higher office buildings, but it appears that those who were experienced a much stronger, slower shaking. One person in the top floor of a six storey building at Ilam experienced great alarm sufficient to cause him to take shelter under his desk. His wife in a house on a hill suburb did not notice the shaking at all.

The historic record provides little indication of what the consequences for modern Christchurch are likely to be from a major seismic event. The earthquake with greatest effects in Christchurch (intensity MM VII - VIII) was in 1869 and produced general damage to masonry in what was then a small town of generally small, timber framed buildings (refer 4.4 above). Significantly, most damage was reported from the area north of the Avon River and east of Papanui Road, which shows up clearly on our maps (Figure 7.19) as an area of extreme amplification for response accelerations.

From 1840 to 1930 the number of Christchurch earthquakes with felt intensity ≥MM VII was at least five, but since 1930 there has been none. Thus there has not been shaking at a damaging level in the past 60 years, in a period where the population of the urban area has grown from about 120,000 to 300,000, and the infrastructure and services have grown substantially in extent, size and complexity. As a further result of this growth, Christchurch now has taller structures which are susceptible to longer period shaking.

In addition to a level of shaking comparable to the 1869 event, Christchurch will at some time in the future almost certainly experience shaking of an intensity that has not been experienced in the period since European settlement. The consequences of such a major event are uncertain. Liquefaction of loose, saturated sands and silty sands is likely to occur, and could affect a substantial portion of the city, resulting in loss of foundation support to structures, disturbance to ground surfaces, damage to buried services, and lateral ground spreading adjacent to rivers and estuaries. The liquefaction hazard is discussed further in Chapter 9. Earthquakes are known to initiate landslides and rockfalls, which could affect the hill suburbs and Lyttelton. Slope instability hazards are discussed in detail in Chapter 10. General subsidence from compaction of granular soils could conceivably occur in Christchurch and has been observed in a number of large earthquakes, such as in Alaska (1964) and Nigata, Japan (1964). In general all works of construction, such as buildings, overhead and underground services, roads, railways, bridges and embankments, are potentially vulnerable to damage from various causes. Potential damage is discussed briefly in Chapter 11.

CHAPTER 9: GROUND INSTABILITY: LIQUEFACTION

9.1 INTRODUCTION

Liquefaction is a common consequence of moderate to large earthquakes. It can be defined as

> *"the transformation of a granular material from a solid state into a liquefied state as a consequence of increased pore pressures"*

(Youd, 1973).

If the liquefaction in any event is sufficiently severe and extensive, loss in ground strength may result in damage to any proximate structures. Bearing capacity failure will cause buildings or superficial structures to settle and tilt, and buoyant, buried structures may float upwards. Liquefaction of a confined, subsurface stratum can cause both vertical and lateral displacement of surficial blocks of soil, and if the area is on a gentle slope, or close to a free face such as an incised river channel or open drain, then lateral spread of several metres can occur.

Liquefaction hazard is site dependent; certain soil profiles are more liquefiable than others. Liquefaction is most likely to occur in saturated, relatively uniform fine sands or coarse silts in a loose state, at depths less than 10 to 15 m below ground level, where the groundwater level is within about 5 m of the ground surface.

A detailed study of the liquefaction potential in Christchurch has not yet been carried out. However the near surface soil profiles compiled as part of this report clearly indicate that at least one-third of the Christchurch urban area overlies a general soil profile dominated by sand, and having a high water table (generally within 2 m of the ground surface). In these areas liquefaction is likely to be a hazard. Unfortunately few specific sites have sufficient data available on the uniformity characteristics of the sands and in situ densities to allow definitive analysis through the city at present.

This section is therefore a review of the limited information pertaining to the liquefaction hazard in Christchurch. It concludes that this hazard is significant, and indicates areas of further research required to more accurately define the hazard. It was not within the scope of this study to carry out this work or to complete a comprehensive analysis for the city.

9.2 HISTORICAL EVIDENCE FOR LIQUEFACTION IN NORTH CANTERBURY

There is little historical evidence of liquefaction occurring in Christchurch. The most destructive earthquake effects experienced in Christchurch since European settlement were during an earthquake (M = 5.5 - 6.0) on 4th June 1869, centred 10 km east of the city. It caused considerable damage to chimneys, masonry and household contents but there are no reports of liquefaction. This may reflect the low population and sparse nature of development at the time, but the size of the earthquake was probably insufficient for widespread liquefaction (Dibble et al, 1980).

Earthquakes have damaged chimneys in Christchurch and the Cathedral spire on at least three occasions: 5th December 1881 (M = 6.25, 80 km from Christchurch near Castle Hill), 1st September 1888 (M = 7.0 - 7.3, 80 km distant on the Hope Fault), and 16th November 1901 (M = 6.5, at a distance of over 100 km). Liquefaction effects were reported in the Hanmer area in 1888 and in the Cheviot, Hurunui and Kaiapoi areas in 1901. A common relationship between earthquake magnitude and epicentral distance (see Figure 9.4) would not predict liquefaction in Christchurch for any of these events, and the 1901 liquefaction at Kaiapoi was at the furthest distance from the epicentre that liquefaction could be expected.

The Motunau earthquake (M = 6.75) on 25th December 1922 caused liquefaction at Waikuku and Leithfield beaches. It appears from press reports that water ejection occurred behind the Waikuku sandhills, and loss of ground support caused a tree to topple, and motor cars to become bogged. Again, the earthquake magnitude and distance were such that liquefaction effects much closer to Christchurch than Waikuku would not be expected.

Other large earthquakes (1929 Arthurs Pass, 1929 Murchison and 1968 Inangahua) were all too distant for liquefaction in Christchurch to be expected.

The most widespread occurrences of liquefaction in New Zealand since 1943 were caused by the 1848 Marlborough, 1855 Wairarapa, 1931 Napier, and 1987 Edgecumbe earthquakes. All these earthquakes occurred in coastal regions with plentiful fine-grained, recent alluvial deposits (Fairless & Berrill 1984). Generally liquefaction in New Zealand has been reported for earthquakes of magnitude 6.9 or greater (Edgecumbe, M = 6.3, was smaller, but unusually shallow).

It is therefore apparent that Christchurch has not experienced earthquake shaking severe enough to cause liquefaction since European settlement.

9.3 LIQUEFACTION SUSCEPTIBILITY

Liquefaction usually occurs in saturated, cohesionless, granular sediment, which is in a relatively loose state and at depths of less than 10 to 15 metres. The potential susceptibility varies according to its depositional history and age. Table 9.1 summarises this information for the type of materials found in the Christchurch area.

Table 9.1: Estimated Susceptibility of Sedimentary Deposits to Liquefaction During Strong Seismic Shaking

Type of Deposit	General Distribution of Cohesionless Sediments in Deposits	Likelihood That Cohesionless Sediments, When Saturated, Would be Susceptible		
		Age <500 Year	Holocene	Pleistocene
River channel	Locally variable	V. high	High	Low
Floodplain	Locally variable	High	Moderate	Low
Alluvial fan and plain	Widespread	Moderate	Low	Low
Dunes	Widespread	High	Moderate	Low
Coastal delta	Widespread	V. high	High	Low
Estuarine	Locally variable	High	Moderate	Low
Beach - high wave energy	Widespread	Moderate	Low	V. low
- low wave energy	Widespread	High	Moderate	Low
Lagoonal	Locally variable	High	Moderate	Low
Foreshore	Locally variable	High	Moderate	Low
Uncompacted fill	Variable	V. High		

(After Youd & Perkins (1978))

Generally, the prediction of liquefaction involves two steps:

(a) Evaluation of liquefaction susceptibility: Identifying those areas or layers which have the characteristics of a liquefiable soil (i.e. loose, saturated, cohesionless and above 15 m depth).

(b) Evaluation of liquefaction opportunity: Identifying the relative probability for earthquake shaking strong enough to generate liquefaction in susceptible materials, based on an appraisal of the regional earthquake potential.

The liquefaction susceptibility and opportunity are considered together to determine the liquefaction potential or the relative likelihood that liquefaction will occur. The liquefaction opportunity is derived directly from the seismic models considered earlier in this report. The liquefaction susceptibility in Christchurch is considered in this section.

(a) Liquefaction occurs in relatively uniform cohesionless fine sands to silty sands, where the permeability is relatively low and drainage slow. Much of the Christchurch area is underlain with sand and fine grained material. The extent of sand and silt underlying Christchurch shown on Figure 9.1. The map delineates soils in four categories:

    (a) Predominantly sand (silty sand to sand), depth 2-10 m.
    (b) Predominantly fine grained - clayey silts to sandy silts, depth 2-10 m.
    (c) Predominantly sand - silty sand to sand, depth 2-5 m.
    (d) Predominantly fine grained - clayey silts to sandy silts, depth 2-5 m.

    This map does not represent areas of liquefiable soils as an appropriate soil size classification is only one of a number of conditions required.

    Figure 9.2 shows typical particle size distribution curves from a number of sites in Christchurch, which can be seen to lie within the envelope for liquefiable soils.

(b) To be susceptible the soil must also be below the groundwater level, and generally the groundwater level must be within a few metres of the ground surface. Christchurch has a high groundwater level, and virtually the whole city east of a line from Halswell - Ilam - Bishopdale has a water table within 2 m of the ground level. Contours of depth to water table are shown on Figure 9.3 (from Talbot et al, 1986).

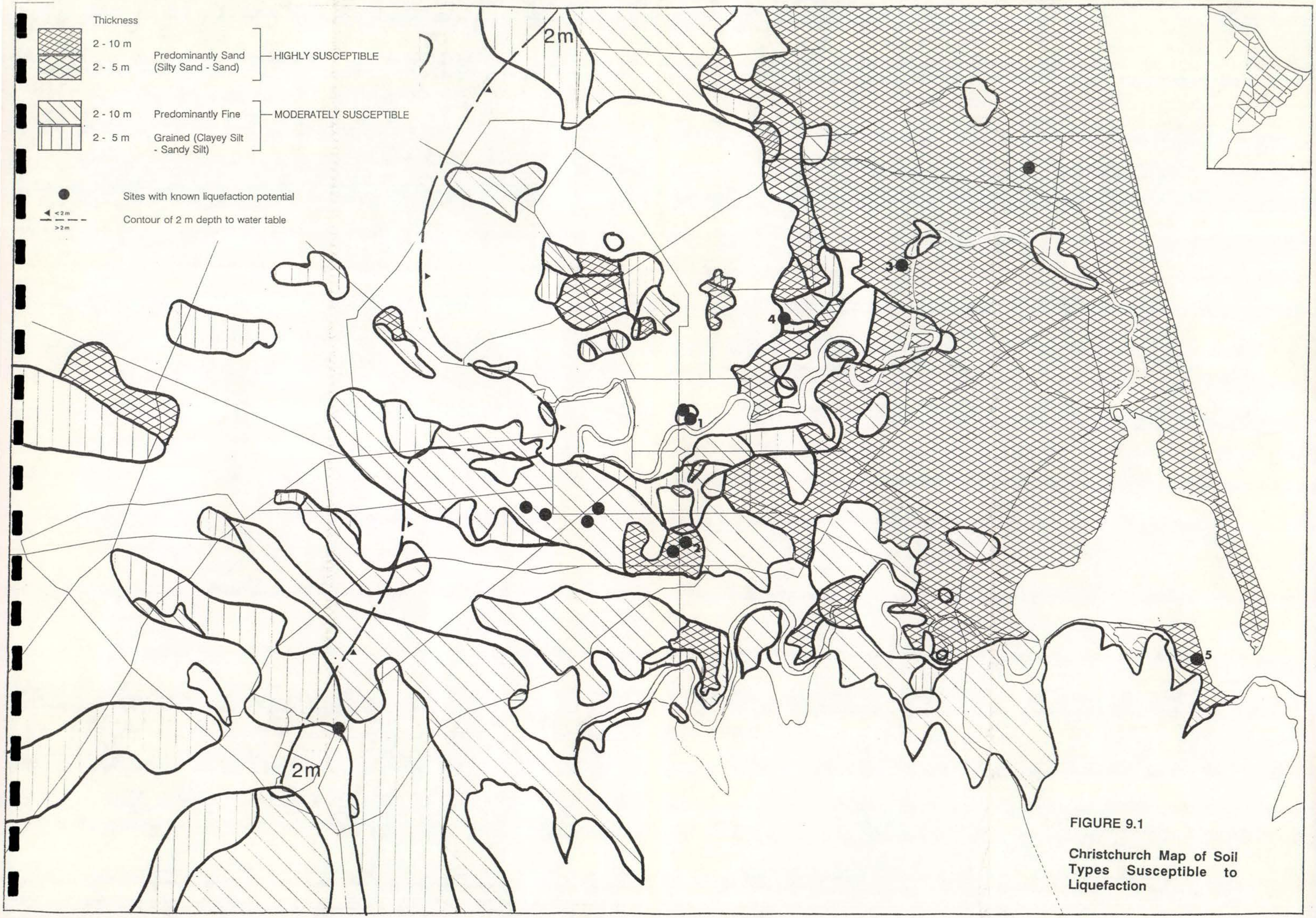
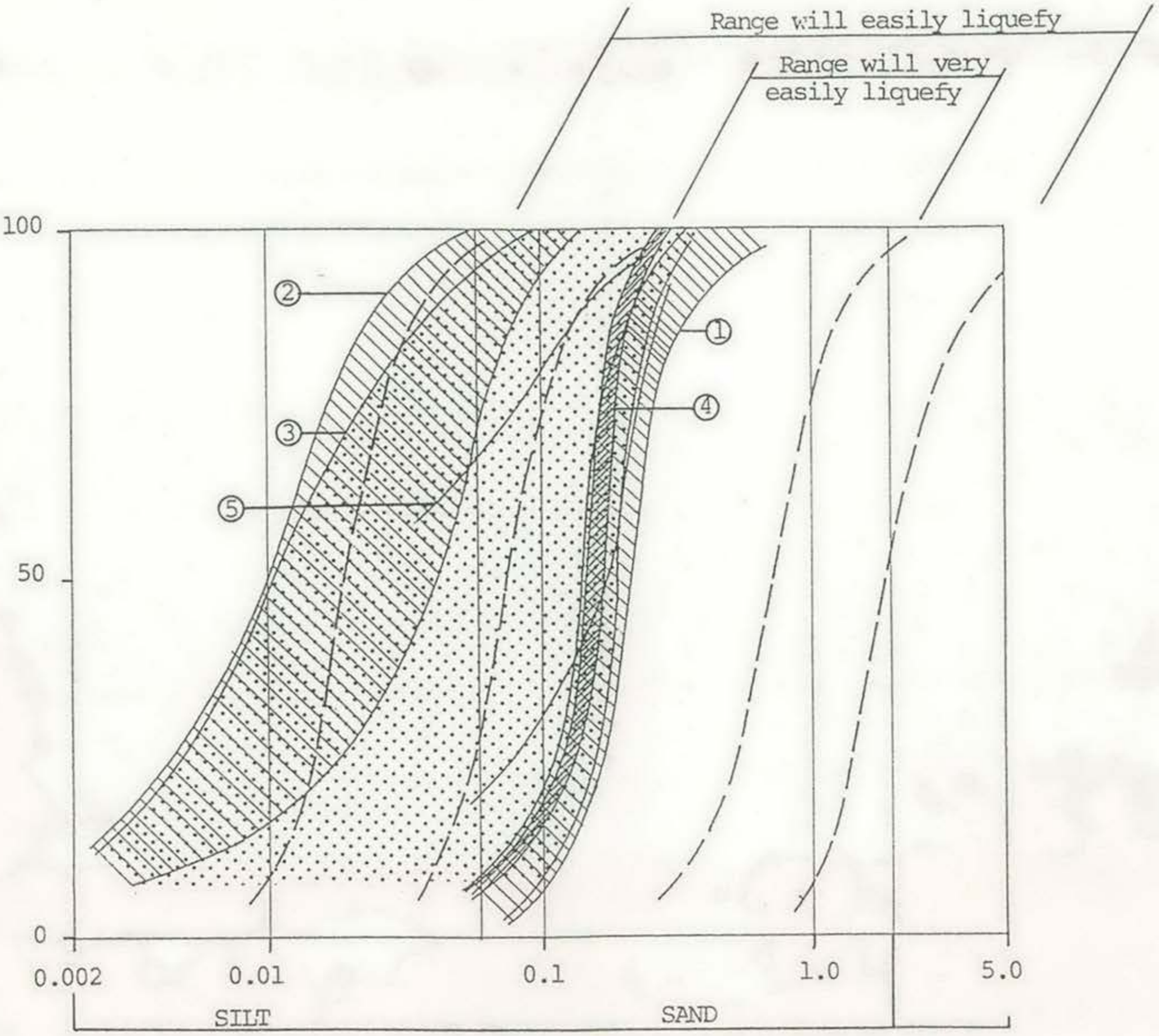


FIGURE 9.1

Christchurch Map of Soil Types Susceptible to Liquefaction



Particle Size Distribution Curve & Liquefaction Susceptibility
Grading curve envelopes are for a limited number of sites
Note: Source for liquefaction Susceptibility, Tsuchida (1971).

SITES (see figure 9.1 for locations)

① Central City, Kilmore Street
Sands between 2m & 13m depth

② Sydenham, Colombo Street
Silts between 1m & 6m depth

③ Burwood
Sands & Silts between 2m & 5m depth

④ Shirley
Sands between 1m & 4m depth

⑤ Redcliffs
Sands between 1m & 2m depth

FIGURE 9.2

Particle Size Distribution Curves and Liquefaction Susceptibility

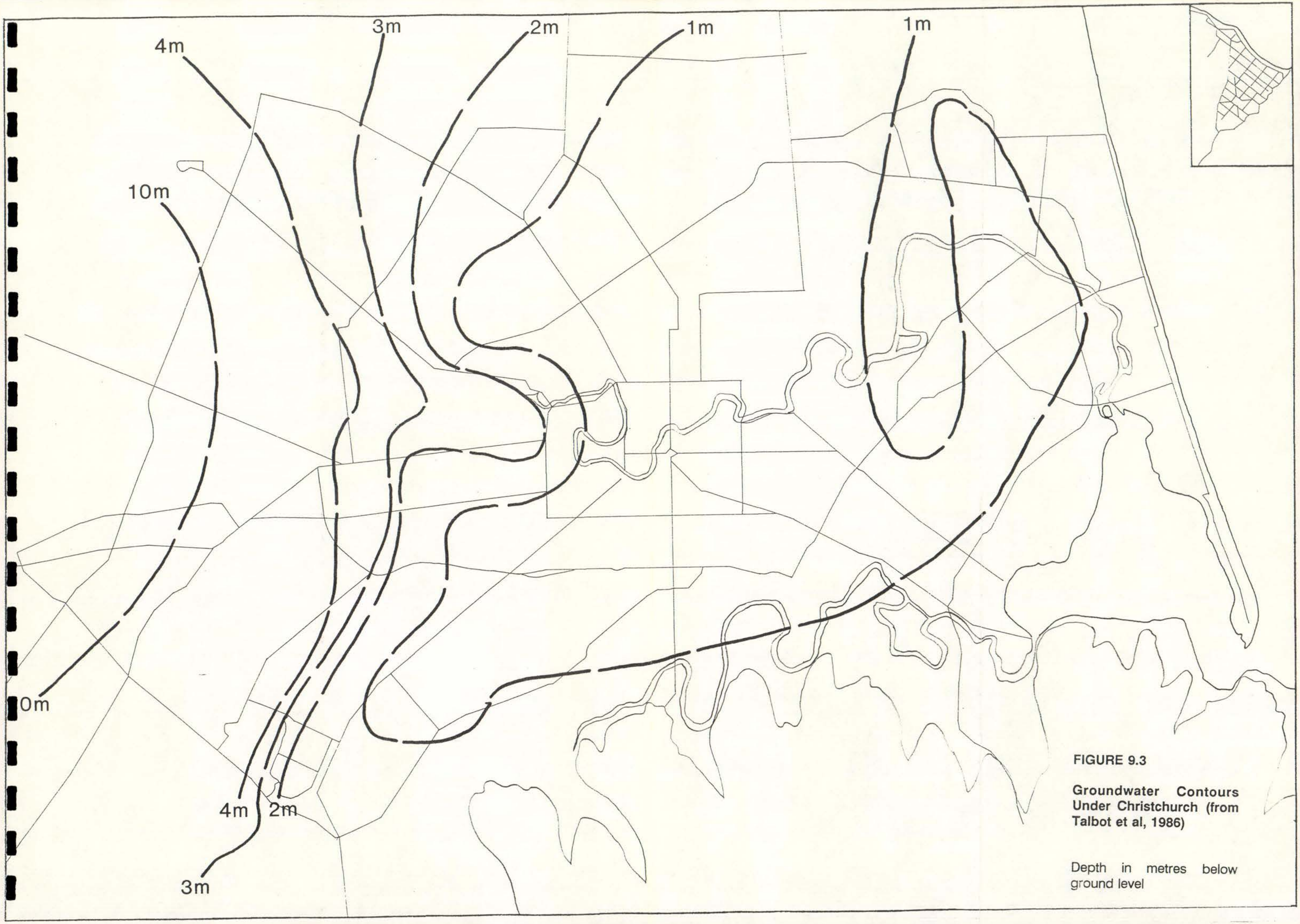


FIGURE 9.3

Groundwater Contours Under Christchurch (from Talbot et al, 1986)

Depth in metres below ground level

(c) Relative Density

Liquefaction is observed only in relatively loose soils. While dense sands and silts may show initial liquefaction, this is rapidly inhibited due to the dilatancy characteristics of such materials. Liquefaction research has commonly related liquefaction potential to in situ test results, in particular the Standard Penetration Test (SPT). The most common methods of liquefaction analysis use SPT data.

As discussed above (Table 9.1), the relative density of the soils will depend on the manner of deposition, and not all the area shown as complying with the general grain size requirement in Figure 9.1 will be liquefiable. Much of the Brighton area soil was probably deposited in a high wave energy beach environment, and former beach deposits of a similar nature now distant from the coast are unlikely to be liquefiable.

There is very limited information on relative densities of soils in Christchurch below the water table except in the central city area. It is therefore not possible to delineate areas of looser soils across the city. A number of sites where test information is available, and where liquefaction hazard has been identified by specific studies carried out by us prior to 1991, are shown on Figure 9.1.

9.4 LIQUEFACTION ANALYSIS

There are four general methodologies for liquefaction prediction, although all overlap somewhat.

- Case history methods, where earthquake magnitude required to produce liquefaction is related to distance from the epicentre.

- In situ soil data methods, using empirical relationships between test data (usually SPT 'N' values) and depth, for given degrees of shaking.

- Methods involving comparison between dynamic shear strength and earthquake induced stress, modified for earthquake magnitude, soil size grading and the thickness of surface confinement, and using ground accelerations.

- Methods using theory of excess pore water pressure generation, relating pore water pressure to dissipated energy, and hence earthquake magnitude and distance.

The first approach is typified by the method proposed by Kuribayashi & Toitsuoka (1975) where Japanese liquefaction observations are included on a plot of earthquake magnitude vs epicentral distance. The line defining the distance from the epicentre to the farthest site of liquefaction is given by

$$\log_{10} R_{max} = 0.77M - 3.6 \quad (R_{max} \text{ in km})$$

New Zealand data has also been plotted (Fairless & Berrill, 1984) and found to approximate this relationship. This gives some confidence in applying this relationship to New Zealand, and the line defining maximum distance for liquefaction is shown in Figure 9.4 (reproduced from Figure 4.5), showing probable earthquake magnitudes for the seismic zones in the Christchurch region. This plot clearly indicates that maximum magnitude earthquakes in the Pegasus, Banks Peninsula, and Canterbury Plains Seismicity Zones, or the Porters Pass Tectonic Zone, and the central section of the Alpine Fault are all capable of inducing liquefaction in susceptible soils within Christchurch.

The second approach is illustrated by the empirical relationship commonly referred to in New Zealand as the "Chinese" method. The method identifies a threshold value of SPT 'N', below which liquefaction can be expected to occur.

$$N_{crit} = N_o [1 + 0.125 (dx - 3) - 0.05 (dw - 2)]$$

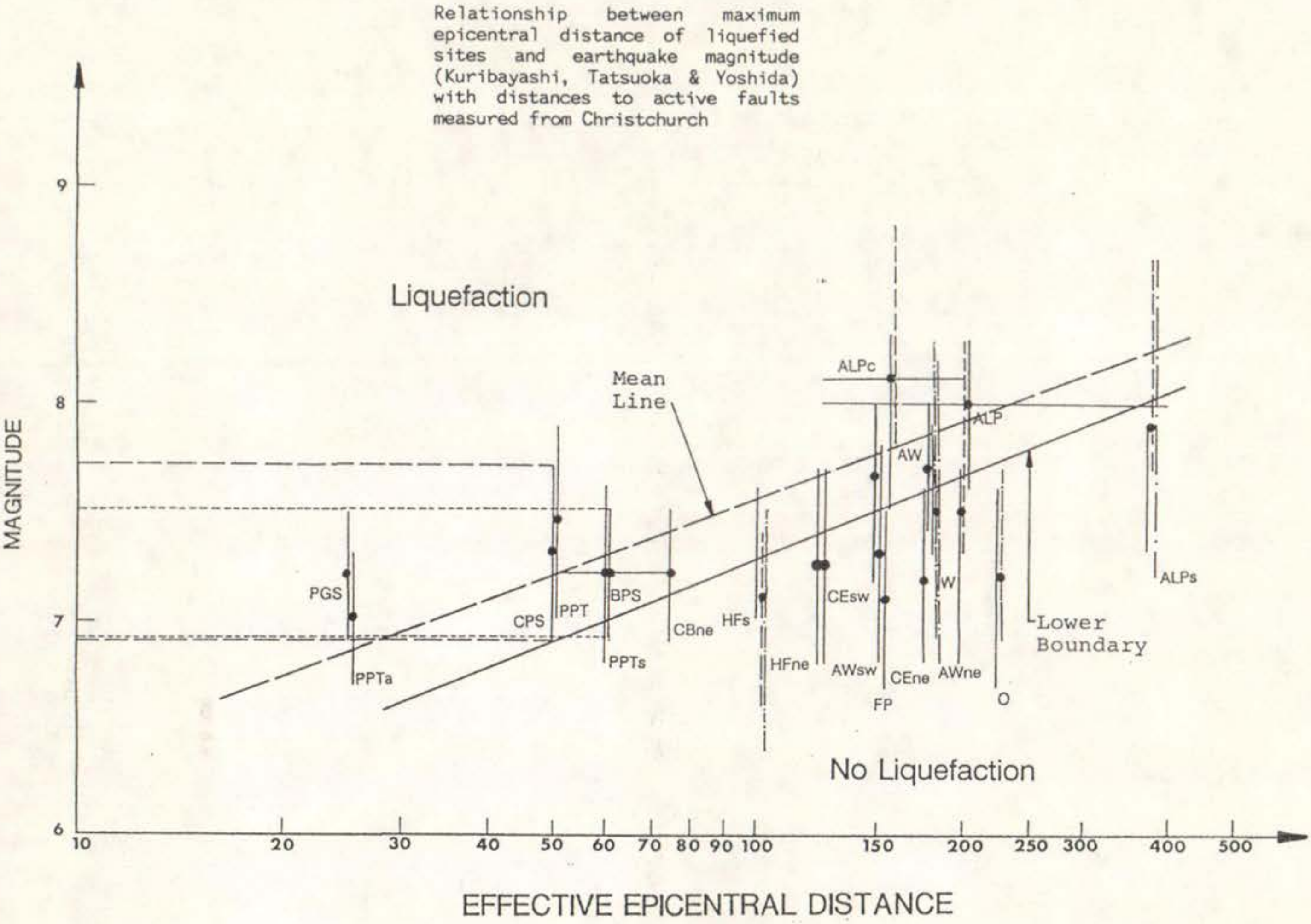
in which dx equals the depth to the layer being considered in metres, dw is the depth to the water table, and N_o is a function of shaking intensity.

A plot of N_{crit} versus depth for intensities MM VII and MM VIII is shown on Figure 9.5 which includes SPT N values (corrected) for two sites: one in Kilmore Street in the central city, and the second near Horseshoe Lake in the eastern city. Clearly the site N values are predominantly less than the N_{crit} values. This method predicts that liquefaction would occur at these two sites for earthquake shaking greater than MM VII (return period of about 20 years allowing for some increase in intensity from deep alluvium amplification).

The third approach compares dynamic shear strength of the soil and earthquake induced stress. The method, described by Seed et al (1983), uses the cyclic stress ratio, which is the ratio of the average cyclic shear stress τ_n developed as a result of the earthquake loading, to the initial vertical effective stress σ_o, and which can be computed from

$$\frac{\tau_n}{\sigma_o} = 0.65 \quad \frac{A_{max}}{g} \cdot \frac{\sigma_o}{\sigma'_o} \cdot r_d$$

where A_{max} = maximum acceleration at the ground surface, σ_o = total overburden pressure on the layer under consideration, σ'_o = initial effective overburden stress, and r_d = stress reduction factor varying from 1 at the ground surface to a value of 0.9 at a depth of 10 m.



Key

CPS	Cant. Plains Seismic
BPS	Banks Pen. Seismic
PGS	Pegasus Seismic
PPT	Porters Pass TZ
PPTa	- Ashley section
PPTs	- Southern section
CBne	Kaiwara
HF	Hope
HFs	- SW section
HFne	- NE section
CE	Clarence/Eliot
CEsw	- SW section
CEne	- NE section
AW	Awatere
AWsw	- SW section
AWne	- NE section
W	Wairau
FP	Fox's Peak
O	Ostler
ALP	Alpine Fault
ALPc	- Central section
ALPs	- South section

FIGURE 9.4

Relationship Between Maximum Epicentral Distance of Liquefied Sites and Earthquake Magnitude (after Kuribayashi et al, 1975)

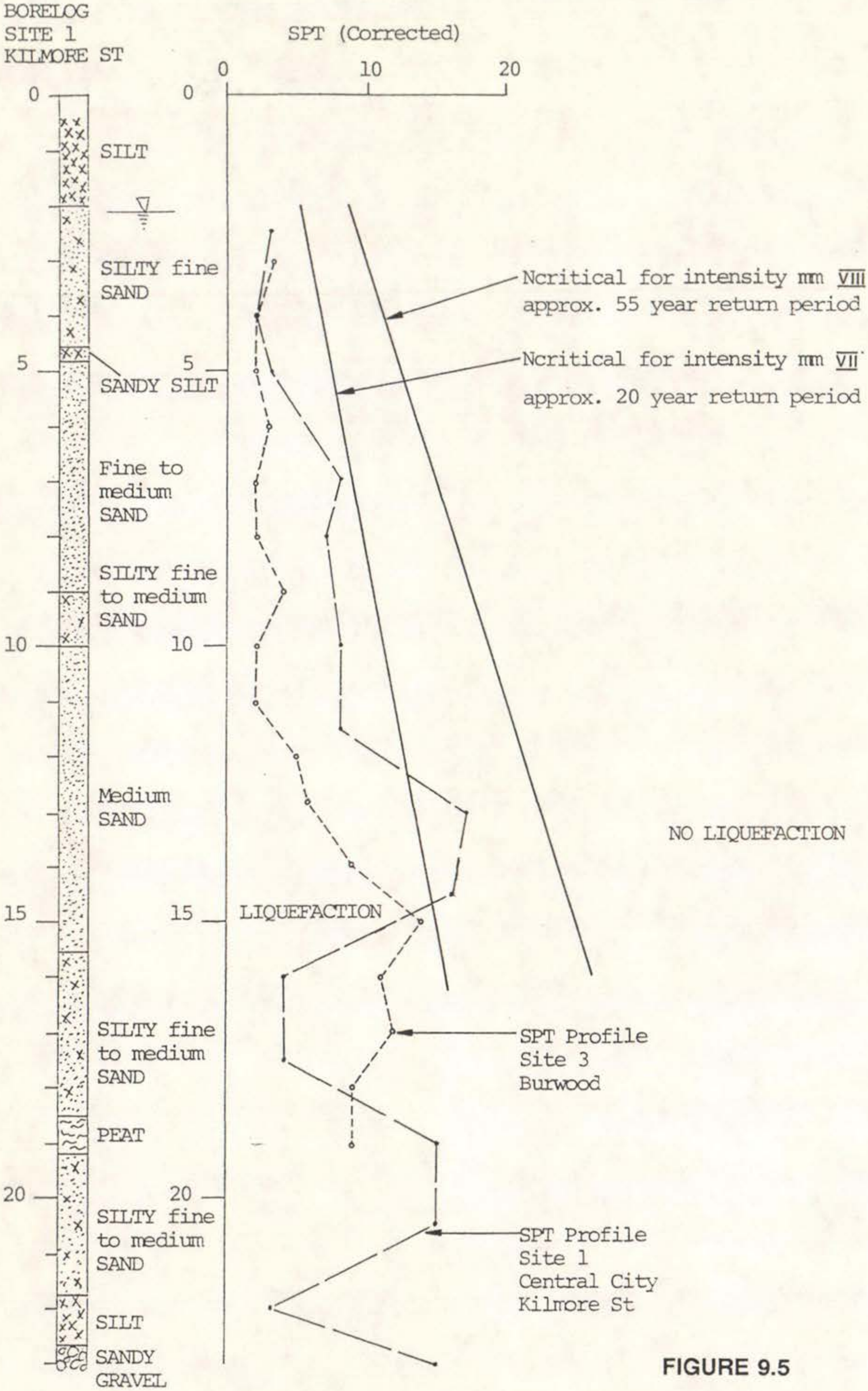


FIGURE 9.5

Liquefaction Analysis - Chinese Method for Two Sites in Christchurch

Correlation with field and test data, using SPT N values as the measure of in situ density, gives a curve dividing cases where liquefaction is likely from those where liquefaction is unlikely. The position of this curve varies with the number of cycles of stress induced by earthquakes of different magnitudes, as shown in Figure 9.6. Figure 9.6 also shows data for the site in Kilmore Street, using a peak ground acceleration of $a_p\backslash g = 0.2$, again indicating that liquefaction is likely to occur. Peak ground accelerations are generally about 40% of the peak spectral acceleration. For an average amplification in Christchurch, a peak spectral acceleration of $a_s\backslash g = 0.5$ has a return period of about 90 years. The probability of a peak ground acceleration of $a_p\backslash g = 0.2$ being equalled or exceeded in any 50 year period is therefore about 40%.

The cyclic stress method has also been adapted for use of in situ test data from the static cone penetration test (CPT) (Robertson & Campanella, 1985). CPT data for the same site in Kilmore Street in shown on Figure 9.7, and confirms that liquefaction is expected for peak ground acceleration $a_p/g = 0.2$.

The fourth method assumes that the pore pressure increase during an earthquake is a function of the density of dissipated seismic energy, where the dissipation rate depends on the initial soil stress and density, characterised by the SPT-N value (Berrill et al, 1988). For level ground, liquefaction occurs when the increase in pore pressure equals the initial vertical effective stress. Extensive liquefaction case history records were used by Berrill et al to determine the values of constants in the proposed relationship for liquefaction prediction. The likelihood of liquefaction occurring is determined by plotting

$$r^2 \, \sigma'^{\,3/2}_{o} \, 10^{-1.5M} \quad v \ N$$

where r is the distance from the earthquake source, σ'_o is the initial effective overburden stress and M is the earthquake magnitude. Liquefiable and non-liquefiable cases are separated by the line

$$450 \ N^{-2} = r^2 \, (\sigma'_o)^{3/2} \, 10^{-1.5M}$$

as shown in Figure 9.8.

Data from the Kilmore Street site are plotted on Figure 9.8 for a number of possible large earthquakes affecting Christchurch and again show that this site has a considerable liquefaction potential. Also of note from this figure is the correlation with the prediction of liquefaction shown on Figure 9.4. The earthquakes shown on Figure 9.4 as likely to produce liquefaction in Christchurch all produce data points in the liquefaction zone of Figure 9.8 while a $M = 7.1$ earthquake on the Hope Fault has only three data points just within the liquefaction zone, consistent with this event being on the lower bound line in Figure 9.4. It must be remembered, however, that these earthquakes are maximum credible earthquakes for each fault.

All the analysis methods for a specific site in Kilmore Street indicate that liquefaction is likely to occur during earthquakes of magnitude $M \geq 7$ in the seismic zones close to Christchurch or magnitude $M \geq 8$ earthquakes on some faults further away.

For this particular site the methods using the cyclic stress approach indicate liquefaction is likely to occur with peak ground accelerations less than $a_p \backslash g = 0.2$, and this acceleration has a return of about 90 years. The empirical relationship of the "Chinese" method suggests liquefaction could occur with intensities of MM VII or greater, with a return period of about 20 years. The relationship between Magnitude and distance can be used in conjunction with the probabilities of occurrence from Table 5.1 to provide another estimate of the likelihood of liquefaction in Christchurch. Using only earthquakes of magnitude greater than the mean line in Figure 9.4 gives an annual frequency of occurrence of 0.0046 (215 year return period) with a probability of liquefaction occurring in any 50 year period of 20%.

There is therefore a wide range in estimated likelihood of liquefaction. Some of this range may be due to the effects of duration in liquefaction; a peak ground acceleration in excess of $a_p \backslash g = 0.2$ may not induce liquefaction unless the shaking is of sufficient duration. However, even for the lower probability of seismic shaking sufficient to cause liquefaction occurring, there is a significant risk to any potentially liquefiable site.

Similar analyses have been carried out for the other sites marked on Figure 9.1, and confirm that liquefaction is also predicted at these sites under these earthquake conditions.

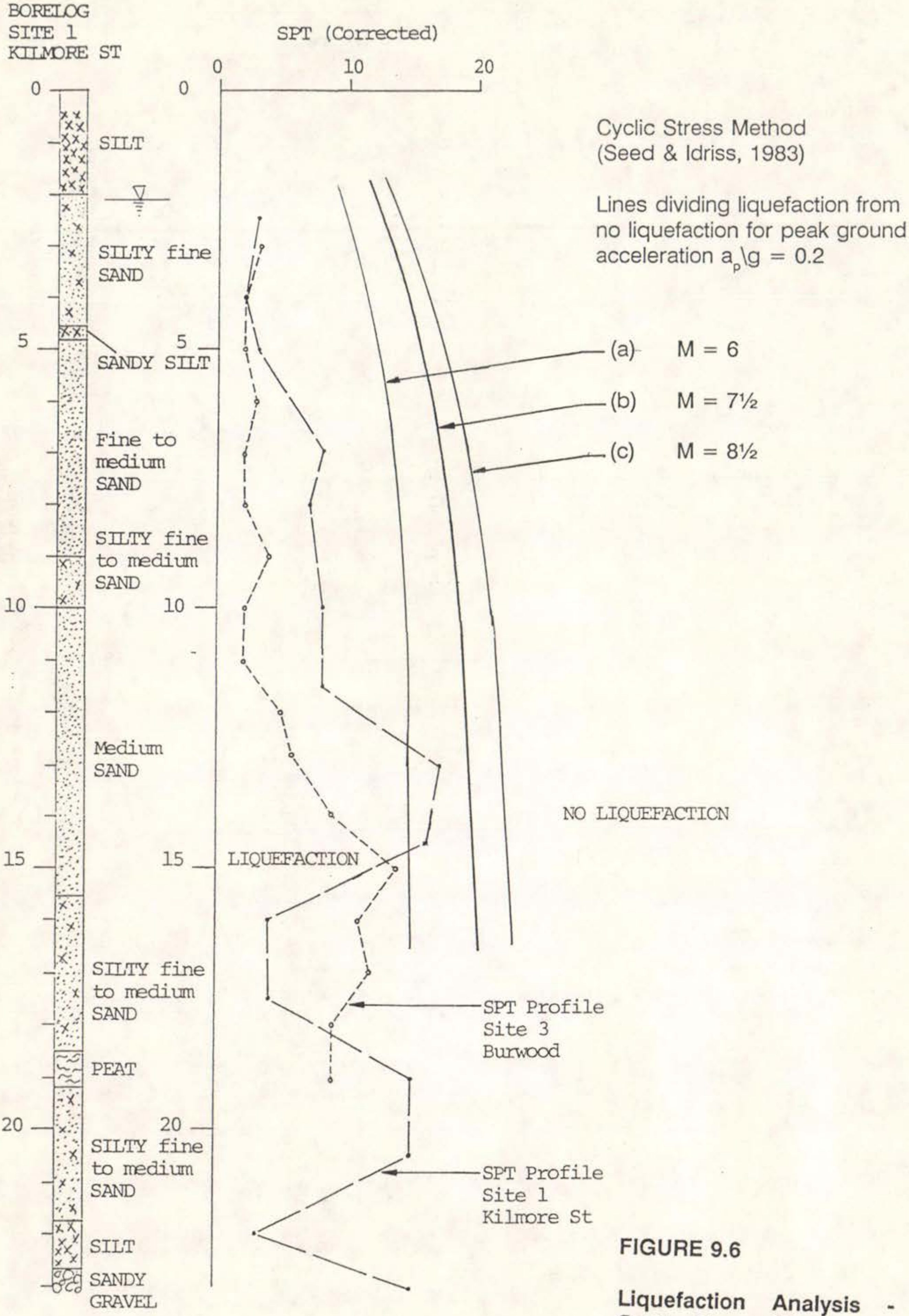
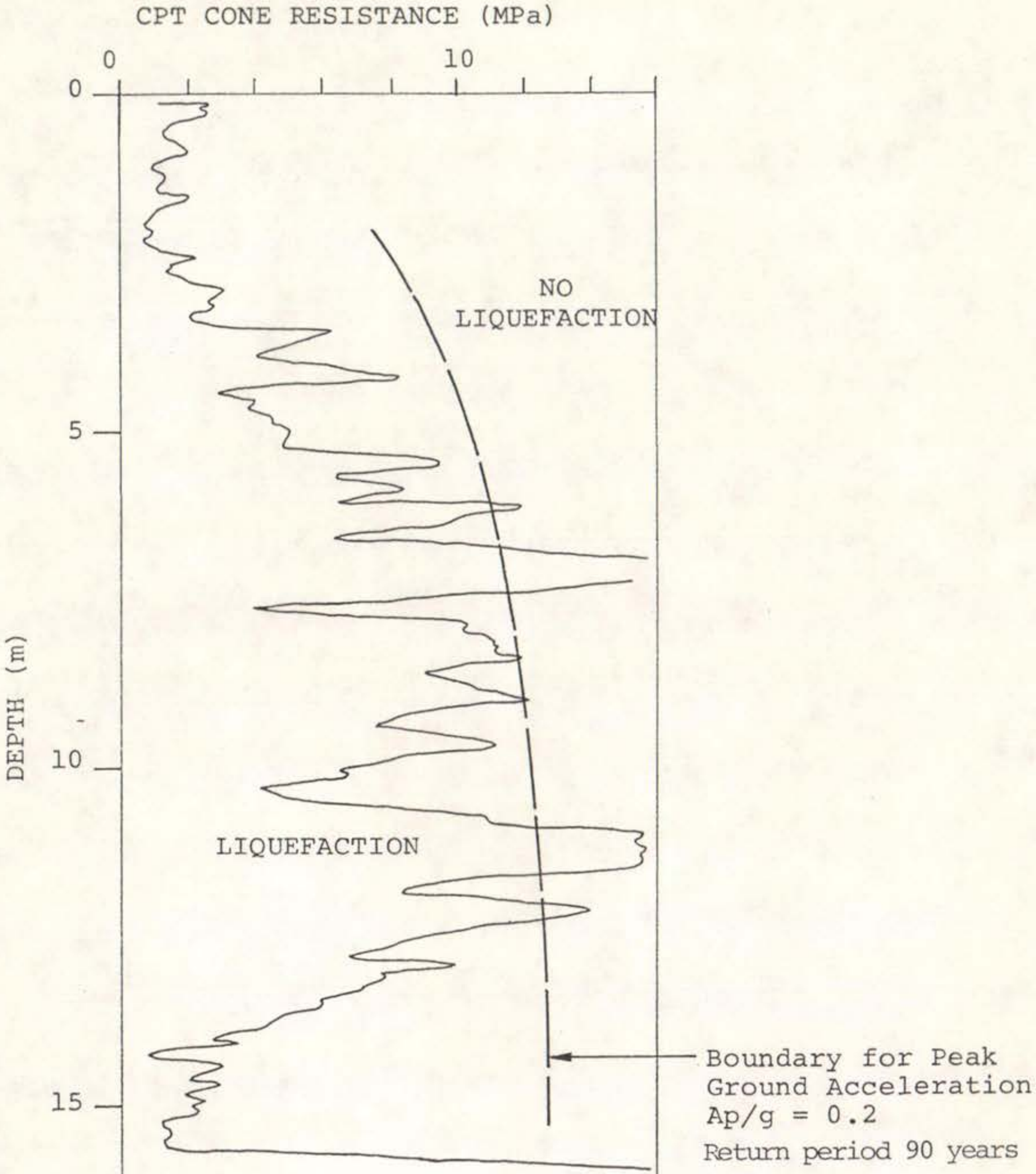


FIGURE 9.6

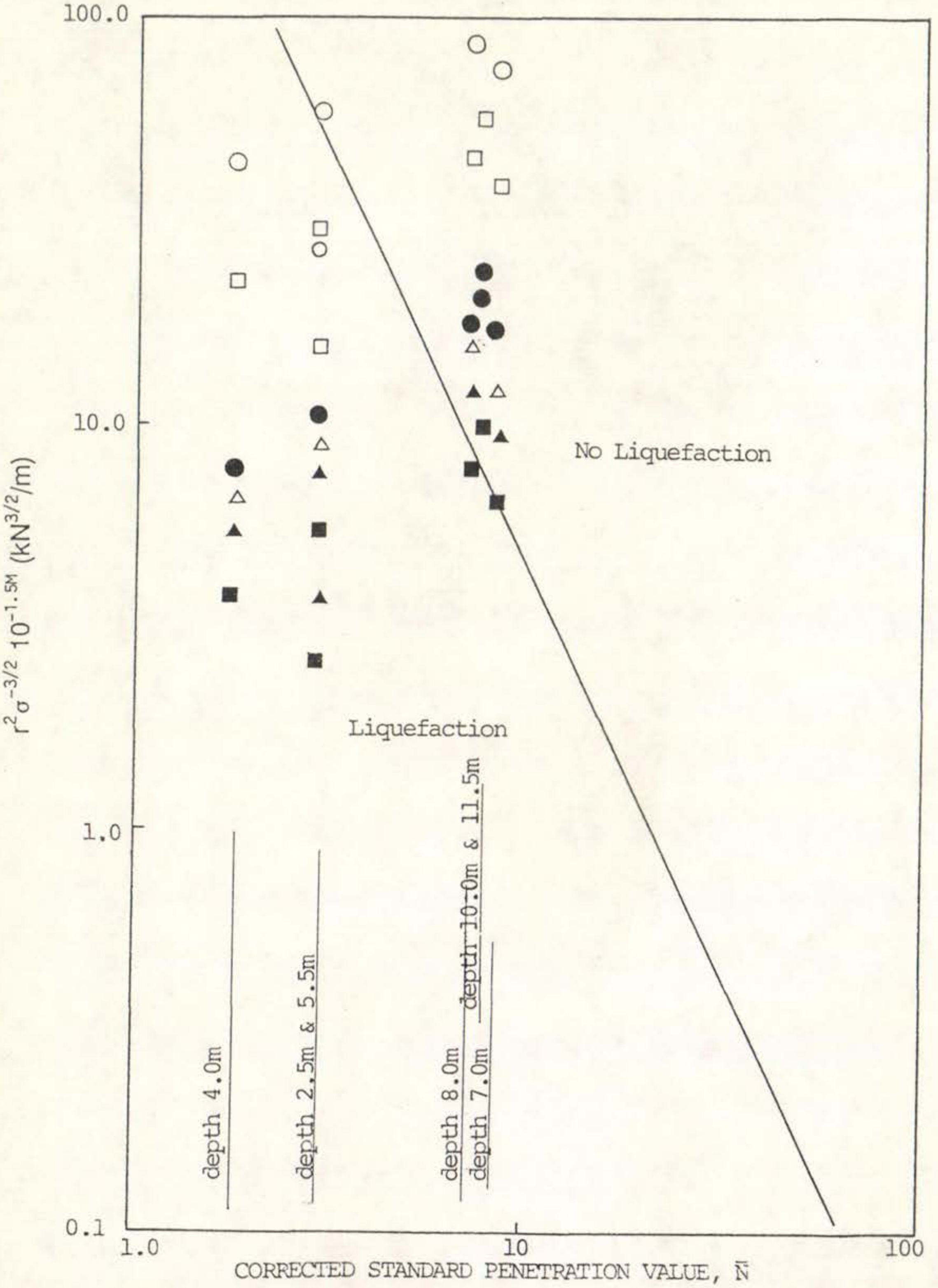
Liquefaction Analysis - Cyclic Stress Method for Standard Penetration Test Data for Two Sites in Christchurch



Cone Penetration Test
(Robertson &Campanella 1985)

FIGURE 9.7

Liquefaction Analysis - Cyclic Stress Method for Core Penetration Test Data for One Site in Christchurch



Data from site 1, Kilmore Street for following seismic events

■ M7.2 @ 25km in Pegasus Bay (liquefaction to 8.0m depth)

▲ M7.5 @ 50km = Porters Pass Tectonic Zone

△ M8.1 @ 150km = Alpine Fault - Central section

● M7 @ 25km = Porters Pass Tectonic Zone -Ashley section

□ M7.2 @ 60km = Banks Peninsula

○ M7.1 @ 100km = Hope Fault - SW Section (liquefaction to 4m Depth)

FIGURE 9.8 Liquefaction Analysis - Seismic Energy Method for One Site in Christchurch

9.5 SUMMARY

Liquefaction is a common effect resulting from intense seismic shaking. It has been observed worldwide, including many examples from New Zealand. Liquefaction has not been observed in Christchurch since European settlement, but a study of historic earthquake records suggests that Christchurch has not been subjected to sufficiently severe shaking during this period to cause liquefaction.

Due to of the limited number of sites in Christchurch at which in situ testing has been carried out, it is not possible at this time to do more than demonstrate that:

(1) A substantial area of Christchurch City is underlain by layers of sand which would be susceptible to liquefaction if sufficiently loose. This area comprises: the area east of Marshland Road, Fitzgerald Avenue and Opawa Road, including the estuary side of Brighton, Heathcote and Sumner, an area through Sydenham, Addington and Riccarton, part of Spreydon, Hoon Hay, and Merivale.

(2) For a number of sites studied within the city area, in situ test data, grain size analyses and liquefaction analyses show that liquefaction is likely to occur during large earthquakes. How far these areas extend, and the intensity of shaking required to initiate liquefaction, are important factors requiring extensive further research.

CHAPTER 10: SLOPE INSTABILITY

10.1 INTRODUCTION

Slope instability has been triggered by virtually all New-Zealand earthquakes with magnitude M > 6.5. The extent to which the damage has extended outside the epicentral region has depended on the focal depth and total energy released, and varied with local topography, geology and groundwater conditions.

To date there has not been a compilation of New Zealand examples of slope instability during earthquakes to assess the most common types but three categories of slope failure have been suggested in a comprehensive international study by Keefer (1984). Categories defined by Keefer include Disrupted Slides and Falls where chaotic masses of small soil or rock blocks move down slope (rock and soil slides, rock and soil falls, soil avalanches); Coherent Slides which are generally deeper and involve larger more coherent blocks (rock and soil slumps, rock and soil block glides, slow earthflows); and Lateral Spreads and Flows involving fluid like flow.

Keefer looked at the relative abundance of these three types in 40 large earthquakes and concluded the first group were the most common. The other two types were less abundant on a numerical basis however larger areas were frequently involved.

Figure 10.1 repeats the plot of maximum credible earthquake v. epicentral distance to Christchurch for all fault zones in the region, presented earlier as Figure 4.5 in Chapter 4, with curves defining the upper stability limit for the stated failure type. Above each line the relevant type of instability can be expected to occur based on average case history data. This is a very general relationship which indicates that slope instability can be expected if earthquakes approaching these magnitudes occur. It may be inferred that the further a potential earthquake event plots above these lines the greater will be the number of slope failures, or the severity of the failures.

The Pegasus Bay, Banks Peninsula and Christchurch Seismicity Zones and the Porters Pass Tectonic Zone, Kaiwara Fault and the central section of the Alpine Fault, all have the potential to generate substantial slope instability in Christchurch for all three categories. Most of the maximum credible earthquakes on the other active faults in the area could generate disrupted and coherent slides, however their position in relation to the curves would suggest that the total number of these may be smaller.

In this chapter the specific soil and rock materials in the Christchurch area are examined in relation to the likely ground accelerations in order to obtain a more comprehensive understanding of the likely response. Sections 2, 3 and 4 consider hill slopes where the soil is reasonably deep while section 5 assesses the consequences for rock slopes with minimal soil cover. Liquefaction induced slope failures are discussed in section 6.

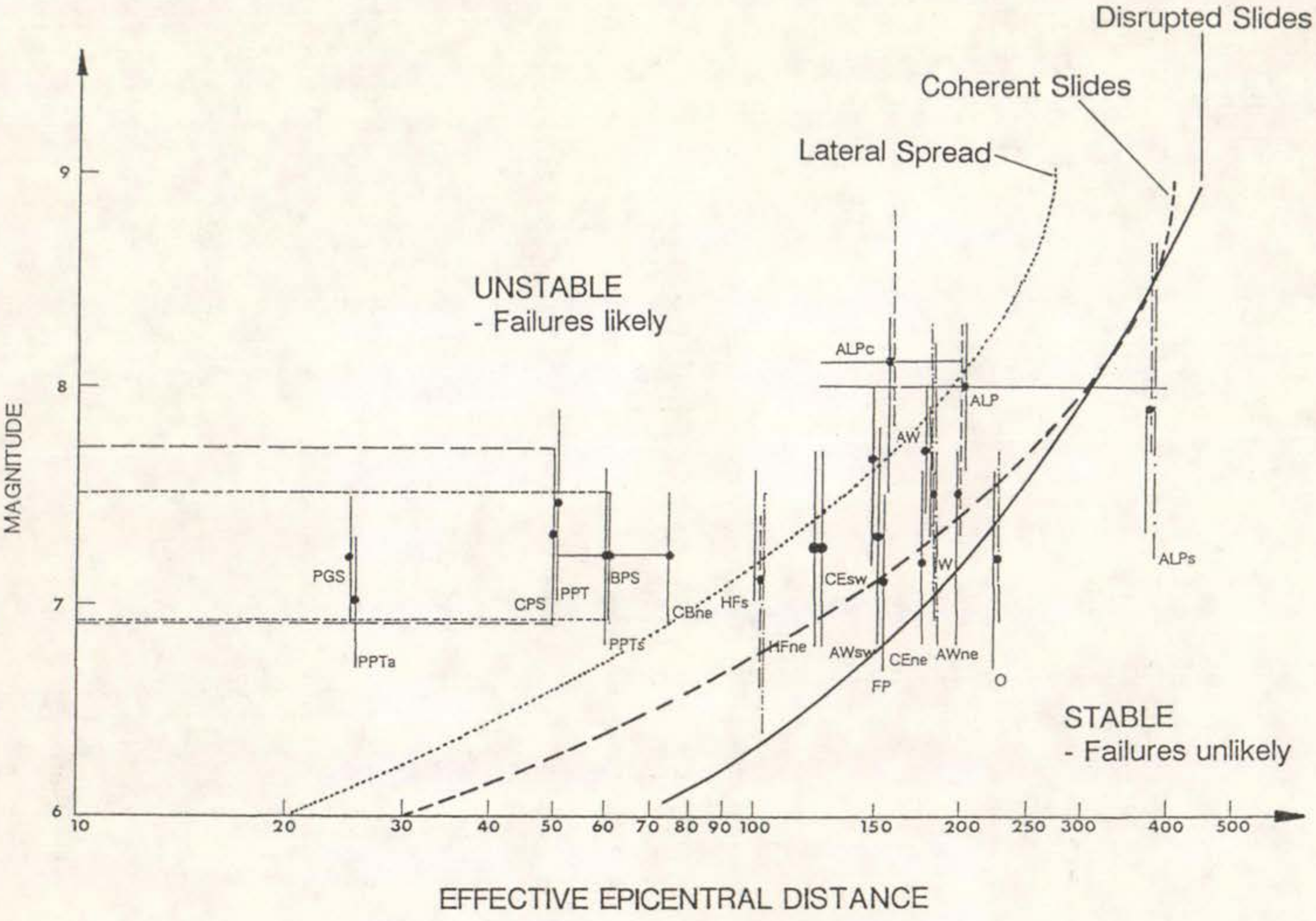
10.2 CHRISTCHURCH HILLSLOPE SOILS

Most of the Christchurch and Lyttelton Harbour hill suburbs are built on loess and mixed colluvium slopes. Volcanic colluvium is generally restricted to the higher areas adjacent to bedrock outcrop and is also found locally in the side slopes of the steeper valleys.

Loess on Banks Peninsula is typically a yellow brown slightly clayey silt with some sand. The material has been subdivided into generic classes by Bell & Trangmar (1988); the term "In Situ Loess" has been adopted for the primary wind deposited material and the term "Loess Colluvium" for the down slope transported erosion product. In practice the two materials are often hard to distinguish in the field and to date no systematic study has been attempted to delineate any significant differences which may exist between the material geotechnical properties. In the absence of such information the general term loess is used in this study to include both types of material.

Important vertical variations in key geotechnical properties do occur in loess profiles. As a result a local classification system, initially proposed by Hughes (1970), has been widely adopted and details are shown in Figure 10.2. Table 10.1 summarises typical loess geotechnical properties with variations between the three layers noted where relevant.

Volcanic Colluvium occurs as brown to red brown silty clay or clayey silt with minor sand and a highly variable gravel component. This gravel may be up to boulder size and constitute 20 - 35 % of the soil mass (most commonly 10 - 20%). In contrast to the loess volcanic colluvium is normally moderately to highly plastic. The material has been derived from in situ volcanics by weathering and slope processes and the clay minerals are often more active varieties than those found in loess. Table 10.2 summarises the limited available geotechnical information.



Key

CPS	Cant. Plains Seismic
BPS	Banks Pen. Seismic
PGS	Pegasus Seismic
PPT	Porters Pass TZ
PPTa	- Ashley section
PPTs	- Southern section
CBne	Kaiwara
HF	Hope
HFs	- SW section
HFne	- NE section
CE	Clarence/Eliot
CEsw	- SW section
CEne	- NE section
AW	Awatere
AWsw	- SW section
AWne	- NE section
W	Wairau
FP	Fox's Peak
O	Ostler
ALP	Alpine Fault
ALPc	- Central section
ALPs	- South section

FIGURE 10.1

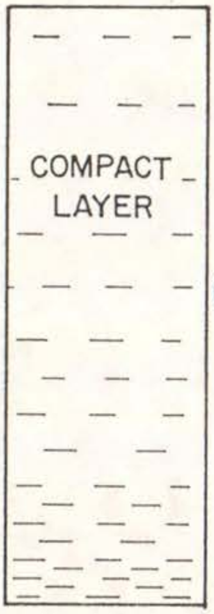
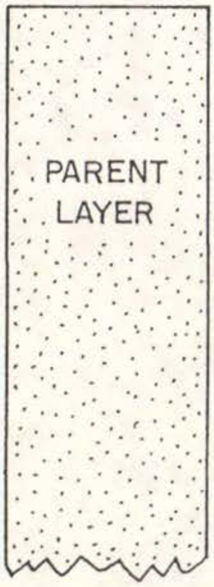
Earthquake Magnitude - Epicentral Distance Relationship With Three Categories of Slope Failure as Defined by Keefer (1984)

Parameter	Typical Range of Values	Source Reference
POROSITY	30 - 40%	Birrel & Packard (1953)
VOID RATIO	0.4 - 0.7	Birrel & Packard (1953) Miller (1971)
ATTERBERG LIMITS	LL 18 - 33 PL 17 - 22 (C layer) PI < 12	Alley (1966) Hughes (1985) Crampton (1985) Yetton (1986)
GRAIN SIZE	(Silt range > 0.002mm & < 0.06mm) Sand ≈ 10% Silt 65 - 80%, Clay 11 - 25%	Alley (1966) Hughes (1985) Crampton (1985) Yetton (1986)
DRY DENSITY	S layer average = 1.54t/m^3 (1.39 - 1.62t/m^3 range) C layer average = 1.64t/m^3 (1.51 - 1.88t/m^3 range) P layer average = 1.55t/m^3 (1.32 - 1.71t/m^3 range)	Evans (1977) Crampton (1985) Yetton (1986)
LINEAR SHRINKAGE	0 - 2% in lower S and P layers > 5% in C layer	Alley (1966) Yetton (1986)
PERMEABILITY	1.5 • 10^{-7} m/s (undisturbed) ≈1•10^{-7}m/s (In-situ test)	Birrel & Packard (1953) Sanders (1986)
INTERNAL ANGLE OF FRICTION	35 - 37° (Residual, Ring shear) 30° (Peak, Triaxial (total)) 30° (Peak, Triaxial (total)) 15 - 25° (Peak, Triaxial (effective))	Salt (1983) Mackwell (1986) McDowell (1989) Alley (1966)
COHESION	0 kPa (effective) 85 - 112kPa (apparent) 0 - 180 kPa (apparent)	Alley (1966) Macwell (1986) McDowell (1989)
COMPRESSION INDEX	Cc = 0.17 (1.7% vol. change dry to saturated)	Birrel & Packard (1953)
Ph	Acidic - 5(S layer) to 7(P layer)	Miller (1971)
SOLUBLE SALT CONCENTRATION	Incr. with depth, from 1meg/l to 60meg/l in P layer	Miller (1971)
EXCHANGEABLE SODIUM %	0.9 in S layer to41 deep in P layer	Hughes (1970)
SEISMIC VELOCITY	250 - 400m/s	Crampton (1985) Yetton (1986) McDowell (1989)
RESISTIVITY	Varying with depth from 90 ohm/m near surface to <10 ohm/m in P layer	Yetton (1986)
CONDUCTIVITY	From 1.0 •10^{-4} mho/cm to 14 •10^{-4} mho/cm with depth	Birrel & Packard (1953) Yetton (1986)

(Modified from Yetton (1986))

TABLE 10.1

Geotechnical Properties of Banks Peninsula Loess

Typical Thick-ness	Symbol (Hughes, 1970)	General Formal Description	Typical Geotechnical Parameters
250mm to 700 mm	 TOPSOIL LOWER SURFACE LAYER S	Grey brown organic SANDY SILT (OL) - Soft, low plasticity	Non erodible, $E_{> 1000}$ (ND1)
		Yellow grey SILT with some sand (ML) - firm, non - slightly plastic	Moderately erodible, $E_{< 380}$ (ND4) Average dry density = 1.54 tm^{-3}
400 mm to 1200 mm	 COMPACT LAYER C	Yellow grey and red brown mottled CLAYEY SILT with minor sand (CL -ML) - stiff to very stiff, slight - moderate plasticity.	Low - Moderately Erodible $E_{1000 - 380}$ (ND1 -ND4) Average dry density = 1.65 tm^{-3} Liquid limit, 24 - 32 Plastic limit, 18 - 20 Plasticity Index, 6 - 12
up to 6 m	 PARENT LAYER P	Light greyish yellow SILT with some sand and minor clay (ML) - stiff, non - slightly plastic.	High Erodibility, often $E_{150 - 50}$ (D2 - D1) Average dry density = 1.55 tm^{-3} Liquid limit, < 25 Plastic limit, non plastic or < 18 Plasticity index, non plastic or < 7

NOTE: ALL THE ABOVE PARAMETERS ARE AVERAGE VALUES OR APPROXIMATE RANGES. EXCEPTIONS ARE KNOWN TO OCCUR.

FIGURE 10.2 **Variation in Loess Geotechnical Properties With Depth**

	Liquid Limit	Plastic Limit	Plasticity Index	% Clay	Activity
Sample 1	64.5	37.5	26.8	14.4	1.9
Sample 2	60.5	36.6	23.9	15.2	1.6

Table 10.2: **Geotechnical index properties for two samples of Volcanic Colluvium from the Akaroa Area (after Mackwell, 1986)**

Volcanic colluvium also exhibits more variable density and permeability than loess, reflecting the greater variety in the original volcanic source material. Most of the variation is inherited from the original mass movement event responsible for the soil formation, therefore the relatively uniform pedological vertical variations noted above for loess are less obvious.

Mixed Colluvium is a category which includes the widely varying admixtures of the two categories discussed above. It ranges from 10 - 90 % loess, with the remainder volcanic colluvium, and as a result the soil varies from a yellow brown clayey silt with rare gravel to dark brown silty clay or clayey silt with some gravel. Properties are intermediate between the loess and volcanic colluvium but abrupt lateral and vertical variations are more common. Table 10.3 summarises the limited geotechnical information.

	Liquid Limit	Plastic Limit	Plasticity Index	% Clay	Activity
Sample 1	40.6	22.5	18.1	19.0	1.0
Sample 2	79.9	37.4	42.4	29.6	1.4

Table 10.3: **Geotechnical index properties for two samples of Mixed Colluvium from the Akaroa Area (after Mackwell, 1986)**

Soil Shear Strengths: Loess

Before any realistic assessment can be made of the likely impact of seismicity as a trigger in slope failure, the shear strength of the soil materials must be reasonably established.

In conjunction with this study, research has been carried out by an engineering geology MSc student at the University of Canterbury under our partial supervision. The results greatly improve current knowledge on the shear strength of loess (Goldwater, 1990). Conclusions relating to shear strength are discussed by Goldwater et al (1990).

All soils are affected similarly by positive pore water pressures which, in a saturated soil, reduce the effective stress and thus the shear strength. However the shear strength of a partially saturated fine grained soil like loess is very sensitive to variations in the degree of saturation. This is the result of strong capillary tensional water forces which become very significant when the voids become unsaturated (refer for example Walker & Mohen, 1987). While these negative pore water pressures cause the effective stress to increase, resulting in an actual increase in the frictional soil strength, they are usually observed as an increase in "apparent soil cohesion". The (true) cohesion is an extremely sensitive factor in controlling overall slope stability; the 'apparent cohesion' is similarly important.

At progressively lower degrees of saturation the true cohesion is supplemented by a significant component of apparent cohesion. A "total cohesion" in excess of 100 kPa is possible in loess. Due to the number of samples requiring testing, Goldwater primarily conducted undrained shear strength tests. Although these tests demonstrate the principal trends they cannot be used directly to obtain the effective angle of friction (ϕ') and effective cohesion (c') for analysis. Figure 10.3 summarises the test results obtained.

Fortunately limited drained direct shear testing of saturated loess also carried out by Goldwater gives some indication of effective stress strength parameters. He concludes that the true cohesion of loess, at least in the upper layers, is relatively low and appears to vary vertically. Values around $c' = 2 - 5$ kPa in the C layer, and as high as $c' = 20$ kPa in the P layer may be realistic. Once saturated the upper layers of the loess are only slightly cohesive and can be considered principally as a frictional material. Frictional strength for all loess types is relatively constant with an angle of friction approximately $\phi = 28.5°$.

To provide an independent check of these suggested cohesion values, published information from existing landslides in loess has been back analysed in this study in order to estimate the cohesion operative at the time of failure. Harvey (1976) carried out a study of rainfall triggered slope failure on the Port Hills which occurred in August 1975. He measured and recorded 519 "slips" in a variety of soil materials, noting that most failure occurred in loess and mixed colluvium.

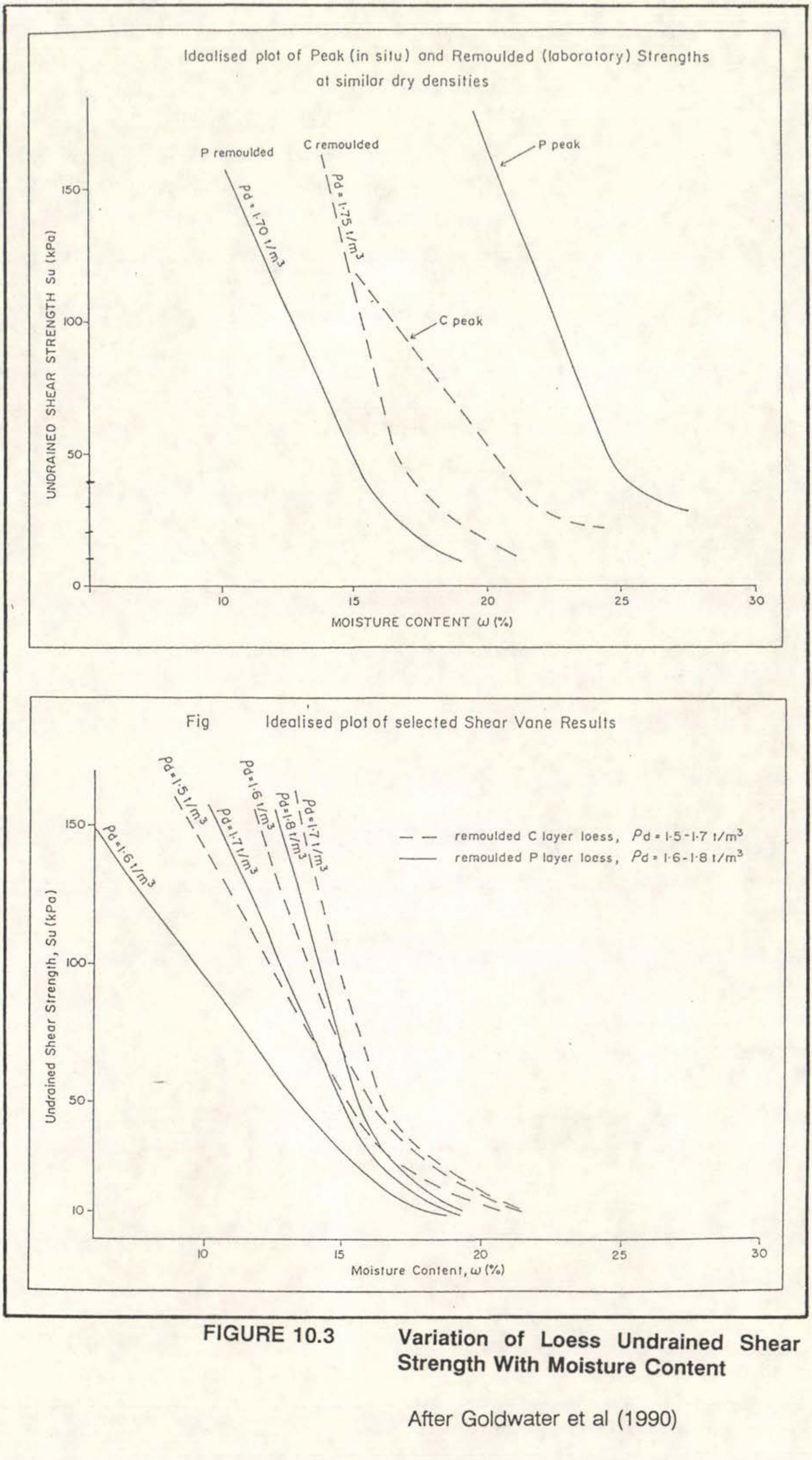


FIGURE 10.3 **Variation of Loess Undrained Shear Strength With Moisture Content**

After Goldwater et al (1990)

He recorded details of 162 loess slides, including the average slope angle on which the failure occurred, and in all cases considered the soil to be fully saturated at the time of failure. The average slope failure depth was observed to be relatively shallow (< 1.0 m).

Harvey found slopes with a relatively sunny aspect had the lowest average failure slope angle, averaging 28 - 29° with a range from 26 and 31°. Shady faces showed a greater range, from 20 - 36°, but had steeper failure slope angles generally, with an average of 30 - 31° degrees. The reason for this difference is not clear but may be related to rainfall variations, soil fissuring on sunny faces, or simply the previous removal by sliding of soil on the shady faces at the most critical angle. The difference in slope angle between the faces is not particularly significant when back analysing this information.

Analysis of these slopes was carried out in this study using the infinite slope assumption discussed in more detail later. Other input parameters adopted include an angle of friction $\phi = 28.5°$ (Goldwater et al, 1990), saturation to the ground surface, and the most common failure depth noted by Harvey of 1.0 m. This gives a maximum cohesion mobilised at failure of approximately $c' = 4$ kPa for failure at 1.0 m. This is consistent with the results $c' = 2 - 5$ kPa suggested by Goldwater for true cohesion in the loess C layer.

Shear Strengths: Volcanic Colluvium

Very little testing has been reported for the geotechnical properties of Banks Peninsula volcanic colluvium generally, and even less for shear strength. The information presented earlier in Table 10.2 indicate a relatively more active and plastic clay mineral component than loess which implies generally lower frictional strengths.

The only available laboratory shear strength information comes from a relatively silty sample of volcanic colluvium in the Akaroa area (unpublished investigation report, Soils & Foundations Ltd). Ring shear tests on a sample of landslide material at Settlers Hill gave an residual angle of friction $\phi = 27°$ at low effective normal stresses, reducing to less than 20° at higher effective stresses (i.e. > 100 kPa). Most volcanic colluvium is more clay rich than this particular sample which is close to the mixed colluvium category. A peak friction angle of 25° has been assumed in the absence of more reliable data.

Back analysing the information of Harvey for slopes underlain by volcanic colluvium (Harvey's categories of "Basalt & Basalt with some loess") suggests a cohesion $c' = 3.9$ kPa, which has been adopted in the analysis below.

Shear Strengths: Mixed Colluvium

No shear strength information of any type is available for mixed colluvium and Table 10.3 above outlines the limited existing geotechnical information. It is reasonable to expect the material strength properties to be intermediate between those of loess and volcanic colluvium. No specific numerical seismic analysis has been carried out here since it can be assumed that these other materials will reasonably bracket the extremes of response.

10.3 SLOPE FAILURE: STATIC CONDITIONS

As noted earlier loess under static conditions is prone to rainfall triggered relatively shallow failures in the upper C and S layers (refer for example Harvey, 1976; Bell & Trangmar ,1988 and Goldwater, 1990). This is the partly the result of variations in shear strength, since the upper horizons are generally weaker than the underlying parent material, but principally this reflects a loss of capillary tension and the build-up of porewater pressure above relatively impermeable lower layers. In cross-section the failures which commonly result are translational slides with a average depth of 1.0 m and an average length around 15 m (Harvey, 1976). Harvey generally observed slides were less frequent in volcanic colluvium than the other materials, however his results suggest their average geometry was essentially similar i.e. an average depth of around 1.2 m and length of 20 m. Once again translational failures with failure surfaces subparallel to the surface were most common. Mixed colluvium was observed to have failed with similar frequency to loess and the analysis of Harvey's data indicates little significant difference in geometry from the other materials.

10.4 SLOPE FAILURE: SEISMIC ANALYSIS

Seismic analysis of slopes has become increasingly sophisticated in recent years with the availability of dynamic analysis employing finite element techniques. While this offers definite advantages by allowing incorporation of a full time history of earthquake accelerations, the extra accuracy of the method is only justified if the input information is of high quality. In our opinion not enough is currently known regarding the dynamic strength of Banks Peninsula soil materials to justify such analyses.

Pseudo-Static Infinite Slope Analysis

A reasonable assessment can be made using the pseudo-static limit equilibrium analysis (e.g.Fell & Jeffery, 1987) where the forces induced by the earthquake accelerations are treated as an equivalent static horizontal force. In this study infinite slope analysis has been used. Goldwater (1990) found that little advantage was gained by adopting other methods (e.g Sarma, 1979), for shallow translational failures typical of the Port Hills which exceeded approximately 5 m length. The infinite slope equation adopted is as follows:

$$FS = \frac{[\delta h\,(\cos^2\beta - k_h \sin\beta\cos\beta) - \delta_w h_w \cos^2\beta]\tan\phi + C}{\delta h\,(\sin\beta\cos\beta + k_h\cos^2\beta)} \tag{10.1}$$

Where:
δ = unit weight of soil
δ_w = unit weight of water
ϕ = angle of friction
C = cohesion
h = vertical depth to failure surface
h_w = height of water above failure surface
β = slope angle
k_h = lateral seismic coefficient

It is important to note the main simplifying assumptions i.e. that no time dependent variation in the earthquake force is incorporated and that the shear strength of the soil is assumed to remain unchanged during the earthquake. These limitations in the method are discussed later in relation to the results.

The pseudo-static approach employs a lateral seismic coefficient k_h to represent earthquake loadings. Terzaghi (1950) proposed k_h values ranging from 0.1 in areas subject to shaking intensity MM IX up to 0.5 (MM ≥X, catastrophic). Hunt (1986) suggests the correlations outlined in Table 10.4 below:

Intensity (MM scale)	k_h
I - IV	0.0
V - VI	0.05
VII	0.10
VIII - IX	0.15
>X	0.25

Table 10.4: Suggested k_h Values (Hunt, 1986)

Most selection of k_h values has been for earth dam design where some amplification in the alluvial foundation materials can often be expected. These values usually include an arbitrary reduction from the expected peak ground accelerations to better reflect the "average" accelerations during the earthquake and are typically obtained by multiplying the expected ground acceleration by 0.3 - 0.4 (Matushka, pers. comm. 1990). Considering the expected peak ground accelerations on the Port Hills, where bedrock is relatively close to the surface and amplification likely to be relatively low, k_h values of 0.05, 0.1 and 0.2 have been considered in this study. The approximate return periods for these accelerations may be assessed from Table 6.2 as 30, 300 and 1500 years respectively.

Influence of Seismic Forces on Failure Geometry

Available information on the geometry of shallow rainfall triggered landslides on the Port Hills has been detailed earlier. The applicability of these geometries in the modelling of slope sensitivity to earthquakes must first be considered, given the possibility that an alternative characteristic deeper "earthquake" geometry may develop.

The infinite slope analysis described by equation 10.1 has been used to produce Figure 10.4, which shows failure depth v factor of safety (FS) for soils with different cohesions inclined on a 30° slope under static conditions. Curves are shown for different idealised dry materials, each with a uniform cohesive strength which does not vary with depth. The trend is clear; in general the cohesion component is most effective in maintaining stability at shallower depths. For constant cohesion, failures should be more frequent at greater depths. Despite this trend the slopes on the Port Hills are observed to typically fail at shallow depths. This reflects the important vertical variations in shear strength, the tendency for progressive top down saturation, and the development of perched water tables between 1 and 2 m.

Results for an idealised dry material with a cohesion of 5 kPa but inclined, in this case, on a 35 degree slope are shown in Figure 10.5. Static analysis predicts this slope would fail at a depth of 3 m. The same material subjected to earthquake acceleration with a seismic coefficient $k_h = 0.1$ would fail at 1.5 m depth. Similarly $k_h = 0.2$ would result in failure at 1.0 m depth. It may be concluded that in a slightly cohesive homogenous soil shallower failure is likely in an earthquake than would occur under static conditions. In the case of a purely frictional soil the factor of safety is not influenced by depth. In reality local soil or water variations with depth are likely to be the dominant factor. If these conditions favour shallow failures under static conditions this propensity for shallow failure is unlikely to change significantly, and may be slightly reinforced by earthquake acceleration. This would seem to be compatible with the observations of Keefer (1984) that relatively shallow "disrupted" failures are the most common during earthquake shaking.

This analysis assumes the soil fails by a comparable mechanism to rainfall induced failure i.e. shear rather than fluidisation or collapse. It is possible, although initial consideration suggests it is unlikely, that P layer loess if saturated may liquefy in some areas and create deeper slope failures. This phenomena has been observed during earthquake shaking in loess on the

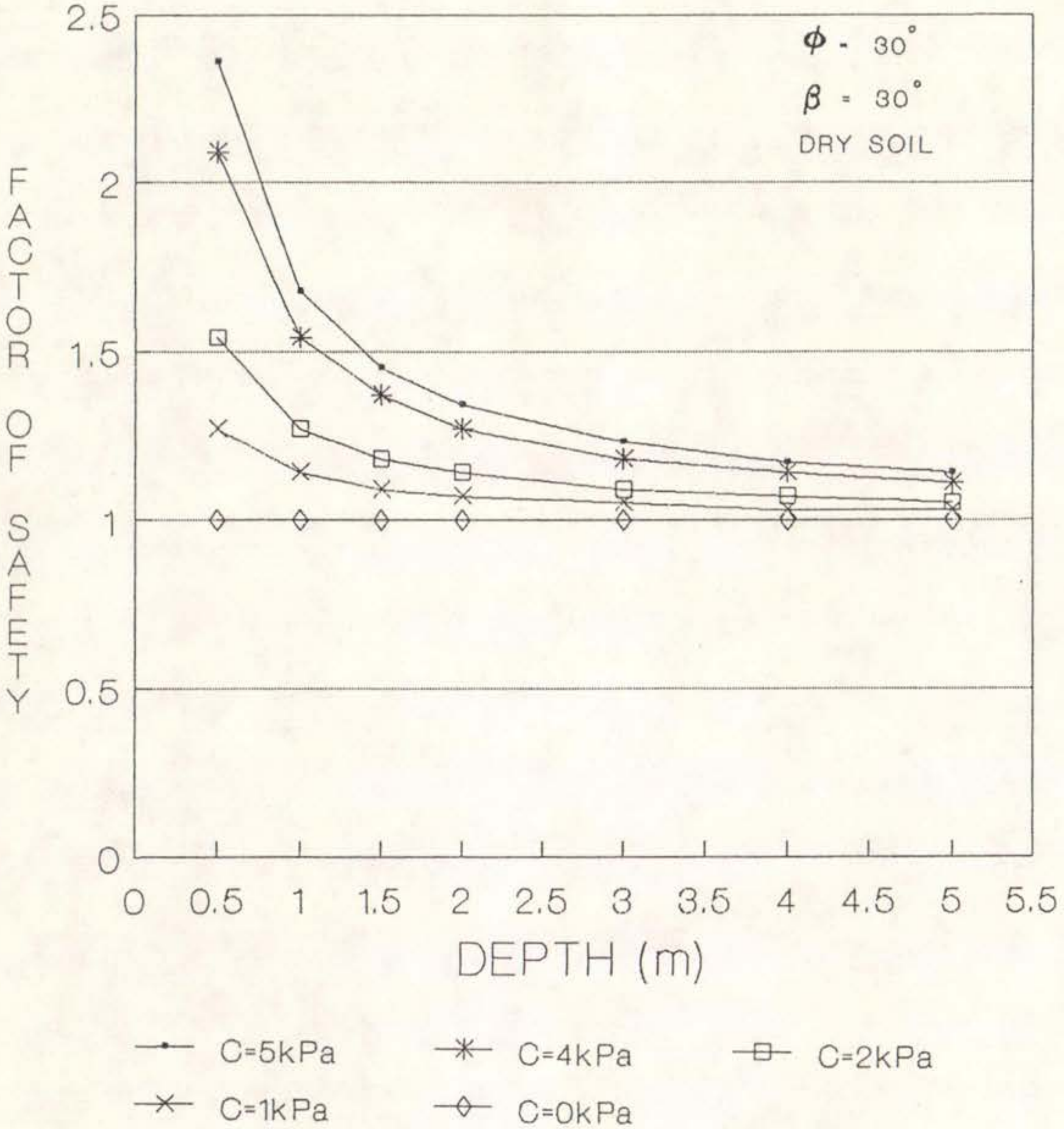


FIGURE 10.4

Factor of Safety - Depth Relationship for Idealised Dry Soils of Various Cohesions

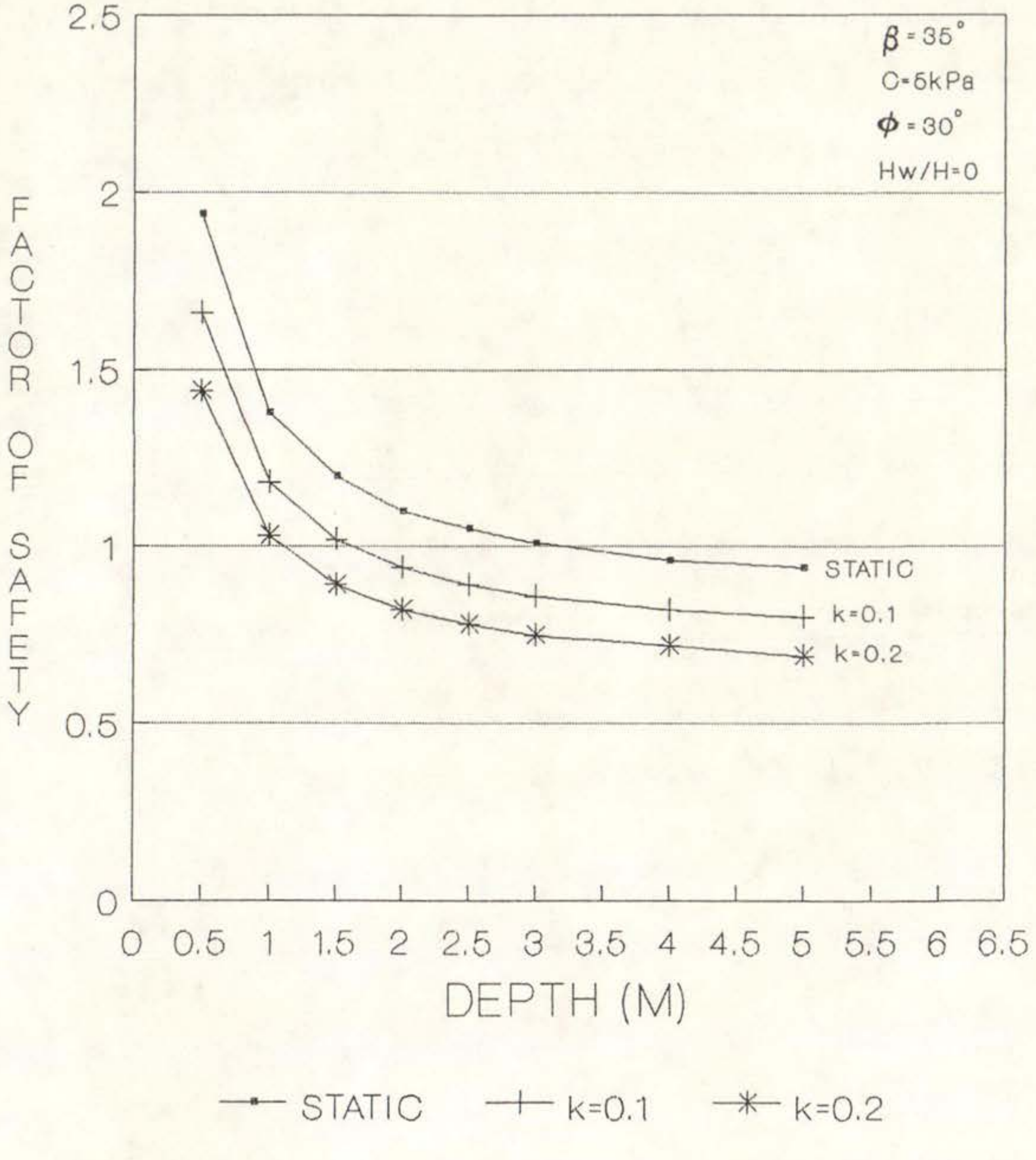


Figure 10.5

Factor of Safety - Depth Relationship for Dry Soil With 5 kPa Cohesion Subject to Various Seismic Accelerations

Eurasian continent (refer for example Seed, 1968) and is discussed later (Section 6).

In conclusion it appears reasonable to use information on slide geometry observed for rainfall triggered failure to model the effect of earthquake acceleration. Harvey observed failure plane depths of around 1.0 m in all the soil materials and this depth has been adopted in the analysis of slopes presented below.

Seismic Analysis - Loess

In Figure 10.6 slope angle v factor of safety is plotted for saturated loess under typical failure conditions, with an angle of friction of 28.5°, a water table at ground level, and a depth of failure of 1.0 m. The cohesion adopted is 4 kPa as discussed earlier.

This figure demonstrates that the critical slope angle (i.e. the slope angle for which FS = 1), reduces from an average of 28.5° observed by Harvey under static conditions, to around 16° under severe shaking ($k_h = 0.2$). With moderate to strong shaking, slopes with angles between 20° and 28.5° would fail under the conditions assumed.

If the water table is below the failure surface and the soil becomes drier a marked increase in stability occurs, due both to the removal of positive pore pressures acting on the failure plane, and to the additional cohesion component that results from negative pore pressures (capillary tension). The extreme case of this capillary effect is in a virtually dry soil, however even at moisture contents of wet - moist commonly encountered a few hundred millimetres above the water table, the increase in cohesion is significant. This improvement in cohesion can be estimated from the effective stress equation of Bishop et al, 1960 (see for example Goldwater et al, 1990), or by other approaches such as that of Fredlund (1981). Numerically there is little difference between the methods and the result in this case, assuming a water table at 1.2m and a failure surface at 1 m, is an additional apparent cohesion of approximately 0.5 kPa. Figure 10.7 presents the plot of slope angle v. FS under these conditions. The critical slope angle has substantially increased for the equivalent k values despite the very small increase in cohesion. For example the critical slope angle for $k_{h = 0.2}$ has increased from around 16° to 32°. If the water table drops to deeper levels (say 3.0 m) the apparent cohesion is much more significant and contributes approximately a further 5 kPa. Figure 10.8 demonstrates the beneficial effect of this. With this degree of additional strength even a severe earthquake in theory has little effect. In reality this is likely to be the case on uniform slopes less than approximately 40° -50°. However extremely dry loess at steep faces often exhibits vertical desiccation cracks which tend to reduce cohesion in the same way joints do in a rock mass. Therefore a strong earthquake in dry conditions is likely to initiate local topple failures in such areas, for example along the base of cut slopes such as road batters.

Volcanic Colluvium

Results of the pseudo-static seismic analysis are presented in Figures 10.9 - 10.11 for the same range of conditions in volcanic colluvium. The essential difference in the assumed parameters for this material is the lower angle of friction. The results are very similar to those for loess and, once again, the critical importance of the soil moisture levels in existence at the time of the earthquake is clear. It is apparent from all analysis results for loess and volcanic soils that cohesion changes are most important in determining slope stability than changes in frictional strength characteristics.

Slope Displacements and Associated Earth Flows

If a slope has a factor of safety less than one during an earthquake event the slope will begin to move. Relatively simple methods exist to model the total displacement which can be expected based on double integration of the time history traces (see for example Newmark, 1965, or more recently Ambraseys & Menu 1988).

An example of the use of Newmark's method to predict the areal extent of landsliding during an earthquake is discussed by Ziony, 1985. In this study of the anticipated disruption to hill areas around Los Angeles, an arbitrary limit of 100 mm of predicted slope displacement was adopted to divide significant "damaging" displacement from insignificant movement. The prediction of displacements, and the corresponding minor increase in the critical slope angle which may result, has not been attempted here for the following reasons:

- The method assumes no reduction in shear strength during movement resulting from either soil remoulding, frictional heating or pore pressure build up and no soil sensitivity is considered. Most soils on Banks Peninsula, particularly loess and mixed colluvium, normally exhibit at least some degree of sensitivity (typically ranging from 2 - 4). As a result landslides are very seldom encountered where displacement has not led to a more rapid failure as a highly mobile earthflow; slides in which movement has ceased after a few tens of millimetres are relatively unusual.

- The calculation of predicted displacements on a scale of millimetres generally implies a much greater level of knowledge of both the material and the earthquake time history than is currently available for the Port Hills.

- Predicting the areal extent of landsliding becomes extremely complex when the shear strength of the material varies laterally in response to relatively subtle changes in soil moisture. Slope angle alone is not a sufficient guide in this situation and the effects of local topography, shading, vegetation and hydrogeology must all be considered.

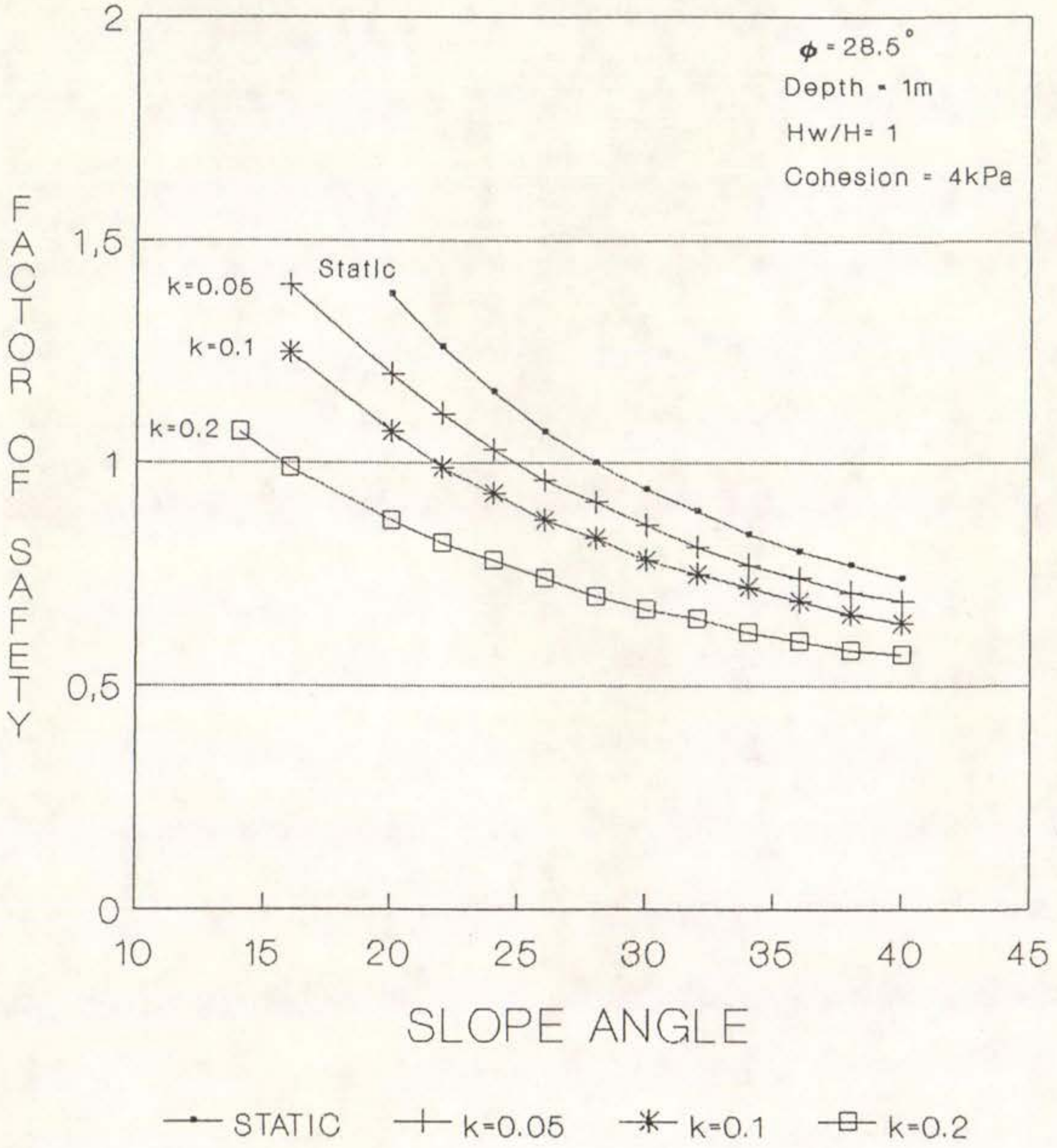


FIGURE 10.6

Factor of Safety - Slope Angle Relationship for a Saturated Loess Slope Subject to Seismic Acceleration

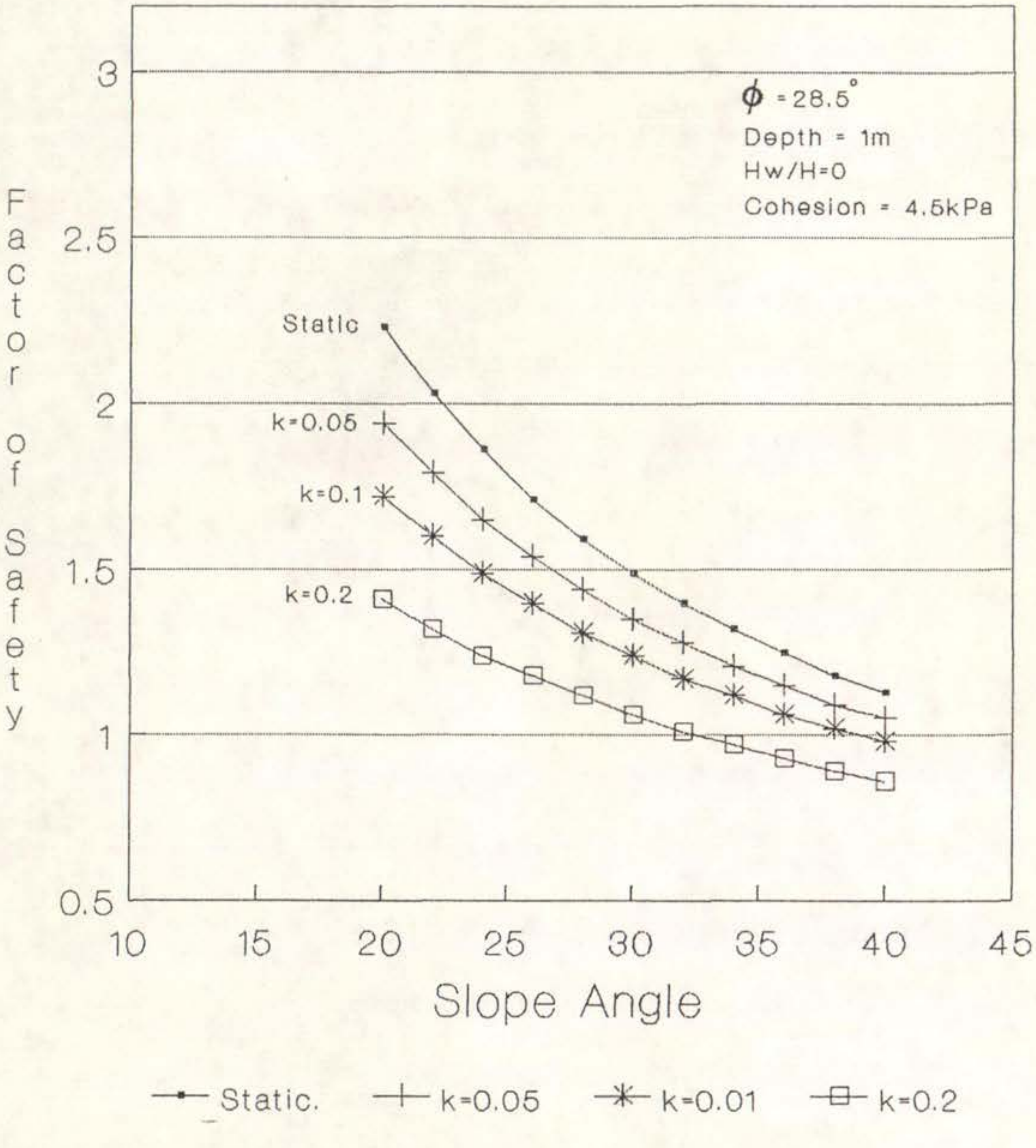


FIGURE 10.7

Factor of Safety - Slope Angle Relationship for Loess Slope with Water Table at 1.2 m Depth Subject to Seismic Acceleration

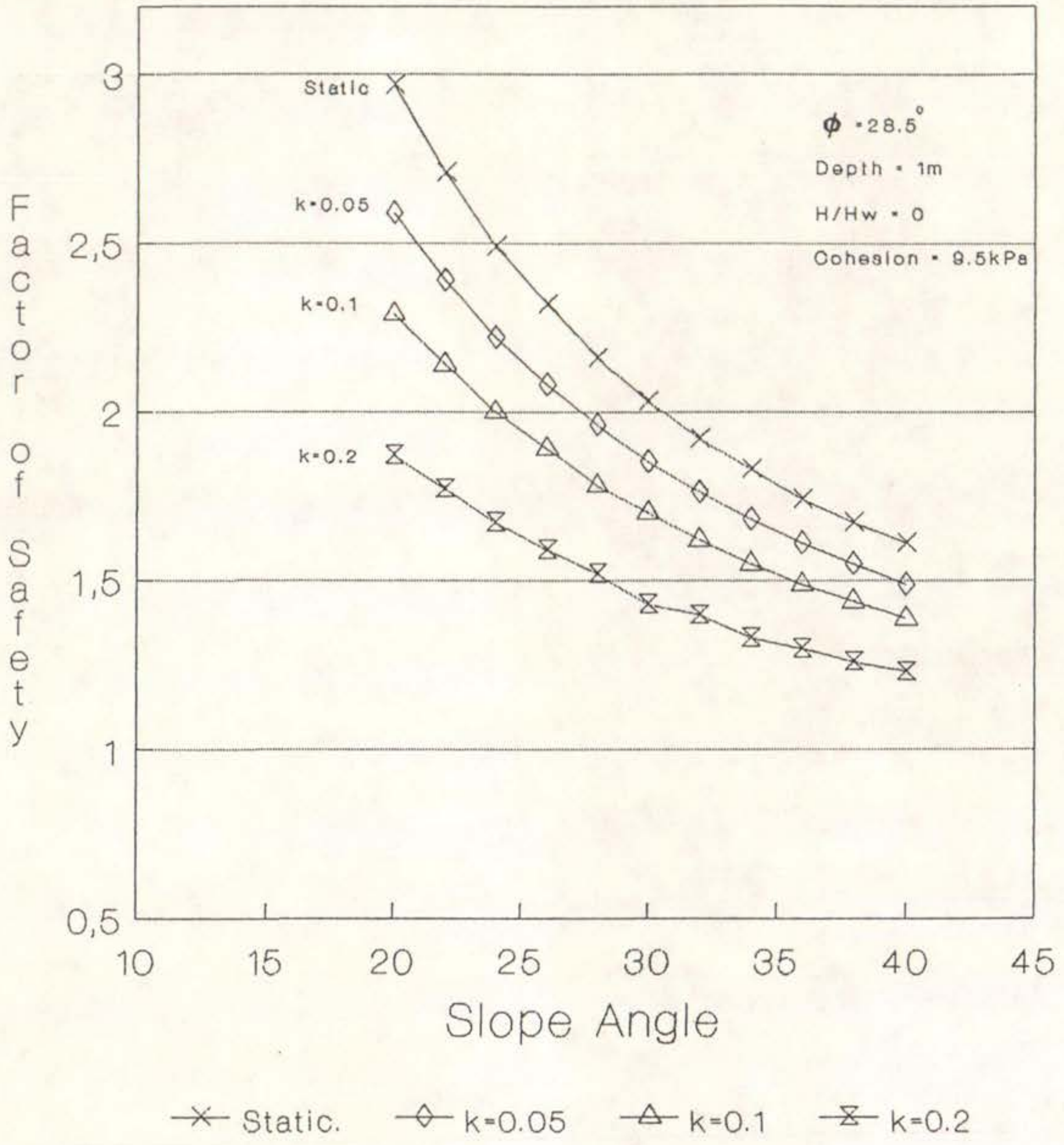


FIGURE 10.8

Factor of Safety - Slope Angle Relationship for Loess Slope with Water Table at 3 m Depth Subject to Seismic Acceleration

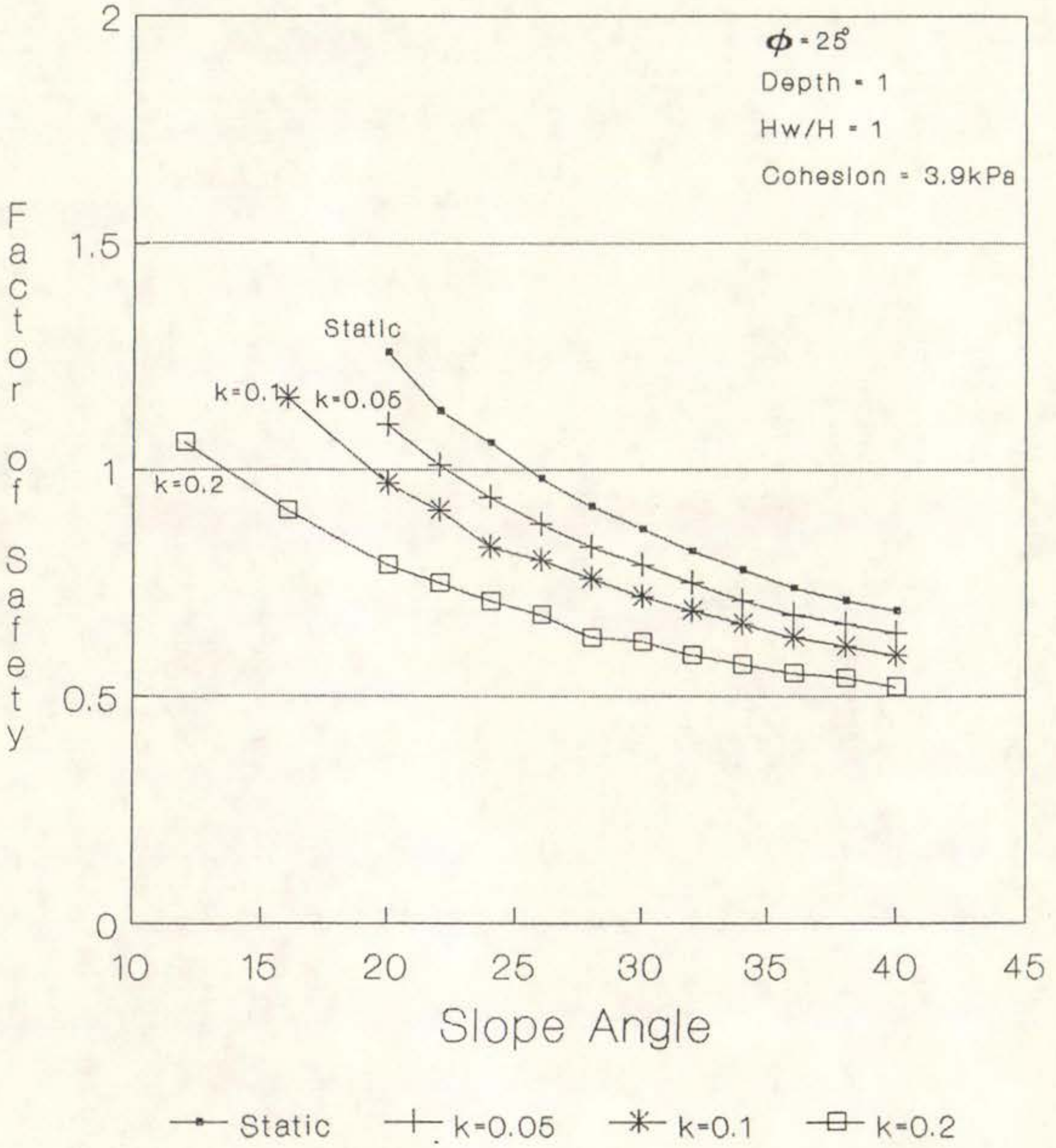


FIGURE 10.9

Factor of Safety - Slope Angle Relationship for a Saturated Volcanic Colluvium Slope Subject to Seismic Acceleration

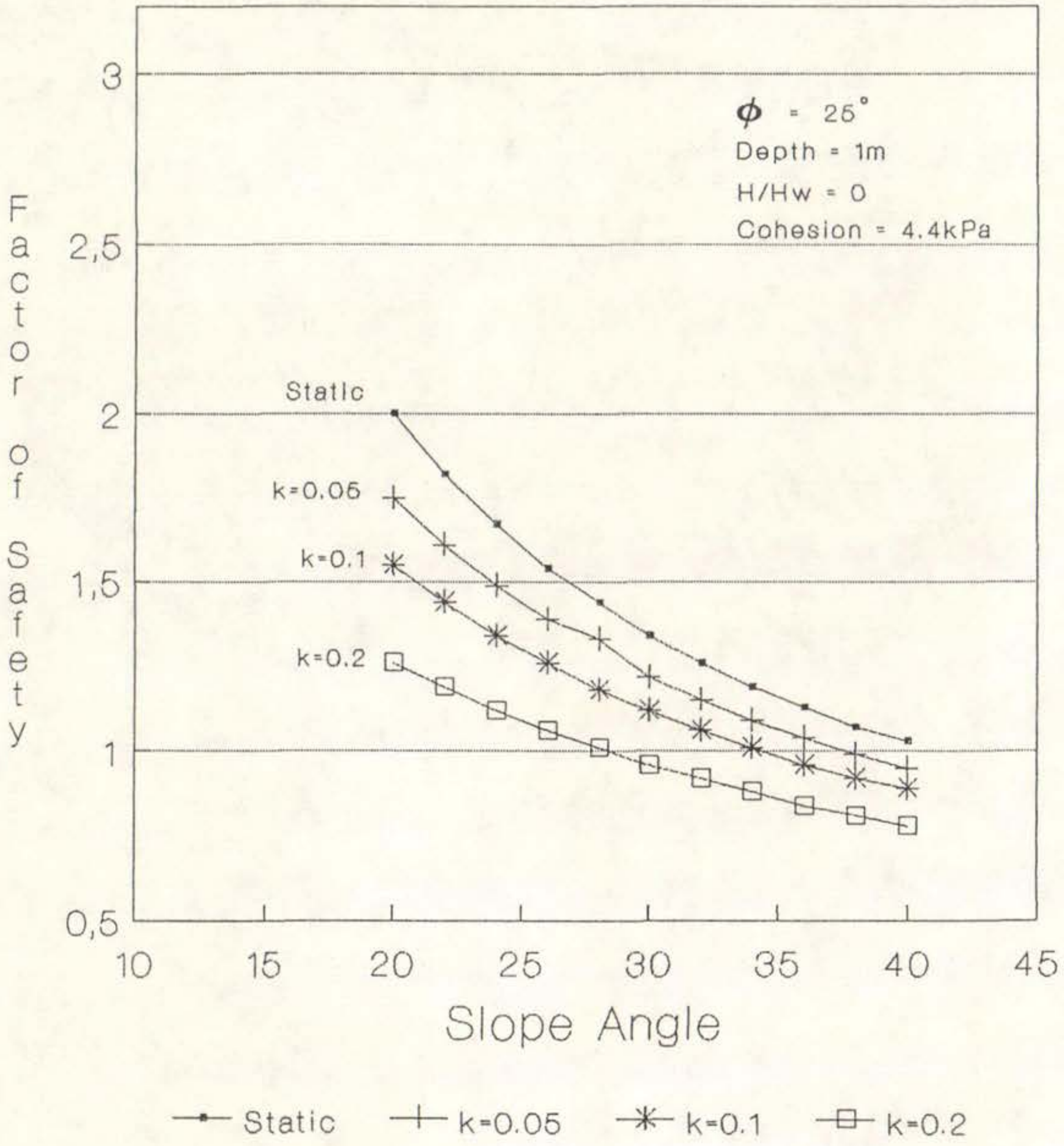


FIGURE 10.10

Factor of Safety - Slope Angle Relationship for a Volcanic Colluvium Slope With a Water Table at 1.2 m Subject to Seismic Acceleration

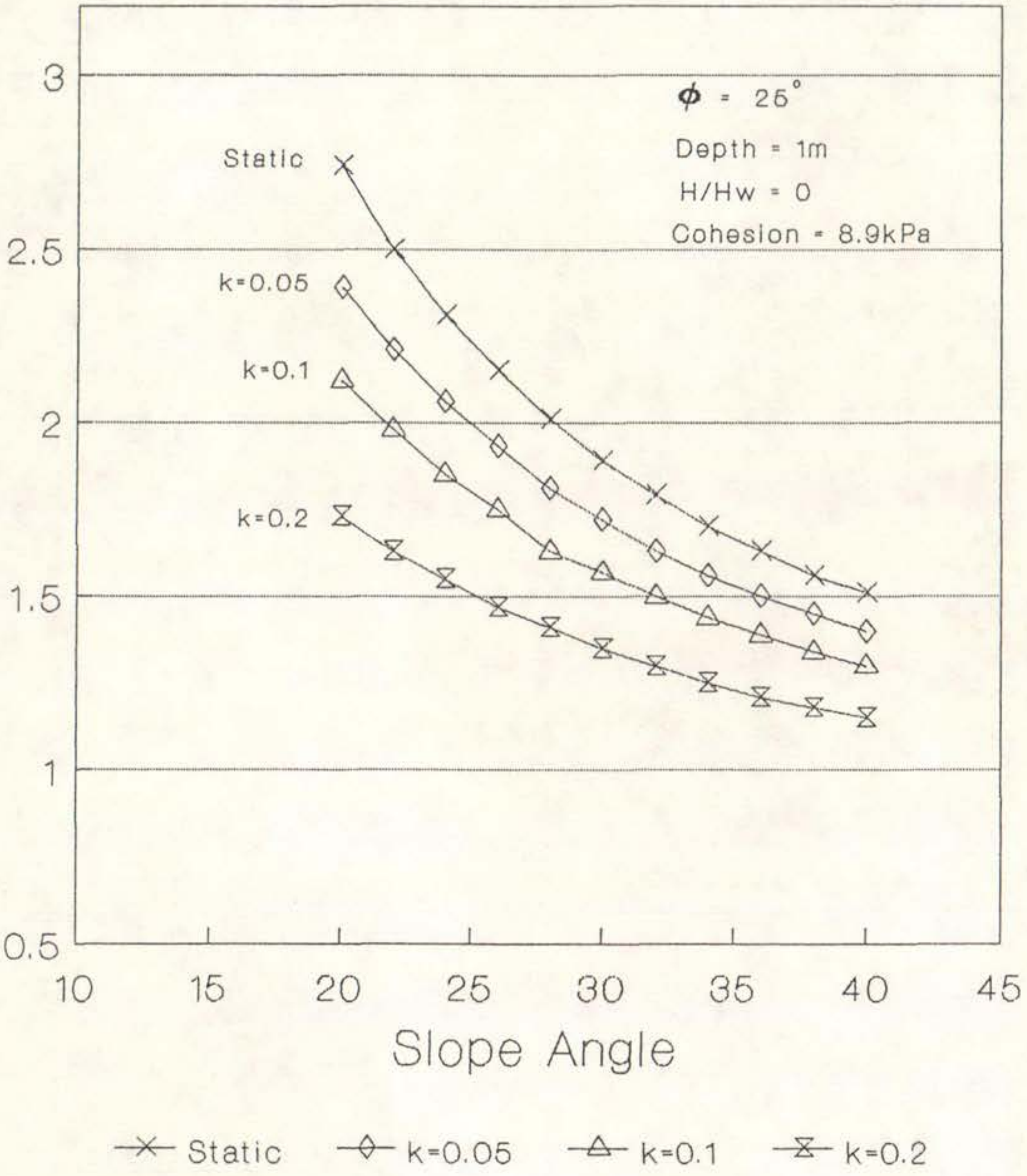


FIGURE 10.11

Factor of Safety - Slope Angle Relationship for a Volcanic Colluvium Slope With a Water Table at 3 m Subject to Seismic Acceleration

Implications for Hill Suburbs

Table 10.5 summarises all the results of slope stability analyses for typical Christchurch hillslopes, considering critical slope angle and the seismic coefficient, k_h. The initial impression from these results is that widespread damage can be expected on moderate to steep soil slopes on the Port Hills (say >20 degrees) if they are fully saturated when a substantial earthquake occurs. However most unsaturated slopes are likely to remain stable even under moderate to strong shaking.

Soil Type	Moisture Conditions	CRITICAL SLOPE ANGLE, for Lateral Seismic Coefficient, k_h			
		0 (Static)	0.05	0.1	0.2
Loess	Saturated to surface	28.5°	25°	22°	16°
	Water table at 1.2 m depth	>45°	45°	39°	33°
	Water table at 3 m depth	>45°	>45°	>45°	>45°
Volcanic Colluvium	Saturated to surface	25.5°	22°	19°	14°
	Water table at 1.2 m depth	42°	38°	34°	28°
	Water table at 3 m depth	>45°	>45°	>45°	>45°
	Return Period (years)		30	300	1500

Table 10.5: Critical Slope Angle (i.e. when FS = 1) for slides on an infinite slope 1 m deep.

Although most hill housing suburbs in Christchurch are located on soil slopes, the majority of these slopes have average angles less than 20°. Slopes between 10 - 20° are typical. Steeper exceptions do exist locally, particularly in the newer housing areas in the eastern hill suburbs. However consideration of gradient alone suggests a relatively large event would be required to cause extensive foundation damage (say intensities on rock in excess of Intensity MM VIII).

This must also coincide with a wet period and suitable local conditions for saturation. Figure 10.12 presents a water balance graph for Christchurch which shows the typical variations in soil moisture levels. Saturation of the ground is typically marked by the onset of a groundwater surplus around the middle of July and continues for about three months which, depending on spring rainfall, frequently extends into October. Even in this late winter wet period soil saturation in these suburban areas cannot be automatically assumed because of the local modifications caused by the "umbrella effect" of development on the soil infiltration characteristics. Typically only localised areas of the hill suburbs do get fully saturated, particularly those areas with undeveloped rural land above them.

The extent of foundation damage and landsliding in residential hill areas is therefore likely to be less than might at first be expected from Table 10.5. Generally the relatively low Christchurch rainfall, and the extremely effective improvement in average soil cohesion imparted by drier areas under houses, roads and driveways in combination is likely to prevent widespread shallow movement. In addition vegetation in gardens will further stabilise local areas.

Of most concern during a substantial earthquake are the suburban areas downhill of steeper rural catchments. The most significant hazard in these cases is not foundation sliding, but inundation by earthflows and debris from steeper areas above the houses. Figures 10.13 and 10.14 show examples of this type of location. In many areas residential development has halted at the base of long steep slopes without a sufficient buffer zone to protect properties against this type of hazard. These steeper areas are wetter as a result of the large upslope catchments and these same areas are also the most at risk from rock roll and rock falls.

10.5 ROCK SLOPES

In situ volcanic materials are more restricted in extent than soil materials in the hill areas of Christchurch. At lower elevations they are confined to the base of the eastern hill suburbs, where their outcropping marks the post glacial shoreline. Intermittent outcrop is common high up on the steeper valley sides and locally along the gently sloping elevated spurs (particularly around

Scarborough, Clifton and Mt Pleasant). Most of these gently inclined areas are located on the old Lyttelton Volcano dip slope.

These in situ volcanic materials can be divided into three groups based on their strength properties. The original andesite lava flow material tends to form the strongest rock mass with very high rock material strengths (uniaxial strengths frequently in excess of 100 MPa) and commonly contain relatively persistent subvertical cooling joints. Other joint orientations sub-parallel to the original flow direction are typically represented and joint spacing is generally closest spaced towards the original flow contacts.

The second group of materials include rubbly lava and agglomerate which typically mark the cooling boundaries of the flows, but can also occur in isolation intergraded with the stronger andesite. Joints are less frequent and often appear to be secondary stress relief features subparallel to the existing topography. The rock material is generally weaker than the main flow material however the absence of primary jointing can frequently mean overall rock mass strengths are comparable.

The third group of materials includes the pyroclastics and laharic materials which are generally weak and can frequently be classified as engineering soil or soft rock. Jointing is relatively uncommon but the softness of this material means it is often preferentially eroded in the steeper exposures.

A coarse colluvium of angular boulders is frequently present below steeper areas of bedrock outcrop. This frequently grades into finer more cohesive volcanic colluvium. The boulders may be scattered down the slopes having rolled some distance from their original source.

Static Failure Modes

Damaging slope failures under static conditions within in situ volcanics are relatively rare, reflecting the material strength and restricted distribution adjacent to housing. In general two types of failure can be recognised:

Fretting, spalling and associated rock falls which generally result from stress relief and weathering, which progressively opens joints in the rock mass creating small isolated falls of a few blocks of material. Occasionally larger falls do occur, for example the rock fall in 1986 which closed the Edwin Mouldey track at Scarborough and damaged the pumping station. Materials most prone to this type of failure belong to the first two rock type groups described above. In particular the rubbly lava and agglomerate appear to weather and spall quite rapidly releasing a regular supply of material. Falls of the stronger andesite are less frequent but more dramatic. In general the eastern suburbs from the estuary to Taylors Mistake are the most susceptible to rockfall damage under static conditions.

Rock roll occurs where the products of the processes described above fall onto a steep slope apron and inertia carries these further, either to the slope base or to some intermediate temporary location. Damage to housing from this process is a relatively infrequent occurrence in Christchurch residential areas but can be dramatic when it does occur. A case occurred from Governors Bay in 1986 where a rock from approximately 200 m upslope passed through the rear wall of a house, and continued out the front, removing some sliding doors in the process.

Seismic Triggered Instability

Site specific seismic analysis methods are available for rock slopes (see for example Hoek & Bray, 1981) and the incorporation of pseudo-static seismic loadings in these methods is not difficult in principle. Unfortunately the local geological complexities and rapid variations in jointing and associated shear strength in rock slopes do not allow quantitative seismic analysis of idealised slopes as a rational guide to likely general behaviour. Similarly the geometry and height of rock slopes in the Christchurch area vary so widely that generalised analysis is not possible. Thus this section is restricted to general comments on likely types of failure.

Topple Failures involve outward rotation of columns or blocks of rock about some relatively fixed base. Harder rock masses with subvertical joints are most prone to this type of failure which does occur locally under static conditions. However an earthquake involving a significant component of outward acceleration is a very effective trigger for this type of failure. The interval between large earthquakes is frequently sufficient to allow progressive joint relaxation and the wedging open of subvertical joints by the action of water, vegetation, frost and salt. The earthquake is then the final trigger event in a progressive process. Figure 10.15 shows an example of a column of andesite potentially vulnerable to topple failure in an earthquake. This type of failure is normally observed on relatively steep rock cliffs and the toppling products do not normally move far out from the base of the cliff. Houses built within 10 - 20 metres of significant rock faces are the most at risk.

Rock Slides are larger failures involving deeper composite slide surfaces normally related to rock mass jointing. If the rock mass is finely jointed the resulting failure may resemble a rotational slide in homogenous soil.

Large failures of this type are not common under static conditions, however we have observed landforms formed by a number of large rock slides in areas of Banks Peninsula. Figure 10.16 shows an example from Purau valley. The location of this feature on a spur, and the absence of local springs, suggests groundwater may not have been a major factor in initiating this slide. We consider it likely that the majority of original failures responsible for landforms on this scale have been earthquake triggered events.

The distinction between topple failures, slides and rock falls is frequently academic as many rock failures involve components of all these processes. The term "rock fall" is used here to imply a substantial free fall component and the process can be expected in steep rock road batters and steep rock cliffs. Weathering and stress relief in the volcanic materials frequently results in a relatively loose outer skin of random jointed material vulnerable to earthquake shaking. This is particularly obvious in exposures of rubbly lava and agglomerate.

Rock Roll can be expected in many of the same locations at risk from earth and debris flow inundation i.e. areas at the base of substantial slopes steeper than 20 - 25° which contain intermittent rock outcrops. The rocks themselves may already exist on temporary locations on the slope, and shaking recommence their downhill movement, or they may be the products of simultaneous earthquake triggered failures in existing outcrops.

In general the various types of failure in rock conform to the "disrupted slides" category of Keefer, 1984 (refer again Figure 10.1). Analysis of the curve marking the threshold for failure in this figure indicates a site shaking intensity of between 5.8 and 6.8. This is relatively low and has a correspondingly high return period of 12 - 20 years (from Figure 5.8). This is substantially different to the 300-year return period deduced earlier for significant sliding in soil slopes in, or adjacent to, residential areas and reflects an important difference between the mechanisms. Exposed rock faces are often steep and progressively lose strength over time by weathering so that a relatively modest earthquake (with a corresponding short return period) can be the final failure trigger. Only a few vulnerable faces will fail with every earthquake, but on a regional basis sizeable areas can be affected on each occasion. In contrast the earlier analysis for the soil slopes in, or adjacent to, residential areas reflects the relatively constant strength of these soils over time and the more moderate slope angles typical of these areas.

In conclusion, areas of rock slope will fail more frequently than typical soil slopes in, or adjacent to, residential areas with return periods corresponding to shaking Intensity VI to VII i.e. 12 - 25 years. The total area of rock face failing may be small, particularly at this lowest threshold of shaking, with different areas failing at each event. The consequences for housing will generally only be serious where houses are very close to steep rock faces, or below more gentle valley sides above which rock outcrops exist and damage would be by rock roll. A relatively small percentage of houses in vulnerable locations may be damaged in any one event.

10.6 LIQUEFACTION INDUCED SLOPE FAILURES AND LATERAL SPREADS

Liquefaction has been suggested as the cause of massive earthflows and damage in loess soils of the Eurasian continent during a number of large earthquakes. Seed (1968) reviews details of damage in the Kansu Earthquake in 1920 and quotes the description of Close & McCormick (1921), made after the event, in which loess "*with the appearance of having shaken loose, clod from clod and grain from grain,..cascaded like water forming vortices, swirls and all convulsions into which a torrent might shape itself*". This ground failure in the province was responsible for the loss of nearly 200,000 lives.

The descriptions of the Kansu material implies a dry soil and has raised the possibility of liquefaction in dry loess presumably resulting in some way from pore air pressures (see for example Casagrande, 1950). Lutenegger (1981) has since suggested the suggested an alternative explanation which assumes the presence at depth of a zone of unstable saturated collapsible loess below more stable material.

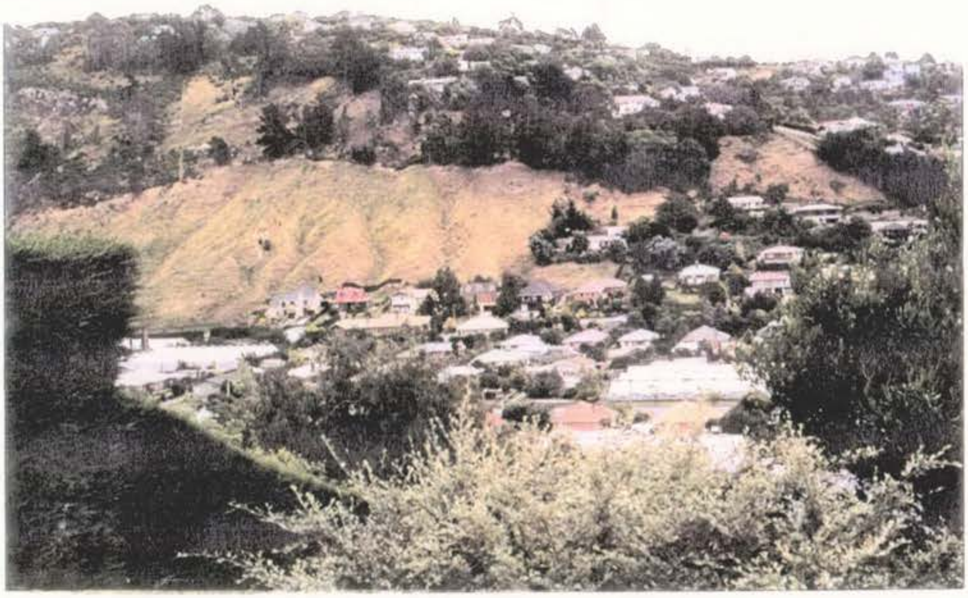
Liquefaction in loess was repeated on a large scale in the Chait Earthquake which occurred in the U.S.S.R. in 1949. These slides occurred on steep mountain slopes covered with thick loess which had been subjected to heavy rain before the earthquake. Earthflows of loess moved into the main valley and inundated 21 villages along the Yasman Valley. Seed (1968) reports similar slides on a smaller scale during five other earthquakes in the region, and in most of these cases the loess had been earlier subjected to heavy rain.

No cyclic triaxial testing or shaking table work has been carried on undisturbed Banks Peninsula loess to date. This would be the best method of establishing the susceptibility of the material to liquefaction during cyclic loading. In the absence of this testing only tentative conclusions can be drawn from a comparison of loess index properties.

Available published information (Lin & Wang, 1988; Lysenko, 1971) suggests the Eurasian loess is often more sandy than Banks Peninsula loess, and has a substantially higher void ratio (reflected in a lower dry density). As a result of the coarser grain size the Eurasian loess plots within the range of potentially liquefiable silty sands based on the standard criteria of Tsuchida (1970). The higher void ratio means the pore medium, be it air or water, is more abundant and shear strengths are likely to be lower. Within a typical vertical profile of Banks Peninsula loess the relatively sandy varieties of the P layer most closely resembles the Eurasian material.

An alternative approach in the assessment of the likelihood of deep seated liquefaction is to look for the local landforms which could be expected from this process if it has occurred in the area in the recent past. Loess deposition had essentially ceased on Banks Peninsula by the Holocene and many of the

Steep slopes above housing with potential to generate significant damaging debris during an earthquake (Sumnervale Drive)



Similar potentially unstable slopes with obvious channels tending to concentrate falling debris during an earthquake (Albert Terrace)

FIGURES 10.13 AND 10.14

Rock face with obvious topple block. This type of column is very sensitive to outward rotation (Sumner area)

Old rock slide scarp. Failures on this scale are not common under static conditions and may have occurred during previous earthquakes

FIGURES 10.15 AND 10.16

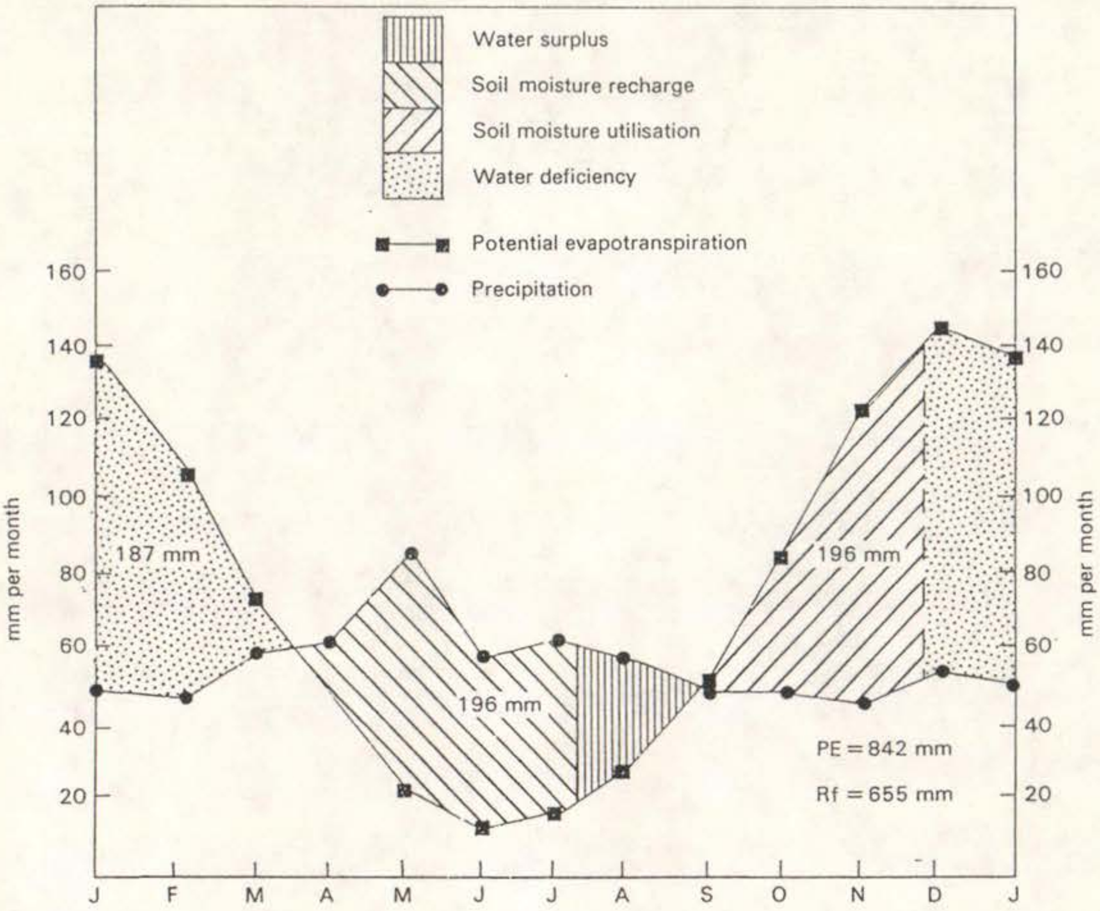


FIGURE 10.12

Average Water Balance for Christchurch (1940 - 1975) (After Trangmar, 1983)

moderately inclined surfaces which were formed have been subjected to only minor local erosion since that time. In many areas gullying is obvious in these surfaces and has resulted from fluvial erosion and associated "conventional" landsliding. The gully floor gradient is generally steep but evenly graded through to the gully head without sudden changes or scarp like steps.

The liquefaction slides in the Eurasian failures are not well described, however the maps of the damage in Seed (1968) suggest sliding was localised along the valley floors, and the side gullies of the lower slopes, where the soil was generally wettest. The slide volumes and descriptions imply deep failures which would leave obvious scarps on the valley floor as nick points in the longitudinal profile of the ephemeral streams. While we are aware of at least two gullies that exhibit this type of landform in the Lyttelton Harbour basin, both in the vicinity of Cass Bay, in general this type of landform is not common and the geomorphic evidence does not suggest liquefaction in loess has occurred on a large scale in Holocene times.

Lateral spreads in alluvial materials in Christchurch are possible where liquefaction occurs in liquefiable areas with sufficient gradient. In the overseas examples of lateral spreads triggered by earthquakes, the slope angles generating movement are very low, as little as 5° in some cases, which reflects the complete loss of strength and fluid behaviour of the basal failure surface.

Figure 9.1, presented earlier in Chapter 9, shows the areas of Christchurch underlain by potentially liquefiable materials based on the limited existing information. Gradients sufficient for lateral spreading exist within these materials along the Avon River downstream of Kerrs Reach; the Heathcote River downstream of Opawa Road; and along all the margins of the Estuary. A few of the smaller creeks and rivers, where they pass through sand areas e.g. Dudley Creek, Linwood Main Drain, Horseshoe Lake etc., are also potentially subject to lateral spreading.

An example from our investigations is shown in Figure 10.17. In this location (adjacent to the Estuary) we carried out specific investigations and identified a shallow layer of liquefiable, saturated loose sand which improved in density with depth. In view of the potential for liquefaction and lateral spreading the new house proposed for the site was built on a piled foundation which extended through the loose material to the firmer sands below.

Given the current limited state of our knowledge of the sand characteristics in these areas, it is not possible to be more specific in our comments. Potential for liquefaction and lateral spreading certainly exists, and future detailed investigations are required to evaluate the level of this risk.

10.7 SUMMARY

Comparison with overseas studies suggests that maximum magnitude earthquakes on a number of active faults and seismicity zones near Christchurch could trigger slope failures. These include large earthquakes on the central section of the Alpine Fault, the Kaiwara Fault, and the Porters Pass Tectonic Zone; and earthquakes within the Pegasus Bay, Banks Peninsula, and Christchurch seismicity zones.

Pseudo-static stability analysis of slopes underlain by soil indicates the critical importance of pre-existing moisture levels in controlling the extent and severity of landsliding. For saturated soil slopes the critical slope angle (i.e. the lowest slope angle at which failure is possible) reduces from around 28° under normal conditions to about 22° in a substantial earthquake (MM Intensity VIII - IX). The predicted return period for this level of shaking on the loess and colluvium covered slopes of Banks Peninsula is about 300 years. The extent to which saturation exists at the time of the earthquake is controlled by a combination of the time of year and various site specific factors. In general the most at-risk period is from July to October, and the most vulnerable housing areas are those on the fringes of existing development below steep hillslopes. Houses in these locations may be damaged by foundation sliding, debris inundation and boulder roll. Damage is also likely within the hill suburbs in areas of local saturation or particularly steep slope angle.

Rock slides, topple failures and falls are likely during a substantial earthquake in the eastern suburbs where high cliffs have been produced by coastal erosion and local quarrying. Damage to houses from these processes may be limited in extent but roading may be more widely affected.

Liquefaction-induced landsliding in loessial soils during earthquakes as been reported overseas but initial consideration suggests this process may not affect Christchurch hill soils. However lateral spreading in alluvial materials following liquefaction is more likely to occur. Potential for this process appears to exist in some areas, in particular, along the lower reaches of the Avon and Heathcote Rivers and around the margins of the Estuary.

Important further work includes a specific study to identify at-risk hill areas, including an owner education and remedial planting programme in the areas so designated; and further investigations along the margins of the rivers and the Estuary to better define the risk of lateral spreading in these areas.

CHAPTER 11: Potential Damage

11.1 Introduction

An earthquake of magnitude 7.5 with an epicentre 60 to 80 km away from a coastal city of 300,000 causes extensive damage to engineering structures, 26 deaths and $1 billion of damage. The buildings are of modern, reinforced concrete and have seismic design provisions, but extensive damage is caused by liquefaction in the alluvial river delta soils.

Another city is located on an alluvial coastal plain around the mouth of a river. Earthquake-induced liquefaction causes most buildings in the central business district to settle 1-2 m, and the streets are inundated with 0.5 m of sandy water. The building settlements cause the adjacent street to heave up with damage to services below the footpath and buried fuel tanks pop to the surface. Lateral spreading of material occurs along the river, causing more damage and ruining some of the town's water supply wells. The underlying aquifer is damaged.

These descriptions could conceivably be describing Christchurch subjected to a large earthquake. In fact, they are Niigata, Japan in 1964, and Dagupan, Phillipines in 1990.

This chapter suggests the type and extent of damage that could result from a major earthquake affecting Christchurch. Any such damage scenario is conjecture, but can be realistically based on earthquake damage experienced elsewhere in the world, combined with knowledge of the local conditions and the type of ground motion that can be expected. For Christchurch, the most significant local conditions are the deep alluvial soils which have a marked effect on the spectral response accelerations (see Chapter 7), and the large areas of the city underlain with soil types susceptible to liquefaction (see Chapter 9). The areas of greatest amplification of the response spectra, and the areas most susceptible to liquefaction are shown on Figure 11.1.

Local conditions apply not only to geological and topographical features of Christchurch, but also to the details of design and construction of the buildings, services and other man-made features of the city. In this respect the 1987 Edgecumbe earthquake provides a useful guide to the performance of New Zealand buildings and services. The area of most intense shaking was largely on the Rangitaiki Plains with a considerable depth of alluvial material overlying bedrock, and similarities to Christchurch in the response spectra, and amplification, can be expected. The Edgecumbe earthquake of magnitude 6.3 was only of moderate size, but maximum intensities of up to MM X were recorded at Edgecumbe. Much of the damage described below is based on the Edgecumbe experience (Leslie, Hunt and Morrison, 1988; Lloyd, 1988; Building Research Association, 1987; Ruscoe, 1989).

In the following sections, possible effects on the various services, housing and structures are considered for an earthquake event producing felt intensities of MM VIII to MM IX in Christchurch. The return period for a felt intensity of MM VIII at bedrock in Christchurch is about 100 years and the probability of it occurring in any 50 year period is about 40%. As this bedrock intensity of MM VIII will produce felt intensities in Christchurch of at least 0 - 1 MM unit higher, as discussed in section 7.3, felt intensities of MM VIII to MM IX (and possibly as high as MM X in some locations) can be expected to occur, on average, about every 100 years (refer Figure 7.5a).

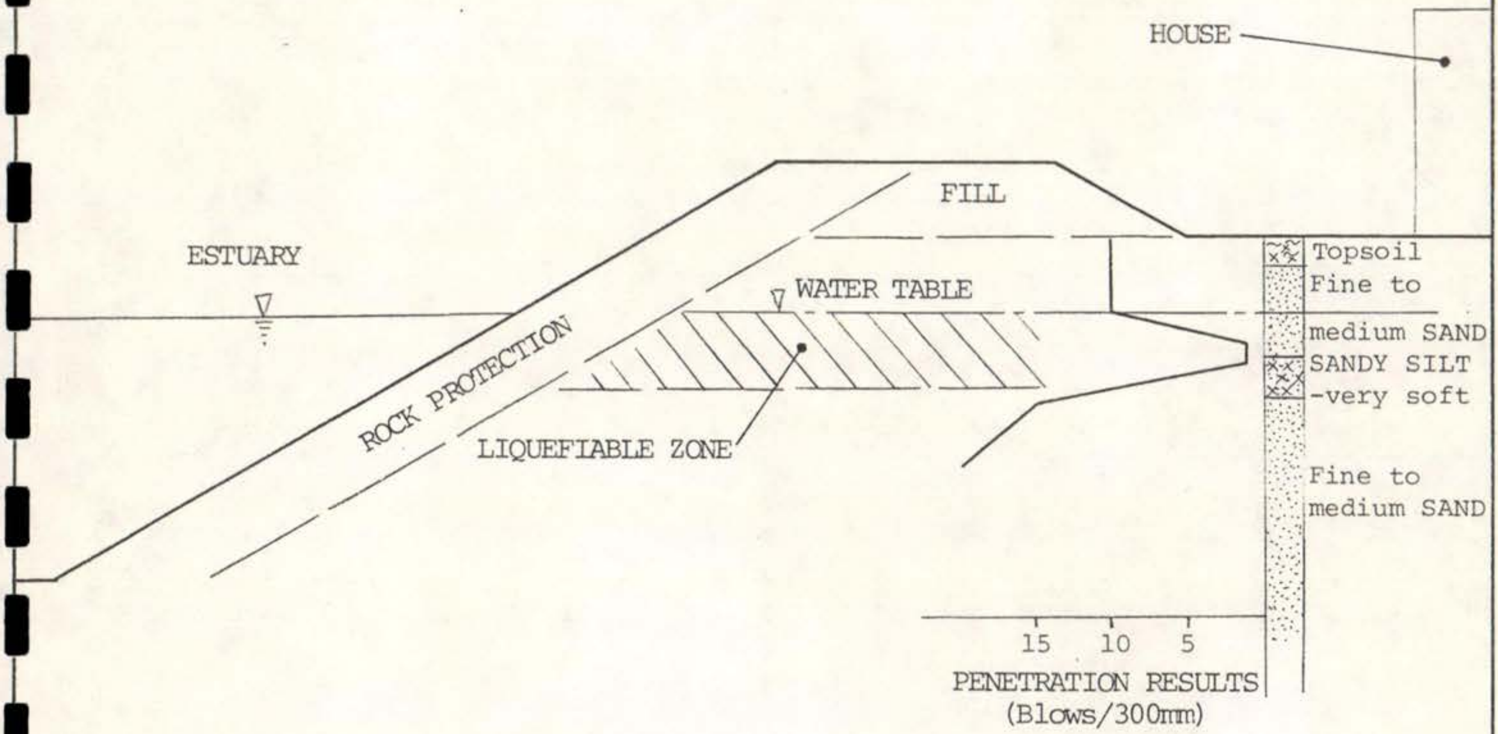
11.2 Structures

The most important aspect to emerge from this study with respect to the design of structures is the effect the deep alluvial soils have on the spectral response accelerations. The response spectra determined for Christchurch which incorporate specific site effects (Figure 7.16) are markedly different from the bedrock response spectra, and the current design spectra (Figure 7.1). Taller buildings, or structures with long periods, may be subjected to considerably larger seismic loads than those for which they were designed, for the same return period event.

The effects of this will vary with the age of a building, the structural system used, the type of construction material, and type and extent of secondary elements such as cladding and partitions.

Few buildings constructed prior to 1935 had any provision for earthquake loading, and most structures from this era with a period of more than about 0.4 seconds can be expected to perform poorly in a large earthquake. Seismic provisions were introduced in 1935 building bylaws, and subsequently upgraded in 1955 (NZSS 95), 1965 (Chap. 8, NZSS 1900) and 1976 (NZS 4203).

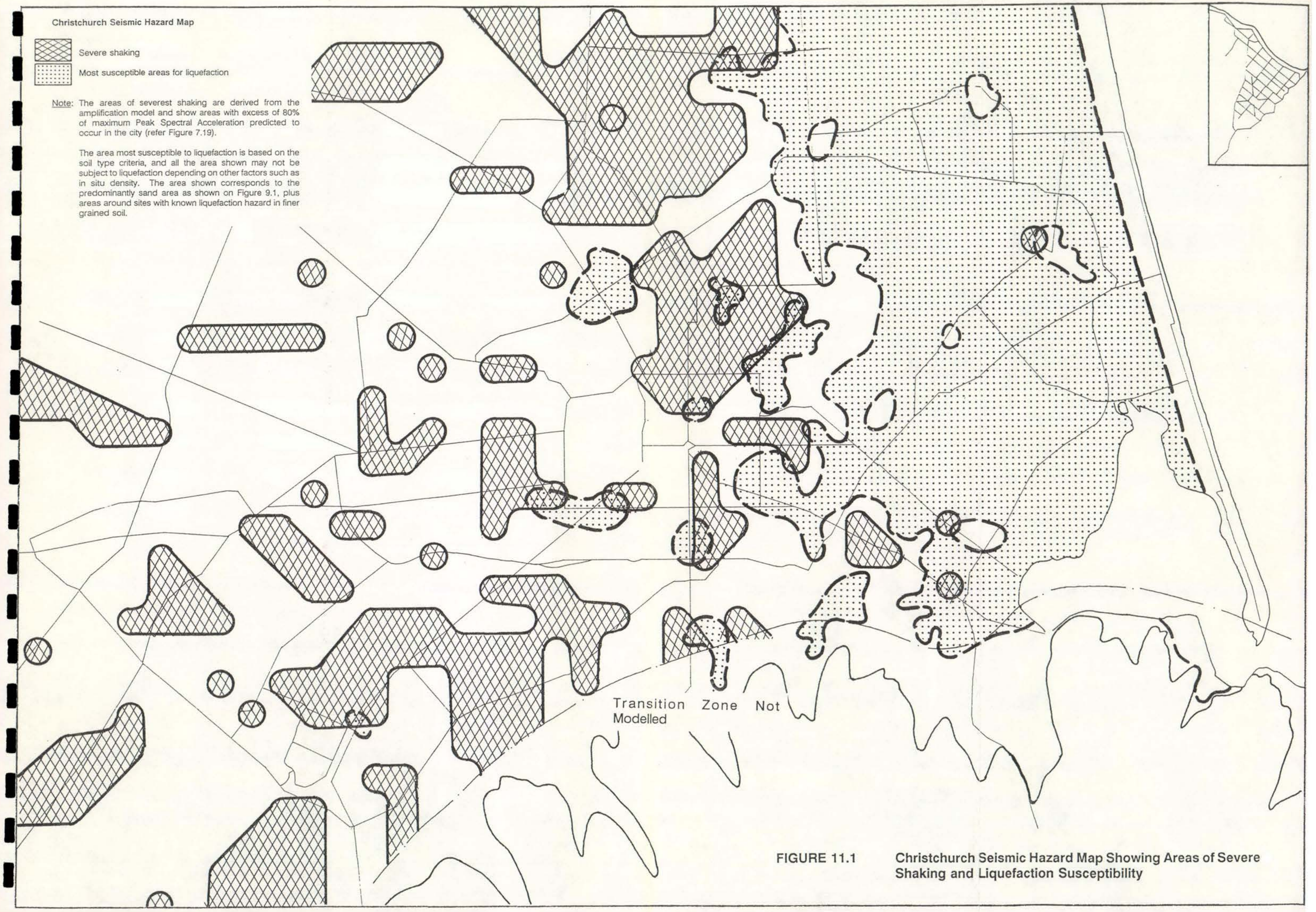
The different structural systems used may mean some buildings will be able to withstand greater seismic loading without collapse than others. Ductile frames in high rise buildings can respond by forming more plastic hinges concurrently over a wider zone in the building than assumed in the design, as such structures can redistribute loads more extensively through the building. This would mean that lateral movement could be excessive, with subsequent pounding of closely adjacent buildings. Shear wall structures do not have such an obvious load redistribution ability, although buildings complying with NZS 4203 are designed with a higher structural type factor. For any structures of



CROSS SECTION - REDCLIFFS FORESHORE

FIGURE 10.17

Investigation Results Indicating Potential for Liquefaction and Lateral Spread at One Location Adjacent to the Estuary



Note: The areas of severest shaking are derived from the amplification model and show areas with excess of 80% of maximum Peak Spectral Acceleration predicted to occur in the city (refer Figure 7.19).

The area most susceptible to liquefaction is based on the soil type criteria, and all the area shown may not be subject to liquefaction depending on other factors such as in situ density. The area shown corresponds to the predominantly sand area as shown on Figure 9.1, plus areas around sites with known liquefaction hazard in finer grained soil.

FIGURE 11.1 Christchurch Seismic Hazard Map Showing Areas of Severe Shaking and Liquefaction Susceptibility a long period and reliant on diagonal bracing, failure of the bracing in response to loads exceeding design loads could result in total collapse.

For buildings designed to behave in a ductile manner, incorporating a "softening" effect to extend the fundamental period of the building after the initial seismic loading, location within Christchurch will also be important. Sites with fine grained material between 10 and 20 m depth have a response spectral shape that has very little reduction in acceleration at longer periods (Type A, Figures 7.16 and 7.17). Sites on coarser material have a more rapid post peak decrease in spectral acceleration (Types B and C). Figure 7.18 shows the general areas within Christchurch conforming to each type of spectral response. Areas in Redwood, Papanui, Riccarton, Ilam, Spreydon and Waltham have the particularly uniform response spectra of Type A.

As indicated in the brief discussion above, the implications of the results of the seismicity and amplification studies for structures are complex. More detailed considerations are outside the scope of this report, further work is needed to evaluate the important ramifications for both existing buildings and future construction.

Aside from the structural performance of large buildings, considerable damage may result from inertial effects of large amplitude movement, or may occur to fittings, furniture and building services. As an example, the 6-storey tower blocks at Whakatane Hospital sustained no structural damage during the 1987 Edgecumbe Earthquake although the two buildings swayed by up to 130 mm. However the movement flung sterile equipment to the floors, which were then flooded with water from the water system which burst. Ceiling tiles and asbestos in the ceilings also shook free. Repair work was anticipated to take up to one year to complete.

11.3 Housing

The deep alluvial soil response in Christchurch will change the bedrock response in both amplitude and period, with the result that little ground motion of 0.2 second period or less will occur (see Section 7.4). For one or two storey houses on the flat, with a short fundamental period of less than 0.2 seconds, it is therefore expected that there will be little damage from resonance. (Houses on the hill will be more vulnerable to resonant damage as they are founded on or close to bedrock). However inertial effects may be considerable, given the large amplitude ground motion likely. Damage to heavy furniture and fittings, hot water cylinders, chimneys and heavy tile roofs may be considerable although structural damage to houses could be minimal.

The 1987 Edgecumbe earthquake probably gives a good indication of the type and extent of damage likely for felt intensities up to MM X. Dowick & Rhoades (1990) report "mean damage ratios" for the Edgecumbe earthquake of:

MM VII	0.0063
MM VIII	0.021
MM IX	0.07
MM IX	c. 0.08 (adjusted where MM X isoseismal exists)

where the mean damage ratio is equal to the total cost of damage divided by the total replacement value of the houses in the chosen intensity zone. It does not include damage to household contents. The return period for intensity MM IX in Christchurch is about 100 to 500 years. These damage ratios for MM >VI are significantly lower than reported from previous studies. This may reflect the probable attenuation effects of the deep alluvium of the Rangitaiki Plains and the similar low spectral response as modelled for Christchurch for the period range less than 0.2 s applicable to most New Zealand houses. No attempt has been made to compares ages and construction types of houses in the Edgecumbe study with Christchurch housing stocks. This, together with research into the Edgecumbe response spectra, would be a valuable addition in predicting likely domestic building damage in Christchurch.

In general, houses constructed in accordance with the Code for Light Timber Framed Construction, NZS 3604, and older well-built houses survived the Edgecumbe earthquake without structural damage. Fewer than 50 houses suffered substantial structural damage that made them uninhabitable, out of a total of 5,300 dwellings in MM VIII and MM IX zones (and 29,000 within MM >VI zone). Of these 50, the damage was mostly to, or because of, poor foundations such as unbraced piles.

Building performance was shown to be very dependent on the local ground conditions, and was illustrated by one of two almost identical, adjacent houses falling off its foundations while the other was displaced only slightly. This sort of variation in soil conditions, inferred from this example, is on too fine a scale to be determined from the soil classification study reported in Chapter 7, but is known to occur in Christchurch. The soft ground in the Edgecumbe earthquake area apparently moved beneath some buildings, compressing the soil around the foundations and damaging services into the building. Again this sort of effect can be expected in Christchurch, particularly in the eastern areas.

Chimneys were particularly vulnerable in the Edgecumbe earthquake, as has been demonstrated before in previous New Zealand earthquakes. Several free-standing stoves and fireboxes were dislodged due to insufficient fixing. Similar damage can be expected in Christchurch for equivalent felt intensities, and if this occurred during winter, house fires would be likely.

11.4 Services

Water Supply and Reticulation

The Christchurch water supply is sourced from the gravel aquifers beneath the city. Severe earthquake shaking is likely to cause at least temporary disruption to this supply. The aquifers may be damaged, and variations in well yield, turbidity and chemical traces can be expected. Well pumps are vulnerable to any power surges that may occur as equipment in the electrical supply is damaged, and the well casings themselves could be damaged and bent by ground displacements.

The water reticulation system is likely to be significantly damaged. During pipeline - soil interaction, pipeline strain increases as the wave propagation speed through a soil decreases. Wave propagation speeds are lower in soft and/or fine grained soils, such as is found in much of Christchurch, and this effect will be accentuated by the depth of alluvium under the city. Pipeline strains can therefore be expected to be relatively high, resulting in high pipe stresses and more frequent joint displacements.

Pipelines parallel to the wave propagation direction, that is pointing generally in the direction of the epicentre, are more vulnerable than pipelines at right angles, so that damage may be patterned through the network in relation to its orientation. Damage to the reticulation is likely to be mainly joint failures; either pipes pulling apart, or if the Edgecumbe earthquake is relevant to Christchurch, predominantly compression failures where the spigot end of one pipe is pushed into, and splits, the socket end of the next. At Edgecumbe, joint failures were widespread, and not necessarily at locations related directly to ground deformations. Pipe sizes 200 mm in diameter or greater survived better than smaller pipe sizes. Pipes of different material (and ages) can also be expected to suffer different degrees of damage. Pipeline fractures from faulting are considered very unlikely. Propagation of any fault beneath Christchurch to create a ground surface trace would almost certainly be averted by the deep alluvium.

Sewerage Reticulation and Treatment

The sewerage system in Christchurch could be seriously damaged in a severe earthquake. The sewerage network all converges on the treatment plant in Bromley which is situated in an area of potentially liquefiable sand. Without in situ density data, it is not possible to be definite about the liquefaction potential, but the information from other similar sites in Christchurch suggests a high probability of this hazard (refer Chapter 9). If liquefaction occurred over even some of the area around Bromley, the following damage could be expected:

- Foundation failure under the larger structures and buildings at the treatment plant resulting in tilting or collapse.

- Flotation of pumping wells, and treatment basins.

- Flotation of the main sewers feeding into the treatment plant.

- Flotation or loss of foundation support to pumping stations in the area, with resultant level misalignments, tilting, and fractured pipework.

- Foundation failure to the oxidation pond embankments, and possible lateral spread to adjacent low areas and the estuary.

With this possible damage to the key components of the system, it is conceivable that Christchurch would be without an operational sewerage system for several months following a major earthquake, with consequent health hazards.

The sewerage reticulation system can be expected to be damaged in a similar way to the water reticulation, but with additional damage in those areas where liquefaction occurred. Sewers are generally constructed at greater depths than water mains, frequently below the water table in Christchurch, and except for pumping mains these generally flow partly full. Flotation under liquefaction conditions in the surrounding soil is probable, with resulting dislocation and breakage at joints and lateral connections (sewer mains came to the ground surface in Niigata in 1964 with the severe liquefaction there). Sewer manholes can similarly undergo movement; some 25% of the manholes in Edgecumbe moved upward relative to the road surface - though whether this was compaction of the soil or flotation of the manholes is not clear.

Liquefaction could also severely damage pumping stations. At Whakatane, an 8 m diameter, 6 m deep pumping station floated upwards some 200 - 300 mm, and an attached chamber rotated and separated from the main caisson. The pumps continued to operate after the initial damage, and pumped a large quantity of sand slurry into the pumping main.

The oxidation ponds could be severely damaged by liquefaction foundation failure or lateral spreading as mentioned above. Earthquake induced wave damage is likely, given the large size of the ponds, with potential overtopping and breaching. The Edgecumbe ponds suffered wave damage, overtopping and minor breaching, and a piping failure through a weakened section.

Drainage and Stormwater

Christchurch has an extensive drainage system incorporating a network of pipelines, open drains, streams and rivers. There are eight major pumping stations. Damage to pipelines, manholes and pumping stations can be expected, similar to the damage discussed above for the sewerage system. Open drains and watercourses could be blocked by bank failures, and lateral spreading of adjacent ground in the eastern areas of the city.

Stopbanks appear to be particularly vulnerable to earthquake damage from bank failure, slumping, lateral spreading towards rivers, and earthquake induced wave and surge action. Older stopbanks in particular are frequently poorly compacted and may slump and settle. In Christchurch, the areas that are stopbanked around parts of the lower Avon and Heathcote Rivers and the estuary are also areas that could experience liquefaction and lateral spreading. Stopbank damage, while not of immediate threat to life in an earthquake, makes the protected areas vulnerable to flooding in the ensuing period. Damage to stopbanks on the Waimakariri River, while outside the immediate area covered in this study would pose a significant flooding hazard to much of Christchurch. However the coarser gravelly soils along the river may reduce this hazard.

It is not possible to predict whether appreciable ground settlement would accompany severe earthquake shaking, but if it did occur, this would present additional problems of drainage and flood protection to a city which is already low-lying.

Transport Routes

Considerable damage to transportation routes can be expected in a severe earthquake, with cracking and settlement affecting surfaces. In the eastern areas of the city where there are considerable depths of soft, saturated soils, ground lurching is possible with the surface thrown into undulating waves which may or may not remain when the ground motion ceases. Similar surface effects can occur over liquefied soils. Figure 9.1 shows areas of soil types susceptible to liquefaction, and a similar area could be subject to ground lurching. Roads on such ground would be extensively fissured and ridged, kerb and channel broken, and footpaths and vehicle crossings destroyed.

There are few road embankments in Christchurch, but the embankments on the tunnel road, the McCormack Bay causeway, the length of Dyers Road across the oxidation ponds, and the bridge approaches on Bridge Street, South Brighton can be expected to be damaged by slumping and fissuring. The elevated section of the southern motorway is built over firmer, predominantly gravel soils and is less likely to be damaged. Roads on the hill areas would be subject to slips and rockfalls.

Christchurch has few major bridges, and the majority of these have been built to earthquake resistant designs. Settlement and slumping of bridge approaches can be expected to be widespread, and many bridges supported on shallow foundations could be damaged by bank failure towards the rivers. Foundation failure caused by liquefaction is a possibility for bridges in the eastern areas of the city.

Railways would be affected by ground lurching if it occurred. The embankment in the Heathcote Valley could well suffer slumping and settlement.

Christchurch Airport is located to the north-west of the city on gravel soils. It is unlikely that the runway would be severely damaged, but the relatively large amplitude, long period spectral response may affect the airport buildings. High inertial forces could also damage unsecured equipment. (An airport affected by the Loma Priata earthquake in San Francisco in 1989 was closed for some time by equipment damage although the runway was undamaged).

Energy Supply

Electricity supply can be expected to suffer substantial damage. Although substation equipment has been designed for earthquake resistance in recent years, damage at Edgecumbe in 1987 indicates that failures will still occur. The large amplitude of the spectral response likely in Christchurch means that inertial forces on heavy equipment such as transformers are likely to be larger than previously assumed for design. The possibility of liquefaction induced foundation failures also has serious consequences. The Electricorp substation at Bromley is on a deep sand profile, and the Addington substation is close to an identified liquefiable site. There is therefore some risk that the main supply nodes would be seriously damaged, causing major problems in reinstating electrical supply even if the reticulation was little damaged.

Power reticulation is likely to suffer some damage. The Edgecumbe earthquake resulted in about 10% of house leads being pulled out, and some pole failures. The long period, large amplitude displacement predicted for Christchurch would increase this potential damage.

The LPG pipeline from Lyttelton to Woolston was the subject of a special seismic and geotechnical study (Dibble & Ansell & Berrill, 1980; Luxford & Bell, 1986) and can be expected to survive a severe earthquake with little damage.

The oil tank farm in Lyttelton is built on very soft reclaimed land, with loose fill overlying soft marine sediments. Both the results of Chapter 7 of this study, and an acceleration response spectra developed for the submarine section of the LPG pipeline (Luxford & Bell, 1986) suggest that large amplification effects can be expected on the reclamation, and extreme damage is likely during a severe earthquake. In addition to the risk of fire and explosion this damage could have grave environmental consequences from large spillages of oil into Lyttelton Harbour. Elsewhere, buried petrol tanks at service stations located in areas of liquefaction may float towards the ground surface, and could rupture. Extensive damage to connections is likely.

11.5 Tsunamis

Fifteen tsunamis are reported to have been detected on the New Zealand coastline since 1848 (Brown et al, 1991), with twelve of these affecting the east coast. The deep bays around Banks Peninsula have experienced particularly severe effects, due to resonance caused by the bathometric configuration from the Chatham Rise to the east coast, and to focusing of waves propagating up the bays.

Any major earthquake in the Pacific Ocean region is capable of generating tsunamis which could cause damage at New Zealand. In 1868, earthquakes near the west coast of South America produced 6 m high waves in Lyttelton Harbour. Ships broke their moorings and were capsized. In 1960 an earthquake in Chile generated 5.5 m waves in the harbour, and damaging waves in the Heathcote/Avon Estuary. However for distant earthquakes, the International Tsunami Warning Centre in Hawaii is usually able to provide sufficient warning to at least allow precautions to be taken to protect those living near the coast.

Locally generated tsunamis probably represent a greater hazard to areas around Christchurch. In 1947 a small to moderate earthquake near Gisborne (M = 5.4) caused waves up to 10 m high along the nearby coast. It is unlikely that a significant tsunami following an earthquake close to Christchurch could be predicted or detected in time to allow any precautions to be taken.

Prediction of tsunamis following earthquakes is very difficult and unreliable. Past reports indicate that large earthquakes are not required to generate very large tsunami wave action; earthquake magnitudes up to M = 6 or greater are likely to occur in future in both onshore and offshore seismicity zones near Christchurch. The most susceptible areas around the city will be those where the coastal buffer zone is narrow and low-lying, or where wave focusing effects are likely to be caused by shoreline geometry.

11.6 Summary

Strong seismic shaking in Christchurch, from either a moderate earthquake with a short epicentral distance, or major earthquake at a more distant location, could be expected to produce widespread damage to the city. The effect of the deep alluvium below the city on the spectral response is likely to subject mid to high rise structures to large amplitude resonant shaking, but may reduce the damage to housing to mainly inertial effects. Services are likely to be severely affected, particularly if liquefaction is widespread, and liquefaction could extensively damage key components of the sewerage system and electrical supply. The type of damage discussed in this chapter is possible for an earthquake event with felt intensities of MM VIII to MM IX in Christchurch. This shaking intensity is estimated to have a return period of about 100 years, with a probability of occurrence within any 50 year period of about 40%.

SECTION IV Conclusions of Earthquake Hazard Study for Christchurch

12. Recommendations for Action and Requirement for Further Work

13. Conclusions

CHAPTER 12: Recommendations for Action and Requirements for Future Work

12.1 Recommendations

This study has highlighted a number of areas that reflect on current engineering design, and which require further action from the general engineering profession. We consider the following recommendations to be the most important:

(1) Review the existing Loadings Code with respect to Christchurch. Christchurch is currently in Zone B but the results from this study suggest Zone A would be more appropriate.

(2) Consider in detail the effect of deep soil response on building performance.

(3) Review local seismic design practices, and particularly consider the need for the inclusion of site specific response spectra in structural design.

(4) Review Christchurch building stock in light of the conclusions of the study, in particular the likely seismic performance of existing multi-storey buildings.

(5) Zone the at-risk hill properties to target an owner education programme encouraging intensive tree planting and possible installation of catchfences and deflection walls.

(6) Undertake a full Lifelines study for Christchurch considering in particular the impact on essential services i.e.

- Water supply and reticulation
- Sewerage system
- Electrical supply
- Telecommunications

(7) Carry out site specific studies of the likely seismic performance of critical and/or high risk facilities from an economic, public health, safety and environmental perspective. We suggest the following be included:

- Lyttelton tank farm
- Other port facilities
- Bromley sewerage treatment plant
- Critical pumping stations
- Electrical substations
- Hospitals
- Civil Defence facilities (including firestations)
- Airport
- Bridges on S.H. 1 across Waimakariri and Rakaia Rivers

(8) Undertake a systematic comparison of the housing stock of Christchurch to that of Edgecumbe to validate likely damage ratios and better define potential economic losses.

(9) Consider the likely impact of a substantial earthquake on the regional economy and review existing commercial insurance levels.

12.2 Future Work

The following areas of further research should be undertaken:

(a) A detailed study of Liquefaction Potential including extensive field testing to define variations in sand density. Given the confirmed existence of the other pre-requisites for liquefaction (i.e. saturated sand in the vulnerable size range), soil density is the critical factor in assessing the location and extent of potential problems.

(b) Further geological evaluation of the active faults adjacent to Christchurch including the Porters Tectonic Zone, Pegasus Bay Fault and the Banks Peninsula and Canterbury Plains Seismicity Zones.

(c) A major paleoseismic investigation should be undertaken of the Alpine Fault including a review of the existing conclusions of Adams (1980). The proximity of this fault to many regional South Island centres makes such a study extremely important not just for Christchurch but for the South Island as a whole.

(d) Research should be carried out to establish the validity or otherwise of the attenuation model for intensity used in this study and others for New Zealand.

(e) Research should be continued to derive a reliable attenuation model for seismic accelerations for New Zealand conditions, taking into account the discussions of previous approaches presented in this report.

(f) A second deep borehole should be drilled, this time in the Central City area preferably to the north of the City Centre where the severest amplifications are predicted. This will refine the assumed subsurface sequence used in this study which has been based largely on the Bexley drillhole for depths from 120 m - 500 m.

(g) In conjunction with this drillhole a gravity geophysics survey should be undertaken to define the topography on the bedrock contact beneath the alluvium.

(h) Carry out cyclic triaxial testing of loess and other Christchurch sediments (e.g. reclamation silts, estuarine silts etc.) to confirm assumptions regarding susceptibility to liquefaction.

CHAPTER 13: Conclusions

13.1 Hazard Model for Christchurch

Christchurch is located close to several active faults with potential to generate earthquakes which could cause damage to the city. Analysis of the possible earthquake magnitudes and epicentral distance suggests the most critical faults are the Pegasus Bay Fault, fault strands in the Porters Pass Tectonic Zone (which includes the Porters Pass and Ashley Faults) and the central section of the Alpine Fault. Significant damage could also result from very close earthquakes of moderate magnitudes associated with the relatively shallow active seismicity below the Pleistocene and recent sediments close to Christchurch. The highest intensity earthquake which Christchurch has experienced historically, the 1869 New Brighton earthquake which reached MM Intensity VII - VIII, appears to have been of this type. Other damaging historical earthquakes have reached Intensity VII in the city and include the 1888 Amuri earthquake (associated with rupture on the Hope Fault) and the 1901 and 1922 Cheviot and Motunau earthquakes. No historical earthquakes have been attributed to the most critical capable faults i.e. the Porters Pass Tectonic Zone, Pegasus Bay Fault, or the central section of the Alpine Fault.

To assess the probability of future earthquake shaking in Christchurch, the available records of seismicity have been analysed using the traditional occurrence model ($\log N = a - bM$). The model is generally based on that developed by Smith & Berryman (1983). To improve accuracy seven new, small regions have been employed in northern and offshore Canterbury and the M_{max} values have been reduced in some areas based on the geologic evidence. Low b values found for these tightly-defined zones are consistent with those reported elsewhere for specific fault zones.

The intensity attenuation model of Smith (1978) has been adapted using a functional relationship which avoids the need for discretisation of the intensity - magnitude - distance correlation.

The attenuation model for seismic acceleration has been considered in detail. It has been demonstrated that the modifications made to the Katayama attenuation model for New Zealand conditions may not be justified, at least on the basis of published data. This is a major area requiring further work.

The return periods for various intensities of bedrock shaking in Christchurch have been calculated in this study as:

Intensity	MM VI	12 years
	MM VII	25 years
	MM VIII	100 years
	MM IX	1,200 years
	MM X	6,000 years

When compared with the predictions of the earlier study by Smith & Berryman (1983) the frequency of small and large events are decreased, but there is an increase in the predicted frequency of medium earthquakes (MM VII - MM VIII) (refer to Figure 5.8). The effect of local amplification through the deep alluvium beneath Christchurch will increase felt intensities by 0 to +2 MM units (Figure 7.5a). This increases the frequency of shaking at the ground surface in Christchurch to the following average return periods:

Intensity	MM VI	7 years
	MM VII	20 years
	MM VIII	55 years
	MM IX	300 years
	MM X	in excess of 6,000 years

The effect on the structural response spectra at Christchurch of propagation of incoming seismic waves through the underlying deep alluvium is dramatic. The effect varies laterally, principally reflecting variations in the top 30 metres of sediment.

Over 10,000 borelogs have been synthesised to obtain information on the extent of this lateral variation. Areas of St Albans, Papanui and Redwood, Sydenham, Addington and Halswell have near surface soil profiles that produce particularly strong ground shaking.

The modifications to the response spectra by underlying alluvium are greater than those which would be predicted by the current New Zealand Loadings Code. The effects are generally threefold:

- A total removal of very short period (less than 0.2 sec) acceleration from the response spectrum by hysteretic damping through the deep, relatively soft soil.
- A general amplification of the peak spectral accelerations by up to twice the peak bedrock acceleration.
- A shift in all the spectral values towards the longer period region of the spectrum.

When these effects are compared with the response spectra from the current code for a fully elastic structure on flexible soils in Zone B, the average amplification in Christchurch produces a typical response spectra nearly 30% in excess of the code spectra (for periods between 1 and 2.5 seconds). For the maximum amplification in Christchurch, the code spectra could be exceeded by 50% at 0.7 sec period, and 160% at 1.2 sec period (for 150-year return period accelerations - refer Figure 7.21).

13.2 Potential Consequences and Hazards

The prediction of likely impacts resulting from a moderate to large earthquake affecting the City is hampered by a lack of site specific information. However a number of general conclusions can be reached:

- There is a substantial portion of Christchurch City that is underlain by layers of sand which would be susceptible to liquefaction if the sand is loose. The main sand area extends generally east of Marshland Road, Fitzgerald Avenue and Opawa Road and includes Brighton, Heathcote and Sumner. Other smaller areas exist scattered through the city. In a number of sites where specific test data is available, loose sand has been identified and analysis shows that liquefaction is likely to occur during a large earthquake. How far these areas extend, and the extent of earthquake shaking required to initiate liquefaction, are important questions requiring extensive further research.

- Comparison with overseas studies suggests slope failure could be triggered by large earthquakes on the central segment of the Alpine Fault, the Kaiwara Fault and the Porters Pass Tectonic Zone. Moderate earthquakes within the Pegasus Bay, Banks Peninsula and Christchurch seismicity zones could also generate slope instability.

- Pseudostatic slope stability analysis suggests that in addition to the degree of shaking, pre-existing soil moisture levels will be critical in controlling the extent of damage in hill suburbs. Widespread foundation damage could result if a large earthquake (say intensity VIII - IX, with return period 300 years) coincided with virtual soil saturation, but fortunately these saturated conditions exist for a relatively small proportion of the year. In general the most at risk period is from July to October, and the most vulnerable housing areas are those of the fringes of existing development located below steep hill slopes. Houses in these locations may also be damaged by debris inundation and boulder roll.

- Rock slides, topple failures and falls are likely during a substantial earthquake in the eastern suburbs where high cliffs have been produced by coastal erosion and local quarrying. Damage to houses from these processes may be limited in extent but roading may be more widely affected.

- Liquefaction induced landsliding in loessial soils during earthquakes has been reported overseas but initial consideration suggests this process is unlikely to affect Christchurch hill soils. Lateral spreading in alluvial materials subject to liquefaction is more likely to occur. Potential for this process appears to exist in some areas, in particular, along the lower reaches of the Avon and Heathcote Rivers and around the margins of the Estuary.

13.3 Potential Damage

Severe seismic shaking comparable to the historical recorded maximum (Intensity VII - VIII) will produce widespread damage to the city during a future event. The effect of the deep alluvium below the city on the spectral response is likely to subject mid to high rise structures to severe resonant shaking, but may reduce the damage to housing to mainly inertial effects. Services are likely to be seriously affected from a combination of settlement, ground lurching and liquefaction. If liquefaction is widespread, extensive damage is likely to key components of the sewerage system and electrical supply located in the eastern city. Major damage with potentially serious environmental consequences may occur at the oil tank farm in Lyttelton, which is built on loose fill overlying soft marine sediments.

Substantial further work is required to assess the type and extent of potential damage. A Lifeline study similar to that recently completed in Wellington will be an important part of this work.

Acknowledgements

This study was funded by the EQC, without whose support this work would not have been possible. We have been very much assisted by early results and discussion relating to a concurrent EQC funded project entitled "Structural Evolution and Seismotectonic Hazard Assessment of the Active Porters Pass Tectonic Zone" undertaken by Mr Hugh Cowan, Dr Jarg Pettinga and Mrs Jocelyn Campbell of the Geology Department, University of Canterbury. Their assistance in this and many other geological aspects of the work is gratefully acknowledged.

In addition, Dr J.B. Berrill and Dr R.O. Davis of the Civil Engineering Department, University of Canterbury, assisted us in reviewing and discussing the seismicity, liquefaction and slope stability aspects of the study. The computer modelling of wave propagation was done under the supervision of Dr R.O. Davis using programmes developed at the University.

DSIR assisted with provision of Seismological Observatory information and Dr Euan Smith was helpful in commenting on various aspects of this material.

Borelogs for Christchurch were provided by a number of organisations and consultants whose help is gratefully acknowledged:

Christchurch Drainage Board
Canterbury Regional Council
P.J. Alley
G.L. Evans
Waimairi County Council
Christchurch City Council
Works Corp
Heathcote County Council
Paparua County Council

The major job of borelog compilation and map work was done by M.D. Adamson of Soils & Foundations. Figure preparation was by M.A. Smith and C.D. McCormick. Typing the many drafts and revisions was patiently carried out by Julie Howard. David Brathwaite and Jim Park undertook final reading and comment.

Special thanks go to our various wives and offspring whose lives at times became seriously disrupted by our overwork, in particular, Therese, Jamie, Dorle, William, Lisa and Ben.

References

Adams, J. (1980) "Paleoseismicity of the Alpine Fault Seismic Gap, New Zealand", Geology, 8, pp 72-76.

Adams, J. (1990) "Speculations on the Mode of Failure of the Alpine Fault, New Zealand: A consequence, and not the cause, of great earthquakes." Seismological Research Letters, V. 61, 22, 1990.

Alley, P.J. (1966) "Cashmere Hills Loess". N.Z. Engineering, Vol. 11, No. 10, p 424.

Ambraseys, N.N. and Menu, J.M. (1988) "Earthquake-Induced Ground Displacements", Earthquake Engineering and Structural Dynamics, Vol. 16, pp 985-1006.

Atkins, R.A.E. and Hicks, S.R. (1977) Analysis of the 1969 Ashburton River seismic survey. Geophysics Division Report No. 127, Geophysics Division, Department of Scientific and Industrial Research.

Atkins, R.A.E. and Hicks, S.R. (1979) Geophysical models along Ashburton River, Canterbury, New Zealand. New Zealand Journal of Geology and Geophysics 22, pp 673-677.

Bastings, L. and Hayes, R.C. (1935) "Earthquake Distribution in New Zealand 1848-1934", N.Z. J. Sci. Tech. XVI, pp. 308-12.

Beanland, S. (comp). (1987) "Field Guide to Sites of Active Earth Deformation, South Island, New Zealand", N.Z. Geological Survey record 19.

Bell, D.H. and Trangmar, B.B. (1987) "Regolith Materials and Erosion Processes on the Port Hills, Christchurch, New Zealand", Proc. 5th Int. Field Wkshp on Landslides, Christchurch, August 1987.

Benson, W.N. (1968) "Dunedin District 1:50000", N.Z. Geological Survey Miscellaneous Series Map 1.

Berrill, J.B. (1985a) "Seismic Hazard Analysis and Design Loads", Bulletin NZNSEE, Vol. 18, No. 2, pp. 139-150.

Berrill, J.B. (1985b) "Distribution of Scatter in New Zealand Accelerograph Data", Bulletin NZNSEE, Vol. 18, No. 2, pp. 151-164.

Berrill, J.B., Davis, R.O., Mullenger, G. and Fairless, G.J. (1988) "Seismic Liquefaction Research by the University of Canterbury". Transactions, IPENZ, Vol. 15, No. 2/CE, July 1988.

Berryman, K.R. (1980) "Late Quaternary movement on White Creek Fault, South Island, New Zealand", N.Z. Journal of Geology and Geophysics 23, pp 93-101.

Berryman, K. and Beanland, S. (1988) "Ongoing Deformation of New Zealand: Rates of Tectonic Movement from Geological Evidence", Transactions, Vol. 15, No. 1/GEN.

Berryman, K.R., Cutten, H.N.C., Fellows, D.L., Hull, A.G., Sewell, R.J. (1986) "Alpine Fault Reconnaissance Survey, from Arawata River to Pyke River, South Westland, New Zealand", NZGS Rep. EDI 86/14.

Birrell, K.S. and Packard, R.Q. (1953) "Some Physical Properties of New Zealand Loess", N.Z.J. Sci. Techn., Vol. B35, pp 30-35.

Bishop, A.W., Alpan, I., Blight, E.E., Donald, I.B. (1960) "Factors Controlling the Strength of Partly Saturated Cohesive Soils", Proc. ASCE Research Conf. on Shear Strength of Cohesive Soils, Boulder, Colorado, USA.

Bonilla, M.G., Mark, R.K. and Lienkaemper, J.J. (1984) "Statistical Relations Among Earthquake Magnitude, Surface Rupture Length, and Surface Fault Displacement", Bulletin of the Seismological Society of America, Vol. 74, No. 6, pp 2379-2411.

Bonilla, M.G. and Buchanan, J.M. (1970) "Interim Report on Worldwide Historic Surface Faulting", U.S. Geol. Surv. Open-File Rept, 32 pp.

Brown, L.J., Beetham, R.D., Elder, D.McG, Weeber, J.H. (In Prep.) "Christchurch Geology and its relevance to Earthquake Hazard".

Building Research Association of N.Z. (1987) "Lessons from the 1987 Edgecumbe Earthquake", Building Information Bulletin 258.

Carter, R.M. and Carter, L. (1982) "The Motunau Fault and other structures at the southern edge of the Australian-Pacific plate boundary, offshore Marlborough, New Zealand", Tectonophysics, 88, pp 133-159.

Casagrande, A. (1950) "Notes on the design of earth dams", J. Boston Soc. Civil Eng. 37, pp 405-429.

Clark, R.H., Dibble, R.R., Fyfe, H.E., Lensen, G.J. and Suggate, R.P. (1965) "Tectonic and Earthquake Risk Zoning". Transactions of the Royal Society of N.Z. 1: 113-126.

Close, V. and McCormick, E. (1922) "Where the Mountains Walked", Natl. Geog. Mag. 16, pp 445-467.

Cooper, A.F. and Norris, R.J. (1989) "Sag-pond stratigraphy and broken beech trees in Northern Fiordland - Evidence for seismic events on the Alpine Fault", Paper presented at 1989 NZGG Conf., Auckland, Dec. 4-7, Geol. Soc. of N.Z. Misc. Publ. 43.

Cowan, H.A. (1990) "Late Quaternary Displacements on the Hope Fault at Glynn Wye, North Canterbury", N.Z. Journal of Geology and Geophysics, Vol. 33, pp 285-293.

Cowan, H.A. (1989) "An evaluation of the late Quaternary displacements and seismic hazard associated with the Hope and Kakapo Faults, Amuri District, North Canterbury", Unpublished M.Sc. Engineering Geology thesis, lodged in the Library, University of Canterbury, 239 p.

Cowan, H.A. and McGlone, M. (In Prep.) "Large Prehistoric Earthquakes and Rate of Slip on the Hope Fault at Glynn Wye, North Canterbury".

Cowan, H.A. and Pettinga, J.R. (1990) "Seismic Hazards in the Canterbury Civil Defence Region", Report prepared for Canterbury United Council.

Cowan, H.A. and Pettinga, J.R. (1989) "Transtension and Structural Complexity Along the Hope Fault, Glynn Wye to Hanmer Basin, North Canterbury", Paper presented at 1989 NZGG Conf., Auckland, Dec. 4-7, Geol. Soc. of N.Z. Misc. Publ. 43.

Coyle, S. (1988) "Porters Pass Fault", Unpublished M.Sc. (Geol.) Thesis, University of Canterbury.

Crampton, N.Z. (1985) "Engineering Geological Aspects of the Lyttelton - Woolston LPG Pipeline", M.Sc. Thesis (Eng. Geol.), University of Canterbury, 241 pp.

Davis, R.O. and Berrill, J.B. (1988) "Design Earthquake Response Spectra for the Telecom Site, Hereford Street, Christchurch", Unpublished Report to Soils & Foundations Ltd.

Davis, R.O. and Berrill, J.B. (1988) "Design Earthquake Response Spectra for the Telecom Site, Hereford Street, Christchurch", Unpublished Report to Soils & Foundations (1973) Ltd.

Davis, R.O. and Berrill, J.B. (1983) "Comparison of a Liquefaction Theory with Field Observations", Geotechnique 33, No. 4.

Dibble, R.R.; Ansell, J.H.; and Berrill, J.B. (1980) "Report on a study of seismic risk for B.P. New Zealand Ltd sites at Woolston and Lyttelton", Unpublished Report to B.P. New Zealand Ltd.

Dickenson, G.E. and Adams, R.D. (1967) "A Statistical Survey of Earthquakes in the Main Seismic Region of New Zealand. Part 3 - Geographical Distribution", N.Z. J. Geol. Geophys. 10, pp. 1040-50.

Dowrick, D.J. and Rhoades, D.A. (1990) "Damage Ratios for Domestic Buildings in the 1987 Edgecumbe Earthquake", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 23, No. 2.

Eiby, G.A. (1990) "The Lake Coleridge Earthquake of 1946", Bulletin NZNSEE, Vol. 23, No. 2, pp. 150-158.

Eiby, G.A. (1980) "The Marlborough Earthquakes of 1848", New Zealand DSIR bulletin 225, 82 p.

Eiby, G.A. (1971) "The frequency of earthquake occurrence in New Zealand", Journal of the Royal Society of N.Z. 1(1), pp. 79-82.

Eiby, G.A. (1966) "The Modified Mercalli Scale of earthquake intensity and its use in New Zealand", N.Z. Journal of Geology and Geophysics 9, pp. 122-129.

Esteva, L. (1988) "The Mexico Earthquake of September 19, 1983 - Consequences, Lessons and Impact on Research and Practice", Earthquake Spectra Vol. 4, No. 3.

Evans, G.L. (1977) "Erosion Tests on Loess Silt, Banks Peninsula", Proc. 9th ISSMFE Conf. Tokyo, July 1977, Vol. 2, pp 63-69.

Evernden, J.F., Hibbard, R.R., and Schneider, J.F. (1973) "Interpretation of seismic intensity data", Bulletin of the Seismological Society of America, v. 63, pp 399-422.

Evernden, J.F. and Thomson, J.M. (1985) "Predicting Seismic Intensities". In Evaluating Earthquake Hazards in the Los Angeles Region - An Earth Science Perspective, U.S.G.S. Professional Paper 1360, 505 p.

Evison, F.F. (1971) "Seismicity of the Alpine Fault, New Zealand", International Symposium on Recent Crustal Movements and Associated Seismicity, Royal Society of N.Z., Bulletin 9, pp. 161-165.

Fairless, G.J. & Berrill, J.B. (1984) "Liquefaction during Historic Earthquakes in New Zealand", Bulletin of N.Z. National Society for Earthquake Engineering, Vol. 17, No. 4, December.

Fox, A.N. (1987) Neotectonic History of Lake Tekapo, McKenzie Basin, New Zealand. Unpublished M.Sc. (Geol.) Thesis, University of Canterbury, Christchurch.

Fredlund, D.G. (1981) "The shear strength of unsaturated soil and its relationship to slope stability problems in Hong Kong", Hong Kong Engineer, April, pp 37-45.

Gammon, J.R.A. and Carrucan, M.J. (1987) "Evaluation of Liquefaction Potential for the Tauranga Harbour Crossing", Proceedings Pacific Conference on Earthquake Engineering, New Zealand, August 1987.

Goldwater, S. (1990) "Slope Failures in Loess: A Detailed Investigation, Allandale, Banks Peninsula", M.Sc. Thesis (Geol.), Univ. Canterbury.

Goldwater, S., Elder, D.McG., Yetton, M.D., Bell, D.H. (1990) "Effect of Groundwater on the Strength of Loess", Proceedings of the Groundwater and Seepage Symposium IPENZ, Proc. of Tech. Gps, Vol. 16, Issue 1(G), pp 119-129.

Gregg, D.R. (1964) (1st Ed.) "Geological Map of New Zealand 1: 250 000", Sheet 18 Hurunui.

Gutenberg, B. and Richter, C.F. (1954) "Seismicity of the Earth and Associated Phenomena", Princeton University Press, 310 p.

Harvey, M.D. (1976) "An analysis of the soil slips that occurred on the Port Hills, Canterbury, between 19-25 August 1975", Soil Science Society of N.Z., Palmerston North, Aug. 1976.

Haskell, N.A. (1960) "Curstal Reflection of Plan SH Waves", Jour. Geophys. Research Vol. 65, p 4147.

Hayes, R.C. (1936a) "Intensity distribution in New Zealand earthquakes", N.Z. Journal of Science and Technology XVIII, pp. 508-511.

Hayes, R.C. (1936b) "The Seismicity of New Zealand Cities and Towns", N.Z. J. Sci. Techn. XVIII, pp. 608-12.

Hayes, R.C. (1941) "The Seismicity of New Zealand", N.Z. J. Sci. Techn. XXIII, pp. 49B-52B.

Hayes, R.C. (1943) "On Earthquake Distribution in New Zealand", N.Z. J. Sci. Tech. XXIV, pp. 236B-8B.

Herzer, R.H. and Bradshaw, J.D. (1985) "The Motunau Fault and Other Structures at the Southern Edge of the Australian-Pacific Plate Boundary, Offshore Marlborough, New Zealand - Discussion", Tectonophysics, 115, pp 161-166.

Hoek, E. and Bray, J.W. (1981) "Rock Slope Engineering", Inst. of Min. and Metal., London.

Hogben, G. (1891) "The Earthquakes of New Zealand", Tr. Aust. Assn Adv. Sci. 3, pp 37-57.

Hughes, D.K. (1987) "Shear Modulus Gs", Bulletin NZNSEE, Vol. 20, No. 1, pp 63-65.

Hughes, P.J. (1970) "Tunnel Erosion in Loess of Banks Peninsula", M.Sc. Thesis (Geog.), Univ. Canterbury.

Hull, A.G. and Berryman, K.R. (1986) "Holocene tectonism in the region of the Alpine Fault at Lake McKerrow, Fiordland, New Zealand", Royal Society of New Zealand, Bulletin 28.

Hull, A.G. (1990) "Tectonics of the 1931 Hawkes Bay Earthquake", N.Z. Journal of Geology and Geophysics, Vol. 33, pp 309-320.

Hunt, R.E. (1986) "Geotechnical Engineering Analysis and Evaluation", McGraw-Hill, Inc. New York, 729 p.

Hutchinson, D.L.; Andrews, A.I.; Butcher, G.W.; Kalston, D. (1986) "Draft Revision of NZS 4203: 1984: Seismic Provisions", Bulletin NZNSEE, Vol. 19, No. 3, pp 158-166.

Iwasaki, T. (1986) "Soil Liquefaction Studies in Japan: State of the Art", Soil Dynamics and Earthquake Engineering, Vol. 5, No. 1.

Jennings, D.N., Edwards, M.R., and Franks, C.A.M. (1988) "Some Observations of Sand Liquefaction in the 2 March 1987 Edgecombe, (New Zealand) Earthquake", Proceedings, Fifth Australia-New Zealand Conference on Geomechanics, 1988.

Johnston, M.R. (1979) "Geology of the Nelson Urban Area" (1: 25 000), New Zealand Geological Survey Urban Series Map 1. Map (1 sheet) and notes (52 p), New Zealand Department of Scientific and Industrial Research, Wellington.

Joyner, W.B. and Fumal, T.E. (1985) "Predictive Mapping of Earthquake Ground Motion". In Evaluating Earthquake Hazards in the Los Angeles Region - An Earth Science Perspective. U.S.G.S. Professional Paper 1360, 505 p.

Katayama, T. (1982) "An Engineering Prediction Model of Acceleration Response Spectra and its Application to Seismic Hazard Mapping", Earthquake Engineering and Structural Design, 10, pp. 149-163.

Keefer, D.K. (1984) "Landslides caused by earthquakes", Geological Society of America Bulletin, v. 95, pp 406-421.

Kieckhefer, R.M. (1979), Sheets M31D, N31A, N31C and pts M32A and M32B, Leader Dale (1st Edition); Sheets N31B and N31D, Dillon (1st Edition); Sheets 030C and 031A, Lake McRae (1st Edition) "Late Quaternary Tectonic Map of New Zealand 1:50 000", DSIR.

Kirkaldy, P.H.S. and Thomas, E.G. (1963) Final report on a seismic survey in the Canterbury Plains area of New Zealand. BP Shell & Todd Petroleum Development Ltd Geological/Geophysical Report 27 (unpublished).

Kuribayashi, E. and Tatsuoka, F. (1975) "Brief Review of Liquefaction During Earthquakes in Japan", Soils and Foundations, 15, pp. 81-92.

Lahr, J.C. and Stephens, C.D. (1982) "Alaska Seismic Zone: Possible Example of Non-Linear Magnitude Distribution for Faults [abs.]", Earthquake Notes, v. 53, p 66.

Lensen, G.J. (1978a) "Historic Tectonic Earth Deformation", in Suggate, R.P.; Stevens, G.R.; Te Punga, M.T. (Eds) "The Geology of New Zealand", Government Printer, Wellington, 2 vols, 820 p.

Lensen, G.J. (1978b) Letter to Marlborough Electric Power Board, 28 April 1978.

Lensen, G.J. (1977) "Late Quaternary Tectonic Map of New Zealand 1:2,000,000", (1st ed.), N.Z. Geol. Surv. Misc. Series Map 12, DSIR, Wellington.

Lensen, G.J. (1976) "Sheets N28D, 028C and N29B - Hillersden, and sheets 028D, P28A and P28C - Renwick, late Quaternary tectonic map of New Zealand 1:50 000", Wellington, Department of Scientific and Industrial Research.

Lensen, G.J. (1968) Analysis of progressive fault displacement during downcutting at the Branch River terraces, South Island, New Zealand, Geological Society of America bulletin 79, pp 545-566.

Leslie, P.D., Hunt, J. and Morrison, J.L. (1987) "The Edgecumbe (New Zealand) Earthquake 1987", Local Authority Engineering in New Zealand, Vol. 5, No. 3.

Lew, M., Chieruzzi, R., Campbell, K.W., Duke, M. (1981) "Correlations of Seismic Velocity with Depth", Int. Conf. on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Vol. 1, pp 13-18.

Lin, Z.G. and Wang, S.J. (1988) "Collapsibility and deformation characteristics of deep-seated loess in China", Eng. Geol. Vol. 25, pp 271-282.

Lloyd, K. "Earthquake Effect Upon Services and Roading", Local Authority Engineering in New Zealand, Vol. 5, No. 3.

Lutenegger, A.J. (1981) "Stability of loess in light of the inactive particle theory", Nature 291: 360.

Luxford, N. and Bell, D.H. (1986) "Some Geotechnical Aspects of the Lyttelton - Woolston LPG pipeline, Christchurch", Institution of Professional Engineers N.Z., Transactions Vol. 13, No. 3/CE, Nov. 1986.

Lysenko, M.P. (1971) "Loessial Rocks of the European USSR" (Translated from Russian), Keter Press, Jerusalem, 150 pp.

Mackwell, J. (1986) "Engineering Geological Investigations Wainui-French Farm Area, Akaroa County, Banks Peninsula", M.Sc. Thesis (Eng. Geol.), Univ. Canterbury.

Madin, I. (1988) "Geology and neotectonics of the Upper Manuherikia Basin, Central Otago, New Zealand", N.Z. Geological Survey record 27.

Makgill, K.P. and Norris, R.J. (1983) "Recent movements on the Akatore Fault, East Otago, and its relationship to regional tectonics" (Abstract), Pacific Science Association 15th Congress, Dunedin.

Mark, R.K. and Bonilla, M.G. (1977) "Regression analysis of earthquake magnitude and surface fault length using the 1970 data of Bonilla and Buchanan", U.S. Geol. Surv., Open-File Rept, 77-614, 8 pp.

Martin, G. (1988) "Geotechnical Aspects of Earthquake Engineering - State of the Art Report No. 2", Delivered to 5th Australia - New Zealand Conference, Sydney, and published in Australian Geomechanics.

Matuschka, T., Berryman, K.R., O'Leary, A.J., McVerry, G.H., Mulholland, W.M. and Skinner, R.I. (1985) "New Zealand Seismic Analysis", Bulletin N.Z. Soc. Earthq. Eng., Vol. 18, No. 4, pp. 313-322.

Matuschka, T. (1980) "Assessment of seismic hazards in New Zealand", University of Auckland, Dept of Civil Engineering, Report No. 222.

McDowell, B.J. (1989) "Site Investigations for Residential Development on the Port Hills, Christchurch", M.Sc. Thesis (Eng. Geol.), University of Canterbury.

McKay, A. (1890) "On the earthquakes of September 1888, in the Amuri and Marlborough districts of the South Island", N.Z. Geological Survey report of geological explorations 20, pp 1-16.

McPherson, R.K. (1988) "Experimental Geomorphic Map for Hazard Assessment, Cheviot, Northern Canterbury, New Zealand", Water and Soil Publication No. 118, 66 p.

McVerry, G.H. (1986) "Uncertainties in Attenuation Relations for New Zealand Seismic Hazard Analysis", Bulletin NZNSEE, Vol. 19, No. 1, pp 28-39.

Miller, D.E.K. (1971) "Soil Properties Affecting Tunnel Gully Erosion", M.Agr. Thesis, Lincoln College.

Mitchell, P.H. (1981) "The Distribution of Observed Earthquake Attenuation About the Model Proposed by Katayama et al", Unpublished Report, University of Canterbury.

Mostyn, G.R., Small, J.C. "Methods of Stability Analysis" in Soil Slope Instability and Stabilisation, Walker and Fell (Eds), Balkema, Rotterdam, 440 p.

Mulholland, W.M. (1982) "Estimation of design earthquake motions for New Zealand", University of Canterbury, Department of Civil Engineering, Report 82/9.

Newmark, N.M. (1965) "Effects of earthquakes on dams and embankments", Geotechnique 15(2), pp 149-160.

Peek, R. (1980) "Estimation of seismic risk for New Zealand", University of Canterbury, Department of Civil Engineering, Report 80/21.

Peek, R., Berrill, J.B. and Davis, R.D., (1980) "A seismicity model for New Zealand", Bulletin of the N.Z. National Society for Earthquake Engineering 13, pp. 355-364.

Read, S.A.L. (1984) "Late Quaternary deformation in the Ostler Fault Zone, Mackenzie Basin, South Island, New Zealand" (Abstract), International Symposium on Recent Crustal Movements of the Pacific Region, Wellington, New Zealand.

Reyners, M. (1989) "New Zealand Seismicity 1964-87: an Interpretation", N.Z. Journal of Geology and Geophysics, Vol. 32, pp 307-315.

Reyners, M. (1987) "Subcrustal earthquake in the central South Island, New Zealand, and the root of the Southern Alps", Geology V. 15, pp 1168-1171.

Robertson, P.K., Campanella, R.G. and Wightman, A. (1983) "SPT/CPT Correlations", Journal of the Geotechnical Engineering Division, ASCE, Vol. 109, No. 11.

Robertson, P.K. and Campanella, R.G. (1985) "Liquefaction Potential of Sands Using the CPT", Journal Geotechnical Engineering Division, ASCE, Vol. 111, No. 3.

Ruscoe (1989) "Walking on Jelly, The 1987 Edgecumbe Earthquake", DSIR.

Salt, G. (1983) "The Use of Residual Strengths of Soils in Geotechnical Design", IPENZ Conf., 1983, Hamilton, N.Z.

Sanders, R. (1986) "Hydrogeological Studies of Springs in Akaroa County, Banks Peninsula", M.Sc. Thesis (Eng. Geol.), University of Canterbury.

Sarma, S.K. (1979) "Stability analysis of embankments and slopes", A.S.C.E., Journal of the Geotechnical Engineering Division, V105, GT12, Dec., pp. 1511-1524.

Schwartz, D.P., Coppersmith, K.J., Swan, F.H., III, Somerville, P. and Savage, W.U. (1981) "Characteristic earthquakes on intraplate normal faults" [abs.], Earthquake Notes, v. 52, p. 71.

Schwartz, D.P. and Coppersmith, K.J. (1984) Fault Behaviour and Characteristic Earthquakes: Examples from the Wasatch and San Andreas Faults. J. Geophys. Res. 89, 5681-5698.

Schwartz, D.P. and Coppersmith, K.J. (1986) in "Active Tectonics: Studies in Geophysics". National Academy Press, pp 224, 225.

Seed, H.B. and Idriss, I.M. (1970) "Soil Moduli and Damping Factors for Dynamic Response Analyses", Report No. EERC 70-10, Earthquake Engineering Research Centre.

Seed, H.B. (1968) "Landslides during earthquakes due to soil liquefaction", A.S.C.E., Journal of the Soil Mechanics and Foundations Division, V94, SM5, pp 1055-1122.

Seed, H. Bolton, Idriss, I.M., and Arango Ignacio (1983) "Evaluation of Liquefaction Potential Using Field Performance Data", Journal Geotechnical Engineering Division, ASCE, Vol. 109, No. 3.

Sewell, R.J., Weaver, S.D., and Thiele, B.W. (1988), "Sheet M36 8D - Lyttelton. Geological map of New Zealand 1:50 000". Map (1 sheet) and notes, Wellington, New Zealand, Department of Scientific and Industrial Research.

Singh, S.K., Astiz, L. and Havskov, J. (1981) "Seismic gaps and recurrence periods of large earthquakes along the Mexican subduction zone: A re-examination", Bulletin of the Seismological Society of America, v. 71, pp 827-843.

Slemmons, D.B. (1977) "Faults and Earthquake Magnitude", U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, Misc. Paper S-73-1, Report 6, 129 pp.

Slemmons, D.B. (1982) "Determination of design earthquake magnitude for microzonation", Proceedings of 3rd World Microzonation Conference, Seattle U.S.A. pp 119-130.

Smith, W.D. (1976) "Statistical estimates of the likelihood of earthquake shaking throughout New Zealand", Bulletin of the N.Z. National Society for Earthquake Engineering 9, pp. 213-221.

Smith, W.D. (1978a) "Spatial distribution of felt intensities for New Zealand earthquakes", N.Z. Journal of Geology and Geophysics 21, pp. 293-311.

Smith, W.D. (1978b) "Earthquake risk in New Zealand: Statistical Estimates", N.Z. Journal of Geology and Geophysics 21, pp. 313-327.

Smith, W.D. (1982) "Pitfalls in the Estimation of Seismic Hazard", Bulletin NZNSEE, Vol. 15, No. 2, pp 77-81.

Smith, W.D. and Lensen, G. (1981) "Seismicity Model for New Zealand", Unpublished working paper, Seismic Risk Subcommittee, Standards Association of New Zealand.

Smith, W.D. and Berryman, K. (1983) "Revised Estimates of Earthquake Hazard in New Zealand", Bulletin NZNSEE, Vol. 16, No. 4, pp. 259-272.

Soils & Foundations (1988) "New Telephone Exchange, Christchurch - Geotechnical Report", Unpublished Report to Telecom N.Z. Ltd.

Talbot, J.D., Weeber, J.H., Freeman, M.C., Mason, C.R., and Wilson, D.D. "The Christchurch Artesian Aquifers", Report by Resources Division of the North Canterbury Catchment Board and Regional Water Board, 159 p.

Terzaghi, K. (1950) "Mechanisms of landslides", in Application to Geology to Engineering Practice, Berkley Volume, The Geological Society of America, pp 83-123.

Tsuchida, H. (1971) "Estimation of liquefaction potential of sandy soils", Proceedings of the Third Joint Meeting, US-Japan Panel on Wind and Seismic Effects, UJNR, May.

Van Dissen, R.J. (1989) "Late Quaternary Deformation in the Kaikoura Region, SE Marlborough, New Zealand, Unpublished M.Sc. (Geol.) Thesis, University of Oregon State.

Walker, B.F. and Mohen, F.J. (1987) "Groundwater Prediction and Control, and Negative Pore Water Pressure Effects", in Soil Slope Stability and Stabilisation, Walker and Fell (Eds), Balkema, Rotterdam, 440 p.

Walley, P. (1976) "The estimation of earthquake risk in New Zealand", N.Z. Department of Scientific and Industrial Research, Applied Mathematics Division, Technical Report No. 49, 81 p.

Wellman, H.W. (1985) "Weathering rind ages for the central part of New Zealand" (Abstract) Geological Society of New Zealand Christchurch conference.

Wesnousky, S.G., Scholz, C.H., Shimazaki, K. and Matsuda, T. (1983) Earthquake frequency distribution and the mechanics of faulting, Journal of Geophysical Research, V. 88, pp 9331-9340.

Whitehouse, I.E. and Bradshaw, J.D. (1988) "Reconnaissance bedrock geology of upper Rakaia River, Canterbury", New Zealand Geological Survey Report G135.

Wilson, R.G. and Keefer, D.K. "Predicting Areal Limits of Earthquake-Induced Landsliding" in "Evaluating Earthquake Hazard in the Los Angeles Region - An Earth Science Perspective", Ziony, J.I. (Ed), U.S.G.S. Prof. Paper 1360, 505 p.

Wise, D.U., Pettinga, J.R., Campbell, J.K. and Nicol, A. (In Prep.) "A Surface Slab Model for the Transfer Zone of New Zealand's Alpine Fault".

Yetton, M.D. (1986) "Investigation and Remedial Methods for Subsurface Erosion Control in Banks Peninsula Loess". Unpublished M.Sc. (Geol.) Thesis, University of Canterbury, 231 pp.

Youd, T.L. (1973) Liquefaction, flow, and associated ground failure. U.S. Geological Survey Circular 688.

Youd, T.L. and Perkins, D.M. (1987) "Mapping of Liquefaction Severity Index", Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 11, November 1987.

Youd, T.L. and Perkins, D.M. (1978) "Mapping Liquefaction-Induced Ground Failure Potential", Journal of Geotechnical Engineering, ASCE, Vol. 104, No. 4.

Youngs, R.R. and Coppersmith, K.J. (1985) "Fault Slip Rates and Earthquake Recurrence Relationships". Bulletin of the Seismological Society of America, Vol. 75, No. 4, pp 939-964.

Yousif, H.S. (1987) "The applications of remote sensing to geomorphological neotectonic mapping in North Canterbury, New Zealand", Unpublished PhD thesis, University of Canterbury.

Ziony, J.I. (1985) "Evaluating Earthquake Hazards in the Los Angeles Region - An Earth Science Perspective", U.S. Geological Survey Professional Paper 1360.

APPENDIX A: Modified Mercalli Scale of Intensity of Earthquake Shaking (N.Z. Version, 1965)

APPENDIX A

MODIFIED MERCALLI SCALE OF INTENSITY OF EARTHQUAKE SHAKING

(N.Z. VERSION, 1965)

MM I	Not felt by humans, except in especially favourable circumstances, but birds and animals may be disturbed. Reported mainly from the upper floors of buildings more than 10 storeys high. Dizziness or nausea may be experienced. Branches of trees, chandeliers, doors, and other suspended systems of long natural period may be seen to move slowly. Water in ponds, lakes, reservoirs, etc., may be set into seiche oscillation.
MM II	Felt by a few persons at rest indoors, especially by those on upper floors or otherwise favourably placed. The long-period effects listed under MMI may be more noticeable.
MM III	Felt indoors, but not identified as an earthquake by everyone. Vibration may be likened to the passing of light traffic. It may be possible to estimate the duration, but not the direction. Hanging objects may swing slightly. Standing motorcars may rock slightly.
MM IV	Generally noticed indoors, but not outside. Very light sleepers may be wakened. Vibration may be likened to the passing of heavy traffic, or to the jolt of a heavy object falling or striking the building. Walls and frame of buildings are heard to creak. Doors and windows rattle. Glassware and crockery rattles. Liquids in open vessels may be slightly disturbed. Standing motorcars may rock, and the shock can be felt by their occupants.
MM V	Generally felt outside, and by almost everyone indoors. Most sleepers awakened. A few people frightened. Direction of motion can be estimated. Small unstable objects are displaced or upset. Some glassware and crockery may be broken. Some windows cracked. A few earthenware toilet fixtures cracked. Hanging pictures move. Doors and shutters may swing. Pendulum clocks stop, start, or change rate.

MM VI

Felt by all.
People and animals alarmed.
Many run outside.
Difficulty experienced in walking steadily.

Slight damage to Masonry D.
Some plaster cracks or falls.
Isolated cases of chimney damage.

Windows, glassware, and crockery broken.
Objects fall from shelves, and pictures from walls.
Heavy furniture moved. Unstable furniture overturned.

Small church and school bells ring.
Trees and bushes shake, or are heard to rustle.
Loose material may be dislodged from existing slips, talus slopes, or shingle slides.

MM VII

General alarm.
Difficulty experienced in standing.
Noticed by drivers of motorcars.
Trees and bushes strongly shaken.
Large bells ring.

Masonry D cracked and damaged.
A few instances of damage to Masonry C.

Loose brickwork and tiles dislodged.
Unbraced parapets and architectural ornaments may fall.
Stone walls cracked.
Weak chimneys broken, usually at the roof-line.
Domestic water tanks burst.
Concrete irrigation ditches damaged.

Waves seen on ponds and lakes.
Water made turbid by stirred-up mud.
Small slips, and caving-in of sand and gravel banks.

MM VIII

Alarm may approach panic.
Steering of motorcars affected.

Masonry C damaged, with partial collapse.
Masonry B damaged in some cases.
Masonry A undamaged.

Chimneys, factory stacks, monuments, towers, and elevated tanks twisted or brought down.
Panel walls thrown out of frame structures.
Some brick veneers damaged.
Decayed wooden piles broken.
Frame houses not secured to the foundation may move.

	Cracks appear on steep slopes and in wet ground. Landslips in roadside cuttings and unsupported excavations. Some tree branches may be broken off. Changes in the flow or temperature of springs and wells may occur. Small earthquake fountains.
MM IX	General panic. Masonry D destroyed. Masonry C heavily damaged, sometimes collapsing completely. Masonry B seriously damaged. Frame structures racked and distorted. Damage to foundations general. Frame houses not secured to the foundations shifted off. Brick veneers fall and expose frames. Cracking of the ground conspicuous. Minor damage to paths and roadways. Sand and mud ejected in alluviated areas, with the formation of earthquake fountains and sand craters. Underground pipes broken. Serious damage to reservoirs.
MM X	Most masonry structures destroyed, together with their foundations. Some well built wooden buildings and bridges seriously damaged. Dams, dykes, and embankments seriously damaged. Railway lines slightly bent. Cement and asphalt roads and pavements badly cracked or thrown into waves. Large landslides on river banks and steep coasts. Sand and mud on beaches and flat land moved horizontally. Large and spectacular sand and mud fountains. Water from rivers, lakes, and canals thrown up on the bank.
MM XI	Wooden frame structures destroyed. Great damage to railway lines. Great damage to underground pipes.
MM XII	Damage virtually total. Practically all works of construction destroyed or greatly damaged. Large rock masses displaced. Lines of sight and level distorted. Visible wave-motion of the ground surface reported. Objects thrown upwards into the air.

Categories of Non-wooden Construction

Masonry A.	Structures designed to resist lateral forces of about 0.1 g, such as those satisfying the New Zealand Model Building Bylaws, 1955. Typical buildings of this kind are well reinforced by means of steel or ferro-concrete bands, or are wholly of ferro-concrete construction. All mortar is of good quality and the design and workmanship is good. Few buildings erected prior to 1935 can be regarded as in category A.
Masonry B.	Reinforced buildings of good workmanship and with sound mortar, but not designed in detail to resist lateral forces.
Masonry C.	Buildings of ordinary workmanship, with mortar of average quality. No extreme weakness, such as inadequate bonding of the corners, but neither designed nor reinforced to resist lateral forces.
Masonry D.	Buildings with low standards of workmanship, poor mortar, or constructed of weak materials like mud brick and rammed earth. Weak horizontally.

Windows

Window breakage depends greatly upon the nature of the frame and its orientation with respect to the earthquake source. Windows cracked at MM V are usually either large display windows, or windows tightly fitted to metal frames.

Chimneys

The "weak chimneys" listed under MM VII are unreinforced domestic chimneys of brick, concrete block, or poured concrete.

Water tanks

The "domestic water tanks" listed under MM VII are of the cylindrical corrugated-iron type common in New Zealand rural areas. If these are only partly full, movement of the water may burst soldered and riveted seams.

Hot-water cylinders constrained only by supply and delivery pipes may move sufficiently to break the pipes at about the same intensity.

APPENDIX B: Log of the Bexley Borehole
Location: Pages Road, M35869440
Depth: 433 m

SOILS & FOUNDATIONS
Geotechnical Consulting Engineers
Soils & Foundations (1973) Ltd
Tasmania House
71 Armagh Street, PO Box 451
Christchurch, New Zealand
FAX (03) 67-780
Telephone (03) 798-432

FEATURE. NCCB DRILLHOLE

LOCATION. BEXLEY, CHRISTCHURCH

ATTITUDE/DIRECTION; VERTICAL

9128/1

R.L. GROUND(m). 1.0 a.m.s.l

MACHINE;

GEOLOGICAL UNIT DESCRIPTION	CORE LOSS/ LIFT	DEPTH (m) below GL	GRAPHIC LOG	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED.	DRILL METHOD	DATE/DEPTH m	SAMPLES AND TESTS	DATE	WATER LEVEL	CASING (mm)	PENETRATION (SPT) (uncorrected for overburden)	MOISTURE CONTENT (%)
		0			TOPSOIL and FILL	CABLE TOOL					508mm		
		5 10		ML	Grey SILT with traces of fine sand and shells			▨					
		15		SM/ML	Grey SANDY SILT with a trace of shells -sand , fine			▨					
				SM	Grey SILTY SAND with a trace of shells								
		20		SW	Dark grey fine to medium SAND								
				ML	Grey SILT with a trace of shells								
		25		SM	Grey SILTY fine to medium SAND with a trace of shells								

COMMENTS ▨ SHELL SAMPLE POSITION

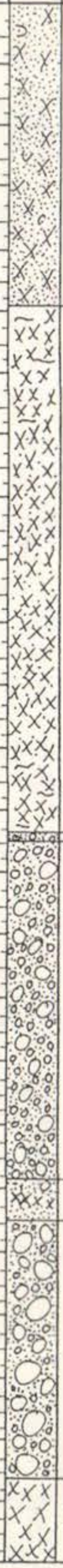
LOGGED MDA
DATE 8/8/89
LENGTH

DRILLER McILLAN
STARTED
FINISHED

SOILS & FOUNDATIONS
Geotechnical Consulting Engineers
Soils & Foundations (1973) Ltd
Tasmania House
71 Armagh Street, PO Box 451
Christchurch, New Zealand
FAX (03) 67-780
Telephone (03) 798-432

FEATURE. NCCB DRILLHOLE
LOCATION. BEXLEY, CHRISTCHURCH
ATTITUDE/DIRECTION; VERTICAL

9128/2
R.L. GROUND(m). 1.0 a.m.s.l
MACHINE;



GEOLOGICAL UNIT DESCRIPTION	CORE LOSS/ LIFT %	DEPTH (m) below GL	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED.	DRILL METHOD	DATE/DEPTH m	SAMPLES AND TESTS	DATE	WATER LEVEL	CASING (mm)	PENETRATION (SPT) (uncorrected for overburden)	MOISTURE CONTENT (%)
		25	SM	Grey SILTY fine to medium SAND with a trace of shells	CABLE TOOL					508mm		
		30 35	ML	Grey SILT -minor organics from 29.9m to 31.8m -worm holes from 29.9m to 31.8m -traces of very fine sand from 31.8m to 37.15m -some stratified organics from 37.15m to 38.3m			⊟ ⊟ ⊟					
			SP	Grey GRAVELLY fine to coarse SAND -gravel , fine to medium, subangular to subrounded greywacke -minor shells			▨ ⊟					
		40	GP	Grey SANDY fine to medium GRAVEL -sand , fine to coarse -gravel , subangular to subround greywacke			⊟		ARTESIAN HEAD OF 1.6M	406mm		
			SM	Grey fine to coarse SAND with some layers of grey SILT and traces of medium gravel			⊟					
		45	GP	Grey SANDY fine to coarse GRAVEL -sand , fine to coarse -gravel , subangular to subround greywacke								
		50	ML	Grey SILT								

COMMENTS ⊟ SAND PUMP SAMPLE POSITTION
▨ SHELL SAMPLE POSITION

LOGGED MDA
DATE 8/8/89
LENGTH

DRILLER McILLAN
STARTED
FINISHED

SOILS & FOUNDATIONS
Geotechnical Consulting Engineers
Soils & Foundations (1973) Ltd
Tasmania House
71 Armagh Street, PO Box 451
Christchurch, New Zealand
FAX (03) 67-780
Telephone (03) 798-432

9128/3

FEATURE. NCCB DRILLHOLE

LOCATION. BEXLEY,CHRISTCHURCH

ATTITUDE/DIRECTION; VERTICAL

R.L. GROUND(m). 1.0 a.m.s.l

MACHINE;

GEOLOGICAL UNIT DESCRIPTION	CORE LOSS/ LIFT %	DEPTH (m) below GL	GRAPHIC LOG	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED.	DRILL METHOD	DATE/DEPTH m	SAMPLES AND TESTS	DATE	WATER LEVEL	CASING (mm)	PENETRATION (SPT) (uncorrected for overburden)	MOISTURE CONTENT (%)
		50		ML	Grey SILT			⊟					
		55		SW	Grey SAND with minor shells and traces of fine gravel			⊟					
		60		SW	Grey fine SAND with traces of silt	Cable Tool					406mm		
				ML	Grey SILT with minor fine sand								
		65		ML	Grey fine SILT -soft			⊟					
				ML	Greyish yellow SILT with minor fine sand			⊟					
		70		GP	Grey SANDY fine to medium GRAVEL with minor yellow silt -sand, fine to coarse -gravel, subrounded to subangular greywacke			⊟					
				SM/ML	Greyish yellow SANDY SILT -sand, fine to coarse			⊟					
				ML	Grey SILT with some organics and traces of medium gravel								
					Brown GRAVELS and SAND?								
		75		ML/PT	Grey SILT with layers of well compressed black and brown peat								

COMMENTS ⊟ SAND PUMP SAMPLE POSITION

LOGGED MDA
DATE 11/8/89
LENGTH

DRILLER McILLAN
STARTED 4/1/89
FINISHED

SOILS & FOUNDATIONS
Geotechnical Consulting Engineers
Soils & Foundations (1973) Ltd
Tasmania House
71 Armagh Street, PO Box 451
Christchurch, New Zealand
FAX (03) 67-780
Telephone (03) 798-432

FEATURE. NCCB DRILLHOLE ,

LOCATION. BEXLEY, CHRISTCHURCH

ATTITUDE/DIRECTION; VERTICAL

9128/4

R.L. GROUND(m). 1.0 a.m.s.l

MACHINE;

GEOLOGICAL UNIT DESCRIPTION	CORE LOSS/ LIFT %	DEPTH (m) below GL	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED.	DRILL METHOD	DATE/DEPTH m	SAMPLES AND TESTS	DATE	WATER LEVEL	CASING (mm)	PENETRATION (SPT) (uncorrected for overburden)	MOISTURE CONTENT (%)
		75 80	ML/PT	Grey SILT with layers of well compressed black and brown peat	Cable Tool		⊟			406mm		
		85	GW	Grey SANDY fine to coarse GRAVEL with minor silt -stained brownish -sand, fine to coarse -gravel, subrounded to subangular			⊟					
			ML	Greyish brown soft SILT with lenses of greyish brown firm silt and minor fine sand			⊟					
		90	ML	Brownish grey soft SILT with lenses of firm silt -minor organics -traces of fine sand			⊟					
			ML	Greyish yellow SILT with minor fine sand and traces of organics			⊟					
		95	ML	Grey,brown stained SANDY fine to coarse GRAVEL with minor silt -sand, fine to coarse -gravel, subangular to subrounded greywacke			⊟					
		100	SP	Brown fine SAND with traces of fine gravel and small lumps of clay			⊟					

COMMENTS ⊟ SAND PUMP SAMPLE POSITION

LOGGED MDA
DATE 11/8/89
LENGTH

DRILLER McILLAN
STARTED 4/1/89
FINISHED

SOILS & FOUNDATIONS
Geotechnical Consulting Engineers
Soils & Foundations (1973) Ltd
Tasmania House
71 Armagh Street, PO Box 451
Christchurch, New Zealand
FAX (03) 67-780
Telephone (03) 798-432

9128/5

FEATURE. NCCB DRILLHOLE

LOCATION. BEXLEY, CHRISTCHURCH

ATTITUDE/DIRECTION; VERTICAL

R.L. GROUND(m). 1.0 a.m.s.l

MACHINE;

GEOLOGICAL UNIT DESCRIPTION	CORE LOSS/ LIFT %	DEPTH (m) below GL	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED.	DRILL METHOD	DATE/DEPTH m	SAMPLES AND TESTS	DATE	WATER LEVEL	CASING (mm)	PENETRATION (SPT) (uncorrected for overburden)	MOISTURE CONTENT (%)
		100 105	SP	Brown fine SAND with traces of fine gravel and small lumps of clay -gravel, angular greywacke	Cable Tool					406mm		
			ML CL	Light brown SILTY CLAY to CLAYEY SILT with a trace of very fine sand								
		110	SP	Brown fine SAND with traces of fine gravel and small lumps of clay -gravel, angular greywacke								
		115	GW	Brownish grey SANDY fine to medium GRAVEL with some silt -sand, fine to coarse -gravel, subrounded to subangular greywacke -stained brown			⊟					
			GW	Brownish grey fine to medium GRAVEL with some fine to coarse sand and a trace of silt -gravel, subrounded to subangular greywacke			⊟					
		120	SW	Brownish grey fine to medium dense SAND with minor silt -stained brown			⊟					
		125	ML	Grey SILT with a trace of fine sand and fine organics								

COMMENTS ⊟ SAND PUMP SAMPLE POSITION

LOGGED MDA
DATE 11/8/89
LENGTH

DRILLER McILLAN
STARTED 4/1/89
FINISHED

SOILS & FOUNDATIONS
Geotechnical Consulting Engineers
Soils & Foundations (1973) Ltd
Tasmania House
71 Armagh Street, PO Box 451
Christchurch, New Zealand
FAX (03) 67-780
Telephone (03) 798-432

9128/6

FEATURE. NCCB DRILLHOLE

LOCATION. BEXLEY, CHRISTCHURCH

ATTITUDE/DIRECTION; VERTICAL

R.L. GROUND(m). 1.0 a.m.s.l

MACHINE;

GEOLOGICAL UNIT DESCRIPTION	CORE LOSS/ LIFT %	DEPTH (m) below GL	GRAPHIC LOG	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED.	DRILL METHOD	DATE/DEPTH m	SAMPLES AND TESTS	DATE	WATER LEVEL	CASING (mm)	PENETRATION (SPT) (uncorrected for overburden)	MOISTURE CONTENT (%)
		125 130 135		ML	Grey SILT with a trace of fine sand and fine organics						406mm		
				ML	Yellowish grey SILT	Cable Tool							
		140		GC	Grey stained yellow fine to medium GRAVELS with some clay and minor fine to medium sand -gravel, subangular to subrounded greywacke			⊟					
		145 150								ARTESIAN HEAD OF 8.6M	305mm		

COMMENTS ⊟ SAND PUMP SAMPLE POSITION

LOGGED MDA
DATE 11/8/89
LENGTH

DRILLER McILLAN
STARTED 4/1/89
FINISHED

SOILS & FOUNDATIONS
Geotechnical Consulting Engineers
Soils & Foundations (1973) Ltd
Tasmania House
71 Armagh Street, PO Box 451
Christchurch, New Zealand
FAX (03) 67-780
Telephone (03) 798-432

FEATURE. NCCB DRILLHOLE
LOCATION. BEXLEY, CHRISTCHURCH
ATTITUDE/DIRECTION; VERTICAL

9128/7

R.L. GROUND(m).1.0 a.m.s.1
MACHINE;

GEOLOGICAL UNIT DESCRIPTION	CORE LOSS/ LIFT %	DEPTH (m) below GL	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED.	DRILL METHOD	DATE/DEPTH m	SAMPLES AND TESTS	DATE	WATER LEVEL	CASING (mm)	PENETRATION (SPT) (uncorrected for overburden)	MOISTURE CONTENT (%)
		150	ML	Light brownSILT with minor fine to medium sand	Cable Tool					305mm		
		155	SP	Light brownish grey fine to medium SAND with traces of silt			⊟					
			GW	Grey fine to coarse GRAVEL with minor coarse sand and traces of light brown silt -gravel, subrounded to subangular greywacke			⊟					
		160	GW	Light brownish grey SANDY fine to coarse GRAVEL with minor silt -sand, fine to medium -gravel,subrounded to subangular greywacke			⊟					
			GP	Grey SANDY fine GRAVEL with minor medium gravel and traces of light brown clayey silt -sand, fine to coarse								
			ML	Light brownish grey SILT with some fine to medium sand and gravel			⊟					
		165	SP	Light brownish grey fine to medium SAND with traces of small lumps of silt			⊟					
		170	SP	Yellowish light brown SILTY fine SAND -very stiff								
		175		Grey SANDY SILT -fine sand			⊟					

COMMENTS ⊟ SAND PUMP SAMPLE POSITION

LOGGED MDA
DATE 10/10/89
LENGTH

DRILLER McILLAN
STARTED 4/1/89
FINISHED

SOILS & FOUNDATIONS
Geotechnical Consulting Engineers
Soils & Foundations (1973) Ltd
Tasmania House
71 Armagh Street, PO Box 451
Christchurch, New Zealand
FAX (03) 67-780
Telephone (03) 798-432

FEATURE. NCCB DRILLHOLE
LOCATION. BEXLEY
ATTITUDE/DIRECTION; VERTICAL

9128/8

R.L. GROUND(m). 1.0 a.m.s.l
MACHINE;

GEOLOGICAL UNIT DESCRIPTION	CORE LOSS/ LIFT %	DEPTH (m) below GL	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED.	DRILL METHOD	SAMPLES AND TESTS	CASING (mm)	PENETRATION (SPT) (uncorrected for overburden)	MOISTURE CONTENT (%)
		175		Grey SANDY SILT -sand, fine	Cable Tool		305mm		
		180	ML						
		185	GW/GM	Grey SANDY fine to medium GRAVEL with minor lumps of silt bound fine to medium gravel with traces of fine sand -sand, fine to coarse -gravel, subrounded to subangular greywacke		⊟			
			ML	Blueish grey SILT with minor fine to medium gravel and traces of fine sand -gravel, subrounded to subangular greywacke -stiff		⊟			
		190	GP	Grey fine to medium GRAVEL with some fine to medium sand -gravel, subrounded to subangular greywacke		⊟			
		195	ML	Grey SILT with minor clay and traces of fine sand -stiff					
			GW	Grey SANDY fine to coarse GRAVEL with traces of silt -sand, fine to coarse -gravel, subrounded to subangular greywacke		⊟			
		200	SP/Pt	Grey fine to medium SAND with wood and traces of silt					

GRAPHIC LOG

COMMENTS ⊟ SAND PUMP SAMPLE POSITION

LOGGED MDA
DATE 12/10/89
LENGTH

DRILLER McILLAN
STARTED 4/1/89
FINISHED

SOILS & FOUNDATIONS
Geotechnical Consulting Engineers
Soils & Foundations (1973) Ltd
Tasmania House
71 Armagh Street, PO Box 451
Christchurch, New Zealand
FAX (03) 67-780
Telephone (03) 798-432

9128/9

FEATURE. NCCB DRILLHOLE
LOCATION. BEXLEY, CHRISTCHURCH
ATTITUDE/DIRECTION; VERTICAL
R.L. GROUND(m). 1.0 a.m.s.l
MACHINE;

GEOLOGICAL UNIT DESCRIPTION	CORE LOSS/ LIFT %	DEPTH (m) below GL	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED.	DRILL METHOD	DATE/DEPTH m	SAMPLES AND TESTS	DATE	WATER LEVEL	CASING (mm)	PENETRATION (SPT) (uncorrected for overburden)	MOISTURE CONTENT (%)
		200	SP/Pt	Grey fine to medium SAND with wood and traces of silt								
		205										
		210	ML SP	Grey SANDY SILT -firm -sand, fine	Cable Tool					305mm		
		215	GW	Grey SANDY fine to coarse GRAVEL with traces of silt -sand, fine to medium -gravel, subrounded to subangular greywacke			⊕					
		220										
			ML	Grey SILT								
		225										

COMMENTS ⊕ SAND PUMP SAMPLE POSITION

LOGGED MDA
DATE 11/10/89
LENGTH
DRILLER McILLAN
STARTED 4/1/89
FINISHED

SOILS & FOUNDATIONS
Geotechnical Consulting Engineers
Soils & Foundations (1973) Ltd
Tasmania House
71 Armagh Street, PO Box 451
Christchurch, New Zealand
FAX (03) 67-780
Telephone (03) 798-432

FEATURE. NCCB DRILLHOLE

LOCATION. BEXLEY, CHRISTCHURCH

ATTITUDE/DIRECTION; VERTICAL

9128/10

R.L. GROUND(m). 1.0 a.m.s.l

MACHINE;

GEOLOGICAL UNIT DESCRIPTION	*CORE LOSS/ LIFT %*	*DEPTH (m) below GL*	*GRAPHIC LOG*	*USC SYMBOL*	*DESCRIPTION OF MATERIAL RECOVERED.*	*DRILL METHOD*	*DATE/DEPTH m*	*SAMPLES AND TESTS*	*DATE*	*WATER LEVEL*	*CASING (mm)*	*PENETRATION (SPT) (uncorrected for overburden)*	*MOISTURE CONTENT (%)*
		225 230		ML	Grey SILT								
				ML	Blueish grey SILT -firm								
		235		GW	Grey SANDY fine to medium GRAVEL -sand, fine to coarse -gravel, subrounded to subangular greywacke -minor silt and some coarse gravel from 232.1m to 234.15m and 234.4m to 235.75m			⊟			305mm		
				ML/SP	Blueish grey medium to coarse SANDY SILT with some fine gravel -gravel, subrounded to subangular greywacke								
		240		GP	Grey SANDY fine to medium GRAVEL with minor silt -sand, fine to coarse -gravel, subrounded to subangular greywacke	Cable Tool							
		245		ML/SP	Grey SANDY SILT -sand, fine to coarse from 240.25 to 240.65m -sand, fine from240.65m to 248.4m -inter bedded with silt from 24.65 to 248.0 with a trace of shells			⊟			254mm		
		250		ML	Blueish grey SILT -firm								

COMMENTS ⊟ SAND PUMP SAMPLE POSITION

LOGGED MDA
DATE 11/10/89
LENGTH

DRILLER McILLAN
STARTED 4/1/89
FINISHED

SOILS & FOUNDATIONS
Geotechnical Consulting Engineers
Soils & Foundations (1973) Ltd
Tasmania House
71 Armagh Street, PO Box 451
Christchurch, New Zealand
FAX (03) 67-780
Telephone (03) 798-432

FEATURE. NCCB DRILLHOLE

LOCATION. BEXLEY, CHRASTCHURCH

ATTITUDE/DIRECTION: VERTICAL

9128/11

R.L. GROUND(m). 1.0 a.m.s.l

MACHINE;

GEOLOGICAL UNIT DESCRIPTION	CORE LOSS/ LIFT %	DEPTH (m) below GL	GRAPHIC LOG	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED.	DRILL METHOD	DATE/DEPTH m	SAMPLES AND TESTS	DATE	WATER LEVEL	CASING (mm)	PENETRATION (SPT) (uncorrected for overburden)	MOISTURE CONTENT (%)
		250 255 260 265		ML	Blueish grey SILT -firm -trace of organics from 253.1m to 266.0m	Cable Tool					254mm		
				ML/SP	Blueish grey SANDY SILT with some clay and minor shell fragments -sand, fine to coarse -soft								
		270 275		ML	Grey SILT -some fine sand from 266.4m to 275.6m -minor clay from 275.6m to 282.4m -minor fine sand from 283.4m to 282.9m								

COMMENTS

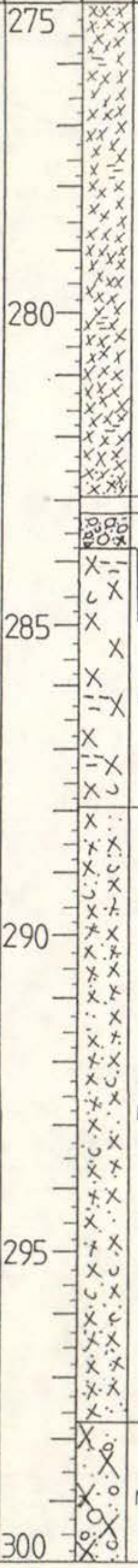
LOGGED MDA
DATE 11/10/89
LENGTH

DRILLER McILLAN
STARTED 1/4/89
FINISHED

SOILS & FOUNDATIONS
Geotechnical Consulting Engineers
oils & Foundations (1973) Ltd
asmania House
71 Armagh Street, PO Box 451
Christchurch, New Zealand
AX (03) 67-780
elephone (03) 798-432

FEATURE. NCCB DRILLHOLE
LOCATION. BEXLEY, CHRISTCHURCH
ATTITUDE/DIRECTION; VERTICAL

9128/12
R.L. GROUND(m). 1.0 a.m.s.l
MACHINE;



GEOLOGICAL UNIT DESCRIPTION	CORE LOSS/ LIFT %	DEPTH (m) below GL	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED.	DRILL METHOD	DATE/DEPTH m	SAMPLES AND TESTS	DATE	WATER LEVEL	CASING (mm)	PENETRATION (SPT) (uncorrected for overburden)	MOISTURE CONTENT (%)
		275	ML	Grey SILT -some fine sand from 266.4m to 275.6m -minor clay from 275.6m to 282.4m -minor fine sand from 283.4m to 282.9m	Cable Tool					254mm		
			GP	Grey GRAVEL with some sand and minor silt, trace of shells -gravel, fine to medium, sub-rounded to rounded -sand, fine								
		285	ML	Grey blue SILT with minor clay and trace of shells -soft								
		290	ML	Grey blue mottled reddish brown SILT with minor sand and trace of shells -sand, fine -firm								
		300	ML	Grey SILT with some sand and trace of gravel -sand, fine -gravel, fine to medium, sub-rounded to rounded -firm								

COMMENTS

LOGGED MDA
DATE 11/10/89
LENGTH

DRILLER McILLAN
STARTED 4/1/89
FINISHED

SOILS & FOUNDATIONS
Geotechnical Consulting Engineers
Soils & Foundations (1973) Ltd
Tasmania House
71 Armagh Street, PO Box 451
Christchurch, New Zealand
FAX (03) 67-780
Telephone (03) 798-432

FEATURE. NCCB DRILLHOLE

LOCATION. BEXLEY, CHRISTCHURCH

ATTITUDE/DIRECTION; VERTICAL

9128/13

R.L. GROUND(m). 1.0 a.m.s.l

MACHINE;

GEOLOGICAL UNIT DESCRIPTION	CORE LOSS/ LIFT %	DEPTH (m) below GL	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED.	DRILL METHOD	DATE/DEPTH m	SAMPLES AND TESTS	DATE	WATER LEVEL	CASING (mm)	PENETRATION (SPT) (uncorrected for overburden)	MOISTURE CONTENT (%)
		300										
			ML	Grey SILT with minor sand -sand, fine								
			ML	Grey SILT with minor shells -very soft								
		305										
			ML	Grey blue SILT with some sand and trace of shells -sand fine -very soft	Cable Tool					254mm		
		310										
		315										
			ML	Grey blue SILT with minor sand and trace of shells -sand fine -silt soft								
		320	ML	Grey blue SILT with minor clay and trace of shells -firm								
			ML	Grey blue mottled orange SILT with minor clay and trace of shells -soft								
		325										

COMMENTS

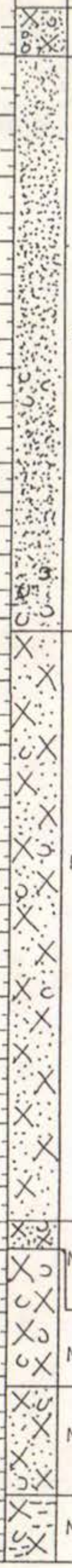
LOGGED PJR
DATE 6/4/90
LENGTH

DRILLER McILLAN
STARTED 4/1/89
FINISHED

SOILS & FOUNDATIONS
Geotechnical Consulting Engineers
Soils & Foundations (1973) Ltd
Tasmania House
71 Armagh Street, PO Box 451
Christchurch, New Zealand
FAX (03) 67-780
Telephone (03) 798-432

FEATURE. NCCB DRILLHOLE
LOCATION. BEXLEY, CHRISTCHURCH
ATTITUDE/DIRECTION; VERTICAL

9128/14
R.L. GROUND(m). 1.0 a.m.s.l
MACHINE;



GEOLOGICAL UNIT DESCRIPTION	CORE LOSS/ LIFT %	DEPTH (m) below GL	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED.	DRILL METHOD	DATE/DEPTH m	SAMPLES AND TESTS	DATE	WATER LEVEL	CASING (mm)	PENETRATION (SPT) (uncorrected for overburden)	MOISTURE CONTENT (%)
		325	SM/ML	GREY SANDY SILT with minor gravel and trace of shells -sand medium -gravel, medium to coarse, sub-rounded to rounded -firm	Cable Tool					254mm		
		330	SP	Grey SAND with trace of shells -sand, fine to medium -soft								
		335 340	ML	Grey SILT with some sand and trace of shells -sand, fine -firm -sand fine to medium from 336.4m -mottled orange from 337m								
		345	ML	Grey mottled orange SILT with some sand and minor shells -sand, fine -soft to firm								
			ML	Grey mottled orange SILT with minor shells -soft to firm								
			ML	Blue grey SILT with some sand and trace of shells -sand, fine, black -soft to firm								
		350	ML	Grey blue SILT with minor clay and trace of shells								

COMMENTS

LOGGED PJR
DATE 6/4/90
LENGTH
DRILLER McILLAN
STARTED 4/1/89
FINISHED

9128/15

FEATURE. NCCB DRILLHOLE

LOCATION. BEXLEY, CHRISTCHURCH

ATTITUDE/DIRECTION; VERTICAL

R.L. GROUND(m). 1.0 a.m.s.l

MACHINE;

GEOLOGICAL UNIT DESCRIPTION	CORE LOSS/ LIFT %	DEPTH (m) below GL	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED.	DRILL METHOD	DATE/DEPTH m	SAMPLES AND TESTS	DATE	WATER LEVEL	CASING (mm)	PENETRATION (SPT) (uncorrected for overburden)	MOISTURE CONTENT (%)
		350	ML		Cable Tool					254mm		
		355	ML	Grey black SILT with some sand and trace of shells -sand, fine -firm -grey mottled orange and no shells from 351.5m								
		360	ML	Grey SILT with some sand, trace of clay and marine organics								
		365 370	ML	Grey SILT with trace of sand -sand, fine -25-30% shells								
			ML	Grey SILT with minor sand and clay and trace of shells -sand fine to medium -soft								
		375										

COMMENTS

LOGGED PJR
DATE 6/4/90
LENGTH

DRILLER McILLAN
STARTED 4/1/89
FINISHED

SOILS & FOUNDATIONS
Geotechnical Consulting Engineers
Soils & Foundations (1973) Ltd
Tasmania House
71 Armagh Street, PO Box 451
Christchurch, New Zealand
FAX (03) 67-780
Telephone (03) 798-432

FEATURE. NCCB DRILLHOLE

LOCATION. BEXLEY, CHRISTCHURCH

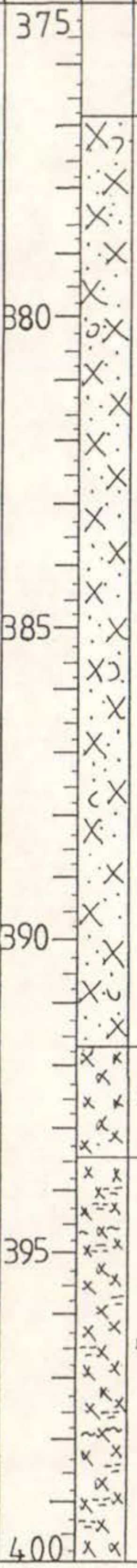
ATTITUDE/DIRECTION; VERTICAL

9128/16

R.L. GROUND(m). 1.0 a.m.s.l

MACHINE;

GEOLOGICAL UNIT DESCRIPTION	CORE LOSS/ LIFT	DEPTH (m) below GL	GRAPHIC LOG	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED.	DRILL METHOD	DATE/DEPTH m	SAMPLES AND TESTS	DATE	WATER LEVEL	CASING (mm)	PENETRATION (SPT) (uncorrected for overburden)	MOISTURE CONTENT (%)
		375				Cable Tool					203mm		
				ML	Grey blue SILT with minor sand and trace of shells -sand, fine -soft -organics from 391.7m -soft to firm from 391.7m								
				ML	Grey black SILT								
				ML	Light green, iron stained SILT with minor clay and trace of organics						No Casing		



COMMENTS
NOTES:
-No samples taken from 391.7 to 433m
-Logged using drillers log

LOGGED PJR
DATE 6/4/90
LENGTH

DRILLER McILLAN
STARTED 4/1/89
FINISHED

SOILS & FOUNDATIONS
Geotechnical Consulting Engineers
Soils & Foundations (1973) Ltd
Tasmania House
71 Armagh Street, PO Box 451
Christchurch, New Zealand
FAX (03) 67-780
Telephone (03) 798-432

FEATURE. NCCB DRILLHOLE

LOCATION. BEXLEY, CHRISTCHURCH

ATTITUDE/DIRECTION; VERTICAL

9128/17

R.L. GROUND(m) 1.0 a.m.s.l

MACHINE;

GEOLOGICAL UNIT DESCRIPTION	CORE LOSS/ LIFT %	DEPTH (m) below GL	GRAPHIC LOG	USC SYMBOL	DESCRIPTION OF MATERIAL RECOVERED.	DRILL METHOD	DATE/DEPTH m	SAMPLES AND TESTS	DATE	WATER LEVEL	CASING (mm)	PENETRATION (SPT) (uncorrected for overburden)	MOISTURE CONTENT (%)
		400											
					No log	Rotary Drill							
		405											
		410											
		415											
		420											
				SP	Grey SAND -Sand, fine								
		425											

COMMENTS

LOGGED PJR
DATE 15/5/90
LENGTH

DRILLER McILLAN
STARTED 4/1/90
FINISHED

SOILS & FOUNDATIONS
Geotechnical Consulting Engineers
Soils & Foundations (1973) Ltd
Tasmania House
71 Armagh Street, PO Box 451
Christchurch, New Zealand
FAX (03) 67-780
Telephone (03) 798-432

FEATURE. NCCB DRILLHOLE
LOCATION. BEXLEY, CHRISTCHURCH
ATTITUDE/DIRECTION; VERTICAL

9128/18
R.L. GROUND(m). 1.0 a.m.s.l
MACHINE;

GEOLOGICAL UNIT DESCRIPTION	*CORE LOSS/ LIFT %*	*DEPTH (m) below GL*	*GRAPHIC LOG*	*USC SYMBOL*	*DESCRIPTION OF MATERIAL RECOVERED.*	*DRILL METHOD*	*DATE/DEPTH m*	*SAMPLES AND TESTS*	*DATE*	*WATER LEVEL*	*CASING (mm)*	*PENETRATION (SPT) (uncorrected for overburden)*	*MOISTURE CONTENT (%)*
		425	∴										
			∴		Grey SAND with trace of organics -Sand, fine								
		430	∴	SP									
		435				Rotary Drill							

COMMENTS

LOGGED PJR
DATE 15/5/90
LENGTH

DRILLER McILLAN
STARTED 4/1/89
FINISHED