# THE EARTHQUAKE HAZARD

# IN

# DUNEDIN

**Soils and Foundations**

EQC

# THE EARTHQUAKE HAZARD IN DUNEDIN

Prepared By

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# EXECUTIVE SUMMARY

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Dunedin is located in one of the least seismically active areas in New Zealand. However, while the frequency of severe earthquake shaking is considerably lower in Dunedin than for Christchurch or Wellington, strong shaking will occur in the city. Even the relatively small earthquake of 1974 caused about $2 million (1993 values) of damage, and larger, though infrequent earthquakes could cause many times this amount in direct damage and major disruption to the local economy.

This study attempts to quantify the seismic hazard of Dunedin by adopting current seismic hazard analysis techniques and applying them specifically to Dunedin. 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 (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 southern South Island by subdividing the area into a number of seismicity zones. In each zone the maximum credible earthquake has been estimated from previous geological studies or, in the absence of such work, 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 1960 - 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 any of the faults near Dunedin and although this is the maximum record available, this time span is still relatively short.

The earthquake hazard in Dunedin is dominated by relatively infrequent moderate to large earthquakes (magnitude up to 7.5) in eastern Otago, and large to very large earthquakes in the much more seismically active but distant Fiordland region.

The calculated probabilities for various intensities of shaking in Dunedin are comparable with those reported previously by others, and are as follows:

| Modified Mercalli Intensity | Approximate Expected Effect | Average Return Period |
|---|---|---|
| Intensity VI | Minimal property damage | 30 years |
| Intensity VII | Some property damage<br>Loss of life unlikely | 100 years |
| Intensity VIII | Significant property damage<br>Loss of life possible | 450 years |
| Intensity IX | Extensive property damage<br>Some loss of life | In excess of 2,500 years |
| Intensity X | Catastrophic property damage<br>Major loss of life | Very small probability |

These intensity return periods are for "average" ground conditions and some parts of the city, such as the reclaimed land and the alluvial area of South Dunedin, are likely to experience shaking of up to one intensity unit higher for the city than the average.

This amplification was observed during the 1974 Dunedin earthquake, the deep relatively soft alluvial soils changes 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 and within the city distinct local variation results in particular from gradational changes in the top 30 m of sediment. These effects have been considered in the Site Response Model.

Variation in shaking intensity can also be expected (and was observed in the 1974 Dunedin earthquake) from topographic focusing and shielding effects. These effects have not been considered in this study, as current knowledge does not allow analysis.

As well as amplifying earthquake shaking, the alluvial soils of South Dunedin are also potentially susceptible to liquefaction. There is insufficient borelog and soil testing data available to define this hazard accurately. Analysis of typical sites indicates that while liquefaction is unlikely to be widespread, some local areas may be vulnerable to liquefaction damage during major earthquakes.

Much of Dunedin is sited on relatively steep topography, with areas of known instability. Large earthquakes could trigger landslide movement, on both existing landslides, where the movement would probably be limited, and generate new landslides. New landslides would occur only if the slope was already at marginal stability. Areas vulnerable to new earthquake triggered movement are difficult to determine, and the best indication is the distribution of known mass movement features. A significant area of South Dunedin is potentially vulnerable, but probably only during earthquakes of a return period in excess of 150 -200 years.

Chapter 8 of the report briefly considers the potential damage to the city in terms of the likely impact of a major earthquake on physical structures. Structures on the alluvial areas of the city should perform adequately provided they have been designed to recent code spectra for flexible soil sites. Damage to engineering lifeline services may be significant. The length of the supply pipelines makes the water supply vulnerable, but this is offset by the number of supply sources and interconnections between them. Reticulation is likely to be damaged in the softer alluvial soil areas. Sewers are similarly vulnerable in the soft soil areas, and a major part of Dunedin sewerage traverses this area.

An in-depth lifelines study for Dunedin has not been attempted. It is recommended that an engineering lifelines study be initiated, and a study of the economic and sociological impact of a major earthquake. Site specific studies should be carried out for key public and service facilities, and a review of planning control on areas of potential mass movement. Further research into the active faults close to Dunedin would be of great benefit in better defining the seismic hazard in Dunedin. Continued compilation of subsurface information with respect to soil amplification effects and liquefaction susceptibility is also necessary.

# CHAPTER 1: INTRODUCTION

## 1.1 BACKGROUND TO THE STUDY

Historical seismicity has shown Dunedin to be in one of the least active areas of New Zealand. The city has experienced shaking intensities greater than MMV on the Modified Mercalli Scale on only seven occasions since 1810. However, the 1974 Dunedin earthquake produced shaking intensities of MMVI over most of the city, and up to MMVII in the worst affected areas of South Dunedin. This event, which caused about $2 million (1993 value) of damage was a reminder that damaging earthquakes can occur in Dunedin, although their frequency may be much lower than in other parts of the country. The earthquake also clearly showed amplification effects from different ground conditions in the city.

A study completed in 1991 on the earthquake hazard in Christchurch (Elder et al, 1991) showed the seismic hazard in that city to be greater than previously thought. This was due to the presence of deep alluvial soils underlying the city, and the recent recognition of a generally higher level of active regional faulting and folding than was previously realised. The Dunedin experience of the 1974 earthquake with the earthquake epicentre probably on the Akatore fault 7 km offshore, indicated that the hazard in Dunedin could have been similarly under-estimated. While the hazard in Dunedin is lower than much of the country, the city is the major population centre in the southern South Island and therefore warrants a detailed evaluation. This evaluation takes into account all the currently available information.

In some areas, little of the 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 within 100 km of Dunedin. 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, particulary at the Geology Department of the University of Otago. In other areas the forms of mathematical models used in hazards prediction are somewhat subjective. Current and future research in these areas may allow their refinement or modification. In all cases, however, this study has either used models previously employed by others for New Zealand studies, or where separate analyses have been developed, these have been justified by available data. The attempted approach has been neither to be unnecessarily conservative (ie alarmist), nor to downgrade predicted effects where there is no reasonable justification for this. Inclusion of probabilistic assessments allow natural variability in model techniques or data to be quantified.

The authors believe the predictions presented in this report represent the best estimates currently available of the seismic hazard in Dunedin. However it is expected that results of research already in progress and the future work recommended here, may refine the predictions in some areas.

## 1.2 REPORT OUTLINE

The report is divided into ten chapters.

Chapter 1 introduces the study, describing previous work of relevance to Dunedin and briefly summarising the analysis philosophy and methods adopted.

In Chapter 2 available information on regional earthquakes, active faults and historical seismicity is described. This is combined to provide a seismicity model which can be used in conjunction with attenuation models to evaluate intensities (Chapter 3) and structural acceleration response spectra (Chapter 4) to determine the effects at Dunedin on a probabilistic basis.

Specific modifications to earthquake effects caused by the variable geologic profile beneath Dunedin are considered in Chapter 5. Compiled soil information from all available borelogs is presented, and ground surface and structural response variations across the city are described. Validation of amplification factors is achieved by comparing predicted amplification factors with those observed in Dunedin during previous earthquakes.

The practical and engineering consequences of the predicted earthquake effects for Dunedin, such as liquefaction and ground displacement and hillslope instability are considered in Chapters 6 and 7. Potential damage or disruption to services and structures is discussed briefly in Chapter 8, although detailed considerations are beyond the scope of this study.

Conclusions drawn from the study are described in Chapters 9 and 10, including recommendations for engineering design in Dunedin (Chapter 9), and for other measures which can readily be adopted in future to mitigate the consequences of a severe earthquake (Chapter 10). Recommendations for further work required to refine or validate the results of this study are outlined.

## 1.3 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.

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. 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 historic data.

The first comprehensive study employing both a seismicity model based on historical seismicity records and an attenuation model to predict felt intensities was reported by Smith (1976, 1978a, 1978b). He estimated return periods for Modified Mercalli intensities in Dunedin of : MMVI, 150 years; MMVII, 500 years. Smith estimated the intensity with a 5% probability of occurrence in Dunedin in 50 years to be MMVII. 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. Wally (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.

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.

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 developed in the earlier work. Return periods for Modified Mercalli intensities in Dunedin were estimated to be: MMVI - 31 years, MMVII - 130 years, MMVIII - 500 years, MMIX - 2200 years.

Berrill (1985a or b) 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 Dunedin at various return periods were:

| | | | | |
|---|---|---|---|---|
| Return period (years) | 50 | 150 | 450 | 1000 |
| Spectral acceleration, $a_s/g$ | 0.15 | 0.3 | 0.55 | 0.75 |

The response spectra calculated form these result using normalised spectral shapes form the basis for the design spectra in the current Design Loadings Code, NZS 4203 : 1992 (Hutchison et al, 1986).

Together with the predictions of intensities made by Smith & Berryman (1983), these have represented the best estimates of earthquake effects for hazard analysis purposes published for Dunedin to date. However when considering the three most important factors in seismic hazard determination, 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 is derived. The remaining two factors - the regional seismicity model and the local amplification due to site-specific ground conditions - need to be readdressed when carrying out a detailed seismic hazard study for Dunedin.

## 1.4 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 this data is 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 are available, this provides very useful information for calibration 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 on 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, it is necessary to make additional allowance for variable site effects caused by the specific geologic profile additional at any site. Some attenuation models include those 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 Dunedin, then a separate deep soil response model is employed to determine the variable effects in Dunedin caused by spatial inhomogeneity in soil types.

# CHAPTER 2 : SEISMICITY : POTENTIAL EARTHQUAKES AFFECTING DUNEDIN

## 2.1 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). In this chapter a revised seismicity model is developed, specifically for use in the prediction of the seismic hazard for Dunedin.

Section 2.2 considers the seismotectonic geology of the region and identifies known active faults with the potential to generate damaging earthquakes at Dunedin. All available information on significant known active faults is tabulated and used to predict the most likely magnitude (wherever possible) or otherwise the maximum credible earthquake which might be generated in each fault zone.

Large historical earthquakes for the period 1810 - 1990 are evaluated in section 2.3.

In section 2.4 all geologic and historic seismicity information is compiled to produce a seismicity model for the region. The model is based on Smith & Berrymans model but some adjustment to the region boundaries has been made, and the seismicity occurrence parameters determined for the two regions mostly likely to experience earthquakes damaging to Dunedin.

## 2.2 ACTIVE FAULTS

Dunedin is located in one of the least seismically active areas in New Zealand, being over 250 km from the main tectonic plate boundary features of the subduction zone south west of Fiordland, the Alpine fault, and the faults of North Canterbury, Marlborough and the Wairarapa. This is shown in figure 2.1, which is a map of late Quarternary Faults as compiled by Lensen. There are, however, active faults and folding in Otago and the Dunedin area.


**FIGURE 2.1**

**Late Quaternary Faults of New Zealand**

(Compiled by G L Lensen)

Figure 2.2 shows known faults of Quaternary age within 200 km of Dunedin. While all of these faults are known to have moved in the last 1.8 million years, of much greater significance are those faults generally referred to as "active faults" which have moved in the last 500,000 years. These in turn can be placed in various classes depending on the date of last movement and the recurrence interval. Unfortunately in many cases information on the date of last movement and recurrence interval is very limited.

It is theoretically possible to predict the probability of future earthquakes occurring from specific data relating to an individual fault, providing this data is known and is well constrained. However the overall hazard at a specific location such as Dunedin is the sum of the risk of rupture on a multitude of capable faults, many of which are poorly understood, and in addition a lot of seismicity occurs away from known surface fault traces. As a result seismic hazard analysis is generally based on a study of the historical seismicity rather than the sum of individual fault recurrence probabilities, but individual fault characteristics are used where known to both set and check the parameters used in seismicity analysis.

In particular the fault characteristics of maximum credible earthquake, date of last movement and recurrence interval, are used:

(a) to predict the maximum magnitude earthquakes which could conceivably occur in a given seismic region

(b) as a guide to the extent of the individual seismicity regions

(c) to check the general recurrence model derived from the historical seismicity data.

The area around Central Otago has benefitted from recent research into active faulting carried out to assist the various Clutha hydroelectric projects in the region. This has provided useful active fault information in this study.

However there are many active faults, particularly in Eastern Otago, which are important with respect to Dunedin earthquake hazard but have had only limited investigation to date. During this project we have been assisted by the University of Otago, in particular Dr Richard Norris, in assessing relevant parameters for the faults which have not been subject to intensive investigation for the hydro projects.

Table 2.1 presents the relevant parameters for the active faults shown in Figure 2.2. The relevant references are listed from which the information has been obtained and it is not proposed here to discuss all the various faults. However in some cases the assessments are very recent or based on research which is not formally published and brief discussion is appropriate:

**Akatore Fault**: This fault trace is very close to Dunedin and the epicentre of the 1974 Dunedin earthquake (M = 5) plots close to the northern end of this feature, as does a smaller 1989 event (M= 4) near the Tairei Mouth.

Original work on this fault (Makgill & Norris,1983) suggested a recurrence interval for the maximum credible earthquake (M = 7.4) of 1500 years. This implied a markedly high activity level on this feature in comparison to the albeit limited information for similar features in the area.

Recent work (Norris, pers.com., 1993) suggests that although the last major event was approximately 1500 years ago there has probably been only one or possibly two other comparable events in the last 12,000 years (i.e since the last loess deposition). Accordingly the recurrence interval has been revised to 5 - 8,000 years (Norris and Koons, 1993). This is compatible with the low rate of strain accumulation measured geodetically (Pearson, 1993). This study identified high strain rates between 1857 and 1909 but these rates reduced considerably in 1909 to 1979 data. Pearson concluded trig point disturbance rather than high geodetic deformation rate was responsible.

**Waihemo Fault**: The Waihemo fault is a branched feature in the Shag Valley forming a termination approximately at right angles to the set of traces parallel to the coast. This has traditionally been viewed as a Quaternary feature but not necessarily a late Quaternary active structure. Recent work by a BSc student at Otago (Hall, 1988) showed about 200mm of sharp displacement of the strath surface (terrace base) beneath the Late Quaternary Ranfurly terrace in the Pigroot Creek area along one branch of the Waihemo Fault. Accordingly we show at least this section of the fault as Late Quaternary.

**Hyde Fault**: Similarly local Late Quaternary movement has recently been confirmed on the Hyde Fault in the vicinity of Middlemarch by G Salton (Norris and Koons, 1993).

**Dunstan Fault**: The Dunstan Fault is very close to the Clyde Power Project and as a result has featured in the investigation in this area. Beanland et al. 1986 favour a recurrence interval of 8000 years for this fault however Norris (pers. com. 1993) considers a shorter recurrence interval is more in keeping with the total displacement. He points out that the average fault movement is approximately 1 mm/year (based on around 2000 m of offset of the Peneplain since the Pleistocene which was approximately 2 million years ago). An 8000 year recurrence interval would imply an 8m displacement per earthquake which is the scale of movement normally associated with very large magnitude earthquakes (M > 8). This displacement is not likely on a such a relatively short feature. Alternatively the recurrence interval could be much shorter (possibly 3 - 5,000 years).

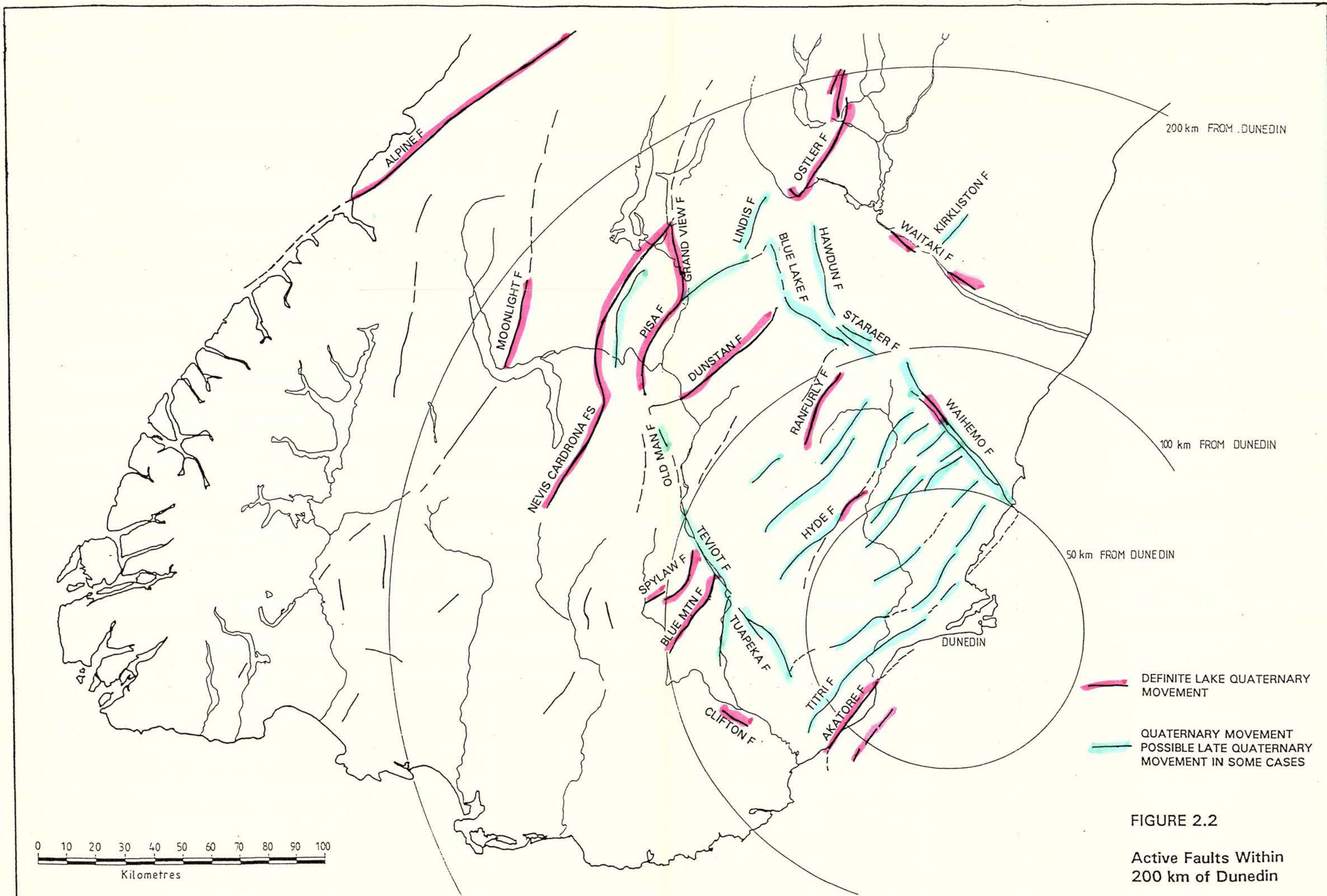


FIGURE 2.2

Active Faults Within 200 km of Dunedin

TABLE 2.1 ACTIVE FAULTS

| SPECIFIC FAULT INFORMATION FROM GEOLOGICAL STUDIES | | | | | | | |
|---|---|---|---|---|---|---|---|
| | | Maximum Credible Earthquake Estimates | | Information on Last Event and Recurrence | | | |
| Fault | Distance from Dunedin | MCE | Source Reference | Date | Magnitude | Recurrence Estimate | Source Reference |
| Alpine Fault | 240 - 260 km | 8.5 | Hancox et at, 1985 | 1650 - 1725 | | | Cooper & Norris, 1990 |
| | | | | | 7.4 - 8.0 | 426 ± 70 yrs | Hull & Berryman, 1986 |
| Moonlight Fault | 175 - 225 km | c 7.5 | Hancox et al, 1985 | | | 2000 - 5000 yrs | Hancox et al, 1975 |
| Nevis - Cardrona F.S | 140 - 170 km | 7.4 | Beanland & Berryman, 1986 | | | <3,600 yrs | Beanland & Barrow Hurlbert, 1988 |
| Dunstan Fault | 120 - 130 kms | 7.6 | Beanland & Berryman, 1986 | | | 5,000 to 8,000 yrs | Norris, pers comm, 1991. Beanland et al, 1986 |
| Spylaw Fault | 90 - 100 kms | 7.2 | Beanland & Berryman, 1986 | | | >3,500 yrs | Beanland & Berryman, 1986 |
| Teviot Fault | 75 - 100 kms | c7.2 | Beanland & Berryman, 1986 | | | >16,000 yrs | Beanland & Berryman, 1986 |
| Blue Mtn No. 1 Fault | 75 - 85 kms | 7.2 | Beanland & Berryman, 1986 | | | >8,000 yrs | Beanland & Berryman, 1986 |
| Clifton Fault | 80 kms | 7.4 | Beanland & Berryman, 1986 | | | Unknown | |
| Hyde Fault | 50 - 60 kms | 7.1 - 7.5 | This study, from fault length | | | 5,000 | Norris & Koons, 1993 |
| Akatone Fault | 10 - 60 kms | 7.4 | Makgill & Norris, 1983 | 1500 BP | 7 - 7.5 | 5 - 8,000 | Norris & Koons, 1993 Makgill & Norris, 1983 |
| Titri Fault | 6 - 60 kms | 7.1 -7.5 | This study, from fault length | | | Unknown | |

TABLE 2.3

## PARAMETERS FOR USE IN SEISMICITY MODEL

| Zone | | Description | Area | Smith & Berryman | | | $M \geq 3$ (1960 - 87) | | $M \geq 4$ (1941 - 87) | | $M \geq 5$ (1941 - 87) | | $M \geq 6$ (1846 - 1987) | | Use * | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| New | Old Smith & Berryman | | Sq. km (1000's) | Mmax | b | $a_4$ | N3 | N3/At | N4 | N4/At | N5 | N5/At | N6 | N6/At | Mmax | b | $a_4$ |
| H | H | Alpine Fault | 24.37 | 7.5<br>8.0 | 1.05<br>1.05 | 0.135<br>0.550 | | | | | | | | | 8.5 | 1.05 | 0.200 |
| L | L | Fiordland | 26.49 | 8.5 | 0.95 | 0.70 | 1483 | 2.073 | 606 | 0.847 | 76 | 0.0624 | 9 | 0.00241 | 8.5 | 1.0 | 0.750 |
| M | M < J | Otago | 64.14 | 8.0 | 1.1 | 0.08<br>-0.11 | 156 | 0.0900 | 64 | 0.0217 | 14 | 0.00475 | 2 | 0.00022 | 7.5 | 0.830 | 0.030 |
| N | N | Offshore Fiordland | 31.93 | 8.5 | 1.0 | 0.60 | | | | | | | | | 8.5 | 1.0 | 0.060 |
| O | O | Stewart Island | 19.30 | 8.0 | 1.1 | 0.08 | | | | | | | | | 8.0 | 1.077 | 0.041 |
| HF | < G,I | Hope | 2.50 | 8.5 | 1.10 | 0.40<br>-0.60 | | | | | | | | | 7.8 | 0.5 | 0.17 |
| CBne | < G | NE Canterbury | 2.60 | 8.5 | 1.10 | 0.600 | | | | | | | | | 8.0 | 0.5 | 0.12 |
| CBnw | < I | NW Canterbury | 3.04 | 8.5 | 1.10 | 0.400 | | | | | | | | | 7.5 | 0.7 | 0.17 |
| PGS | < G | Pegasus Seismic | 4.04 | 8.5 | 1.10 | 0.600 | | | | | | | | | 6.6 | 0.7 | 0.06 |
| PPT | < I,J | Porters Pass TZ | 5.08 | 8.5 | 1.10 | 0.110<br>-0.400 | | | | | | | | | 7.5 | 0.8 | 0.22 |
| CPS | < J,K | Canterbury Plains | 4.07 | 8.5 | 1.10 | 0.03<br>-0.110 | | | | | | | | | 6.6 | 0.6 | 0.05 |
| BPS | < G,K | Banks Peninsula | 13.18 | 8.5 | 1.10 | 0.030<br>0.600 | | | | | | | | | 6.6 | 0.7 | 0.009 |
| $CB_{SE}$ | < K | SE Canterbury | 9.50 | 8.0 | 1.10 | 0.030 | | | | | | | | | 8.0 | 1.1 | 0.03 |
| $CB_{SW}$ | < J | SW Canterbury | 25.13 | 8.0 | 1.10 | 0.110 | | | | | | | | | 8.0 | 1.0 | 0.11 |

* Note: Parameters for regions H and HF to $CB_{sw}$ are as determined by Elder et al

Parameters for regions N, O are as determined by Smith & Berryman

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**Offshore Trace**: Figure 2.2 shows a short offshore active trace east of the Akatore Fault. This fault has recently been described as part of a MSc study (Johnstone, 1990).

Table 2.1 notes estimates of maximum credible earthquakes for the Hyde and Titri Faults of M = 7.1 - 7.5 which conform reasonably well to the regional pattern. These estimates are based on empirical correlations between fault length and likely earthquake magnitude derived from analysis of the data from Bonilla et al 1984 and outlined in Elder et al, 1991. Figure 2.3 shows the relationship in graphical form.

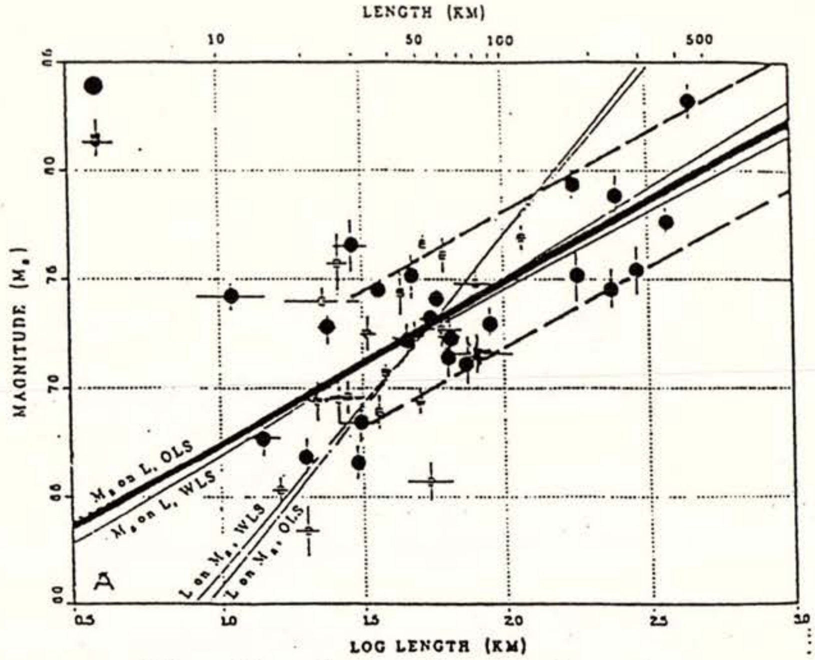
The maximum credible earthquake magnitudes are plotted at their effective epicentral distances from Dunedin on Figure 2.4. Error bars are used to indicate the range of magnitude uncertainty where this has been reported in previous studies, and a solid circle shown for the adopted mean value at the median radial distance of the fault from Dunedin.

These faults with magnitudes close to the upper bound are the most critical faults in terms of energy likely to reach Dunedin. These faults include the Akatore and Titri faults close to Dunedin, and the Alpine Fault. This figure includes the maximum magnitude earthquakes able to be generated with the seismicity regions described in Section 2.4. The seismically active Fiordland area, although not containing active faults, is also clearly significant.

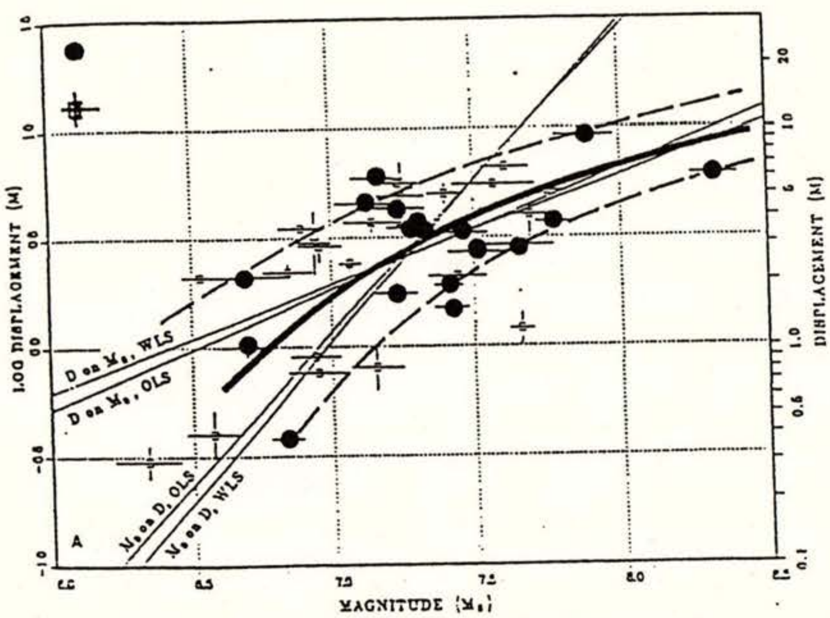
## 2.3 HISTORICAL EARTHQUAKES

Table 2.2 lists the historical earthquakes which are predicted to have caused an intensity greater than MMV in Dunedin. The bulk of the information comes from the Seismological Observatory, Institute of Geological and Nuclear Sciences. Some additional information comes from Adams and Kean (1974).

The epicentral locations are shown in Figure 2.5 showing that four out of the seven were located in Fiordland. Figure 2.6 shows the historical record in histogram form. The most significant earthquake for Dunedin was the 1974 Dunedin earthquake. The shaking intensity in Dunedin was a result of the earthquakes proximity, not its size as the earthquake magnitude was lower than any of the others. This histogram illustrates what the intensity predictions in Chapter 3 also show, that Dunedin is shaken most often by larger distant earthquakes, but that the less frequent but nearby earthquakes are the potentially damaging ones. Figure 2.6 compares the historical earthquakes recorded to date for Dunedin and Christchurch in histogram form. The relative historical seismic inactivity of Dunedin is apparent.




(a) **Magnitude - Rupture Length**



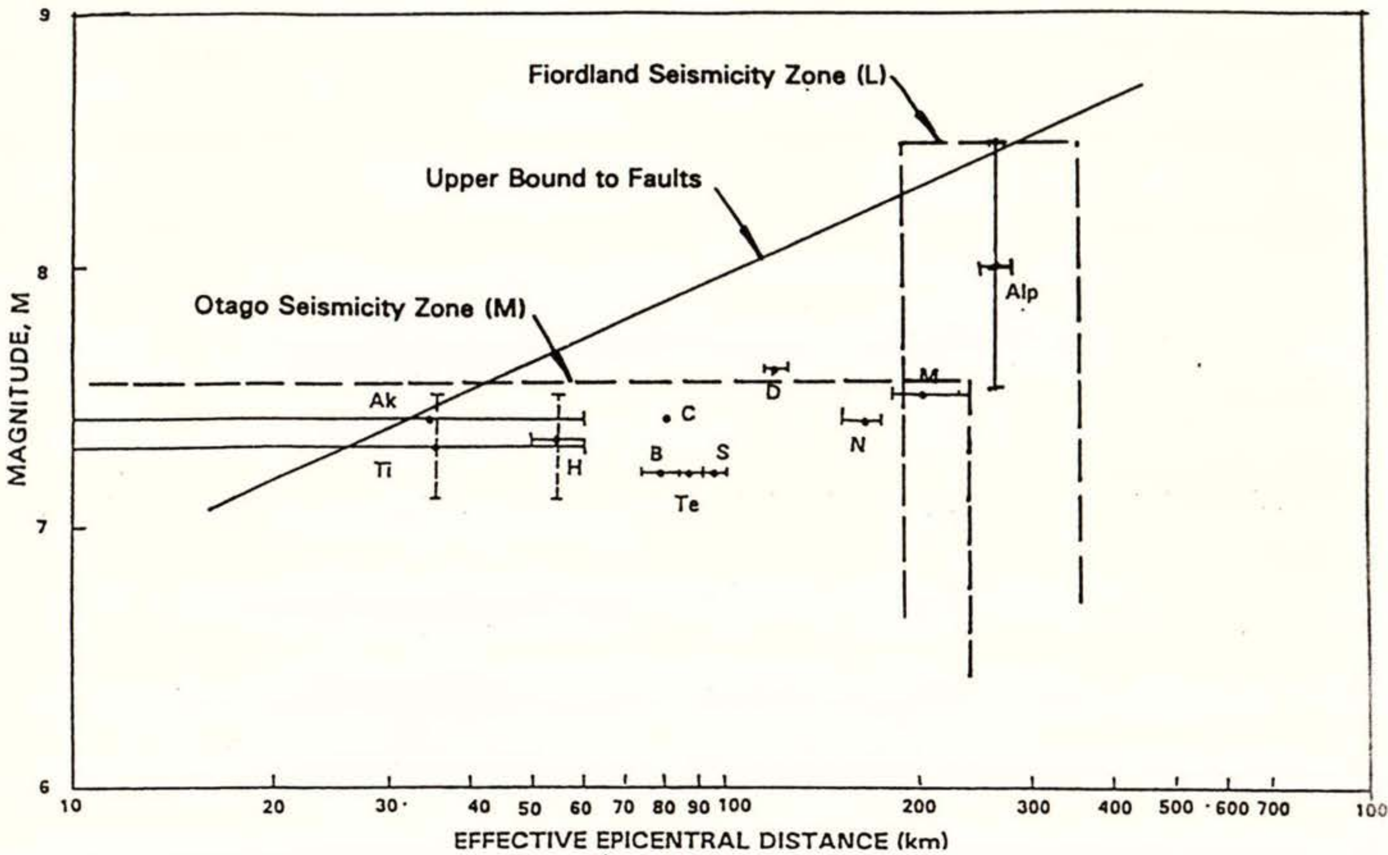

(b) **Magnitude - Fault Displacement**

**Key**

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

**FIGURE 2.3**

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




KEY

| | |
|---|---|
| Ak | Akatore Fault |
| Alp | Alpine Fault |
| B | Blue Mountain No. 1 Fault |
| C | Clifton Fault |
| D | Dunstan Fault |
| H | Hyde Fault |
| M | Moonlight Fault |
| N | Nevis - Cardrona Fault |
| S | Spylaw Fault |
| Te | Teviot Fault |
| Ti | Titri Fault |

Horizontal lines are measured radially off distances from Dunedin

Vertical error bars indicate range of magnitude uncertainty from Titri to Hyde Faults

FIGURE 2.4

Earthquake Magnitude - Effective Epicentral Distance from Dunedin

## TABLE 2.2 HISTORICAL EARTHQUAKES RESULTING IN MM INTENSITY > 5 IN DUNEDIN

| Year | Date | Latitude of Epicentre | Longitude of Epicentre | Magnitude | Distance From Dunedin | Observatory Predicted Intensity at Dunedin | Felt Intensity at Dunedin | Common Name | Information Source |
|---|---|---|---|---|---|---|---|---|---|
| 1817 | | 46 | 167 | 6 - 7.5 | 270 | 5.3 | | | Observatory |
| 1826 | | 45 | 167 | >7.5 | 290 | 5.8 | | Duskey Sound | Observatory |
| 1876 | Feb 26 | 45.2 | 170.9 | 5.8 | 82 | 5.1 | | Oamaru | Observatory |
| 1943 | Feb 17 | 45.2 | 167.9 | 6.9 | 217 | 5.3 | | Fiordland | Observatory |
| 1943 | May 8 | 44.5 | 169.9 | 6.2 | 160 | | V | South Westland | Adams & Kean |
| 1960 | May 25 | 44.2 | 167.7 | 7.0 | | | V | Fiordland | Adams & Kean |
| 1974 | April 9 | 45.97 | 170.52 | 4.9 | 10 | 5.3 | VI - VII | Dunedin | Adams & Kean 1974 |

Note: The Oamaru Earthquake is recorded as two earthquakes of M5.8 on February 26 with a further earthquake of M5.8 on April 11.

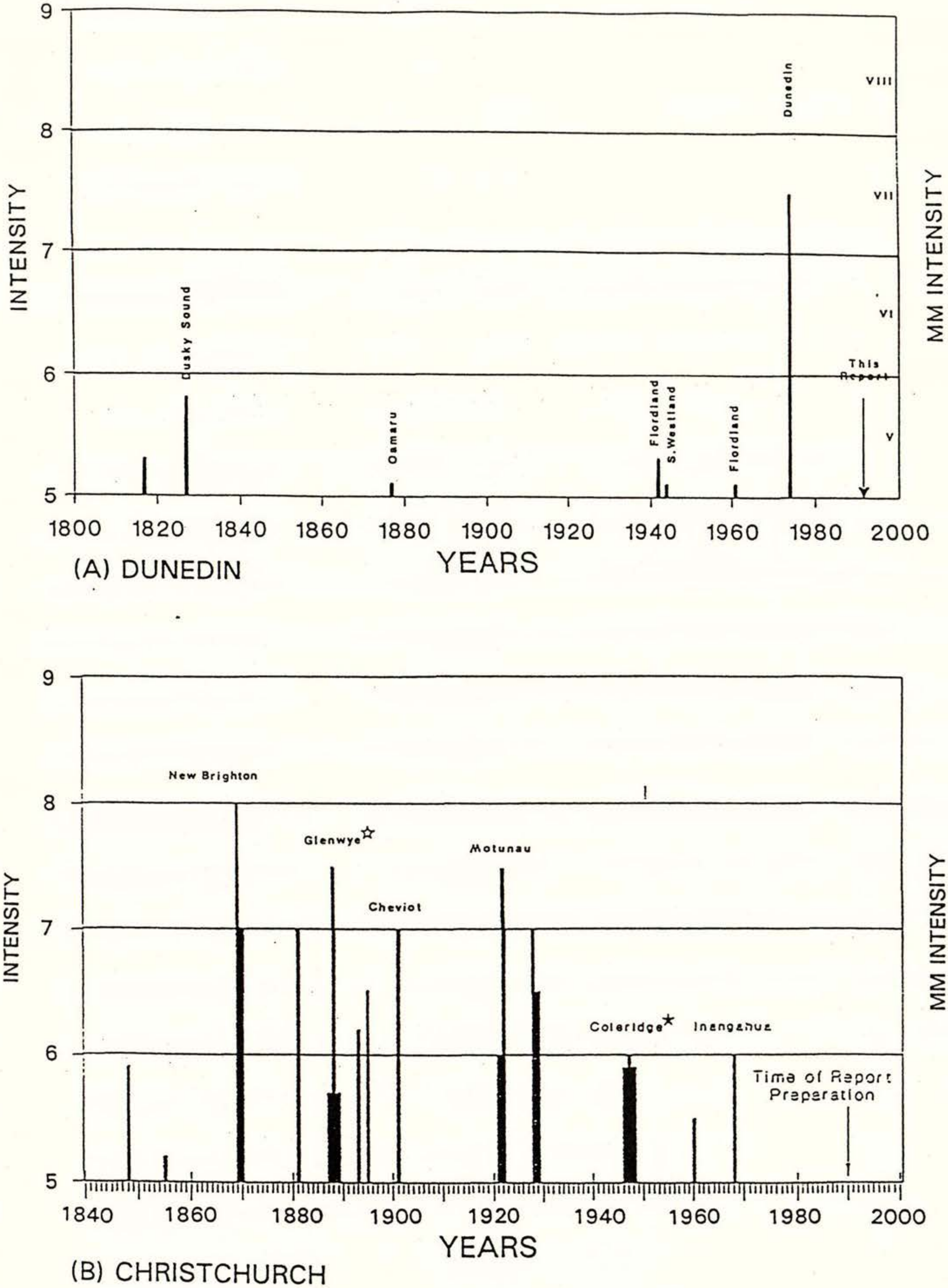
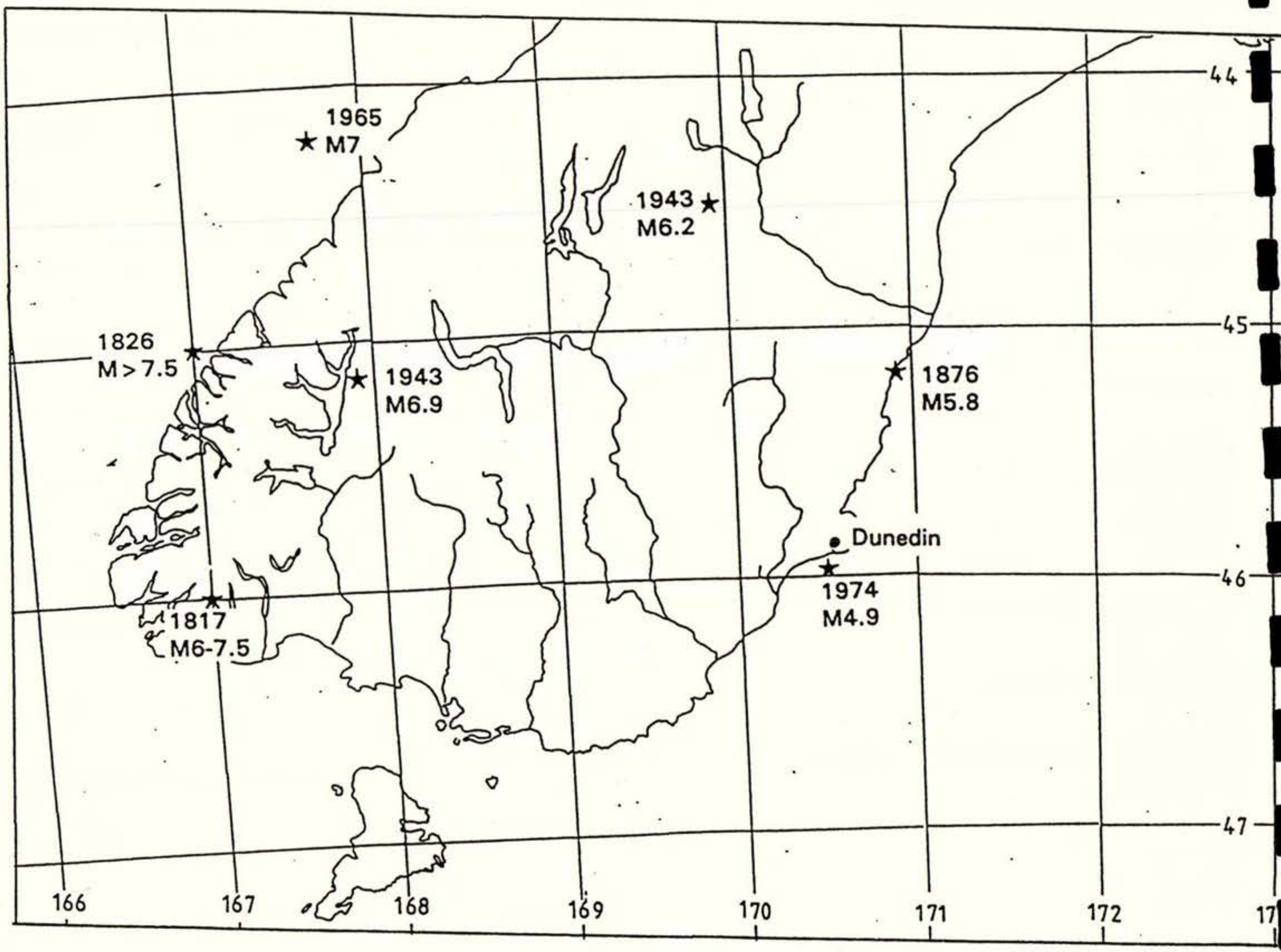



FIGURE 2.6

HISTORICAL EARTHQUAKE RECORD FOR DUNEDIN AND CHRISTCHURCH




## FIGURE 2.5

**Historical Earthquakes Resulting in MM Intensities Greater than 5 in Dunedin; Epicentral Locations**

Sources : various (See Table 2.2)

Information regarding the felt effects in Dunedin from these historical earthquakes has not been researched, because MMV shaking, while generally felt by most people, would have caused virtually no damage. The 1974 Dunedin earthquake is the only one the city has experienced since its founding that has caused damage. The earthquake was reported by Adams & Kean (1974). Although the earthquake was of only moderate magnitude (M = 4.9) the proximity to the city (7 km offshore to the South of St Kilda) produced shaking intensities of MMVI over most of the city, and up to MMVII in the worst affected areas of south Dunedin. Damage surveys showed concentrations in South Dunedin, correlating with the soft sediments in South Dunedin. Ground accelerations of 0.12g at the Dunedin Central Post Office, and 0.27 g at the St Clair Telephone Exchange were recorded.

The Earthquake and War Damages Commission received 3000 claims for damage - a particularly high number for an earthquake of this magnitude. Damage amounted to about $250,000 in 1974, equivalent to nearly $2 million in 1993 values.

## 2.4 SEISMICITY MODEL FOR SOUTHERN SOUTH ISLAND

The seismicity model used for this study is essentially that of Smith & Berryman (1983) with some adjustments to their seismic zone boundaries and parameter values and included seismicity data since their work. It is based on the traditional model describing the rate of occurrence of earthquakes of different magnitude (Gutenberg and Richter, 1954)

$$\log N = a - bM \tag{2.1}$$

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, $M_{max}$:

$$N = N_0[10^{b(Mo - M)} - 10^{b(Mo - Mmax)}] \tag{2.2}$$

$N_0$ is the number of earthquakes of magnitude $M_0$ or greater, where $M_0$ 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 2.2 becomes:

$$N = a_4\,[10^{b(4 - M)} - 10^{b(4 - Mmax)}] \tag{2.3}$$

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.

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 $M \geq 6.5$.

The distant location of Dunedin with respect to the more activity seismic regions in Fiordland and to the Southwest of Fiordland means that there is only a small contribution to the hazard in Dunedin from outside Smith & Berrymans Region M. In this study, we reviewed the seismicity model for Regions L and M only (the seismicity regions are shown on figure 2.8). Use was made of a list of earthquakes in the southern South Island provided from the computer catalogue held by the Seismological Observatory.

The epicentres for earthquake events between 1960 and 1987 were plotted on a map to enable the seismic activity patterns to be observed. As can be seen in Figure 2.7 there is a small increase in activity towards the north and west of Otago before reaching the much more active region of Fiordland. There is no discernable correlation between seismic activity and known active faults. On this basis the whole of Otago has been kept as single seismic zone as:

- There is no geological differentiation to warrant a subdivision.

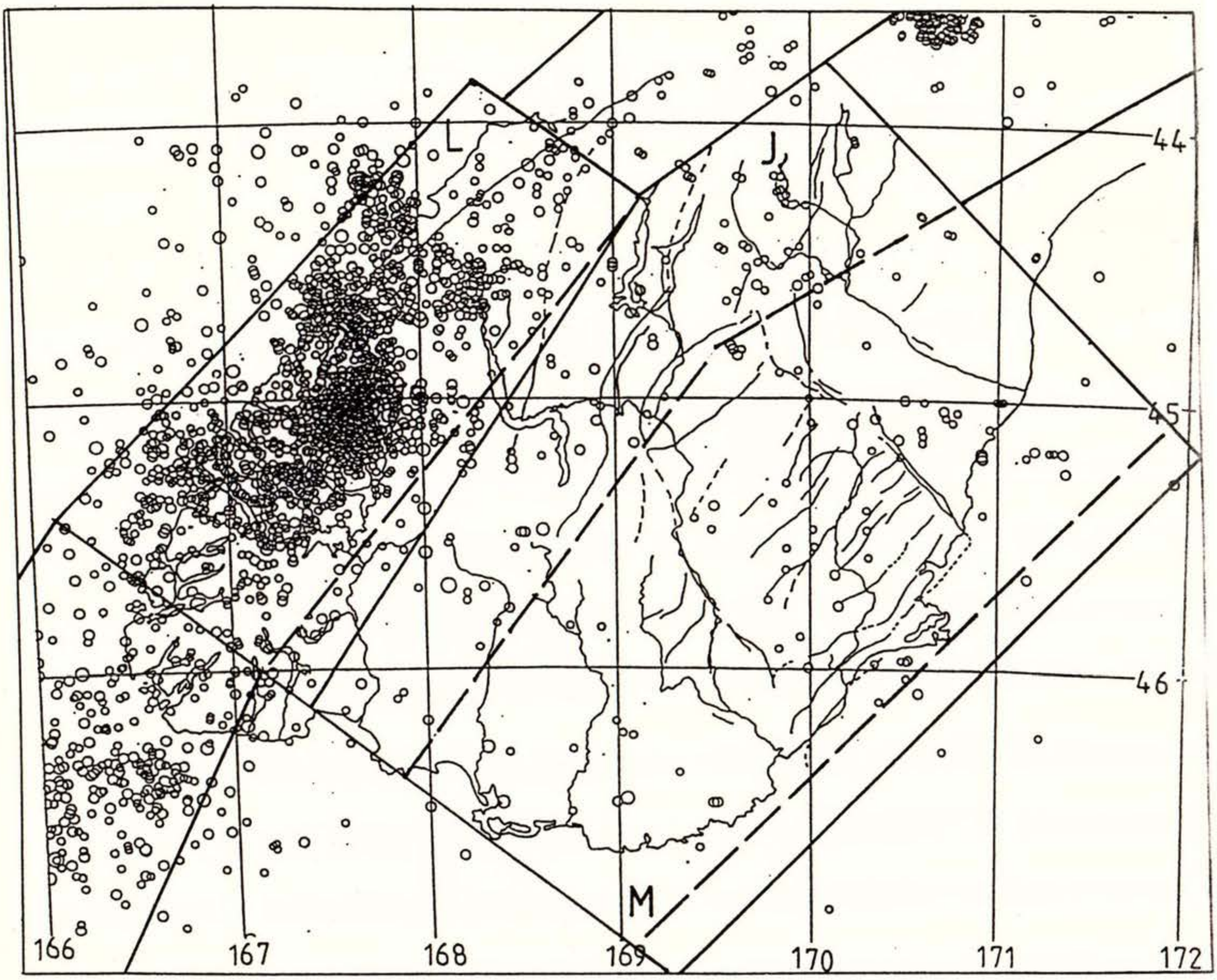
- The total amount of data for all of Otago is small : to subdivide the region into small zones would substantially increase the difficulties of sensibly assessing the seismicity patterns from the limited data.

For this study of earthquake hazard in Dunedin, the deletion of Smith & Berryman region J is compensated for by increasing the size of the Fiordland region L with its eastern boundary moved to the east. The small increase in seismicity to the north and west of the region M will have the effect of increasing the seismicity of eastern Otago within the model. This reflects some recent geological evidence that the eastern Otago faults are more active than considered previously.

The following changes have been made to Smith & Berrymans model for this study:

- Region J has been removed as a separate region, to be incorporated largely into region M.

- Region M has been extended over region J to the west, and an additional 14 km added to the southeast, further from the coast, to include most of the offshore seismicity.




Earthquake Epicentres:

° $3 \leq M < 4$

o $4 \leq M < 5$

o $5 \leq M < 6$

O $6 \leq M < 7$

Seismicity Zone Boundaries

— This Study

- - Smith & Berryman (1983)

FIGURE 2.7

OTAGO - SOUTHLAND SHALLOW SEISMICITY (1960-1987)

- The boundary between the revised region M and region L has been amended to better fit the edge of the increased seismicity in Fiordland.

- Canterbury seismicity has been modelled using the zones as determined by Elder et al. Because of the distance from Dunedin the form of model used for this area has little effect on the hazard prediction for Dunedin.

The seismicity regions used are shown in Figure 2.8.

In this study the following earthquake data has been used, with occurrence frequencies calculated at each magnitude step:

| | | |
|---|---|---|
| $M \geq 3$ | } | |
| $M \geq 4$ | } | Instrumental data recorded 1960 - 1987 |
| $M \geq 5$ | } | |
| $M > 6$ | | Published instrumental data recorded 1941 - 1991 |
| $M > 6.5$ | } | Published earthquake data recorded 1846 - 1991 |
| $M > 7$ | } | |

Data since 1987 has not been included for smaller sized earthquakes because of the increase in sensitivity of the seismograph network in that year.

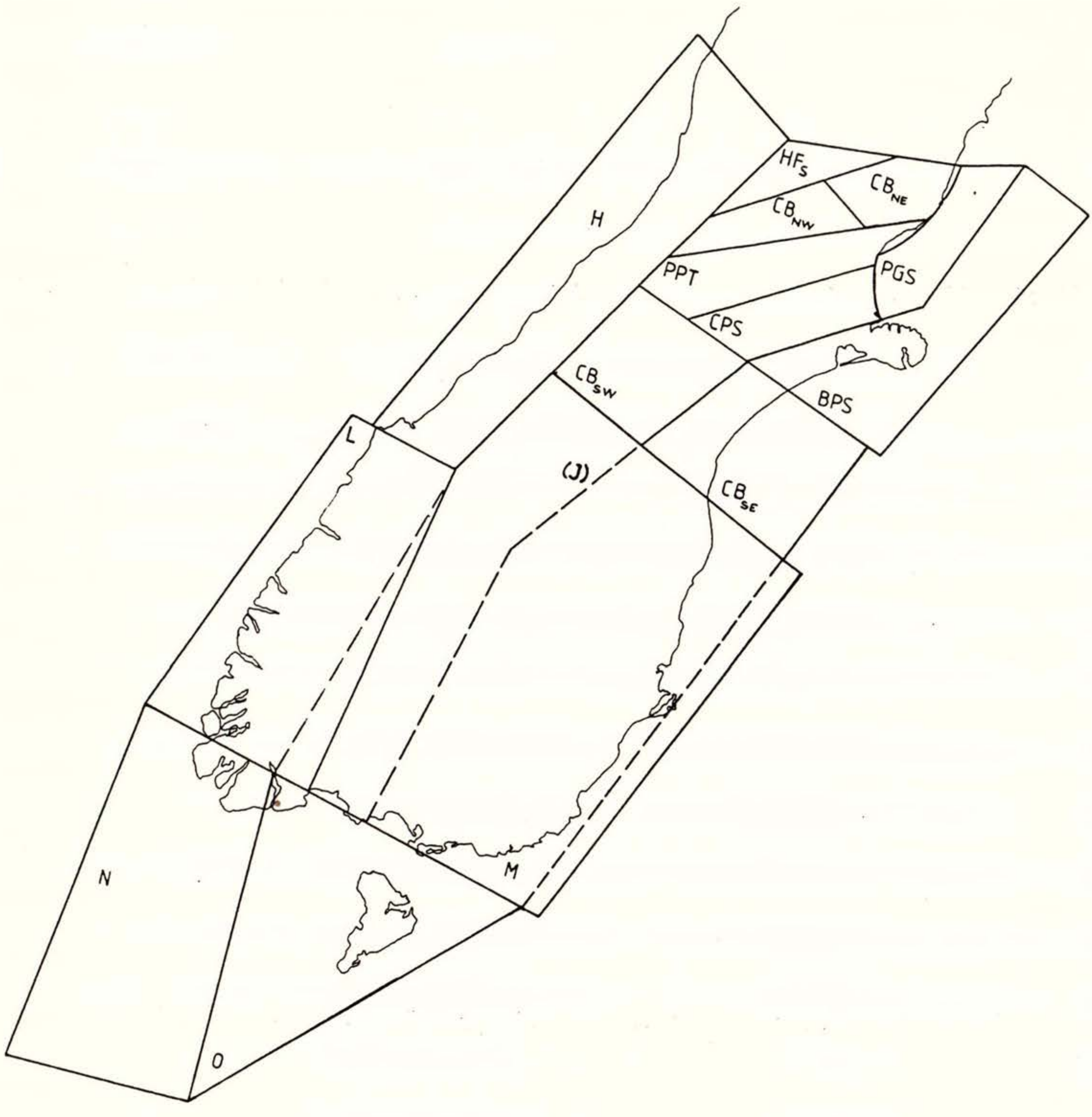
Several important facts have been considered when fitting seismicity models to the data. Some of these are discussed by Smith (1982).

- 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 earthquake record for southern New Zealand, pre installation of the seismograph network in the 1930's is thought to be incomplete, because of the low population in the Fiordland area.

- 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.




Seismicity Zone Boundaries

— This Study

- - Smith & Berryman (1983)

**FIGURE 2.8**

**Seismicity Zones Used for Dunedin Seismic Model**

- 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.

Occurrence frequency data are shown on Figure 2.9 for Regions L (Fiordland) and M (Otago). Parameter values are given in Table 2.3, for data from the various data periods, and as given in Smith & Berryman (1983),

The parameters for regions L and M have been assessed from the data as outlined above, and the following values determined as a best fit. Corresponding values from Smith & Berryman are shown for comparison. Smith & Berryman determined the b value and then assessed the $a_4$ values, whereas in this study and $a_4$ and b values have been determined together

**REGION L (FIORDLAND)**

| **This Study** | **b** | **$a_4$** |
|---|---|---|
| 1946 - 1987 | 0.923 | 0.175 |
| 1941 - 1987 | 1.034 | 0.675 |
| 1960 - 1987 | 1.319 | 1.250 |
| Chosen | 0.95 | 0.75 |

**Smith & Berryman**

| | | |
|---|---|---|
| 1840 - 1982 | 0.95 | 0.253 |
| 1942 - 1982 | 0.95 | 0.640 |
| 1965 - 1982 | 0.95 | 0.662 |
| Chosen | 0.95 | 0.70 |

**REGION M (OTAGO)**

| **This Study** | **b** | **$a_4$** |
|---|---|---|
| 1846 - 1987 | 0.886 | 0.015 |
| 1941 - 1987 | 0.792 | 0.026 |
| 1960 - 1987 | 0.856 | 0.028 |
| Chosen | 0.85 | 0.03 |

**Smith & Berryman**

| | | |
|---|---|---|
| 1940 - 1982 | 1.1 | 0.042 |
| 1965 - 1982 | 1.1 | 0.020 |
| Chosen | 1.1 | 0.08 |

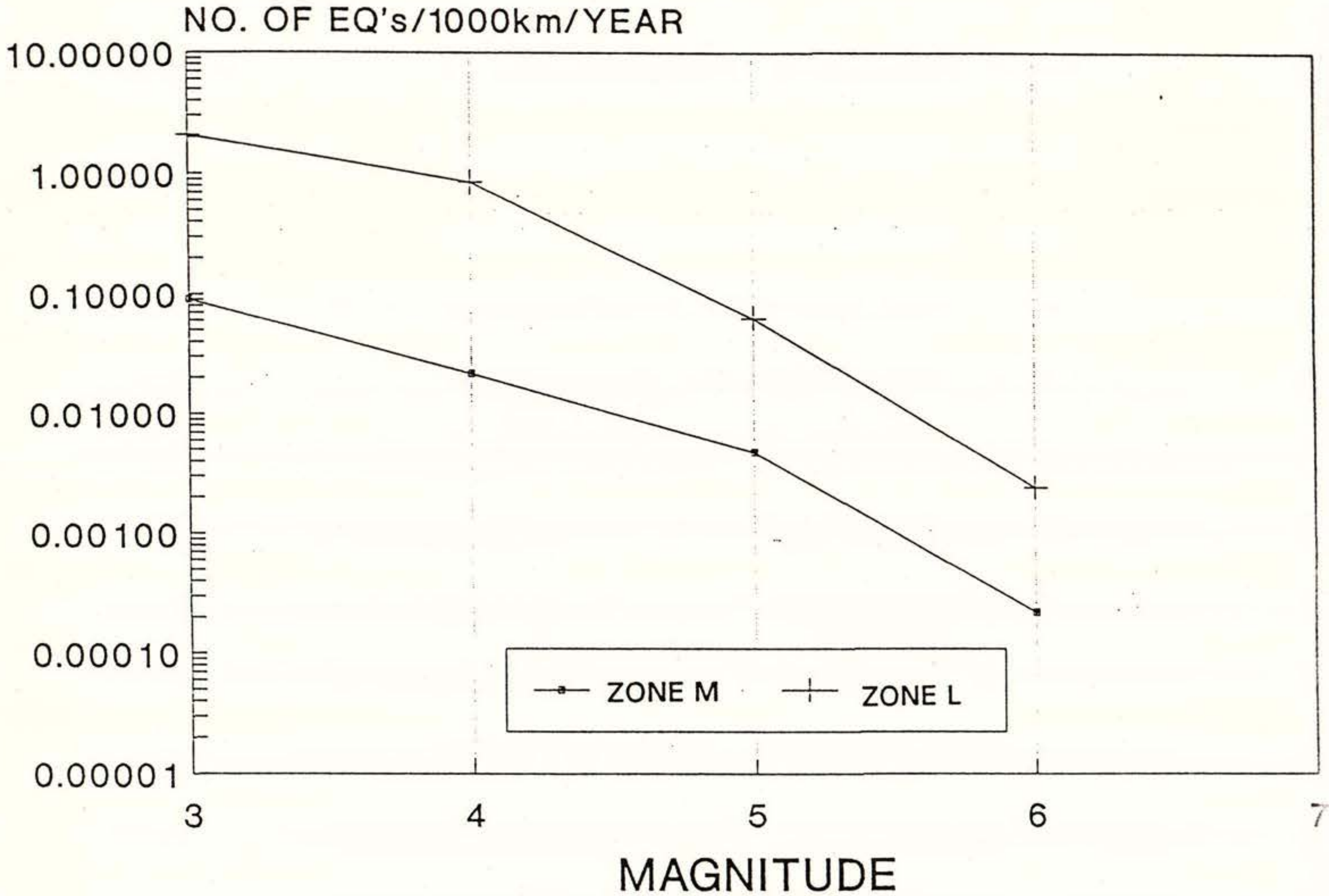



**FIGURE 2.9**

**Earthquake Occurrence Frequency**

In the Christchurch study (Elder et al 1991) Smith & Berrymans model was subdivided into several small seismicity regions in North Canterbury, with assigned b values as low as 0.5. Those regions were assessed on the basis of seismicity patterns and the faulting of the region. The low b values are necessary in the moderate earthquake range to reconcile geologic and seismicity data for individual faults or tightly constrained fault zones, as discussed in that study.

In this study, the seismicity regions in the southern South Island have been kept large as discussed above. Statistical studies for large regions containing many fault zones demonstrate that the b value is constant and generally close to 1.0.

The b values calculated for Region L (Fiordland) are similar to that used by Smith & Berryman, and their b and $a_4$ values have been used in this study. The seismicity in Region M dominates the earthquake hazard in Dunedin, and that data indicates a lower b value than Smith & Berryman's value. We have used a b value of 0.85 in this study, and also ran the probability calculations with a value of 1.0 for comparison. The choice of b value made little difference to the predicted hazard in Dunedin.

Each region, and the parameters used in the seismicity model are summarised below:

| | |
|---|---|
| Region H: | Alpine Fault. Region H of Smith & Berryman. Parameters $b = 1.05$, $a_4 = 0.2$ are as used by Elder et al, to model fault activity for $M > 6.5$ and predict a recurrence interval of about 500 years for $M > 8$. |
| Region L: | Fiordland. Region L of Smith & Berryman, but with the southeast boundary moved eastward to align more closely with the change in regional seismicity, and the geological structure. |
| Region M: | Otago. Region M and part of region J of Smith & Berryman. Eastern boundary extended 14 km further offshore to include offshore seismicity. |
| Region N: | Offshore southwest from Fiordland. As defined by Smith & Berryman. |
| Region O: | Stewart Island. As defined by Smith & Berryman. |
| Regions in Canterbury: | PGS, BPS, PPT, CPS, $CB_{NE}$ $CB_{NW}$ HF, $CB_{SE}$ $CB_{SW}$ All these regions and the seismicity parameters are as defined by Elder et al (1991). |

## 2.5 SEISMICITY : SUMMARY

A seismicity model has been developed for the southern South Island, taking into account the available geologic, tectonic and seismicity evidence. The model is based on that of Smith & Berryman (1983) with an increase in size of the Otago seismicity region, and the incorporation of the Canterbury seismicity model of Elder et al (1991) for the central South Island. Seismicity occurrence parameters were reassessed for the Otago and Fiordland Regions.

# CHAPTER 3 : INTENSITY PREDICTIONS

## 3.1 INTRODUCTION

The seismicity model developed in this study has been described in chapter 2. In order to assess ground shaking effects at Dunedin 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 which has been used to calculate intensities is described.

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 along with the New Zealand 1991 revision. 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 provided by Dr Euan Smith at the observatory.

## 3.2 MODEL FOR PREDICTION OF INTENSITIES PRODUCED BY SOUTH ISLAND EARTHQUAKES

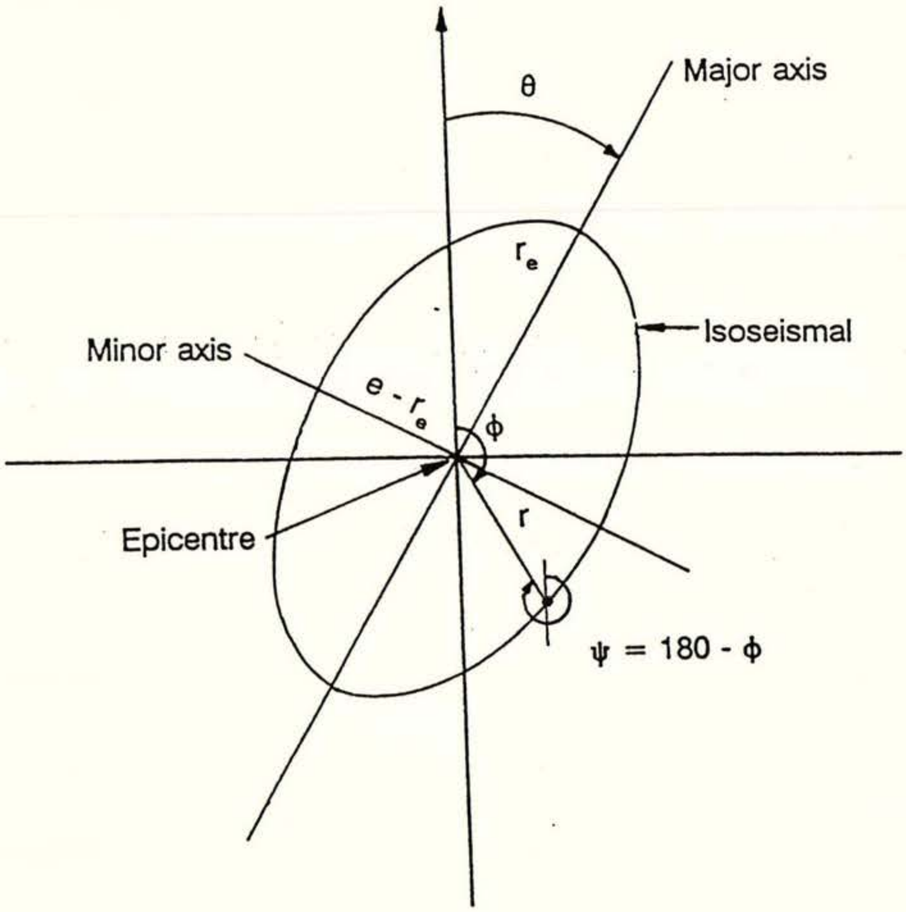
The intensity attenuation and prediction model presented in this study is a refinement of that developed by Smith (1978 a,b) for 'average ground'.

### Isoseismals

Isoseismals, or curves connecting locations of equal felt intensity from a given earthquake, are generally elliptical. The equation of an elliptical isoseismal can be expressed in polar co-ordinates r, $\theta$ as shown in Figure 3.1 as:

$$r_e^2/r^2 = \frac{1}{e^2 \sin^2(\phi - \theta) + \cos^2(\phi - \theta)} \quad (3.1)$$

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)
- $\phi$ = orientation of point east of north about centre
- $\theta$ = orientation of major isoseismal axis east of north




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 3.1**

**Definition of Elliptical Isoseismal**

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 3.2. 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.

For the southern part of the South Island, isoseismal information with respect to eccentricity is limited. In many instances there is a lack of information in more than one or two quadrants in many instances because of epicentral locations close to the west and south coastlines. A review of 14 isoseismal plots for earthquakes in the region gave 50% with major axis orientation between N32 - 55° E, and 50% between N125 - 155° E. The majority of these earthquakes were centred in the Fiordland area. This is in broad agreement with Smiths general orientation of N40° E and the eccentricity parameter shown in Figure 3.2.

The scatter of orientation, and poor data base can also be used to argue, as has Dowrick (1991), that no convincing systematic pattern of isoseismal orientation may be readily described.

Elder et al (1991) examined the major axis orientation for central and northern South Island and subdivided the northern South Island into regions according to the mean orientation of significant faults, on the basis that orientation of the major axis is close to that of the general trend of faulting in the epicentral area.

There is very little isoseismal information for earthquakes in the Otago area to check this. Faults close to Dunedin tend to follow the general 40° orientation, but there are also numerous faults at right angles. However, faults may be a minor contribution to seismic hazard in Central Otago, compared with large folding (Downes, pers comm 1992). The folding is again of approximately 40° orientation.

The values of N40° E for isoseismal axis orientation and Smiths values for the eccentricity ratio have been used for this study.

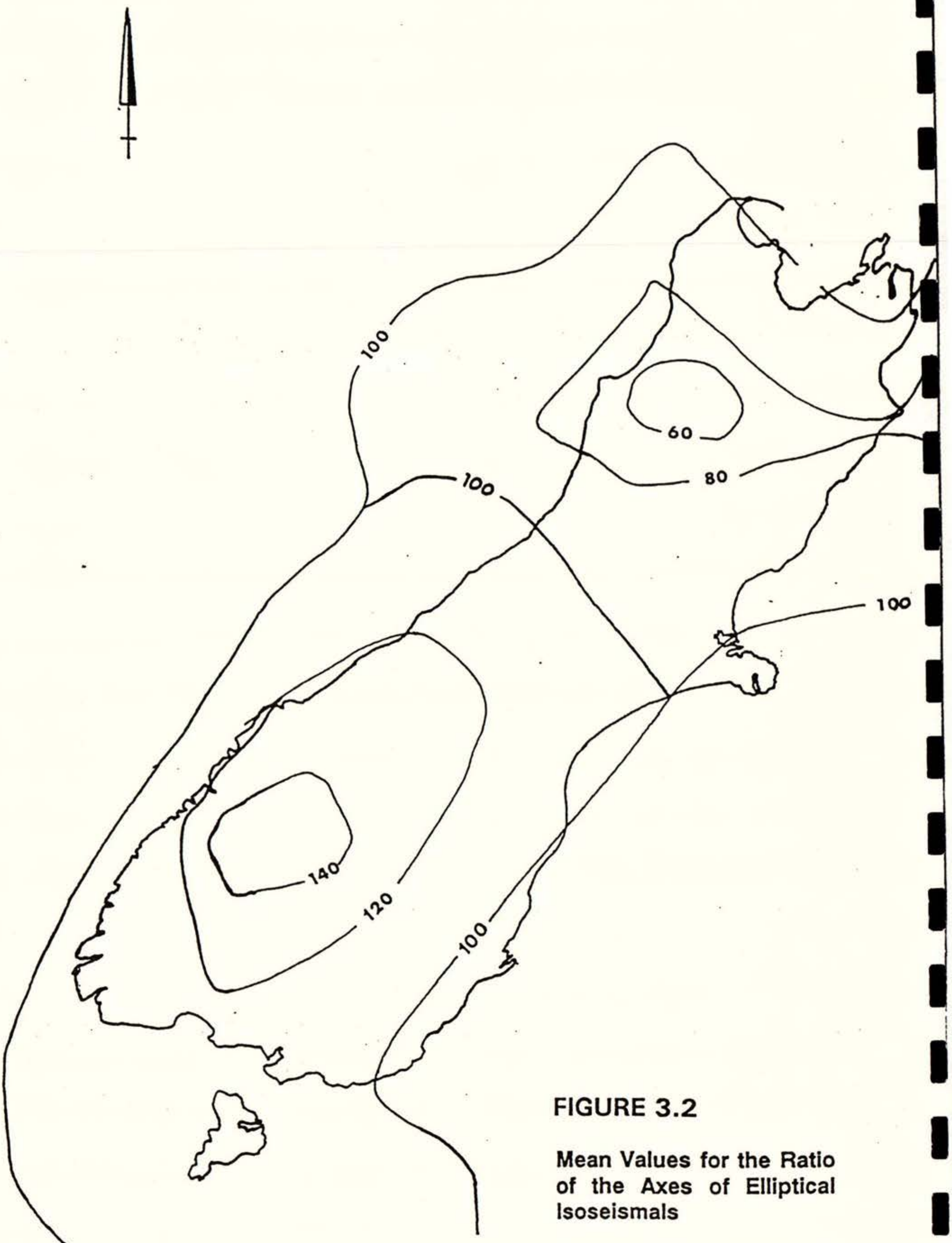



**FIGURE 3.2**

**Mean Values for the Ratio of the Axes of Elliptical Isoseismals**

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

**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 Dunedin, 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 200 km of Dunedin; 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. Smiths region B covers all the northern South Island and the eastern side of southern South Island, and region C covers south Westland and Fiordland. Dowrick (1991) suggests that the attenuation in region C is likely to be the same as the rest of the country (ie region B).

Almost all earthquakes significantly affecting Dunedin will either originate in Smith's region B, or will propagate to Dunedin predominately through region B. In this study only Smith's type B attenuation has been considered. If region C attenuation is used for earthquakes in the active Fiordland area, the effect on Dunedin is greatly reduced and the return period for all intensities MMVI - MMIX approximately doubles.

Smith proposed an intensity attenuation relationship with magnitude and epicentral distance of the form:

$$I = C(r_e).M + D(r_e) \qquad (3.2)$$

where $C(r_e)$ and $D(r_e)$ were tabulated as discrete functions of $r_e$. Elder et al (1991) refined this relationship to a continuous function so as to simplify analysis. They found the intensity 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:

$$I = I_o - k \log (r_e + d) \qquad (3.3)$$

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

$$I_o = 0.6319\ M^2 + 9.661\ M - 60.15 \qquad (3.4a)$$
$$k = 5.586\ M - 26.87 \qquad (3.4b)$$
$$d = 143\ M - 768 \qquad (3.4c)$$

all for $M \geq 5.5$.

Although this approach is purely empirical, it is logical to define the constants in equation 3.3 as functions of magnitude since this will allow the form of the attenuation equation to reflect the type and size of earthquake. The quality of this fit to the curves proposed by Smith is shown in Figure 3.3. The maximum error is about one tenth of an intensity unit.

For $M < 5.5$ the parameters in equation 3.3 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 3.1 and 3.3 gives the equation of any isoseismal of given intensity, I as a function of location and earthquake magnitude

$$r = \frac{10^{(Io-I)/k} - d}{\left[ \frac{1}{e^2} \sin^2 (\phi-\theta) + \cos^2 (\phi-\theta) \right]^{1/2}} \qquad (3.5)$$

where $I_o$, k, d are given by equations 3.4 which are implicit in the earthquake magnitude, M. In order to estimate the probability of occurrence of a given intensity at Dunedin it is necessary to calculate, for each source region, the probability of an earthquake occurring with sufficient magnitude to cause that intensity at Dunedin. This can be done by solving equation 3.2 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 Dunedin. This is done by using the transforming angle $\phi = 180 - \psi$, as shown in Figure 3.2, to give

$$r' = \frac{10^{(Io-I)/k} - d}{\left[ \frac{1}{e^2} \quad \sin^2 (\psi-180-\theta) - \cos^2 (\psi-180-\theta) \right]^{1/2}} \qquad (3.6)$$

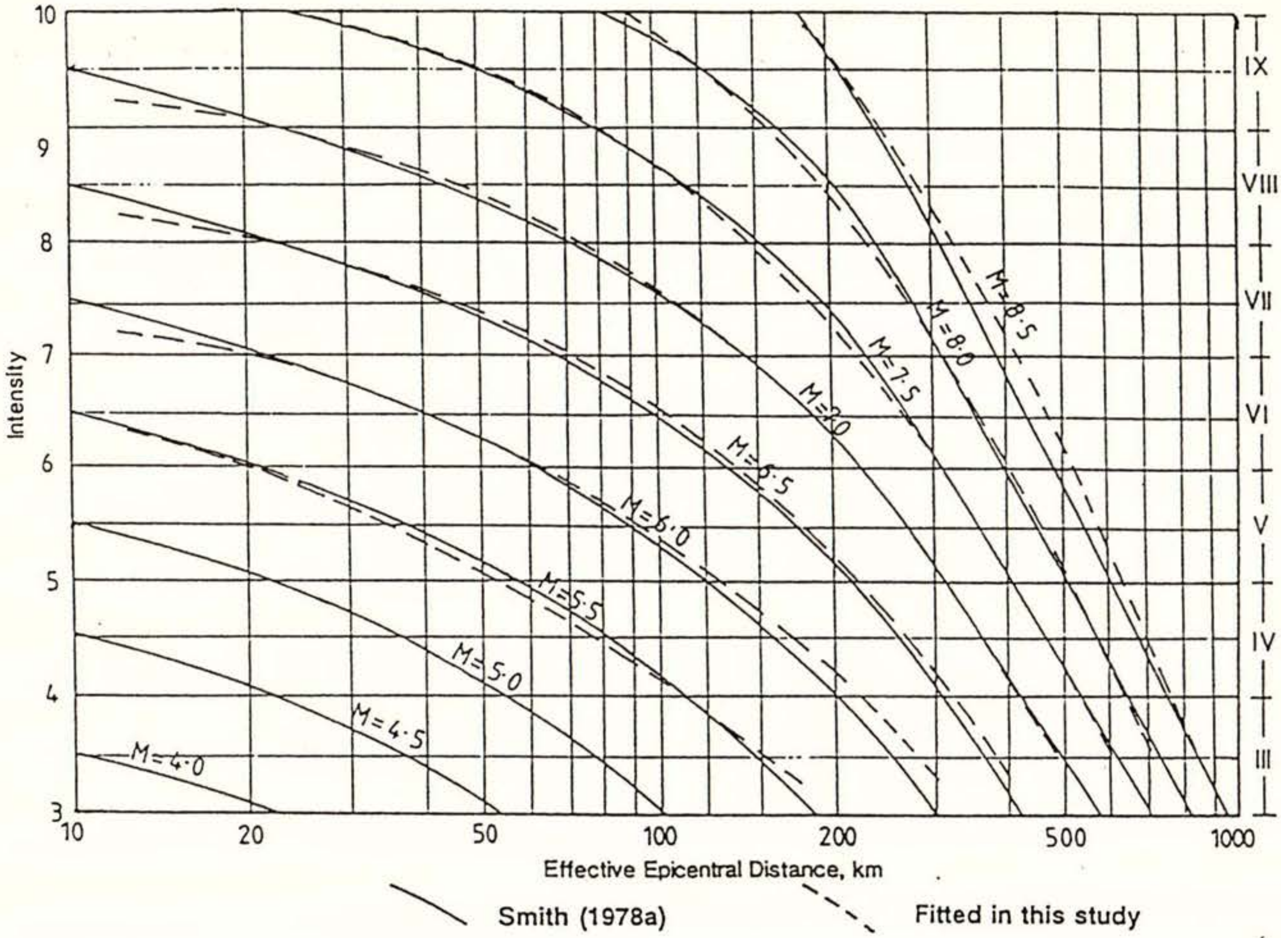



FIGURE 3.3

Attenuation Relationship for Intensity with Epicentral Distance Along Major Isoseismal Axis where $I_o$, k, d are given by equations 3.4 and $\phi$ is the orientation of each respective potential epicentre about the location of given felt intensity (e.g. Dunedin). To determine the probability of intensity $I_{Dunedin}$ occurring at Dunedin it is only necessary to determine magnitude M (causing $I_{Dunedin}$) 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 3.4 (a) to (e) for intensities I = 6, 7, 8, 9, 10 at Dunedin. 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.

## 3.3 INTENSITIES AT DUNEDIN: PROBABILITY AND RECURRENCE

In order to predict the earthquake hazard at Dunedin 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:

$$p(i \geq I) = 1 - e^{(-Nt)}$$

The return period, $\tau$, is the inverse of the annual exceedance probability, $p_1$

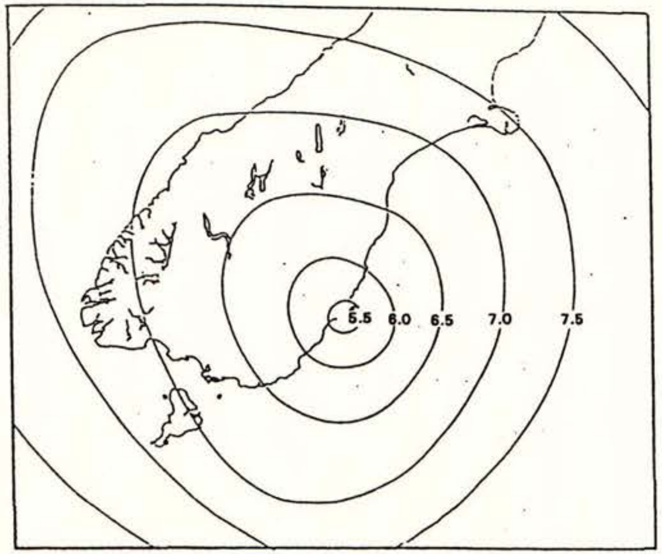
$$\tau = p_1^{-1} = [1 - e^{(-Nt)}]^{-1} \qquad (3.8)$$

The probability of exceedance $i \geq I$ in $\tau$ years is 63%, while if $N_1t$ is very small then the probability, p is approximately $N_1t$.

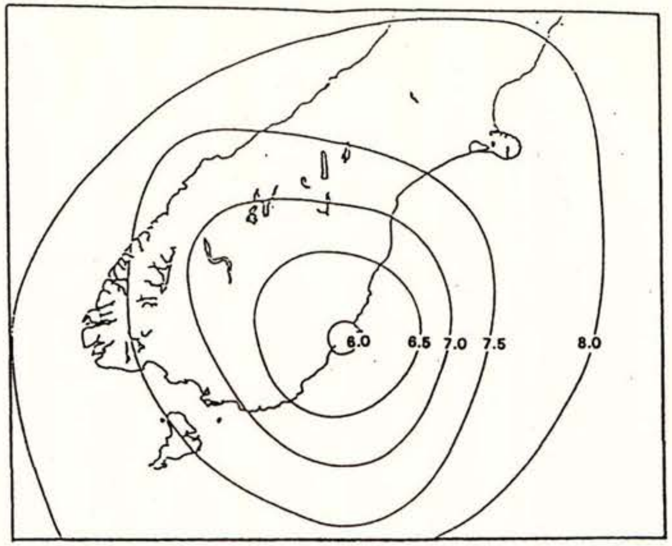
Using the seismicity model from Chapter 2, for each region k

$$N_k = A_{4k} [10^{b(4-M)} - 10^{b(4-Mmax)}] \qquad (3.9)$$

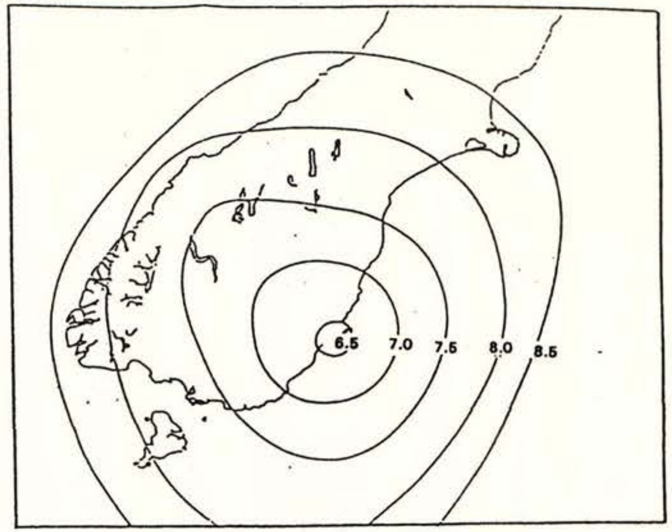
which gives the number of earthquakes per 1000 $km^2$ per year with magnitude $\geq$ M, together with the intensity attenuation model from equation 3.6 of this chapter, the probabilities of different intensities are calculated as follows:



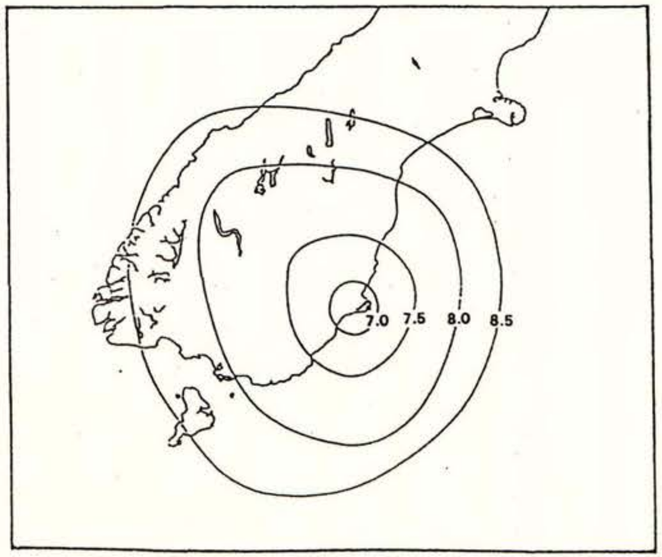

(a) Magnitudes Required for I = 6 at Dunedin



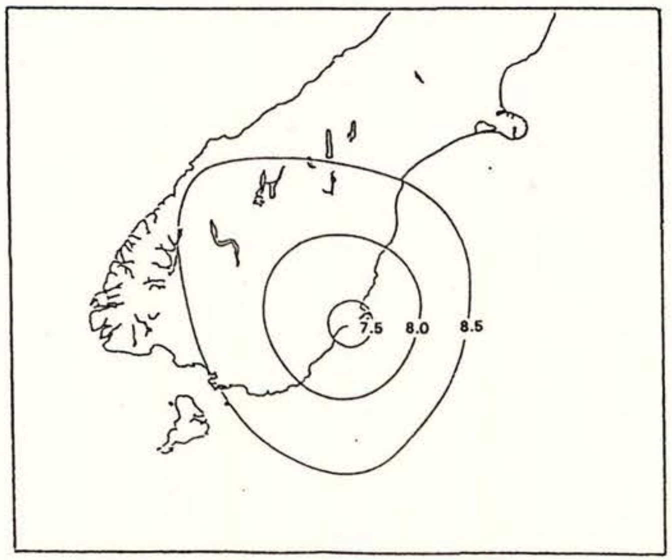

(b) Magnitudes Required for I = 7 at Dunedin




(c) Magnitudes Required for I = 8 at Dunedin




(d) Magnitudes Required for I = 9 at Dunedin




(e) Magnitudes Required for I = 10 at Dunedin

FIGURE 3.4

Isosource Maps showing earthquake magnitudes and distances required to generate different shaking intensities at Dunedin

# TABLE 3.1 Intensity Probabilities at Dunedin

| 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 | M | 0.13 | 5.50 | 6.00 | 6.40 | 6.92 | 7.40 | 7.50 | 0.83 | 0.03 | 0.000217 | 0.000080 | 0.000035 | 0.000010 | 0.000001 |
| 2 | | 0.13 | 5.50 | 6.00 | 6.40 | 6.92 | 7.40 | 7.50 | 0.83 | 0.03 | 0.000217 | 0.000080 | 0.000035 | 0.000010 | 0.000001 |
| 3 | | 0.17 | 5.50 | 6.00 | 6.40 | 6.92 | 7.40 | 7.50 | 0.83 | 0.03 | 0.000284 | 0.000105 | 0.000046 | 0.000013 | 0.000001 |
| 4 | | 0.17 | 5.50 | 6.00 | 6.40 | 6.92 | 7.40 | 7.50 | 0.83 | 0.03 | 0.000284 | 0.000105 | 0.000046 | 0.000013 | 0.000001 |
| 5 | | 0.17 | 5.50 | 6.00 | 6.40 | 6.92 | 7.40 | 7.50 | 0.83 | 0.03 | 0.000284 | 0.000105 | 0.000046 | 0.000013 | 0.000001 |
| 6 | | 0.17 | 5.50 | 6.00 | 6.40 | 6.92 | 7.40 | 7.50 | 0.83 | 0.03 | 0.000284 | 0.000105 | 0.000046 | 0.000013 | 0.000001 |
| 7 | | 0.17 | 5.50 | 6.00 | 6.40 | 6.92 | 7.40 | 7.50 | 0.83 | 0.03 | 0.000284 | 0.000105 | 0.000046 | 0.000013 | 0.000001 |
| 8 | | 0.17 | 5.50 | 6.00 | 6.40 | 6.92 | 7.40 | 7.50 | 0.83 | 0.03 | 0.000284 | 0.000105 | 0.000046 | 0.000013 | 0.000001 |
| 9 | | 0.17 | 5.50 | 6.00 | 6.40 | 6.92 | 7.40 | 7.50 | 0.83 | 0.03 | 0.000284 | 0.000105 | 0.000046 | 0.000013 | 0.000001 |
| 10 | | 0.17 | 5.50 | 6.00 | 6.40 | 6.92 | 7.40 | 7.50 | 0.83 | 0.03 | 0.000284 | 0.000105 | 0.000046 | 0.000013 | 0.000001 |
| 11 | | 0.17 | 5.50 | 6.00 | 6.40 | 6.92 | 7.40 | 7.50 | 0.83 | 0.03 | 0.000284 | 0.000105 | 0.000046 | 0.000013 | 0.000001 |
| 12 | | 0.17 | 5.50 | 6.00 | 6.40 | 6.92 | 7.40 | 7.50 | 0.83 | 0.03 | 0.000284 | 0.000105 | 0.000046 | 0.000013 | 0.000001 |
| 13 | | 0.17 | 5.50 | 6.00 | 6.40 | 6.92 | 7.40 | 7.50 | 0.83 | 0.03 | 0.000284 | 0.000105 | 0.000046 | 0.000013 | 0.000001 |
| 14 | | 0.17 | 5.50 | 6.00 | 6.40 | 6.92 | 7.40 | 7.50 | 0.83 | 0.03 | 0.000284 | 0.000105 | 0.000046 | 0.000013 | 0.000001 |
| 15 | | 0.17 | 5.50 | 6.00 | 6.40 | 6.92 | 7.40 | 7.50 | 0.83 | 0.03 | 0.000284 | 0.000105 | 0.000046 | 0.000013 | 0.000001 |
| 16 | | 0.12 | 5.50 | 6.00 | 6.40 | 6.92 | 7.40 | 7.50 | 0.83 | 0.03 | 0.000200 | 0.000074 | 0.000032 | 0.000009 | 0.000001 |
| 17 | | 0.17 | 5.75 | 6.20 | 6.70 | 7.10 | 7.65 | 7.50 | 0.83 | 0.03 | 0.000174 | 0.000070 | 0.000023 | 0.000007 | - |
| 18 | | 0.33 | 5.75 | 6.20 | 6.70 | 7.10 | 7.65 | 7.50 | 0.83 | 0.03 | 0.000337 | 0.000135 | 0.000045 | 0.000014 | - |
| 19 | | 0.33 | 5.75 | 6.20 | 6.70 | 7.10 | 7.65 | 7.50 | 0.83 | 0.03 | 0.000337 | 0.000135 | 0.000045 | 0.000014 | - |
| 20 | | 0.33 | 5.75 | 6.20 | 6.70 | 7.10 | 7.65 | 7.50 | 0.83 | 0.03 | 0.000337 | 0.000135 | 0.000045 | 0.000014 | - |
| 21 | | 0.33 | 5.75 | 6.20 | 6.70 | 7.10 | 7.65 | 7.50 | 0.83 | 0.03 | 0.000337 | 0.000135 | 0.000045 | 0.000014 | - |
| 22 | | 0.33 | 5.75 | 6.20 | 6.70 | 7.10 | 7.65 | 7.50 | 0.83 | 0.03 | 0.000337 | 0.000135 | 0.000045 | 0.000014 | - |
| 23 | | 0.33 | 5.75 | 6.20 | 6.70 | 7.10 | 7.65 | 7.50 | 0.83 | 0.03 | 0.000337 | 0.000135 | 0.000045 | 0.000014 | - |
| 24 | | 0.33 | 5.75 | 6.20 | 6.70 | 7.10 | 7.65 | 7.50 | 0.83 | 0.03 | 0.000337 | 0.000135 | 0.000045 | 0.000014 | - |
| 25 | | 0.33 | 5.75 | 6.20 | 6.70 | 7.10 | 7.65 | 7.50 | 0.83 | 0.03 | 0.000337 | 0.000135 | 0.000045 | 0.000014 | - |
| 26 | | 0.33 | 5.75 | 6.20 | 6.70 | 7.10 | 7.65 | 7.50 | 0.83 | 0.03 | 0.000337 | 0.000135 | 0.000045 | 0.000014 | - |
| 27 | | 0.33 | 5.75 | 6.20 | 6.70 | 7.10 | 7.65 | 7.50 | 0.83 | 0.03 | 0.000337 | 0.000135 | 0.000045 | 0.000014 | - |
| 28 | | 0.24 | 5.75 | 6.20 | 6.85 | 7.10 | 7.65 | 7.50 | 0.83 | 0.03 | 0.000245 | 0.000099 | 0.000022 | 0.000010 | - |
| 29 | | 0.68 | 6.00 | 6.35 | 6.85 | 7.40 | 7.85 | 7.50 | 0.83 | 0.03 | 0.000421 | 0.000203 | 0.000063 | 0.000005 | - |
| 30 | | 0.94 | 6.10 | 6.35 | 6.85 | 7.40 | 7.80 | 7.50 | 0.83 | 0.03 | 0.000475 | 0.000281 | 0.000086 | 0.000007 | - |
| 31 | | 0.94 | 6.00 | 6.35 | 6.85 | 7.40 | 7.80 | 7.50 | 0.83 | 0.03 | 0.000582 | 0.000281 | 0.000086 | 0.000007 | - |
| 32 | | 0.94 | 6.00 | 6.35 | 6.85 | 7.40 | 7.80 | 7.50 | 0.83 | 0.03 | 0.000582 | 0.000281 | 0.000086 | 0.000007 | - |
| 33 | | 0.94 | 6.00 | 6.35 | 6.85 | 7.40 | 7.80 | 7.50 | 0.83 | 0.03 | 0.000582 | 0.000281 | 0.000086 | 0.000007 | - |
| 34 | | 0.94 | 6.00 | 6.35 | 6.85 | 7.40 | 7.80 | 7.50 | 0.83 | 0.03 | 0.000582 | 0.000281 | 0.000086 | 0.000007 | - |
| 35 | | 0.94 | 6.10 | 6.35 | 6.85 | 7.40 | 7.80 | 7.50 | 0.83 | 0.03 | 0.000475 | 0.000281 | 0.000086 | 0.000007 | - |
| 36 | | 0.94 | 6.10 | 6.35 | 6.85 | 7.40 | 7.80 | 7.50 | 0.83 | 0.03 | 0.000475 | 0.000281 | 0.000086 | 0.000007 | - |
| 37 | | 0.94 | 6.15 | 6.35 | 6.85 | 7.40 | 7.80 | 7.50 | 0.83 | 0.03 | 0.000428 | 0.000281 | 0.000086 | 0.000007 | - |
| 38 | | 0.90 | 6.15 | 6.30 | 6.85 | 7.40 | 7.80 | 7.50 | 0.83 | 0.03 | 0.000410 | 0.000299 | 0.000083 | 0.000007 | - |
| 39 | | 0.76 | 6.15 | 6.70 | 7.20 | 7.70 | 8.10 | 7.50 | 0.83 | 0.03 | 0.000227 | 0.000103 | 0.000022 | - | - |
| 40 | | 1.50 | 6.30 | 6.65 | 7.20 | 7.70 | 8.10 | 7.50 | 0.83 | 0.03 | 0.000499 | 0.000228 | 0.000043 | - | - |
| 41 | | 1.50 | 6.25 | 6.55 | 7.20 | 7.70 | 8.00 | 7.50 | 0.83 | 0.03 | 0.000555 | 0.000288 | 0.000043 | - | - |
| 42 | | 1.50 | 6.25 | 6.55 | 7.10 | 7.65 | 8.00 | 7.50 | 0.83 | 0.03 | 0.000555 | 0.000288 | 0.000064 | - | - |
| 43 | | 1.50 | 6.25 | 6.50 | 7.10 | 7.65 | 7.95 | 7.50 | 0.83 | 0.03 | 0.000555 | 0.000323 | 0.000064 | - | - |
| 44 | | 1.50 | 6.25 | 6.50 | 7.10 | 7.65 | 7.90 | 7.50 | 0.83 | 0.03 | 0.000555 | 0.000323 | 0.000064 | - | - |
| 45 | | 1.50 | 6.10 | 6.40 | 7.20 | 7.65 | 8.00 | 7.50 | 0.83 | 0.03 | 0.000499 | 0.000402 | 0.000043 | - | - |
| 46 | | 1.50 | 6.30 | 6.45 | 7.20 | 7.65 | 8.10 | 7.50 | 0.83 | 0.03 | 0.000499 | 0.000361 | 0.000043 | - | - |
| 47 | | 2.56 | 6.35 | 6.65 | 7.25 | 7.70 | 8.15 | 7.50 | 0.83 | 0.03 | 0.000765 | 0.000390 | 0.000059 | - | - |
| 48 | | 1.37 | 6.35 | 6.70 | 7.25 | 7.75 | 8.15 | 7.50 | 0.83 | 0.03 | 0.000409 | 0.000185 | 0.000031 | - | - |
| 49 | | 3.64 | 6.65 | 7.00 | 7.50 | 7.85 | 8.35 | 7.50 | 0.83 | 0.03 | 0.000554 | 0.000217 | - | - | - |
| 50 | | 1.67 | 6.50 | 6.50 | 7.40 | 7.85 | 8.25 | 7.50 | 0.83 | 0.03 | 0.000359 | 0.000359 | 0.000013 | - | - |
| 51 | | 1.67 | 6.45 | 6.85 | 7.35 | 7.85 | 8.25 | 7.50 | 0.83 | 0.03 | 0.000401 | 0.000154 | 0.000021 | - | - |
| 52 | | 1.67 | 6.40 | 6.80 | 7.25 | 7.85 | 8.15 | 7.50 | 0.83 | 0.03 | 0.000448 | 0.000175 | 0.000038 | - | - |
| 53 | | 1.67 | 6.40 | 6.75 | 7.25 | 7.85 | 8.15 | 7.50 | 0.83 | 0.03 | 0.000448 | 0.000199 | 0.000038 | - | - |
| 54 | | 1.67 | 6.45 | 6.65 | 7.30 | 7.85 | 8.25 | 7.50 | 0.83 | 0.03 | 0.000401 | 0.000254 | 0.000029 | - | - |
| 55 | | 1.67 | 6.50 | 6.70 | 7.35 | 7.85 | 8.25 | 7.50 | 0.83 | 0.03 | 0.000359 | 0.000225 | 0.000021 | - | - |
| 56 | | 4.32 | 6.75 | 7.25 | 7.70 | 8.10 | 8.50 | 7.50 | 0.83 | 0.03 | 0.000515 | 0.000099 | - | - | - |
| 57 | | 2.29 | 6.65 | 7.20 | 7.55 | 7.95 | 8.40 | 7.50 | 0.83 | 0.03 | 0.000348 | 0.000066 | - | - | - |
| 58 | | 2.31 | 6.55 | 6.15 | 7.40 | 7.90 | 8.25 | 7.50 | 0.83 | 0.03 | 0.000444 | 0.001052 | 0.000018 | - | - |
| 59 | | 2.43 | 6.65 | 6.90 | 7.50 | 7.90 | 8.35 | 7.50 | 0.83 | 0.03 | 0.000370 | 0.000195 | - | - | - |
| 60 | | 4.88 | 6.75 | 6.85 | 7.70 | 8.10 | 8.50 | 7.50 | 0.83 | 0.03 | 0.000582 | 0.000449 | - | - | - |
| 61 | | 1.21 | 6.70 | 6.85 | 7.10 | 8.00 | 8.40 | 7.50 | 0.83 | 0.03 | 0.000161 | 0.000111 | 0.000052 | - | - |
| 62 | O | 9.72 | 7.00 | 7.50 | 7.85 | 8.25 | 8.70 | 8.00 | 1.08 | 0.04 | 0.000215 | 0.000048 | 0.000009 | - | - |
| 63 | | 9.58 | 7.60 | 7.80 | 8.85 | 9.00 | 9.00 | 8.00 | 1.08 | 0.04 | 0.000033 | 0.000012 | - | - | - |
| 64 | N | 31.93 | 7.80 | 8.00 | 8.70 | 9.20 | 9.50 | 8.50 | 1.00 | 0.06 | 0.000243 | 0.000131 | - | - | - |
| 65 | L | 7.25 | 7.10 | 7.65 | 8.00 | 8.30 | 8.70 | 8.50 | 1.00 | 0.75 | 0.004147 | 0.001045 | 0.000372 | 0.000101 | - |
| 66 | | 6.52 | 6.85 | 7.55 | 7.90 | 8.20 | 8.65 | 8.50 | 1.00 | 0.75 | 0.006753 | 0.001224 | 0.000461 | 0.000154 | - |
| 67 | | 5.93 | 6.80 | 7.40 | 7.75 | 8.15 | 8.50 | 8.50 | 1.00 | 0.75 | 0.006908 | 0.001630 | 0.000650 | 0.000174 | - |
| 68 | | 5.65 | 6.80 | 7.40 | 7.75 | 8.15 | 8.50 | 8.50 | 1.00 | 0.75 | 0.006582 | 0.001553 | 0.000620 | 0.000166 | - |
| 69 | H | 5.33 | 6.95 | 7.35 | 7.85 | 8.25 | 8.65 | 8.50 | 1.05 | 0.20 | 0.000832 | 0.000304 | 0.000077 | 0.000017 | - |
| 70 | | 5.88 | 7.25 | 7.40 | 8.25 | 8.40 | 8.80 | 8.50 | 1.05 | 0.20 | 0.000433 | 0.000294 | 0.000018 | 0.000006 | - |
| 75 | | 6.29 | 7.55 | 7.85 | 8.50 | 9.20 | 9.00 | 8.50 | 1.05 | 0.20 | 0.000212 | 0.000090 | - | - | - |
| 76 | | 6.88 | 8.00 | 8.40 | 9.00 | 9.60 | 9.50 | 8.50 | 1.05 | 0.20 | 0.000061 | 0.000007 | - | - | - |
| 71 | CBsw | 3.32 | 7.00 | 7.20 | 7.90 | 8.40 | 8.70 | 8.00 | 1.00 | 0.11 | 0.000329 | 0.000194 | 0.000009 | - | - |
| 72 | | 3.82 | 7.00 | 7.20 | 7.95 | 8.40 | 8.70 | 8.00 | 1.00 | 0.11 | 0.000378 | 0.000223 | 0.000005 | - | - |
| 73 | CBse | 5.05 | 6.80 | 7.20 | 7.75 | 8.30 | 8.60 | 8.00 | 1.10 | 0.03 | 0.000120 | 0.000040 | 0.000005 | - | - |
| 74 | | 4.45 | 6.75 | 7.20 | 7.75 | 8.20 | 8.50 | 8.00 | 1.10 | 0.03 | 0.000121 | 0.000035 | 0.000005 | - | - |
| 77 | HF | 2.50 | 8.00 | 8.40 | 8.80 | 10.00 | 9.60 | 7.80 | 0.50 | 0.17 | - | - | - | - | - |
| 78 | CBnw | 3.05 | 7.80 | 7.62 | 8.60 | 9.60 | 9.40 | 7.50 | 0.70 | 0.17 | - | - | - | - | - |
| 79 | PPT | 5.09 | 7.65 | 7.80 | 8.35 | 9.00 | 9.20 | 7.50 | 0.80 | 0.22 | - | - | - | - | - |
| 80 | CPS | 4.07 | 7.45 | 7.70 | 8.30 | 8.80 | 9.10 | 6.60 | 0.60 | 0.05 | - | - | - | - | - |
| 81 | BPS | 13.19 | 7.55 | 7.75 | 8.40 | 9.00 | 9.40 | 6.60 | 0.70 | 0.01 | - | - | - | - | - |
| 82 | CBne | 2.61 | 7.85 | 8.40 | 8.80 | 10.00 | 9.60 | 8.00 | 0.50 | 0.12 | 0.000590 | - | - | - | - |
| 83 | pgs | 4.03 | 7.80 | 8.50 | 8.70 | 9.80 | 9.70 | 6.60 | 0.70 | 0.06 | - | - | - | - | - |
| | | | | | | | | | | Total | 0.05159 | 0.01915 | 0.00496 | 0.00104 | 0.00002 |

- Divide each seismicity region into subregions of sufficiently small area that a mean distance may be used from Dunedin to a central node, j, in the subregion. For each node, the area ($A_j$), attenuation parameters ($\theta_j$, $e_j$), distance and orientation from Dunedin ($r_j$, $\psi$), and seismicity parameters ($a_k$, $b_k$, $M_{max\ k}$) are determined.

- For each intensity value I selected at Dunedin the earthquake magnitude $M_j$ at node j which will cause intensity I is calculated from equation 3.6.

- The annual frequency of occurrence $N_j$ of earthquakes with magnitude $M_j$ or greater is calculated from equation 3.9 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 3.1 and exceedance probabilities are summarised in Table 3.2, where the relative contributions to the overall probability from each seismicity region are also shown.

Return periods calculated in this study are essentially the same as those previously reported by Smith & Berryman as shown in Figure 3.5. The changes in the seismicity model for Otago, and the different seismicity parameters used are not of significance to the overall seismic hazard.

Regions M (all of Otago and Southland) and Fiordland dominate the seismic hazard in Dunedin. Intensity 10 shaking could only be produced by a maximum magnitude earthquake occuring within 10 - 20 km of the city, and has a very low probability of occurrence. Intensity 9 shaking could be produced either by a large earthquake occuring within 90 km of the city, or a very large earthquake in Fiordland. Lower intensities are likely from either region M taken overall, or from Fiordland. The contribution to the hazard from the Alpine Fault, and the marine areas south of New Zealand are all very small. Earthquakes in Canterbury are too far away to contribute any hazard.

**TABLE 3.2 Probabilities of Occurrence of Different Intensities on Average Ground at Dunedin, and Relative Contributions from each Seismicity Region**

Percentage Contribution to Total Probability from each Seismicity Zone

| SEISMICITY REGION | INTENSITY AT DUNEDIN 6 | 7 | 8 | 9 | 10 |
|---|---|---|---|---|---|
| M | 45.81 | 64.34 | 55.03 | 40.87 | 100.00 |
| O | 0.48 | 0.32 | 0.18 | - | - |
| N | 0.47 | 0.68 | - | - | - |
| L | 47.28 | 28.47 | 42.38 | 56.95 | - |
| H | 2.98 | 3.63 | 1.91 | 2.18 | - |
| CBsw | 1.37 | 2.18 | 0.29 | - | - |
| CBse | 0.47 | 0.39 | 0.20 | - | - |
| BPS | - | - | - | - | - |
| CPS | - | - | - | - | - |
| PPT | - | - | - | - | - |
| PGS | - | - | - | - | - |
| CBnw | - | - | - | - | - |
| CBne | 1.14 | - | - | - | - |
| HF | - | - | - | - | - |
| TOTALS | 100.00 | 100.00 | 100.00 | 100.00 | 100.00 |
| Annual Exceedance $N_1$ | 0.052 | 0.019 | 0.005 | 0.001 | 0.00002 |
| Return Period (yrs) | 20 | 52 | 200 | 950 | >60,000 |
| Probability (%) of Exceedance in: | | | | | |
| 50 years | 92.9 | 62.0 | 22.0 | 5.1 | 0.1 |
| 150 years | 100 | 94.5 | 52.6 | 14.5 | 0.3 |
| 450 years | 100 | 100 | 89.3 | 37.5 | 0.9 |
| 1000 years | 100 | 100 | 99.3 | 64.8 | 2.0 |
| Modified Mercalli Intensity | MMVI | MMVII | MMVIII | MMIX | |
| Return Period (yrs) | 30 | 100 | 450 | >2,500 | |

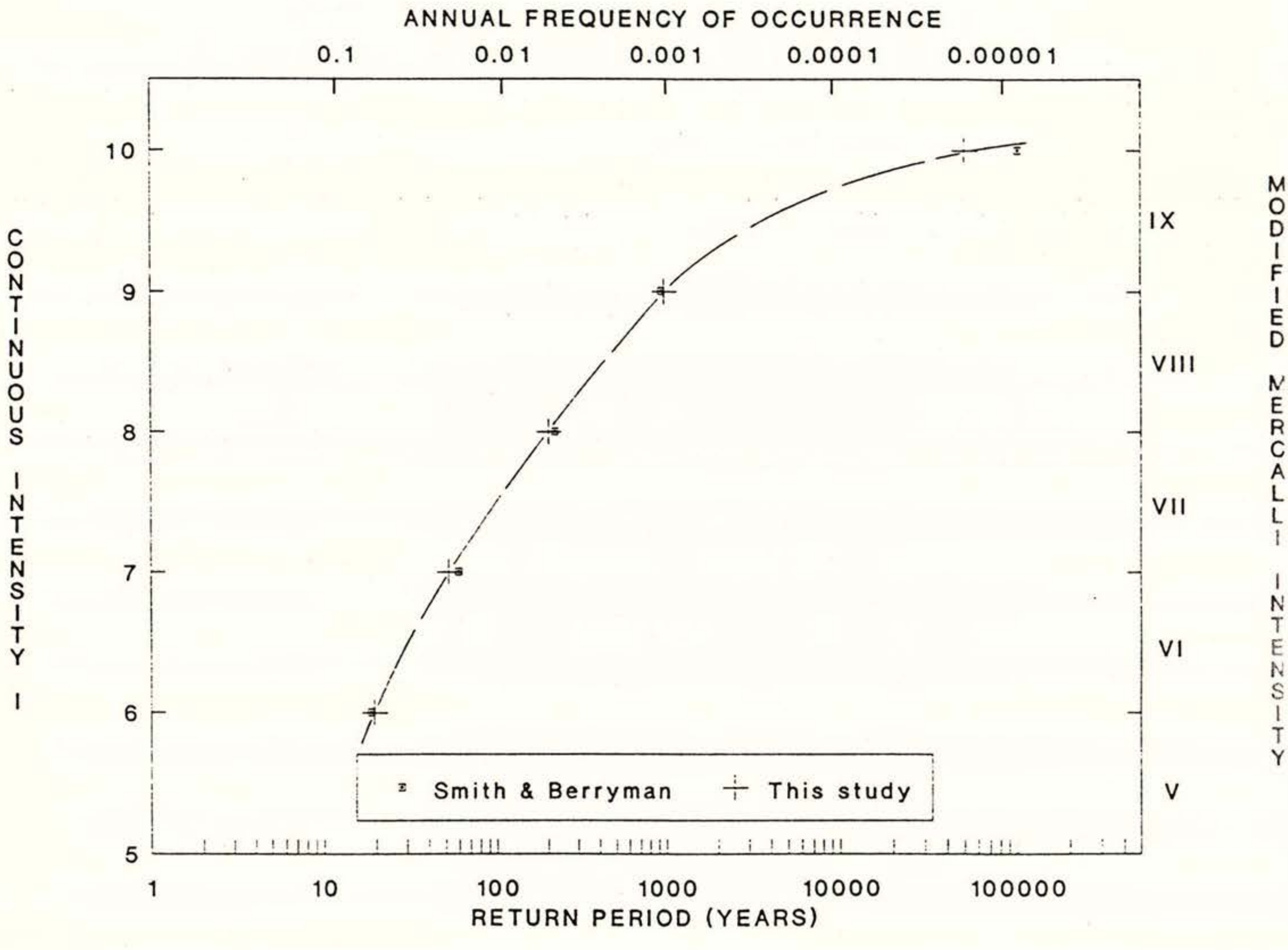



**FIGURE 3.5**

**Occurrence Frequencies for Seismic Intensities at Dunedin (No correction for ground amplification effects)**

It is common to consider the effect of an earthquake with a given, low probability of exceedance in a time comparable to the design life of typical structures. From Table 3.2 the continuous intensity with 10% probability of exceedence in 50 years is about I = 8.5, corresponding to a Modified Mercalli Intensity of VIII.

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 geological conditions. This effect is discussed further in Chapter 5.

## 3.4 SUMMARY

The intensity attenuation model developed by Smith (1978a or b) and refined by Elder et al (1991) has been used with the seismicity model proposed in Chapter 2 to estimate exceedance probabilities for different intensities at Dunedin, for 'average' ground conditions. Site-Specific intensity amplification or reduction is considered in Chapter 5.

The earthquake hazard in Dunedin is dominated by the Fiordland and Otago Regions. The Alpine Fault is at a distance where it contributes little to the hazard. Very high intensities (MMX or greater) will only be generated by a maximum magnitude earthquake occurring on a fault within 10 - 20 km of the city.

The return periods for different intensities of shaking are similar to those previously reported by Smith & Berryman, despite significant differences in the seismicity model used.

# CHAPTER 4: RESPONSE SPECTRA PREDICTION

## 4.1 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 a predicted time history of ground acceleration, velocity and displacement. However the most widespread general methods for structural design, including the current NZ Loadings Code NZS 4203:1992, 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 calculations from available strong motion records. More recent approaches, including that on which the current NZ 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, ground conditions at the site studied, and the period of the fundamental mode,

In this chapter a standard model of this type is used to predict acceleration response spectra for bedrock conditions at Dunedin. 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 2.

## 4.2 MODEL FOR PREDICTION OF RESPONSE SPECTRA

Two spectral acceleration attenuation models were considered in this study. The model that was perceived to most accurately predict the spectral response in Dunedin was used to develop a consistent risk response spectra. The models used were those proposed by Katayama (1982) and Kawashima (1990).

The following sections briefly outline these models, discuss modifications and presents comparisons between the models, and summarise the reasons for adopting the Kawashima Model for use in this study.

Both these models have been developed by Japanese researchers from Japanese data. It is considered that similarities between the tectonic environment of Japan and New Zealand mean that the results are reasonably applicable.

The Katayama model has been studied extensively (Peek 1980, Peek et al 1980, Mulholland 1982, Berrill 1985, McVerry 1986) and modifications have been suggested to tune the model to New Zealand conditions. These are discussed subsequently. The more recent Kawashima model has not been subjected to the same level of review.

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

$$a_s = f_m.f_r.f_{gc} \tag{4.1}$$

| | | |
|---|---|---|
| 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, ie 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, $\sigma_{10}$, of this normal distribution is a function of period, varying in the appropriate range $\sigma_{10}$ = 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.

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 1.445 at T = 0.1 second to 1.0 at T = 0.7 seconds. This modification was made to the Katyama model for comparison with the Kawashima model.

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 (fr). These factors were not incorporated into the model, though the significance of attenuation rate is discussed later.

The Kawashima model predicts acceleration $a_s$, at 5% of critical damping for natural periods T = 0.1 - 3 seconds using the following equation:

$$a_s = a(T_k, GC_i) \times 10^{b\,(Tk,\,GCi)m} \times (\nabla + 30)^{c(Tk,\,GCi)} \tag{4.2}$$

| | | | |
|---|---|---|---|
| Where | a and b | - | are constants which are a function of natural period (Tk) and ground condition (GCi) |
| | c | - | is generally taken as -1.1178 |
| | m | - | magnitude |
| | $\nabla$ | - | epicentral distance (km) |

Kawashima determined and tabulated values for a and b for discrete values of period and for three ground conditions. The scatter in predicted data is also well represented by a log normal distribution with the standard deviation for ground condition 1 (rock) being $\sigma_{10}$ = 0.216.

In developing his model Kawashima modified the original accelerogram data to compensate for the low sensitivity of the Japanese SMAC accelerograph at the high frequency range. Further modification is therefore not required.

The two models described above were used to determine the predicted bedrock response spectra at Dunedin. The predictions were compared and evaluated. This process is discussed in the following sections.

## 4.3 BEDROCK RESPONSE SPECTRA AT DUNEDIN; PROBABILITIES AND RECURRENCE

Probabilities of different spectral response accelerations occurring at Dunedin were determined using a similar analysis to that described in Chapter 3 for intensities. The same set of subregions and nodes were used, together with the seismicity model described in Chapter 2. Only peak spectral accelerations (T = 0.2 for Katyamama and T = 0.15 for Kawashima) were considered since other values scale directly from the peak values as a result of the assumption of uniform spectral shape.

The calculation process used to determine the occurrence probability of a target spectral acceleration due to seismic activity at a particular node was as follows:

- The attenuation equations were rearranged so that for a specified epicentral distance the magnitude required to produce the target spectral acceleration was calculated.

- The seismicity model was used to determine the expected number of such events.

- The total number of events causing the target spectral acceleration were summed over all nodes.

- The percentage contribution to the total probability from each seismicity zone was calculated.

An upper and lower bound limit was placed on magnitude calculated in the first step. The upper bound limit being Mmax and the lower M = 5.25. This value was suggested by Matuschka (1985) as it was considered that structures designed in accordance with current design codes would not be susceptible to damage from the effects of earthquakes with magnitudes less than this.

The spreadsheets developed for the above calculation process are summarised in Table 4.1 (Katyama) and Table 4.2 (Kawashima).

Table 4.3 summarises the percentage contribution to total probability from each seismicity zone for the two attenuation models.

The summary in Table 4.3, reveals that the two models predict markedly different contributions from the various seismicity zone. The Katyama model predicts that with increasing spectral acceleration in Dunedin, the contribution to the total probability from the Fiordland area (Zone C) significantly increases. The Kawashima model however, predicts that activity on the nearby faults (zone M) dominates the expected spectral accelerations in Dunedin. The models also give quite different probabilities for larger accelerations.

TABLE 4.1 **Response Spectra Probabilities Based on Katyama Attenuation Model**

| NODE | ZONE | AREA | Asp/g 0.1 | Asp/g 0.2 | Asp/g 0.3 | Asp/g 0.4 | Asp/g 0.5 | Mmax | b | a4 | N for As/g values of; (excludes probabilistic enhancement) 0.1 | 0.2 | 0.3 | 0.4 | 0.5 | DISTANCE rj |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | M | 0.13 | 5.25 | 5.25 | 6.05 | 6.56 | 6.90 | 7.5 | 0.83 | 0.030 | 0.00035 | 0.00035 | 0.00007 | 0.00002 | 0.00001 | 20 |
| 2 | | 0.13 | 5.25 | 5.25 | 6.05 | 6.56 | 6.90 | 7.5 | 0.83 | 0.030 | 0.00035 | 0.00035 | 0.00007 | 0.00002 | 0.00001 | 20 |
| 3 | | 0.17 | 5.25 | 5.25 | 6.05 | 6.56 | 6.90 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00010 | 0.00003 | 0.00001 | 20 |
| 4 | | 0.17 | 5.25 | 5.25 | 6.05 | 6.56 | 6.90 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00010 | 0.00003 | 0.00001 | 20 |
| 5 | | 0.17 | 5.25 | 5.25 | 6.05 | 6.56 | 6.90 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00010 | 0.00003 | 0.00001 | 20 |
| 6 | | 0.17 | 5.25 | 5.25 | 6.05 | 6.56 | 6.90 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00010 | 0.00003 | 0.00001 | 20 |
| 7 | | 0.17 | 5.25 | 5.25 | 6.05 | 6.56 | 6.90 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00010 | 0.00003 | 0.00001 | 20 |
| 8 | | 0.17 | 5.25 | 5.25 | 6.05 | 6.56 | 6.90 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00010 | 0.00003 | 0.00001 | 20 |
| 9 | | 0.17 | 5.25 | 5.25 | 6.05 | 6.56 | 6.90 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00010 | 0.00003 | 0.00001 | 20 |
| 10 | | 0.17 | 5.25 | 5.25 | 6.05 | 6.56 | 6.90 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00010 | 0.00003 | 0.00001 | 20 |
| 11 | | 0.17 | 5.25 | 5.25 | 6.05 | 6.56 | 6.90 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00010 | 0.00003 | 0.00001 | 20 |
| 12 | | 0.17 | 5.25 | 5.25 | 6.05 | 6.56 | 6.90 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00010 | 0.00003 | 0.00001 | 20 |
| 13 | | 0.17 | 5.25 | 5.25 | 6.05 | 6.56 | 6.90 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00010 | 0.00003 | 0.00001 | 20 |
| 14 | | 0.17 | 5.25 | 5.25 | 6.05 | 6.56 | 6.90 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00010 | 0.00003 | 0.00001 | 20 |
| 15 | | 0.17 | 5.25 | 5.25 | 6.05 | 6.56 | 6.90 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00010 | 0.00003 | 0.00001 | 20 |
| 16 | | 0.12 | 5.25 | 5.25 | 6.05 | 6.56 | 6.90 | 7.5 | 0.83 | 0.030 | 0.00033 | 0.00033 | 0.00007 | 0.00002 | 0.00001 | 20 |
| 17 | | 0.17 | 5.25 | 5.91 | 6.64 | 7.05 | 7.32 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00013 | 0.00003 | 0.00001 | 0.00000 | 35 |
| 18 | | 0.33 | 5.25 | 5.99 | 6.70 | 7.10 | 7.37 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00021 | 0.00004 | 0.00001 | 0.00000 | 37.5 |
| 19 | | 0.33 | 5.25 | 5.99 | 6.70 | 7.10 | 7.37 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00021 | 0.00004 | 0.00001 | 0.00000 | 37.5 |
| 20 | | 0.33 | 5.25 | 5.99 | 6.70 | 7.10 | 7.37 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00021 | 0.00004 | 0.00001 | 0.00000 | 37.5 |
| 21 | | 0.33 | 5.25 | 5.99 | 6.70 | 7.10 | 7.37 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00021 | 0.00004 | 0.00001 | 0.00000 | 37.5 |
| 22 | | 0.33 | 5.25 | 5.99 | 6.70 | 7.10 | 7.37 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00021 | 0.00004 | 0.00001 | 0.00000 | 37.5 |
| 23 | | 0.33 | 5.25 | 5.99 | 6.70 | 7.10 | 7.37 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00021 | 0.00004 | 0.00001 | 0.00000 | 37.5 |
| 24 | | 0.33 | 5.25 | 5.99 | 6.70 | 7.10 | 7.37 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00021 | 0.00004 | 0.00001 | 0.00000 | 37.5 |
| 25 | | 0.33 | 5.25 | 5.99 | 6.70 | 7.10 | 7.37 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00021 | 0.00004 | 0.00001 | 0.00000 | 37.5 |
| 26 | | 0.33 | 5.25 | 5.99 | 6.70 | 7.10 | 7.37 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00021 | 0.00004 | 0.00001 | 0.00000 | 37.5 |
| 27 | | 0.33 | 5.25 | 5.99 | 6.70 | 7.10 | 7.37 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00021 | 0.00004 | 0.00001 | 0.00000 | 37.5 |
| 28 | | 0.24 | 5.25 | 5.99 | 6.70 | 7.10 | 7.37 | 7.5 | 0.83 | 0.030 | 0.00065 | 0.00015 | 0.00003 | 0.00001 | 0.00000 | 37.5 |
| 29 | | 0.68 | 5.28 | 6.66 | 7.21 | 7.52 | 7.73 | 7.5 | 0.83 | 0.030 | 0.00175 | 0.00010 | 0.00002 | | | 70 |
| 30 | | 0.94 | 5.28 | 6.66 | 7.21 | 7.52 | 7.73 | 7.5 | 0.83 | 0.030 | 0.00242 | 0.00014 | 0.00003 | | | 70 |
| 31 | | 0.94 | 5.28 | 6.66 | 7.21 | 7.52 | 7.73 | 7.5 | 0.83 | 0.030 | 0.00242 | 0.00014 | 0.00003 | | | 70 |
| 32 | | 0.94 | 5.28 | 6.66 | 7.21 | 7.52 | 7.73 | 7.5 | 0.83 | 0.030 | 0.00242 | 0.00014 | 0.00003 | | | 70 |
| 33 | | 0.94 | 5.28 | 6.66 | 7.21 | 7.52 | 7.73 | 7.5 | 0.83 | 0.030 | 0.00242 | 0.00014 | 0.00003 | | | 70 |
| 34 | | 0.94 | 5.28 | 6.66 | 7.21 | 7.52 | 7.73 | 7.5 | 0.83 | 0.030 | 0.00242 | 0.00014 | 0.00003 | | | 70 |
| 35 | | 0.94 | 5.28 | 6.66 | 7.21 | 7.52 | 7.73 | 7.5 | 0.83 | 0.030 | 0.00242 | 0.00014 | 0.00003 | | | 70 |
| 36 | | 0.94 | 5.28 | 6.66 | 7.21 | 7.52 | 7.73 | 7.5 | 0.83 | 0.030 | 0.00242 | 0.00014 | 0.00003 | | | 70 |
| 37 | | 0.94 | 5.28 | 6.66 | 7.21 | 7.52 | 7.73 | 7.5 | 0.83 | 0.030 | 0.00242 | 0.00014 | 0.00003 | | | 70 |
| 38 | | 0.90 | 5.23 | 6.63 | 7.19 | 7.51 | 7.71 | 7.5 | 0.83 | 0.030 | 0.00252 | 0.00014 | 0.00003 | | | 68 |
| 39 | | 0.76 | 5.84 | 7.01 | 7.48 | 7.74 | 7.92 | 7.5 | 0.83 | 0.030 | 0.00065 | 0.00004 | 0.00000 | | | 105 |
| 40 | | 1.50 | 5.84 | 7.01 | 7.48 | 7.74 | 7.92 | 7.5 | 0.83 | 0.030 | 0.00129 | 0.00009 | 0.00000 | | | 105 |
| 41 | | 1.50 | 5.84 | 7.01 | 7.48 | 7.74 | 7.92 | 7.5 | 0.83 | 0.030 | 0.00129 | 0.00009 | 0.00000 | | | 105 |
| 42 | | 1.50 | 5.84 | 7.01 | 7.48 | 7.74 | 7.92 | 7.5 | 0.83 | 0.030 | 0.00129 | 0.00009 | 0.00000 | | | 105 |
| 43 | | 1.50 | 5.84 | 7.01 | 7.48 | 7.74 | 7.92 | 7.5 | 0.83 | 0.030 | 0.00129 | 0.00009 | 0.00000 | | | 105 |
| 44 | | 1.50 | 5.84 | 7.01 | 7.48 | 7.74 | 7.92 | 7.5 | 0.83 | 0.030 | 0.00129 | 0.00009 | 0.00000 | | | 105 |
| 45 | | 1.50 | 5.84 | 7.01 | 7.48 | 7.74 | 7.92 | 7.5 | 0.83 | 0.030 | 0.00129 | 0.00009 | 0.00000 | | | 105 |
| 46 | | 1.50 | 5.84 | 7.01 | 7.48 | 7.74 | 7.92 | 7.5 | 0.83 | 0.030 | 0.00129 | 0.00009 | 0.00000 | | | 105 |
| 47 | | 2.56 | 5.89 | 7.04 | 7.51 | 7.77 | 7.94 | 7.5 | 0.83 | 0.030 | 0.00196 | 0.00013 | | | | 110 |
| 48 | | 1.37 | 5.92 | 7.06 | 7.52 | 7.78 | 7.95 | 7.5 | 0.83 | 0.030 | 0.00100 | 0.00007 | | | | 112 |
| 49 | | 3.64 | 6.22 | 7.25 | 7.66 | 7.90 | 8.05 | 7.5 | 0.83 | 0.030 | 0.00144 | 0.00008 | | | | 145 |
| 50 | | 1.67 | 6.16 | 7.21 | 7.63 | 7.87 | 8.03 | 7.5 | 0.83 | 0.030 | 0.00075 | 0.00005 | | | | 137.5 |
| 51 | | 1.67 | 6.16 | 7.21 | 7.63 | 7.87 | 8.03 | 7.5 | 0.83 | 0.030 | 0.00075 | 0.00005 | | | | 137.5 |
| 52 | | 1.67 | 6.16 | 7.21 | 7.63 | 7.87 | 8.03 | 7.5 | 0.83 | 0.030 | 0.00075 | 0.00005 | | | | 137.5 |
| 53 | | 1.67 | 6.16 | 7.21 | 7.63 | 7.87 | 8.03 | 7.5 | 0.83 | 0.030 | 0.00075 | 0.00005 | | | | 137.5 |
| 54 | | 1.67 | 6.16 | 7.21 | 7.63 | 7.87 | 8.03 | 7.5 | 0.83 | 0.030 | 0.00075 | 0.00005 | | | | 137.5 |
| 55 | | 1.67 | 6.16 | 7.21 | 7.63 | 7.87 | 8.03 | 7.5 | 0.83 | 0.030 | 0.00075 | 0.00005 | | | | 137.5 |
| 56 | | 4.32 | 6.45 | 7.39 | 7.77 | 7.99 | 8.13 | 7.5 | 0.83 | 0.030 | 0.00104 | 0.00004 | | | | 180 |
| 57 | | 2.29 | 6.45 | 7.39 | 7.77 | 7.99 | 8.13 | 7.5 | 0.83 | 0.030 | 0.00055 | 0.00002 | | | | 180 |
| 58 | | 2.31 | 6.39 | 7.36 | 7.75 | 7.96 | 8.11 | 7.5 | 0.83 | 0.030 | 0.00063 | 0.00003 | | | | 170 |
| 59 | | 3.43 | 6.39 | 7.36 | 7.75 | 7.96 | 8.11 | 7.5 | 0.83 | 0.030 | 0.00094 | 0.00004 | | | | 170 |
| 60 | | 4.88 | 6.45 | 7.39 | 7.77 | 7.99 | 8.13 | 7.5 | 0.83 | 0.030 | 0.00117 | 0.00004 | | | | 180 |
| 61 | | 1.21 | 6.39 | 7.36 | 7.75 | 7.96 | 8.11 | 7.5 | 0.83 | 0.030 | 0.00033 | 0.00001 | | | | 170 |
| 62 | O | 9.72 | 6.65 | 7.52 | 7.87 | 8.07 | 8.20 | 8 | 1.077 | 0.041 | 0.00054 | 0.00005 | 0.00001 | | | 220 |
| 63 | | 9.58 | 6.97 | 7.72 | 8.03 | 8.20 | 8.31 | 8 | 1.077 | 0.041 | 0.00023 | 0.00002 | | | | 320 |
| 64 | N | 31.93 | 7.07 | 7.78 | 8.07 | 8.23 | 8.34 | 8.5 | 1 | 0.060 | 0.00158 | 0.00026 | 0.00010 | 0.00005 | 0.00003 | 360 |
| 65 | L | 7.25 | 6.83 | 7.63 | 7.96 | 8.14 | 8.26 | 8.5 | 1 | 0.750 | 0.00784 | 0.00109 | 0.00043 | 0.00022 | 0.00013 | 270 |
| 66 | | 6.52 | 6.76 | 7.59 | 7.92 | 8.11 | 8.23 | 8.5 | 1 | 0.750 | 0.00841 | 0.00111 | 0.00043 | 0.00022 | 0.00013 | 248 |
| 67 | | 5.93 | 6.71 | 7.56 | 7.90 | 8.09 | 8.22 | 8.5 | 1 | 0.750 | 0.00858 | 0.00109 | 0.00042 | 0.00022 | 0.00013 | 235 |
| 68 | | 5.65 | 6.73 | 7.57 | 7.91 | 8.10 | 8.22 | 8.5 | 1 | 0.750 | 0.00781 | 0.00101 | 0.00039 | 0.00020 | 0.00012 | 240 |
| 69 | H | 5.33 | 6.74 | 7.58 | 7.91 | 8.10 | 8.23 | 8.5 | 1.05 | 0.200 | 0.00140 | 0.00017 | 0.00006 | 0.00003 | 0.00002 | 243 |
| 70 | | 5.88 | 6.85 | 7.64 | 7.97 | 8.15 | 8.26 | 8.5 | 1.05 | 0.200 | 0.00118 | 0.00015 | 0.00006 | 0.00003 | 0.00002 | 275 |
| 75 | | 6.29 | 6.99 | 7.74 | 8.03 | 8.20 | 8.31 | 8.5 | 1.05 | 0.200 | 0.00089 | 0.00013 | 0.00005 | 0.00002 | 0.00001 | 327 |
| 76 | | 6.88 | 7.13 | 7.82 | 8.10 | 8.26 | 8.36 | 8.5 | 1.05 | 0.200 | 0.00069 | 0.00011 | 0.00004 | 0.00002 | 0.00001 | 390 |
| 71 | CBsw | 3.32 | 6.70 | 7.55 | 7.89 | 8.09 | 8.21 | 8 | 1 | 0.11 | 0.00070 | 0.00007 | 0.00001 | | | 232 |
| 72 | | 3.82 | 6.73 | 7.57 | 7.91 | 8.10 | 8.22 | 8 | 1 | 0.11 | 0.00075 | 0.00007 | 0.00001 | | | 240 |
| 73 | CBse | 5.05 | 6.55 | 7.45 | 7.82 | 8.03 | 8.16 | 8 | 1.1 | 0.03 | 0.00023 | 0.00002 | 0.00000 | | | 198 |
| 74 | | 4.45 | 6.48 | 7.41 | 7.79 | 8.00 | 8.14 | 8 | 1.1 | 0.03 | 0.00025 | 0.00002 | 0.00000 | 0.00000 | | 185 |
| 77 | HF | 2.5 | 7.11 | 7.81 | 8.09 | 8.25 | 8.35 | 7.8 | 0.5 | 0.19 | 0.00727 | | | | | 380 |
| 78 | CNnw | 3.05 | 7.03 | 7.76 | 8.06 | 8.22 | 8.33 | 7.5 | 0.6 | 0.3 | 0.00656 | | | | | 345 |
| 79 | PPT | 5.09 | 6.96 | 7.72 | 8.02 | 8.19 | 8.30 | 7.5 | 0.8 | 0.3 | 0.00411 | | | | | 315 |
| 80 | CPS | 4.07 | 6.89 | 7.67 | 7.99 | 8.16 | 8.28 | 7.5 | 0.8 | 0.06 | 0.00080 | | | | | 290 |
| 81 | BPS | 13.19 | 6.95 | 7.71 | 8.01 | 8.19 | 8.30 | 7.5 | 0.6 | 0.015 | 0.00180 | | | | | 310 |
| 82 | CBne | 2.61 | 7.13 | 7.82 | 8.10 | 8.26 | 8.36 | 8 | 0.5 | 0.15 | 0.00676 | 0.00089 | | | | 390 |
| 83 | PGS | 4.03 | 7.10 | 7.80 | 8.09 | 8.25 | 8.35 | 7.5 | 0.6 | 0.08 | 0.00188 | | | | | 376 |
| | | | | | | | | | | Totals | 0.134186 | 0.018412 | 0.004245 | 0.001671 | 0.000843 | |

## TABLE 4.2 Response Spectral Probabilities Based on Kawashima Attenuation Model

| NODE | ZONE | AREA | Asp/g 0.1 | Asp/g 0.2 | Asp/g 0.3 | Asp/g 0.4 | Asp/g 0.5 | Mmax | b | a4 | N for As/g values of; (excludes probabilistic enhancement) 0.1 | 0.2 | 0.3 | 0.4 | 0.5 | DISTANCE rj |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | M | 0.13 | 5.25 | 5.25 | 5.25 | 5.62 | 6.07 | 7.5 | 0.83 | 0.030 | 0.00035 | 0.00035 | 0.00035 | 0.00017 | 0.00007 | 20 |
| 2 | | 0.13 | 5.25 | 5.25 | 5.25 | 5.62 | 6.07 | 7.5 | 0.83 | 0.030 | 0.00035 | 0.00035 | 0.00035 | 0.00017 | 0.00007 | 20 |
| 3 | | 0.17 | 5.25 | 5.25 | 5.25 | 5.62 | 6.07 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00046 | 0.00022 | 0.00009 | 20 |
| 4 | | 0.17 | 5.25 | 5.25 | 5.25 | 5.62 | 6.07 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00046 | 0.00022 | 0.00009 | 20 |
| 5 | | 0.17 | 5.25 | 5.25 | 5.25 | 5.62 | 6.07 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00046 | 0.00022 | 0.00009 | 20 |
| 6 | | 0.17 | 5.25 | 5.25 | 5.25 | 5.62 | 6.07 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00046 | 0.00022 | 0.00009 | 20 |
| 7 | | 0.17 | 5.25 | 5.25 | 5.25 | 5.62 | 6.07 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00046 | 0.00022 | 0.00009 | 20 |
| 8 | | 0.17 | 5.25 | 5.25 | 5.25 | 5.62 | 6.07 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00046 | 0.00022 | 0.00009 | 20 |
| 9 | | 0.17 | 5.25 | 5.25 | 5.25 | 5.62 | 6.07 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00046 | 0.00022 | 0.00009 | 20 |
| 10 | | 0.17 | 5.25 | 5.25 | 5.25 | 5.62 | 6.07 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00046 | 0.00022 | 0.00009 | 20 |
| 11 | | 0.17 | 5.25 | 5.25 | 5.25 | 5.62 | 6.07 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00046 | 0.00022 | 0.00009 | 20 |
| 12 | | 0.17 | 5.25 | 5.25 | 5.25 | 5.62 | 6.07 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00046 | 0.00022 | 0.00009 | 20 |
| 13 | | 0.17 | 5.25 | 5.25 | 5.25 | 5.62 | 6.07 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00046 | 0.00022 | 0.00009 | 20 |
| 14 | | 0.17 | 5.25 | 5.25 | 5.25 | 5.62 | 6.07 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00046 | 0.00022 | 0.00009 | 20 |
| 15 | | 0.17 | 5.25 | 5.25 | 5.25 | 5.62 | 6.07 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00046 | 0.00022 | 0.00009 | 20 |
| 16 | | 0.12 | 5.25 | 5.25 | 5.25 | 5.62 | 6.07 | 7.5 | 0.83 | 0.030 | 0.00033 | 0.00033 | 0.00033 | 0.00016 | 0.00006 | 20 |
| 17 | | 0.17 | 5.25 | 5.25 | 5.66 | 6.24 | 6.69 | 7.5 | 0.83 | 0.030 | 0.00046 | 0.00046 | 0.00021 | 0.00006 | 0.00002 | 35 |
| 18 | | 0.33 | 5.25 | 5.25 | 5.75 | 6.33 | 6.78 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00090 | 0.00034 | 0.00010 | 0.00004 | 37.5 |
| 19 | | 0.33 | 5.25 | 5.25 | 5.75 | 6.33 | 6.78 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00090 | 0.00034 | 0.00010 | 0.00004 | 37.5 |
| 20 | | 0.33 | 5.25 | 5.25 | 5.75 | 6.33 | 6.78 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00090 | 0.00034 | 0.00010 | 0.00004 | 37.5 |
| 21 | | 0.33 | 5.25 | 5.25 | 5.75 | 6.33 | 6.78 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00090 | 0.00034 | 0.00010 | 0.00004 | 37.5 |
| 22 | | 0.33 | 5.25 | 5.25 | 5.75 | 6.33 | 6.78 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00090 | 0.00034 | 0.00010 | 0.00004 | 37.5 |
| 23 | | 0.33 | 5.25 | 5.25 | 5.75 | 6.33 | 6.78 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00090 | 0.00034 | 0.00010 | 0.00004 | 37.5 |
| 24 | | 0.33 | 5.25 | 5.25 | 5.75 | 6.33 | 6.78 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00090 | 0.00034 | 0.00010 | 0.00004 | 37.5 |
| 25 | | 0.33 | 5.25 | 5.25 | 5.75 | 6.33 | 6.78 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00090 | 0.00034 | 0.00010 | 0.00004 | 37.5 |
| 26 | | 0.33 | 5.25 | 5.25 | 5.75 | 6.33 | 6.78 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00090 | 0.00034 | 0.00010 | 0.00004 | 37.5 |
| 27 | | 0.33 | 5.25 | 5.25 | 5.75 | 6.33 | 6.78 | 7.5 | 0.83 | 0.030 | 0.00090 | 0.00090 | 0.00034 | 0.00010 | 0.00004 | 37.5 |
| 28 | | 0.24 | 5.25 | 5.25 | 5.75 | 6.33 | 6.78 | 7.5 | 0.83 | 0.030 | 0.00065 | 0.00065 | 0.00024 | 0.00007 | 0.00003 | 37.5 |
| 29 | | 0.68 | 5.25 | 5.87 | 6.68 | 7.26 | 7.71 | 7.5 | 0.83 | 0.030 | 0.00185 | 0.00055 | 0.00010 | 0.00001 | | 70 |
| 30 | | 0.94 | 5.25 | 5.87 | 6.68 | 7.26 | 7.71 | 7.5 | 0.83 | 0.030 | 0.00255 | 0.00076 | 0.00013 | 0.00002 | | 70 |
| 31 | | 0.94 | 5.25 | 5.87 | 6.68 | 7.26 | 7.71 | 7.5 | 0.83 | 0.030 | 0.00255 | 0.00076 | 0.00013 | 0.00002 | | 70 |
| 32 | | 0.94 | 5.25 | 5.87 | 6.68 | 7.26 | 7.71 | 7.5 | 0.83 | 0.030 | 0.00255 | 0.00076 | 0.00013 | 0.00002 | | 70 |
| 33 | | 0.94 | 5.25 | 5.87 | 6.68 | 7.26 | 7.71 | 7.5 | 0.83 | 0.030 | 0.00255 | 0.00076 | 0.00013 | 0.00002 | | 70 |
| 34 | | 0.94 | 5.25 | 5.87 | 6.68 | 7.26 | 7.71 | 7.5 | 0.83 | 0.030 | 0.00255 | 0.00076 | 0.00013 | 0.00002 | | 70 |
| 35 | | 0.94 | 5.25 | 5.87 | 6.68 | 7.26 | 7.71 | 7.5 | 0.83 | 0.030 | 0.00255 | 0.00076 | 0.00013 | 0.00002 | | 70 |
| 36 | | 0.94 | 5.25 | 5.87 | 6.68 | 7.26 | 7.71 | 7.5 | 0.83 | 0.030 | 0.00255 | 0.00076 | 0.00013 | 0.00002 | | 70 |
| 37 | | 0.94 | 5.25 | 5.87 | 6.68 | 7.26 | 7.71 | 7.5 | 0.83 | 0.030 | 0.00255 | 0.00076 | 0.00013 | 0.00002 | | 70 |
| 38 | | 0.90 | 5.25 | 5.82 | 6.63 | 7.21 | 7.66 | 7.5 | 0.83 | 0.030 | 0.00244 | 0.00080 | 0.00014 | 0.00002 | | 68 |
| 39 | | 0.76 | 5.25 | 6.58 | 7.39 | 7.97 | 8.42 | 7.5 | 0.83 | 0.030 | 0.00206 | 0.00014 | 0.00001 | | | 105 |
| 40 | | 1.50 | 5.25 | 6.58 | 7.39 | 7.97 | 8.42 | 7.5 | 0.83 | 0.030 | 0.00407 | 0.00027 | 0.00001 | | | 105 |
| 41 | | 1.50 | 5.25 | 6.58 | 7.39 | 7.97 | 8.42 | 7.5 | 0.83 | 0.030 | 0.00407 | 0.00027 | 0.00001 | | | 105 |
| 42 | | 1.50 | 5.25 | 6.58 | 7.39 | 7.97 | 8.42 | 7.5 | 0.83 | 0.030 | 0.00407 | 0.00027 | 0.00001 | | | 105 |
| 43 | | 1.50 | 5.25 | 6.58 | 7.39 | 7.97 | 8.42 | 7.5 | 0.83 | 0.030 | 0.00407 | 0.00027 | 0.00001 | | | 105 |
| 44 | | 1.50 | 5.25 | 6.58 | 7.39 | 7.97 | 8.42 | 7.5 | 0.83 | 0.030 | 0.00407 | 0.00027 | 0.00001 | | | 105 |
| 45 | | 1.50 | 5.25 | 6.58 | 7.39 | 7.97 | 8.42 | 7.5 | 0.83 | 0.030 | 0.00407 | 0.00027 | 0.00001 | | | 105 |
| 46 | | 1.50 | 5.25 | 6.58 | 7.39 | 7.97 | 8.42 | 7.5 | 0.83 | 0.030 | 0.00407 | 0.00027 | 0.00001 | | | 105 |
| 47 | | 2.56 | 5.27 | 6.66 | 7.48 | 8.06 | 8.51 | 7.5 | 0.83 | 0.030 | 0.00668 | 0.00038 | 0.00000 | | | 110 |
| 48 | | 1.37 | 5.30 | 6.70 | 7.51 | 8.09 | 8.54 | 7.5 | 0.83 | 0.030 | 0.00335 | 0.00019 | | | | 112 |
| 49 | | 3.64 | 5.80 | 7.19 | 8.01 | 8.59 | 9.03 | 7.5 | 0.83 | 0.030 | 0.00337 | 0.00011 | | | | 145 |
| 50 | | 1.67 | 5.69 | 7.09 | 7.90 | 8.48 | 8.93 | 7.5 | 0.83 | 0.030 | 0.00190 | 0.00007 | | | | 137.5 |
| 51 | | 1.67 | 5.69 | 7.09 | 7.90 | 8.48 | 8.93 | 7.5 | 0.83 | 0.030 | 0.00190 | 0.00007 | | | | 137.5 |
| 52 | | 1.67 | 5.69 | 7.09 | 7.90 | 8.48 | 8.93 | 7.5 | 0.83 | 0.030 | 0.00190 | 0.00007 | | | | 137.5 |
| 53 | | 1.67 | 5.69 | 7.09 | 7.90 | 8.48 | 8.93 | 7.5 | 0.83 | 0.030 | 0.00190 | 0.00007 | | | | 137.5 |
| 54 | | 1.67 | 5.69 | 7.09 | 7.90 | 8.48 | 8.93 | 7.5 | 0.83 | 0.030 | 0.00190 | 0.00007 | | | | 137.5 |
| 55 | | 1.67 | 5.69 | 7.09 | 7.90 | 8.48 | 8.93 | 7.5 | 0.83 | 0.030 | 0.00190 | 0.00007 | | | | 137.5 |
| 56 | | 4.32 | 6.23 | 7.62 | 8.44 | 9.02 | 9.47 | 7.5 | 0.83 | 0.030 | 0.00166 | | | | | 180 |
| 57 | | 2.29 | 6.23 | 7.62 | 8.44 | 9.02 | 9.47 | 7.5 | 0.83 | 0.030 | 0.00088 | | | | | 180 |
| 58 | | 2.31 | 6.11 | 7.51 | 8.32 | 8.90 | 9.35 | 7.5 | 0.83 | 0.030 | 0.00113 | | | | | 170 |
| 59 | | 3.43 | 6.11 | 7.51 | 8.32 | 8.90 | 9.35 | 7.5 | 0.83 | 0.030 | 0.00168 | | | | | 170 |
| 60 | | 4.88 | 6.23 | 7.62 | 8.44 | 9.02 | 9.47 | 7.5 | 0.83 | 0.030 | 0.00188 | | | | | 180 |
| 61 | | 1.21 | 6.11 | 7.51 | 8.32 | 8.90 | 9.35 | 7.5 | 0.83 | 0.030 | 0.00059 | | | | | 170 |
| 62 | O | 9.72 | 6.64 | 8.04 | 8.85 | 9.43 | 9.88 | 8 | 1.077 | 0.041 | 0.00055 | | | | | 220 |
| 63 | | 9.58 | 7.44 | 8.83 | 9.65 | 10.23 | 10.68 | 8 | 1.077 | 0.041 | 0.00006 | | | | | 320 |
| 64 | N | 31.93 | 7.70 | 9.09 | 9.91 | 10.48 | 10.93 | 8.5 | 1 | 0.060 | 0.00032 | | | | | 360 |
| 65 | L | 7.25 | 7.08 | 8.47 | 9.28 | 9.86 | 10.31 | 8.5 | 1 | 0.750 | 0.00440 | 0.00001 | | | | 270 |
| 66 | | 6.52 | 6.89 | 8.29 | 9.10 | 9.68 | 10.13 | 8.5 | 1 | 0.750 | 0.00608 | 0.00010 | | | | 248 |
| 67 | | 5.93 | 6.78 | 8.18 | 8.99 | 9.57 | 10.02 | 8.5 | 1 | 0.750 | 0.00722 | 0.00016 | | | | 235 |
| 68 | | 5.65 | 6.83 | 8.22 | 9.03 | 9.61 | 10.06 | 8.5 | 1 | 0.750 | 0.00620 | 0.00012 | | | | 240 |
| 69 | H | 5.33 | 6.85 | 8.25 | 9.06 | 9.64 | 10.09 | 8.5 | 1.05 | 0.200 | 0.00106 | 0.00002 | | | | 243 |
| 70 | | 5.88 | 7.11 | 8.51 | 9.32 | 9.90 | 10.35 | 8.5 | 1.05 | 0.200 | 0.00061 | | | | | 275 |
| 75 | | 6.29 | 7.49 | 8.88 | 9.70 | 10.27 | 10.72 | 8.5 | 1.05 | 0.200 | 0.00025 | | | | | 327 |
| 76 | | 6.88 | 7.87 | 9.27 | 10.08 | 10.66 | 11.11 | 8.5 | 1.05 | 0.200 | 0.00009 | | | | | 390 |
| 71 | CBsw | 3.32 | 6.75 | 8.15 | 8.96 | 9.54 | 9.99 | 8 | 1 | 0.11 | 0.00061 | | | | | 232 |
| 72 | | 3.82 | 6.83 | 8.22 | 9.03 | 9.61 | 10.06 | 8 | 1 | 0.11 | 0.00059 | | | | | 240 |
| 73 | CBse | 5.05 | 6.43 | 7.82 | 8.63 | 9.21 | 9.66 | 8 | 1.1 | 0.03 | 0.00032 | 0.00000 | | | | 198 |
| 74 | | 4.45 | 6.29 | 7.68 | 8.50 | 9.07 | 9.52 | 8 | 1.1 | 0.03 | 0.00040 | 0.00001 | | | | 185 |
| 77 | HP | 2.5 | 7.82 | 9.21 | 10.02 | 10.60 | 11.05 | 7.8 | 0.5 | 0.19 | | | | | | 380 |
| 78 | CNnw | 3.05 | 7.60 | 9.00 | 9.81 | 10.39 | 10.84 | 7.5 | 0.6 | 0.3 | | | | | | 345 |
| 79 | PPT | 5.09 | 7.41 | 8.80 | 9.62 | 10.19 | 10.64 | 7.5 | 0.8 | 0.3 | 0.00046 | | | | | 315 |
| 80 | CPS | 4.07 | 7.23 | 8.62 | 9.44 | 10.02 | 10.46 | 7.5 | 0.8 | 0.06 | 0.00025 | | | | | 290 |
| 81 | BPS | 13.19 | 7.37 | 8.77 | 9.58 | 10.16 | 10.61 | 7.5 | 0.6 | 0.015 | 0.00030 | | | | | 310 |
| 82 | CBne | 2.61 | 7.87 | 9.27 | 10.08 | 10.66 | 11.11 | 8 | 0.5 | 0.15 | 0.00062 | | | | | 390 |
| 83 | PGS | 4.03 | 7.79 | 9.19 | 10.00 | 10.58 | 11.03 | 7.5 | 0.6 | 0.08 | | | | | | 376 |
| | | | | | | | | | | Totals | 0.135394 | 0.028103 | 0.012241 | 0.004798 | 0.001814 | |

**TABLE 4.3 Relative Contributions to Spectral Acceleration Probabilities from each seismicity zone by Kawashima and Katyama Attenuation Models**

**(a) Kawashima Model**

| SEISMICITY MODEL | AS/G AT DUNEDIN 0.1 | 0.2 | 0.3 | 0.4 | 0.5 |
|---|---|---|---|---|---|
| M | 77.6 | 98.5 | 100 | 100 | 100 |
| O | 0.5 | - | - | - | - |
| N | 0.2 | - | - | - | - |
| L | 17.6 | 1.4 | - | - | - |
| H | 1.5 | 0.1 | - | - | - |
| CBsw | 0.9 | - | - | - | |
| CBse | 0.5 | - | - | - | - |
| BPS | 0.2 | - | - | - | - |
| CPS | 0.2 | - | - | - | - |
| PPT | 0.3 | - | - | - | - |
| PGS | - | - | - | - | - |
| CBnw | - | - | - | - | - |
| CBne | 0.5 | - | - | - | - |
| HF | - | - | - | - | - |
| TOTALS | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 |
| Annual Exceedance | 0.135 | 0.0281 | 0.0122 | 0.0048 | 0.0018 |
| Return Period (yrs) | 7.4 | 35 | 80 | 210 | 550 |

**(b) Katyama Model**

| SEISMICITY MODEL | AS/G AT DUNEDIN 0.1 | 0.2 | 0.3 | 0.4 | 0.5 |
|---|---|---|---|---|---|
| M | 47.6 | 66.0 | 52.4 | 38.3 | 29.5 |
| O | 0.6 | 0.3 | 0.2 | - | - |
| N | 1.2 | 1.4 | 2.4 | 3.1 | 3.2 |
| L | 24.3 | 23.4 | 39.3 | 52.1 | 60.2 |
| H | 3.1 | 3.0 | 5.0 | 6.5 | 7.1 |
| CBsw | 1.1 | 0.8 | 0.5 | - | - |
| CBse | 0.4 | 0.2 | 0.2 | - | - |
| BPS | 1.3 | - | - | - | - |
| CPS | 0.6 | - | - | - | - |
| PPT | 3.1 | - | - | - | - |
| PGS | 1.4 | - | - | - | - |
| CBnw | 4.9 | - | - | - | - |
| CBne | 5.0 | 4.9 | - | - | - |
| HF | 5.4 | | | | |
| TOTALS | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 |
| Annual Exceedance | 0.134 | 0.0184 | 0.0042 | 0.0017 | 0.0008 |
| Return Period (yrs) | 7.5 | 55 | 235 | 600 | 1,200 |

The apparent large differences in the relative zone contributions and the probabilities of the two models is a reflection in the different attenuation rates embodied within the models. This can be illustrated by way of Figure 4.1. Here combinations of magnitude and distance to produce a specific spectral accelerations are plotted for the two models. The curves for the Katyama are very flat implying very little attenuation with distance. As mentioned previously McVerry (1986) argued that the Katyama model underestimates the rate of attenuation in New Zealand. He also reports that the rate of attenuation in the Fiordland area is greater than any other seismicity zone in New Zealand. He predicts that at an epicentral distance of 100 km and for a magnitude of M = 6.5, the spectral acceleration in Fiordland would be half those expected elsewhere in New Zealand. With the Fiordland area being between 200 and 300 km from Dunedin, and with an expected large attenuation with distance, the contribution to the total probability from the Fiordland zone is expected to be low (as predicted by the Kawashima model).

The Kawashima model shows less attenuation with distance for earthquakes within 70 - 150 km of the site than the Katyama model. This increases comparatively the effect of the near field seismicity, particularly of smaller (and more frequent) earthquakes, hence the overall probability of a given acceleration is increased.

The Kawashima model was selected for use in this study for the following reasons:

- The rates of attenuation with distance are believed to more closely resemble those expected in the lower South Island.

- This model has been developed from a larger data base (394 records compared with 100 for the Katyama model).

- The spread of data about the predicted mean is smaller (as measured by the standard deviation).

The probabilities of occurrence of different spectral accelerations as estimated by the Kawashima model are shown in Table 4.4. Inclusion of the probabilistic enhancement factor increases the annual exceedance as shown in the lower part of the Table (explanation of the probabilistic enhancement factor is included in the next section). This information is shown graphically in Figure 4.2. Also on Figure 4.2 is the peak spectral acceleration occurrence predicted by Matushka et al (1985). This study predicts shorter return periods for low accelerations and longer return periods for high accelerations than the earlier study.

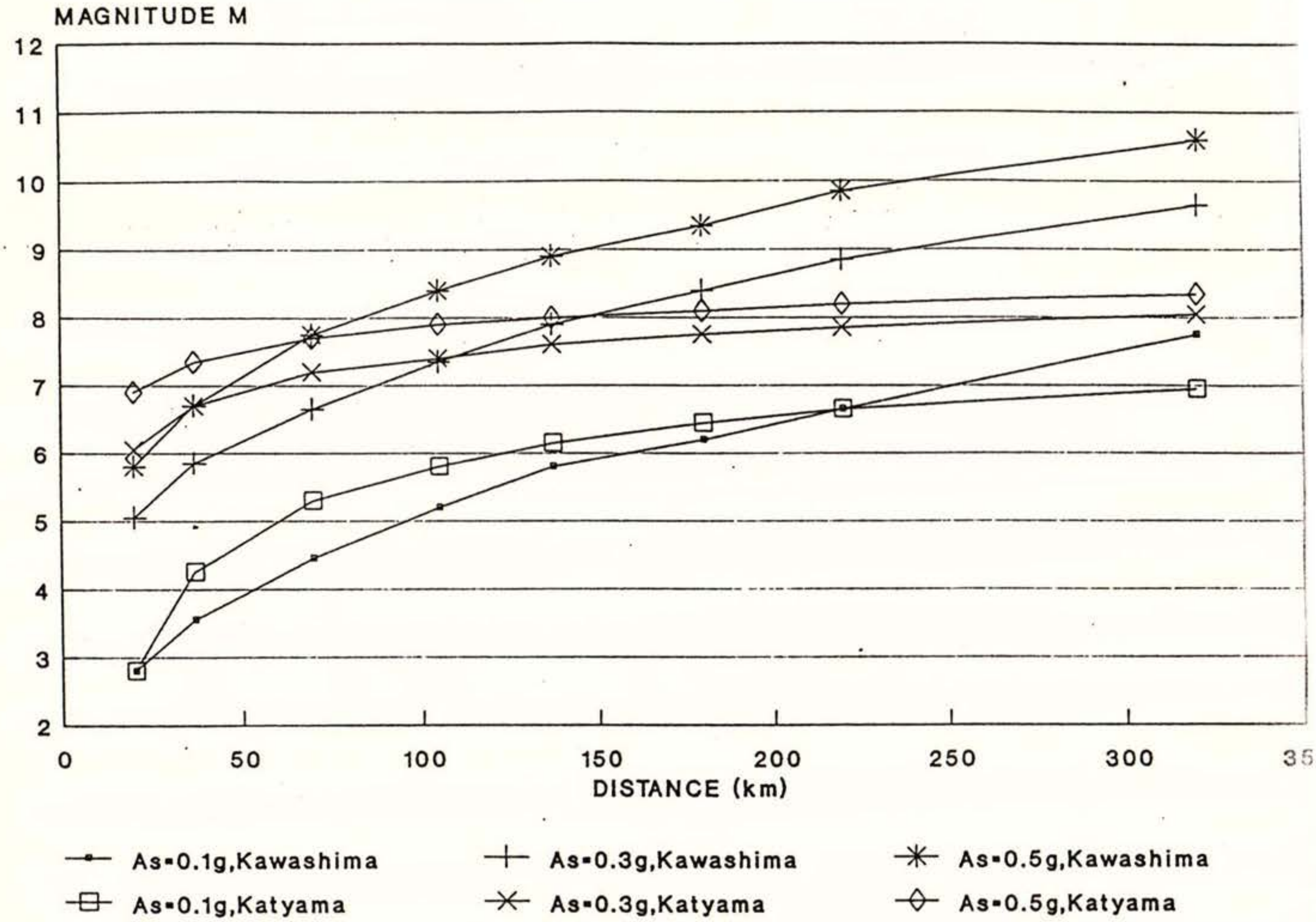



**FIGURE 4.1**

**Comparison of Katyama and Kawashima Attenuation Models for 3 Peak Spectral Accelerations at Dunedin**

## TABLE 4.4 Probabilities of Occurrence of Different Spectral Accelerations at Dunedin and Relative Contributions form each Seismicity Region

Percentage Contribution to Total Probability from each Seismicity Zone

| SEISMICITY | INTENSITY AT DUNEDIN | | | | |
|---|---|---|---|---|---|
| REGION | 0.1 | 0.2 | 0.3 | 0.4 | 0.5 |
| M | 77.6 | 98.5 | 100 | 100 | 100 |
| O | 0.5 | - | - | - | - |
| N | 0.2 | - | - | - | - |
| L | 17.6 | 1.4 | - | - | - |
| H | 1.5 | 0.1 | - | - | - |
| CBsw | 0.9 | - | - | - | - |
| CBse | 0.5 | - | - | - | - |
| BPS | 0.2 | - | - | - | - |
| CPS | 0.2 | - | - | - | - |
| PPT | 0.3 | - | - | - | - |
| PGS | - | - | - | - | - |
| CBnw | - | - | - | - | - |
| CBne | 0.5 | - | - | - | - |
| HF | - | - | - | - | - |
| TOTALS | 100.00 | 100.00 | 100.00 | 100.00 | 100.00 |

**(a) Probability for 50% confidence that accelerations less than that shown**

| | | | | | |
|---|---|---|---|---|---|
| Annual Exceedance $N_1$ | 0.135 | 0.0281 | 0.0122 | 0.0048 | 0.0018 |
| Return Period (yrs) | 7.4 | 35 | 80 | 210 | 550 |

**(b) Probability for 84% confidence that Accelerations less than that shown**

| | | | | | |
|---|---|---|---|---|---|
| Annual Exceedance | 0.756 | 0.130 | 0.0548 | 0.0213 | 0.0080 |
| Return Period (yrs) | 1.3 | 7.7 | 18 | 46 | 125 |

| Probability (%) of Exceedance in: | | | | | |
|---|---|---|---|---|---|
| 50 years | 100.0 | 99.9 | 93.8 | 66.3 | 33.0 |
| 150 years | 100.0 | 100.0 | 100.0 | 96.2 | 70.0 |
| 450 years | 100.0 | 100.0 | 100.0 | 100.0 | 97.3 |
| 1000 years | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 |

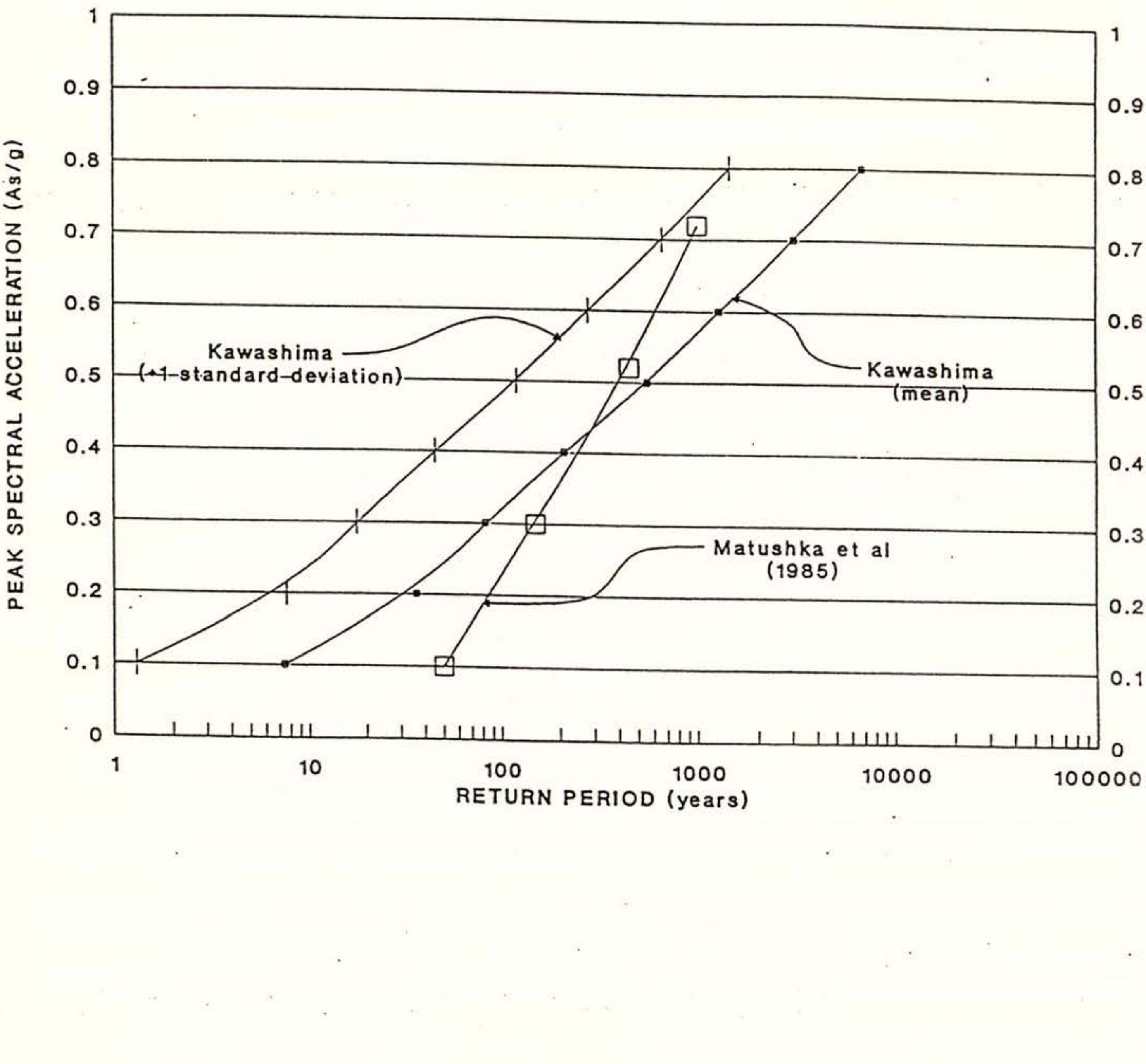



**FIGURE 4.2**

**Annual Occurrence Frequencies and Return Periods for Bedrock Peak Spectral Accelerations**

## 4.4 UNIFORM RISK RESPONSE SPECTRA

A response spectra was developed for the 450 year return period event at Dunedin by using the spreadsheet illustrated in Table 4.2. The target spectral acceleration was varied until the computed probability equalled 0.0022.

This provides one point on the response spectra for a fundamental period of 0.15 seconds. The accelerations for other periods were then calculated by scaling, and assuming that the shape of the response spectra was similar to that derived by Kawashima for a magnitude 7 earthquake at a epicentral distance of 40 km. The validity of this assumption was evaluated by developing a spreadsheet for a period of 0.5 seconds. The spectral acceleration calculated via this approach was found to be within 10% of that developed from the scaled spectra.

The response spectra developed by the above approach provides what can be considered the average 450 year return period response spectra. In probabilistic terms, it is equally likely that the spectral accelerations will be greater or lower than those calculated. To place some probability boundaries on the spectra, it is necessary to consider the scatter of the data around the predicted attenuation model.

McVerry (1986) discussed the issue of probabilistic enhancement and development the enhancement factor of exp $[2.3^2 b\ \sigma^2/2c]$ where $\sigma$ is the standard deviation and c is typically 0.25.

The 450 year return period response spectra was recalculated as described above, but with the above probabilistic enhancement factor included. The response spectra for plus and minus one standard deviation from the normal, and the mean spectra are shown in Figure 4.3. As the log of the ratios of the observed and predicted accelerations using Kawashima attenuation model are normally distributed, it is possible to state with 84% confidence the spectral accelerations associated with a 450 year event will be smaller than the upper response spectra, or with 63% confidence that it lies between the upper and lower spectra.

Also shown on Figure 4.3 is the 450 year return period for rock subsoil conditions at Dunedin (Z = 0.6) with a ductitily factor of one, from NZS 4203:1992. For periods in the 0.15 to 0.3 second range, the Kawashima plus one standard deviation spectra is similar to that of the untruncated NZS 4203:1992 spectra. At large periods there are significant differences, the explanation for these are as follows:

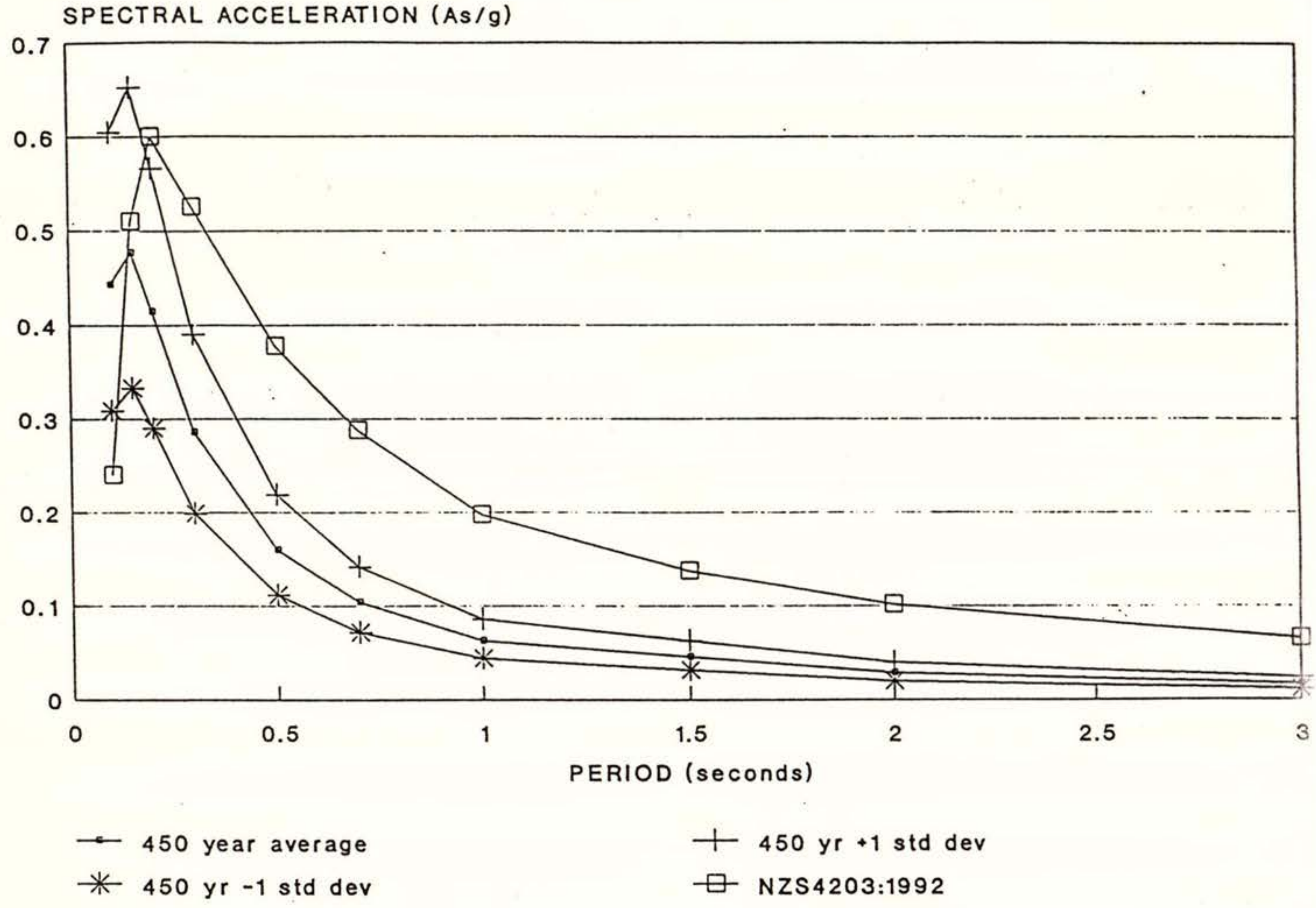



FIGURE 4.3

**Response Spectra at Bedrock for Dunedin 450 year Return Period**

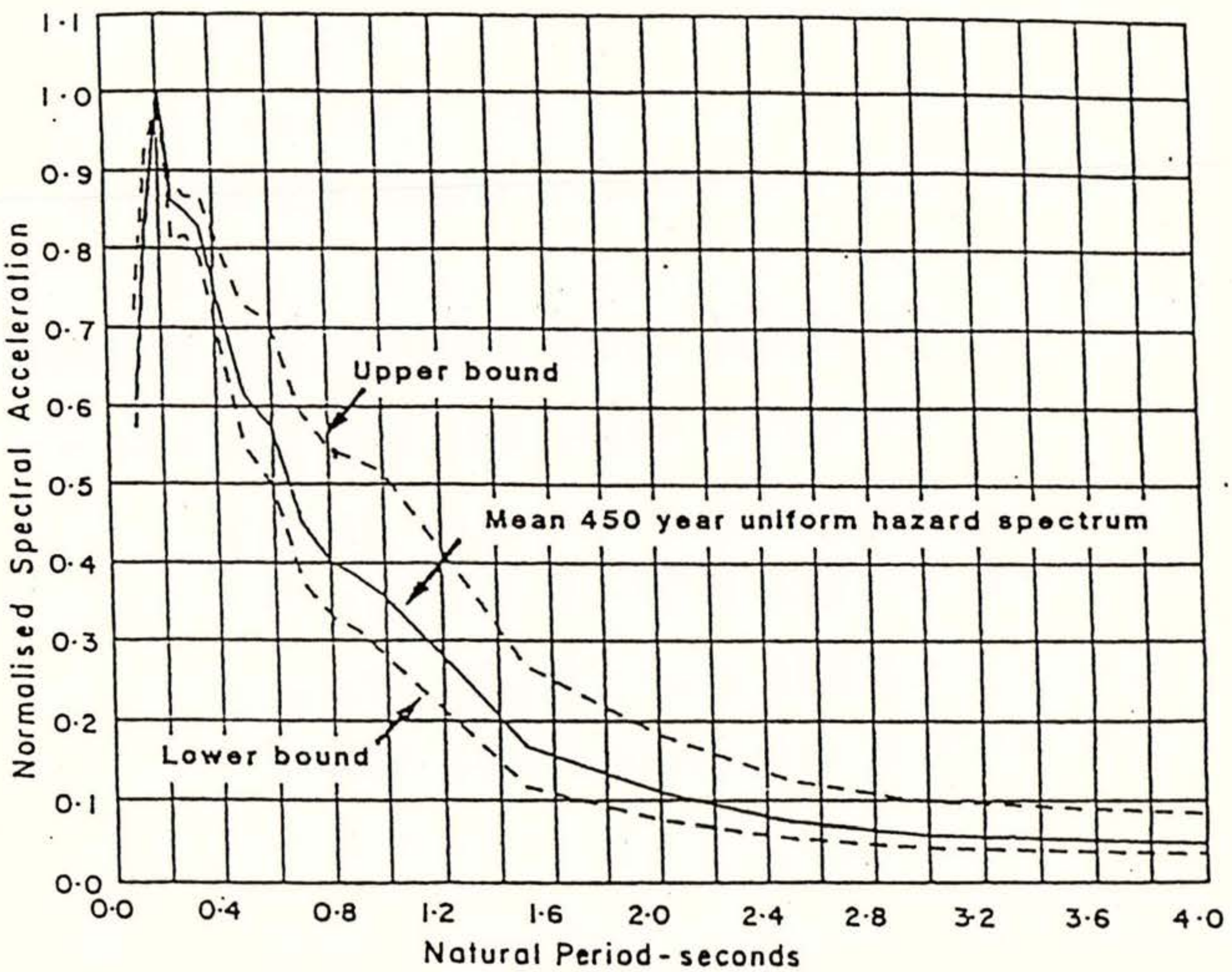



**FIGURE 4.4**

**Normalised 450 year Return Period Spectra**
For New Zealand
(Matushka et al 1985)

- The spectral shape used in NZS 4203 was generated using the modified Katyama model. An envelope was developed for the 450 year uniform hazard as illustrated in Figure 4.4. The upper bound envelope was used in deriving the NZS 4203 spectra. However, Matuschka (1985) reports that in low seismicity areas (such as Dunedin) the shape tends towards the lower bound curve.

- The spectral acceleration presented for bedrock in NZS 4204 have been increased by 50% above those predicted by Katyama as it was felt that there is insufficient data from New Zealand earthquakes to use the model with confidence.

- For periods greater than one second, the NZS 4203 spectra was modified to have constant velocity spectral ordinates. This tends to increase the expected spectral accelerations for periods greater than one second.

In this study the mean and 'plus one standard deviation' 450 year uniform hazard spectra were used as input into the deep alluvium propagation model described in Chapter 5.

## 4.5 SUMMARY

Both the Katayama and Kawashima attenuation models were considered, and the Kawashima model selected for use with the seismicity model from Chapter 2 to predict the response spectra for bedrock at Dunedin. The expected spectral accelerations in Dunedin are dominated by activity on faults close to the city. The return periods predicted for different peak spectral accelerations are shorter for low accelerations and longer for high accelerations than those previously predicted for Dunedin.

# CHAPTER 5 : PREDICTED INFLUENCE OF DUNEDIN GEOLOGY

## 5.1 INTRODUCTION

Damage to structures at a given epicentral distance during a particular earthquake has been observed regularly since 1908 to vary considerably with ground conditions at each location (eg 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 New Zealand loadings code, NZS 4203:1992 incorporates some allowance for variations in subsoil conditions, with different basic seismic hazard acceleration coefficients for rock or very stiff soil sites, intermediate soil sites, and flexible or deep soil sites. In the code commentary, it is noted that none of these subsoil categories satisfactorily account for resonant subsoil response which has been a factor in causing severe damage in a number of earthquakes around the world. Site conditions that favour subsoil resonance are sharp impedance contrasts between the near-surface soil layers and underlying material, and the presence of cohesive soil with a high plasticity index. The most vulnerable locations are often where recent lake, swamp, or marine deposits overlie coarser gravel or rock. The code commentary notes that earthquake motions at such sites may far exceed those implied by the code spectra for some spectral periods.

Dunedin City is in part built over recent marine sediments, and reclaimed land around the head of the harbour. Marked amplification effects were observed in the 1974 Dunedin earthquake, and it is apparent that subsoil response effects can be expected within the city.

This Chapter describes a preliminary analysis of such effects.

## 5.2 GEOLOGY OF DUNEDIN

The geology of Dunedin is shown in a simplified form in Figure 5.1. North Dunedin, the city end of the Peninsula, and most of the hills separating the harbour from the Kaikorai Valley are all underlain by volcanic rock. South Dunedin is underlain with sedimentary formation including Caversham sandstone, Abbotsford mudstone and Green Island Sand. Holocene estuarine, flood plain and swamp deposits overlie the older formations in the Kaikorai Valley, and around the head of the harbour. It is these latter deposits, particularly those adjoining the harbour, which are of most concern with respect to seismic response.

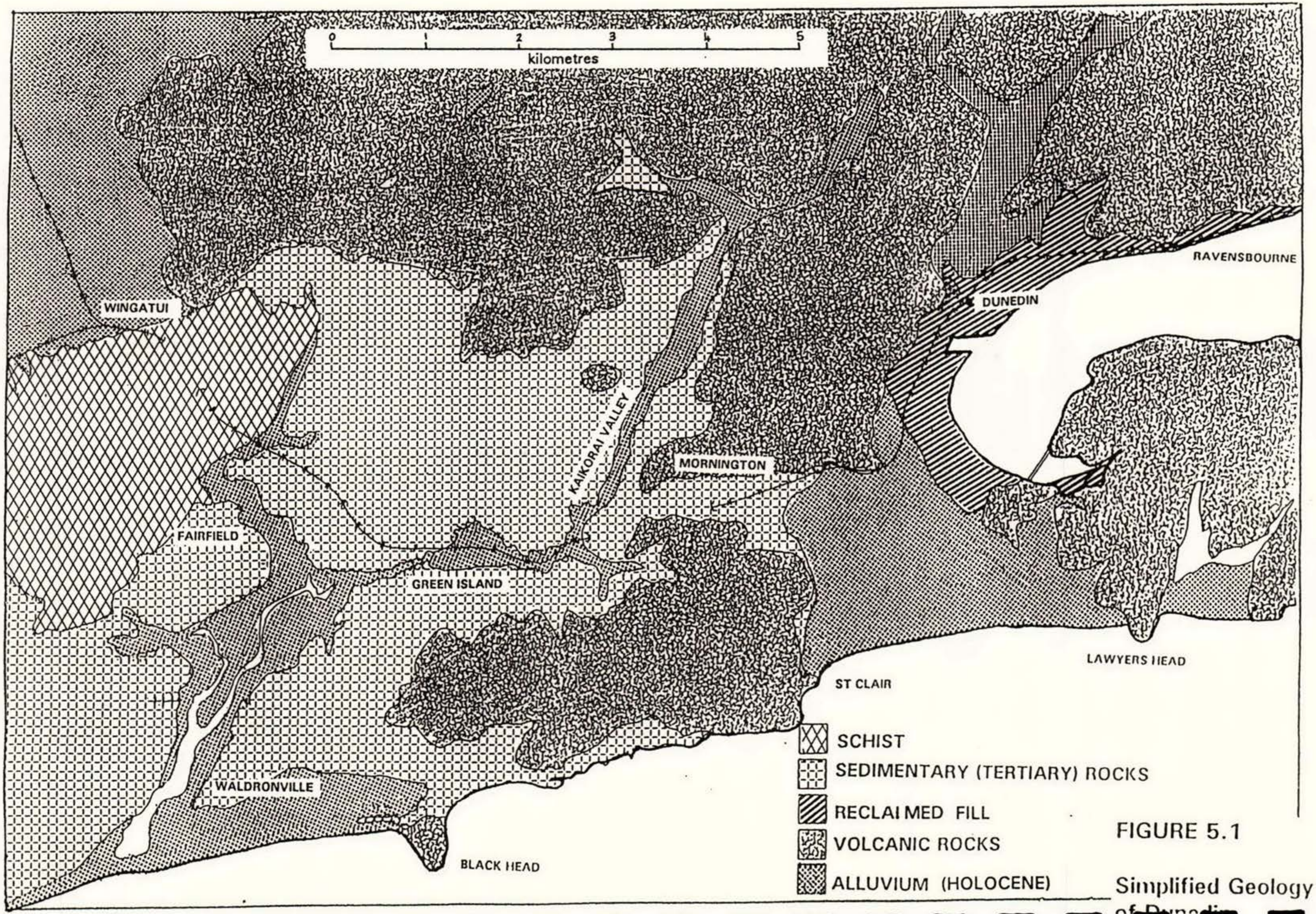



FIGURE 5.1

Simplified Geology of Dunedin

The sediments have been deposited since the last glacial period, when the sea level was about 100 m lower than at present. The area now occupied by Otago harbour was two valleys with a watershed running between Portobello and Port Chalmers. South Dunedin was a broad valley, draining to the south. It may have been partly infilled with fluvial gravels, sands and silts. As the sea level rose, the valley was flooded and marine or estuarine sediments were deposited.

In order to better assess the site specific seismic response in Dunedin, it was necessary to construct a three dimensional model of the geologic profile beneath the city. As many borehole records as possible were obtained in an attempt to determine the geologic profile. In the event, boreholes for a total of only 479 sites were obtained. (This compares with over 15,000 logs available for Christchurch). Many of the borelogs were also for widely separated sites away from the main alluvial areas, and records for about 200 sites were eventually used. Records were obtained from the sources listed below. The co-operation of the holders or owners in assisting us with the transfer of these records is gratefully acknowledged.

Dunedin City Council
Hanlon & Partners Limited
Works Consultancy Services
Duffill Watts & King Limited
Rankine Hill Limited
DSIR - NZ Geological Survey
Royds Garden Limited

A resistivity survey was commissioned as part of this study to help determine the thickness of the holocene sediments, particularly in South Dunedin where there are very few boreholes, and even fewer records giving a depth to rock. The resistivity survey was carried out by Christopher Pearson, University of Otago Consulting Group, and the report is attached as Appendix B.

The borelog data was used in the following way:

1. All the borelogs were fitted and allocated a number.

2. The borelogs were summarised by taking representative soil types for 5 depth ranges.

3. The borelog locations were plotted on a city street map at a scale of 1:10,000.

4. Five reproductions of the map were prepared and assigned to represent layers beneath the city at the following depths beneath the ground surface

0 - 2 metres
2 - 5 metres
5 - 10 metres
10 - 20 metres
20 - 30 metres

5. The representative soil type for each depth layer from step 2 was then plotted on the appropriate map for each borehole, for the following predominant soil type

| | | |
|---|---|---|
| R | - | Rock |
| G | - | Predominantly gravel |
| S | - | Predominantly sand |
| M | - | Predominantly fine grained silts or clays |
| P | - | Predominantly peat |
| F | - | Predominantly fill |

It had originally been intended to define areas of each predominant soil type occurring within each depth layer. However the scarcity of data, and the variability been boreholes made such an approach unrealistic, and the representative soil types were left as defined by each borelog. The soil type maps are reproduced in this report as Figures 5.2 to 5.6. Some additional information of soil types has been provided by the restivity survey which indicated an appreciable thickness of near surface marine mud and silt around the head of the harbour between Portsmouth Drive and Andersons Bay Road (Refer to Figure 3, Appendix B). This correlates well with the predominantly silt soils logged in this area to 20 m depth.

The depth of bedrock is shown in Figure 5.7. The depth contours are compiled from borelogs where these extended to rock, and from the resistivity survey information. There is little information on bedrock depths, and the depth contours can only be regarded as tentative and approximate. Where deeper borelogs are available, it is often difficult to determine whether the material logged is alluvium, or highly weathered insitu basement. This figure shows the original stream channel that formed the now drowned valley lay closer to Waverley than to the Roslyn hills in the area of the head of the harbour, passed diagonally across the valley close to the line of Midland Street and Macandrew Road, before swinging south east close to the Kew Hills and passing under the present beachline opposite the Forbury Park Raceway. A secondary buried valley appears to extend northwest under the north Dunedin reclamation opposite the end of Logan Park.

The bedrock is thought to be Caversham Sandstone at least under part of South Dunedin, and probably the area of deepest sediment in St Kilda. Where the sediment is shallower, and for the whole of the central and north Dunedin area, the bedrock is assumed to be volcanic overlying Tertiary sediments. The sediments dip to the east and are probably about 600 m thick under Dunedin. It is assumed that schist extends below the Tertiary sediments under the whole area.

Three cross sections of the soils were also compiled from the borelog data and these are shown in Figures 5.8 to 5.10. Figure 5.8 is a "longitudinal" section from the St Kilda beach at the south, north west on Prince Albert Road to Macandrew Road, diagonally north to Cumberland Street at the Andersons Bay Road corner and hence north east for the length of Cumberland Street to the Water of Leith in North Dunedin. Figure 5.9 is a "cross" section through South Dunedin on Macandrew Road and Midland Street, and Figure 5.10 is a "cross" section on Hanover and Halsey Streets to the north of the central city. The bedrock levels shown on these cross-sections are taken from the depth to bedrock map, Figure 5.10, and are approximate only.

It is clear from these maps and sections, that the stratigraphy beneath Dunedin is very complex and areas of uniform soil profiles are limited in extent. South Dunedin appears to be more uniform in that the soils appear to be generally confined to sand and silt gradings, but the scarcity of borelog data may be misleading.

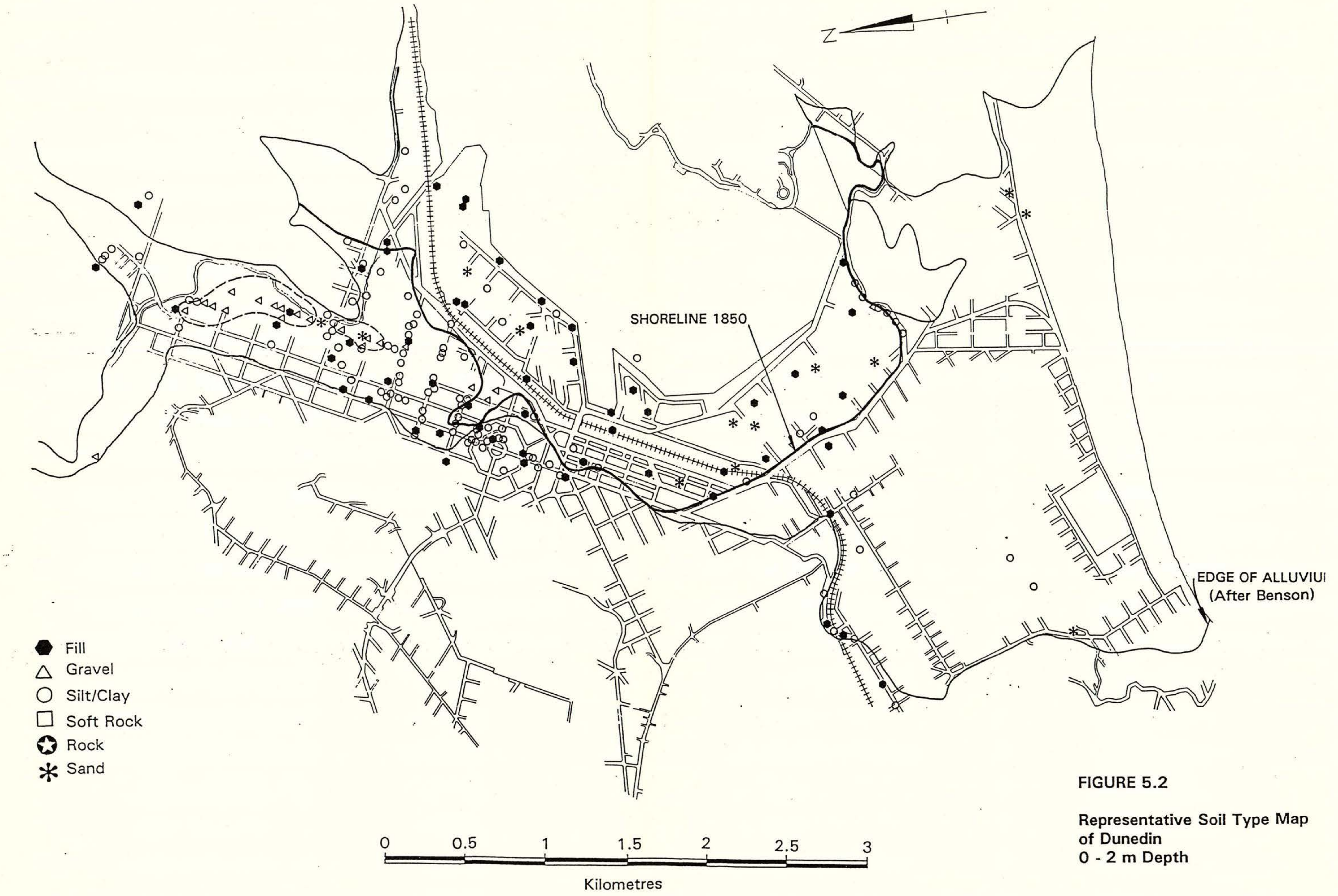



FIGURE 5.2

**Representative Soil Type Map of Dunedin 0 - 2 m Depth**

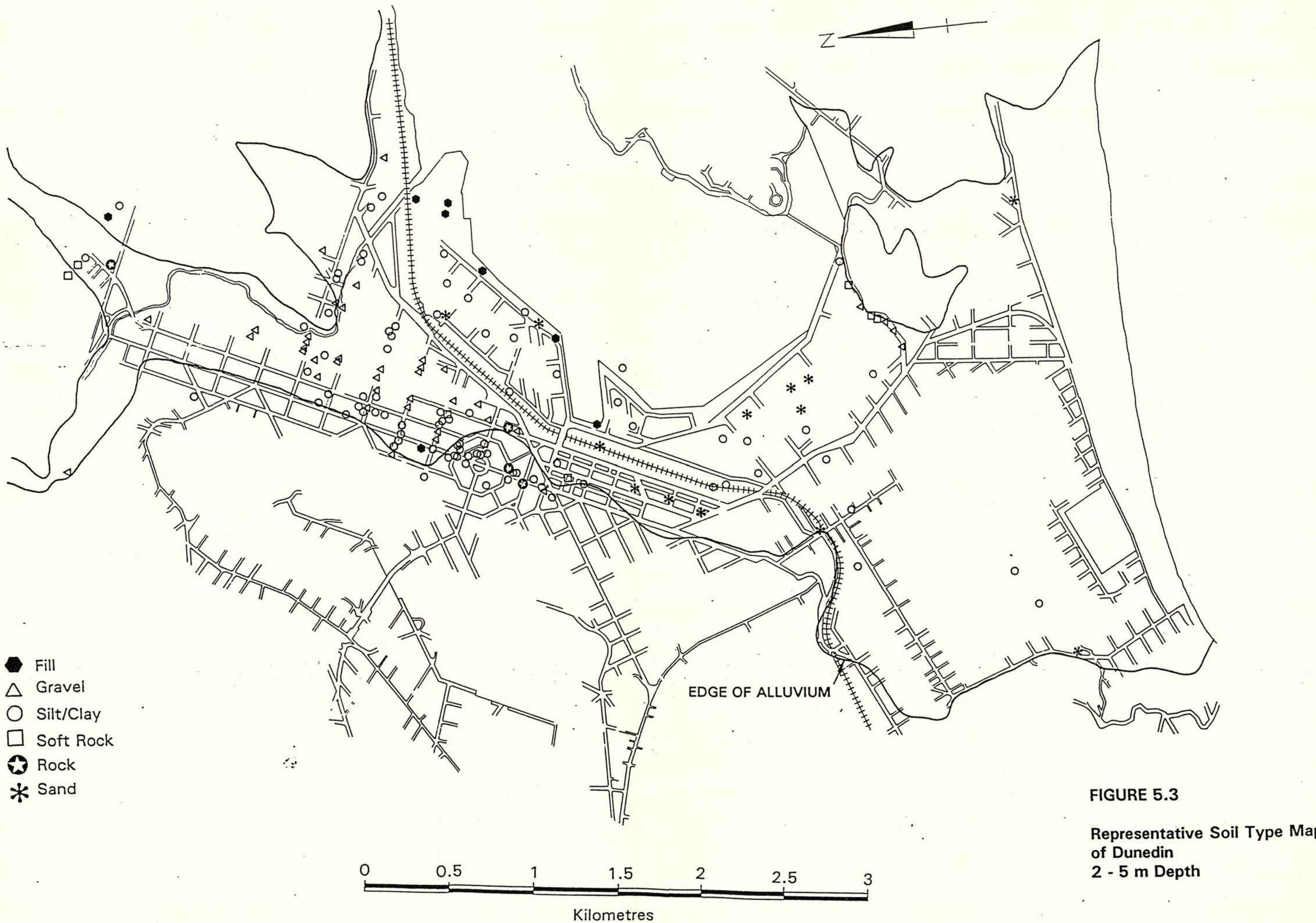



**FIGURE 5.3**

**Representative Soil Type Map of Dunedin 2 - 5 m Depth**

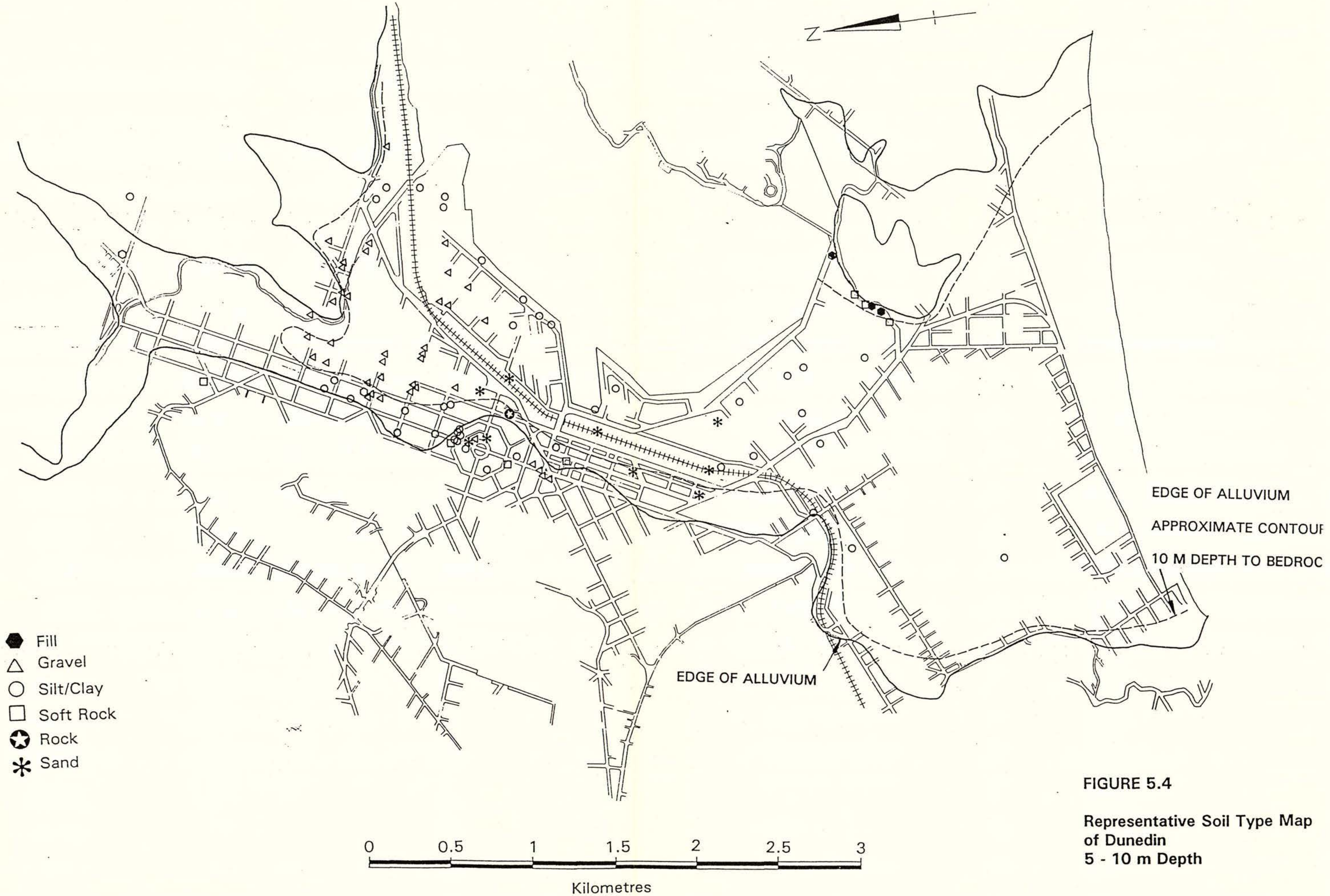



FIGURE 5.4

**Representative Soil Type Map of Dunedin 5 - 10 m Depth**

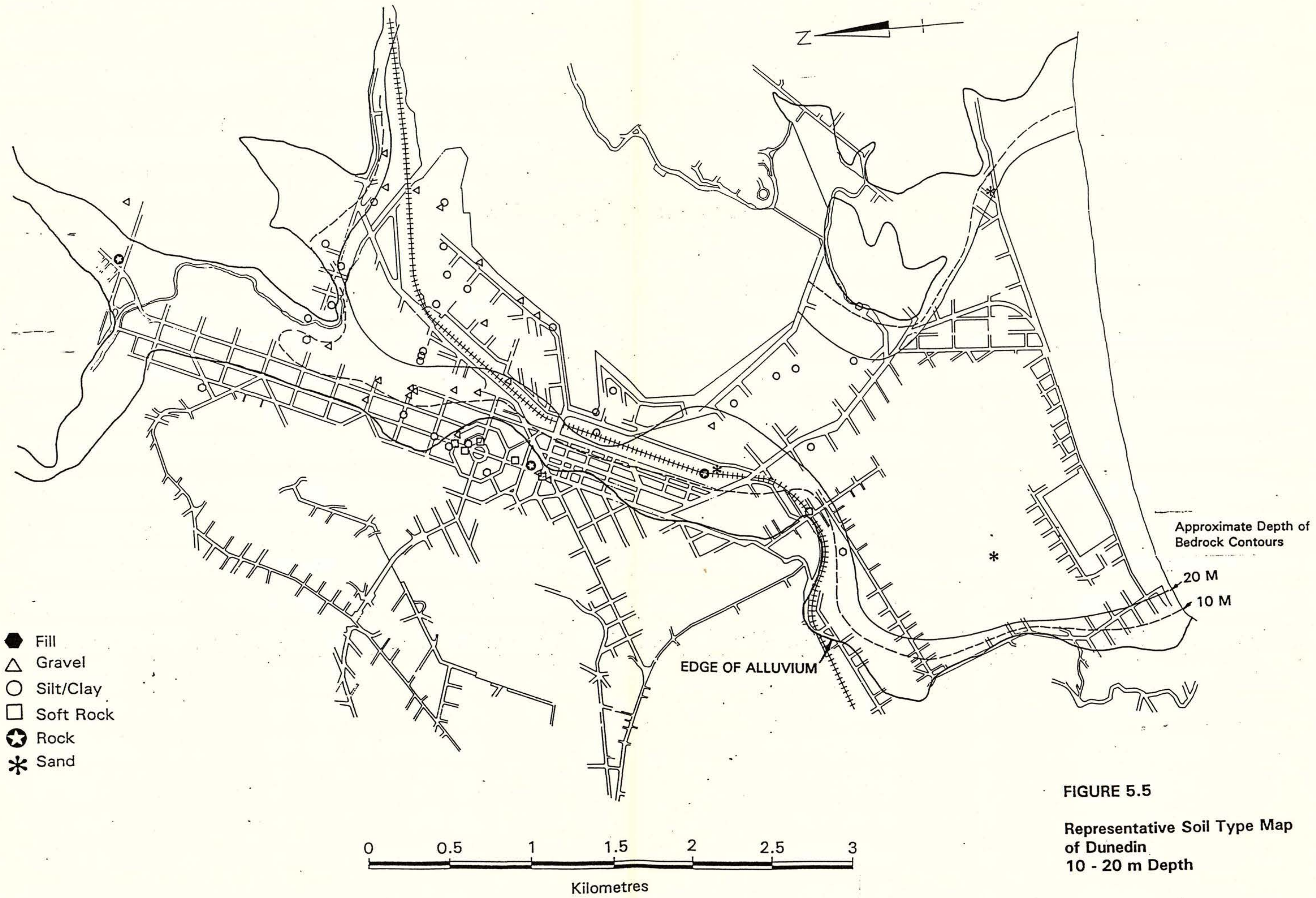



FIGURE 5.5

**Representative Soil Type Map of Dunedin 10 - 20 m Depth**

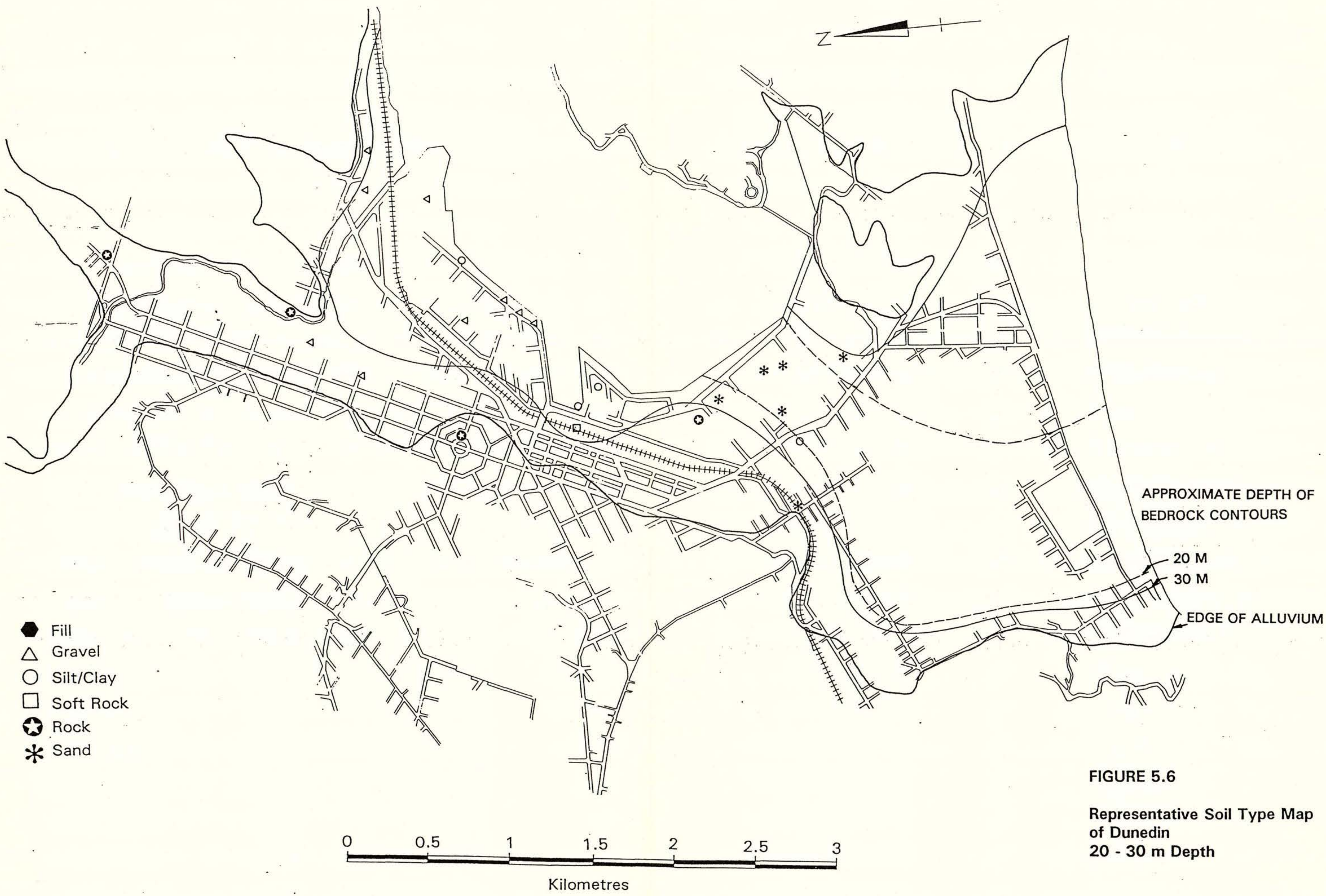



FIGURE 5.6

**Representative Soil Type Map of Dunedin 20 - 30 m Depth**

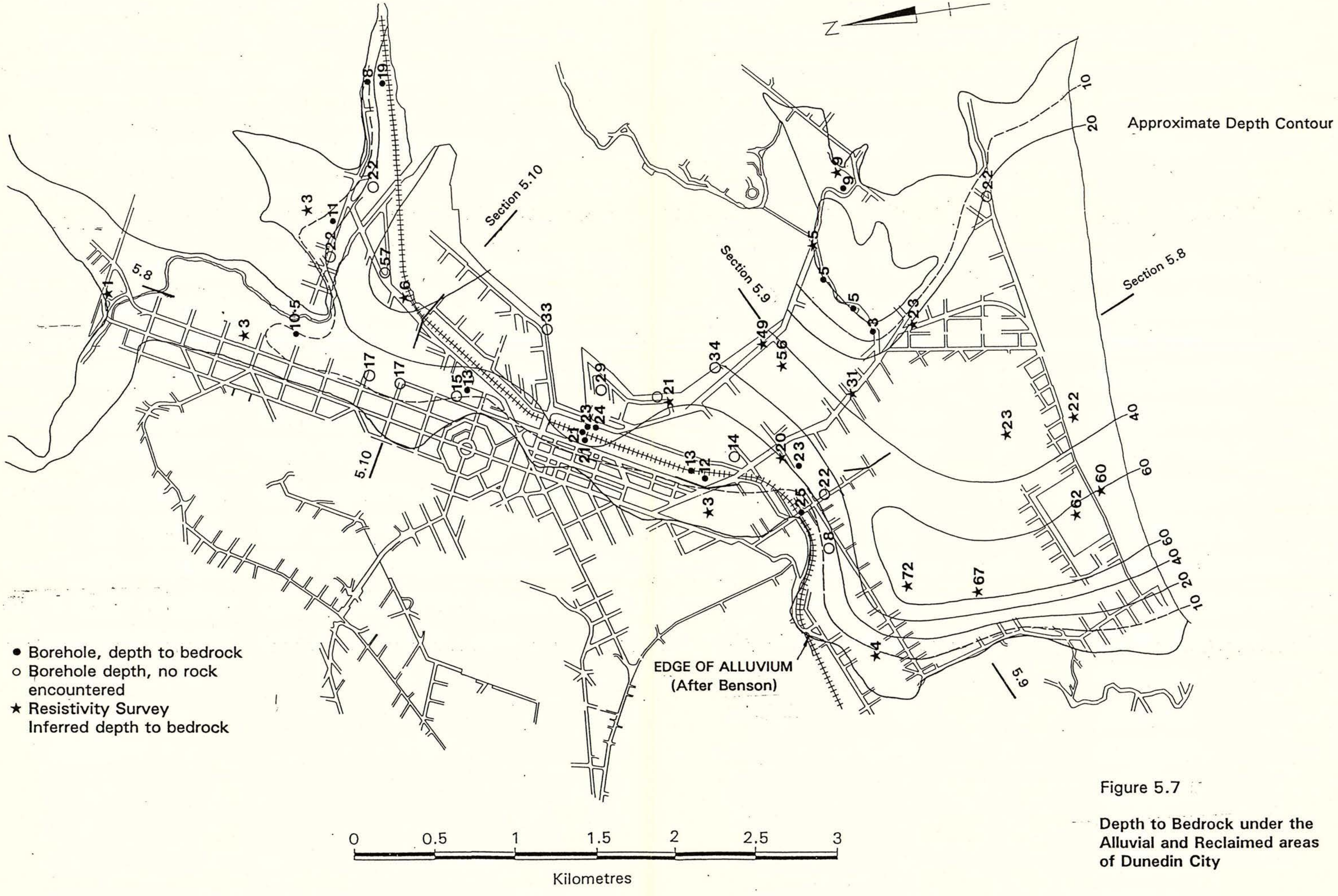



Figure 5.7

Depth to Bedrock under the Alluvial and Reclaimed areas of Dunedin City

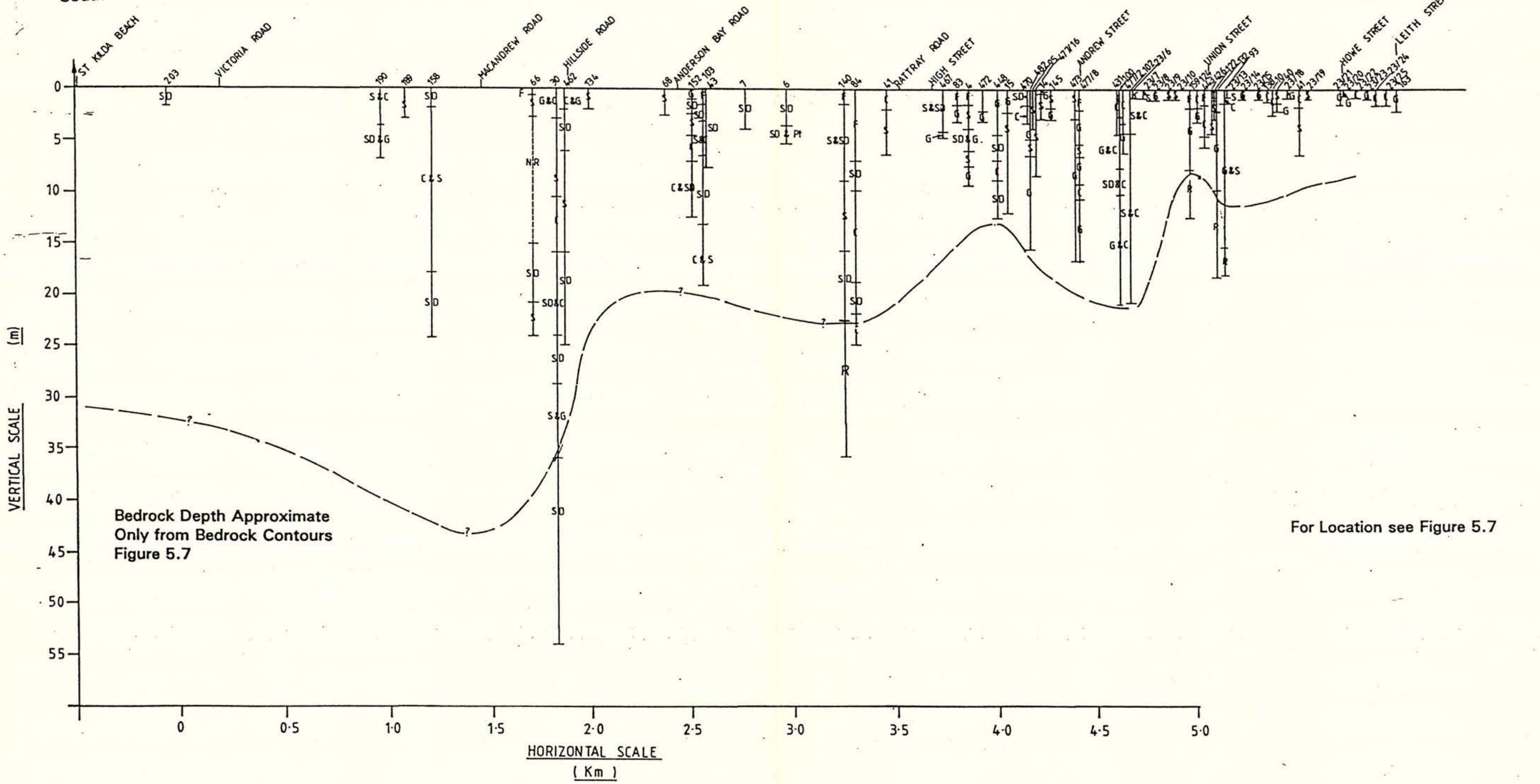
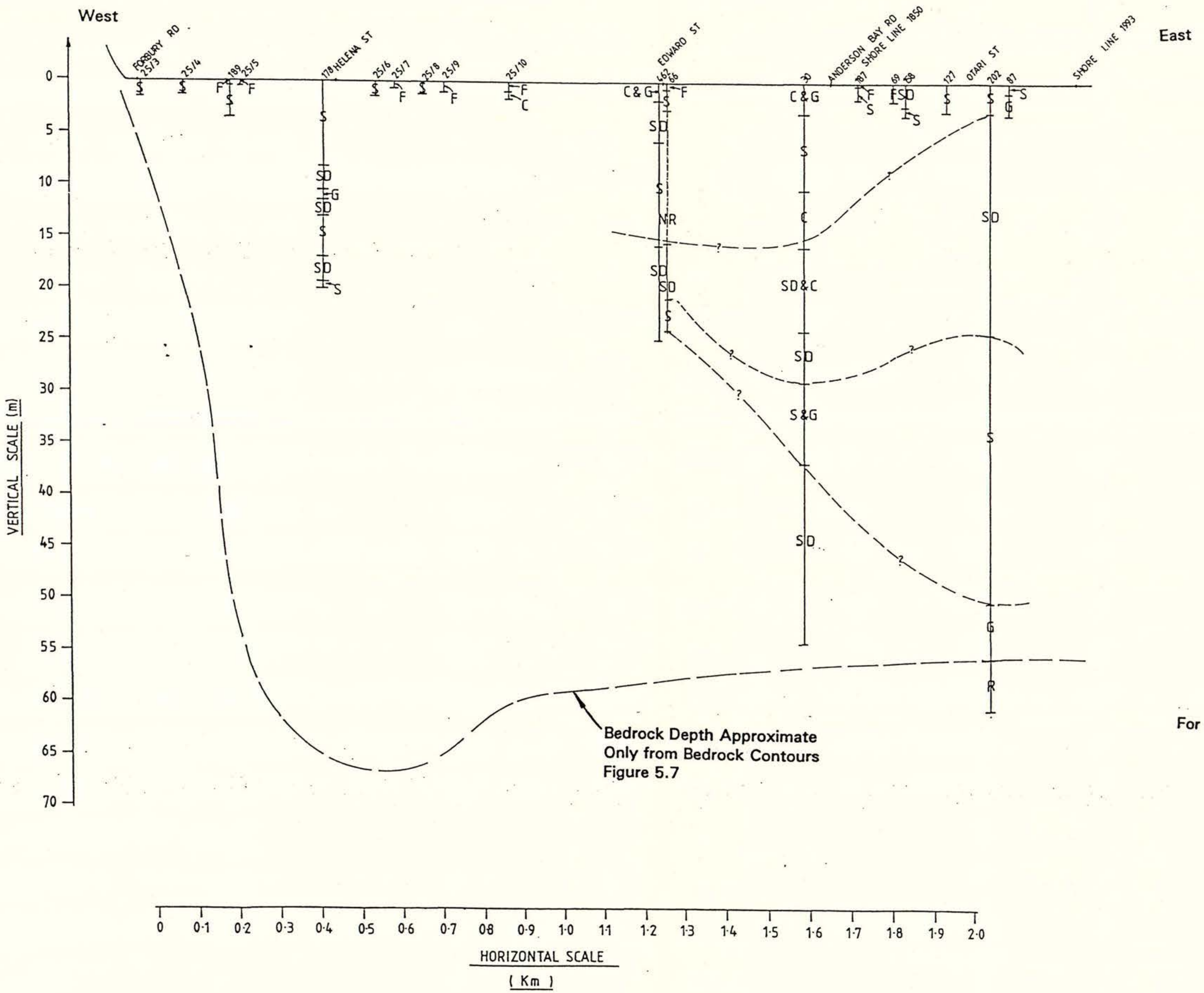



Figure 5.8

**North - South Cross Section Through Dunedin on Prince Albert Road and Cumberland Street**




For Location see Figure 5.7

Figure 5.9

East - West Cross Section
Through North Dunedin on
Macandrew Road and Midland Street

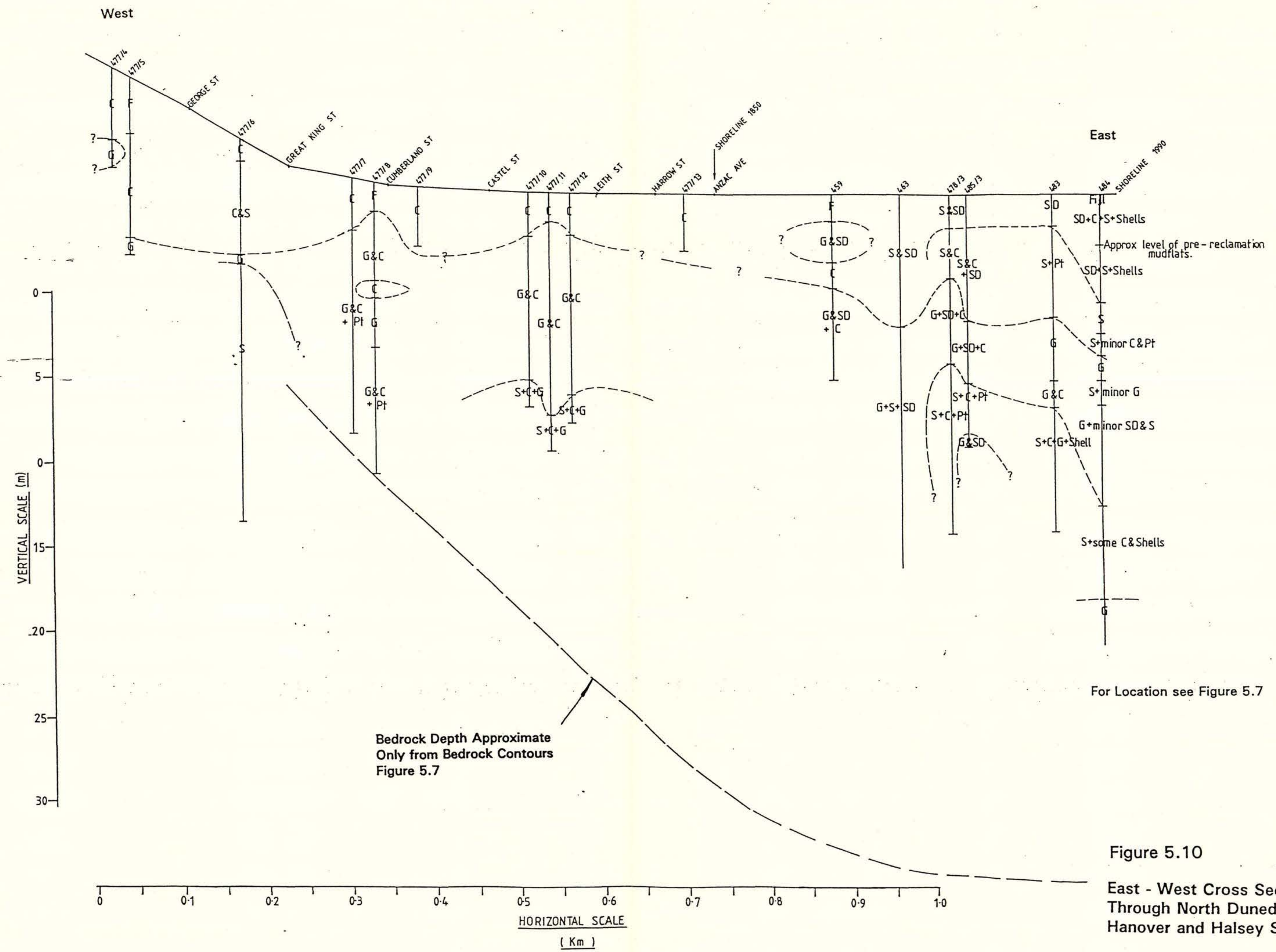



Figure 5.10

East - West Cross Section Through North Dunedin on Hanover and Halsey Streets

## 5.3 DEEP SOIL RESPONSE 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.

- 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. Focusing may be very significant in Dunedin, but unfortunately cannot be sensibly modelled 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 inter layering of peats, silts, sands and gravels beneath Dunedin 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 response of the Dunedin soil profiles were modelled conventionally by considering viscoelastic strain-softening in conjunction with vertically propagating shear waves. The analysis method of Haskell (1960) was employed which was the first which investigated the resonant characteristics of a layer or layers overlying an elastic half space. The method, in theory, applies only for layering of constant thickness, but in practice it is widely used to model more complex geometries. It can also be used to model the case of obliquely incident waves, but it is easily shown that oblique waves will lead to smaller amplifications, and hence the case of vertical incidence is conservative.

All calculations are carried out in the frequency domain. A set of matrices called Haskell-Thompson matrices are derived, and when multiplied together they give the transfer function for the soil profile. The transfer function is simply the ratio of the free surface acceleration to the basement rock acceleration at a given excitation period. Viscous damping is directly incorporated.

Non-linear strain softening will also affect the site response and this is incorporated in the model in an iterative scheme. Assuming viscoelastic behaviour, the strains in each soil layer may be calculated. The peak strain magnitude is then used to predict a reduced shear modulus for that layer, based on data from laboratory tests. An enhanced damping coefficient is also predicted. The relationships between shear modulus, damping, and strain given by Seed and Idriss (1984) is used in the calculations described here. Once altered values for shear modulus and damping coefficient have been obtained, the problem is re-calculated and new peak strains are found for each layer. This process is carried out iteratively until a consistent set of strains are found. Usually only three or four iterations are necessary to force convergence.

The specific method of calculation used is that developed for computer analysis by Dr R O Davis at the University of Canterbury. 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.

Shear Wave Velocity

A number of methods are available for assessing the shear wave (S-wave) velocities in different soil strata.

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 & Tinsely (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 a 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 (eg 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 |
| 2 - 10 | 150 | 250 | 400 |
| 10 - 20 | - | 350 | 600 |
| 20 - 50 | - | 500 | 800 |
| 50 - 100 | - | 700 | 1100 |
| 100 - 200 | - | 1000 | 1500 |
| 200 - 500 | - | 1500 | 2200 |

Soil Property Profile

The lack of good information on the soil profile beneath large areas of Dunedin prevents any general site response modelling for the whole city. A number of representative soil profiles were considered, to provide typical site response spectrum.

From the typical soil maps, four broad groupings of soil profile were made:

| | | |
|---|---|---|
| Zone 1 | - | South Dunedin and St Kilda |
| Zone 2 | - | Reclaimed area south of the central city |
| Zone 3 | - | Reclaimed area adjacent to central and north Dunedin |
| Zone 4 | - | North Dunedin |

Borelogs were selected from within each zone, and had to extend some depth, appear to have an accurate log, and preferably have some test data. A total of 17 soil profiles were compiled from the borelogs data, and the approximately depth to bedrock from Figure 5.7. In most cases the borelogs did not extend to bedrock, and assumed soil profiles had to be constructed. The soil profiles used were of this general form:

| | |
|---|---|
| 0 to between 20 and 60 m | Soil profile from borelogs |
| 20 - 60 m to bedrock | Assumed soil profile |
| Bedrock to 600 m | Volcanics and/or Tertiary sediments |
| 600 m | Basement rock |

Details of layers, densities and shearwave velocity ranges used are shown in Table 5.1

TABLE 5.1 **Deep Soil Response Model**

| Soil/Rock | Thickness (m) | No. of Layers | Density ($t/m^3$) | Shear Wave Velocity (m/s) |
|---|---|---|---|---|
| Holocene Sediments - Borelog profile | 12 - 60 | 5 - 10 | 1.7 - 2.1 | 75 - 600 |
| Holocene Sediments - Assumed Profile | 0 - 50 | 0 - 4 | 1.9 - 2.1 | 400 - 830 |
| Tertiary Sediments and Volcanics | 530 - 580 | 1 | 2 - 2.25 | 1500 - 1700 |
| Basement Schist | | 1 | 2.5 | 3000 |

The upper bound bedrock response spectrum calculated in Chapter 4 (see Figure 4.3) was used as input into the model.

The form of the model was similar to that used previously for the Christchurch Study (Elder et al, 1991). That study considered variation to the input parameters. 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.

For this study the depth to basement rock, and the S-wave velocity in the tertiary sediments were checked for sensitivity. It was found that a change of 25% in the thickness of the Tertiary sediments and volcanics made 10 - 20% difference in the surface response spectrum. Similarly a 10% change in the S-wave velocity in the Tertiary sediments made about 10% change in the surface response spectrum.

## 5.4 SITE EFFECTS PREDICTED BY THE MODEL

A total of 17 soil profiles were input into the deep soil propagation model. The locations of these sites are shown on Figure 5.11. Typical calculated surface response spectrum are shown on Figure 5.12. These are all for an assumed depth to schist basement of 600 m.

The responses can be grouped according to shape as shown on Figure 5.13. This figure also shows the input bedrock spectrum, and the elastic response spectrum for a deep soil or flexible site in Dunedin derived from NZS 4203:1992 (structural ductility factor $\mu$ = 1, zone factor z = 0.6. Type A response shows a pronounced peak at 0.2 - 0.3 seconds, and a large amplification effect from 0.4 to about 0.8 seconds period. In relative terms, these sites can be termed "firm" with softer soils generally less than 10 m in thickness at the surface.

Type B response shows much smaller amplification at periods of less than 0.8 seconds, but a very pronounced amplification and a second peak at 1.1 - 1.5 second period. These sites all exhibit a large impedance contrast between either the tertiary rocks or lower layers of Holocene sediment, and a soft silt layer at 20 - 30 m depth.

Type C response shows a lower peak response acceleration, but pronounced amplification with nearly constant accelerations between 0.4 and 1.3 second period. These sites show a deeper profile of softer soils, than for Type A sites, extending to over 20 m depth.
In some cases, the differences between soil profiles giving different response spectrum is not great. The model used for this study is sensitive to changes in the soil profile, particularly in the 10 - 25 m depth range. Because of this, it is not possible to draw any generalisation in terms of the type of response or likely peak spectral accelerations on an areal basis for Dunedin, given the scarcity of subsurface information, and the complexity of the alluvial stratigraphy.

The soil propagation model had been expected to show greater amplification with greater thickness of alluvium, but no clear pattern is observable from the model with the input parameters used. For one site profile (No. 477) increasing the alluvium depth from 20 m to 70 m increased the peak spectral acceleration by about 5%, whereas for a second site profile (no. 143), the same increase in alluvial depth increased the second peak of the response spectrum by 17% but decreased the first peak by 25%.

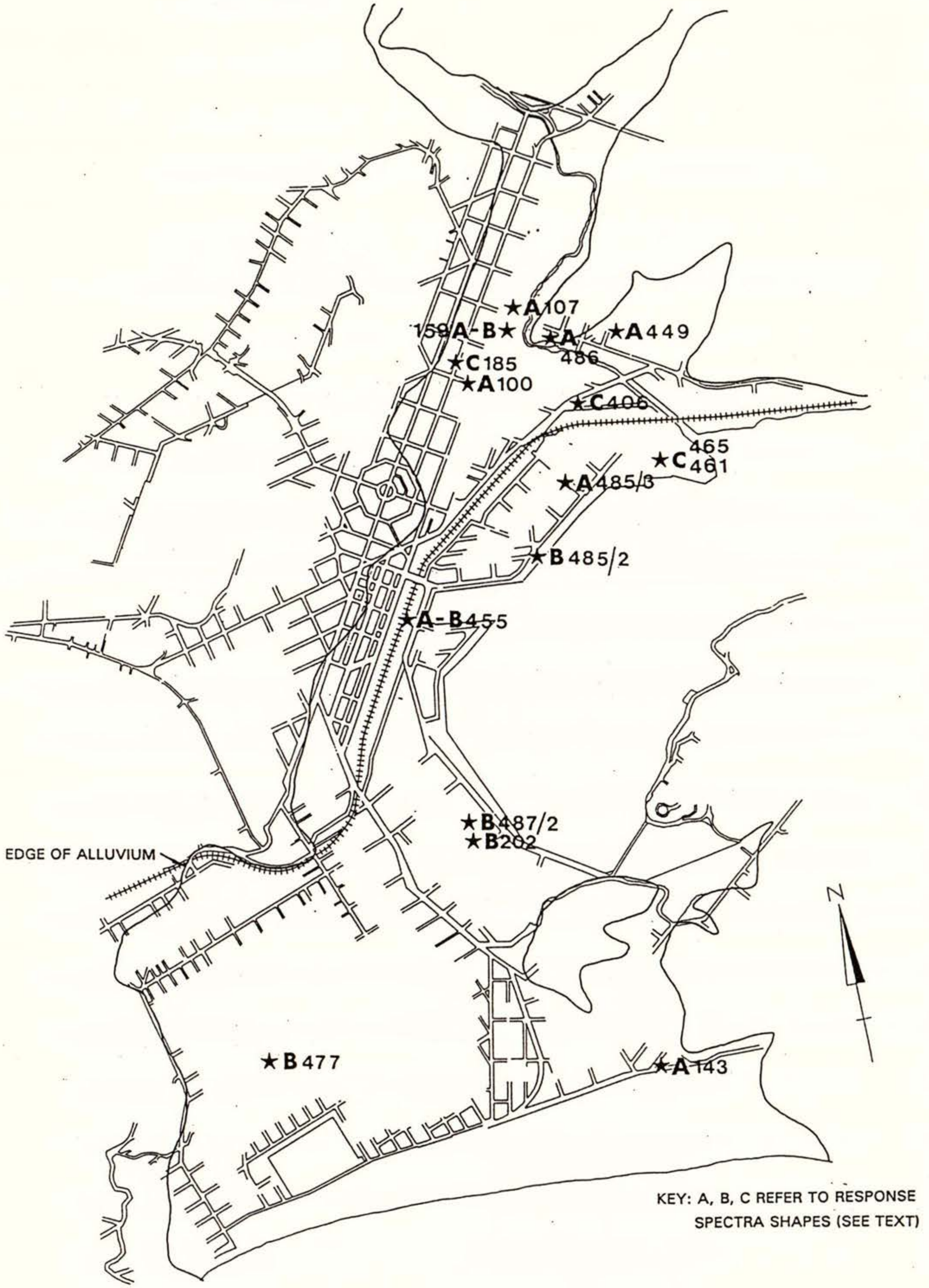



**FIGURE 5.11**

**Location of sites used in Deep Soil Propagation Model**

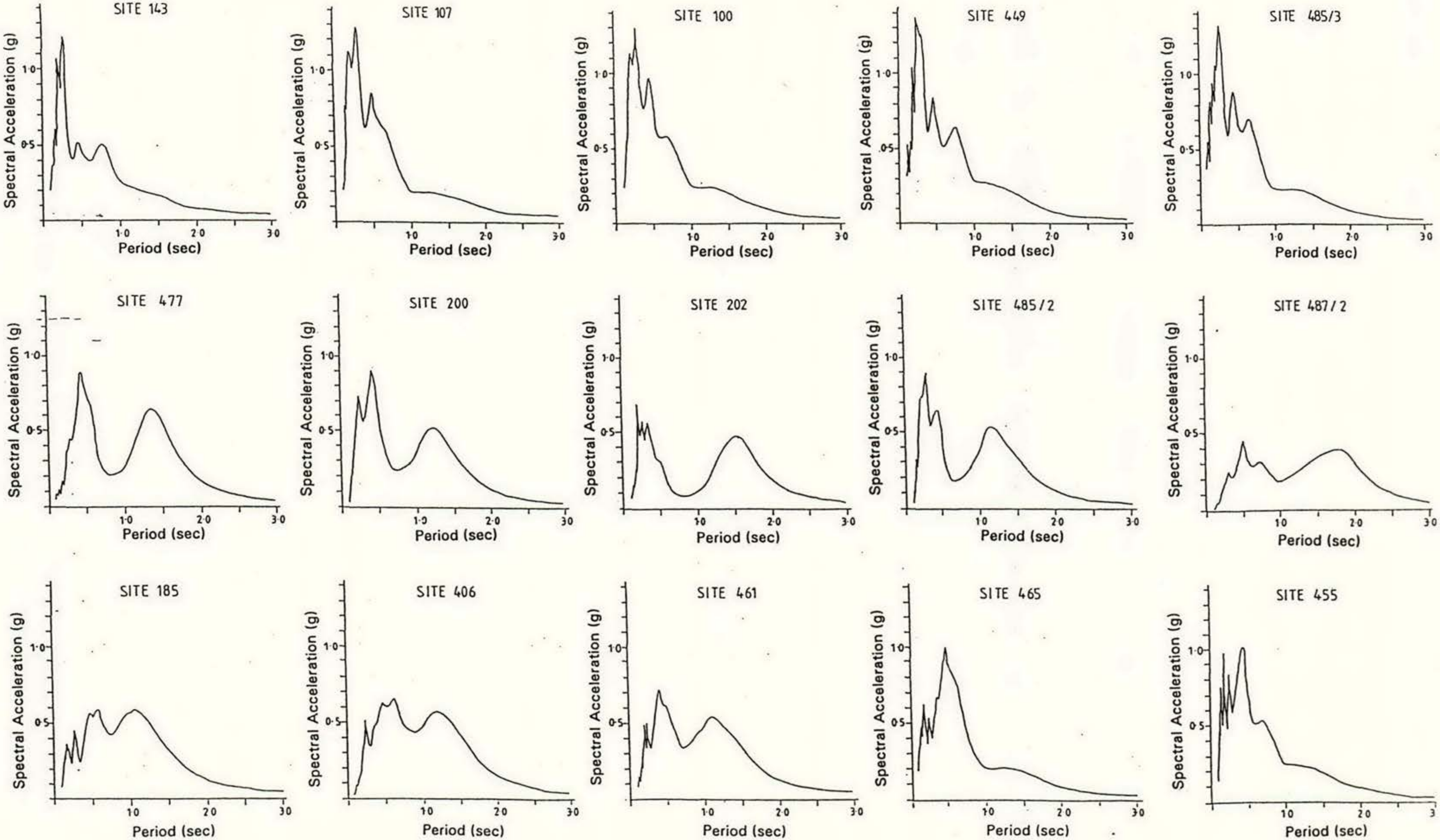



**FIGURE 5.12**

**Response Spectra at the Ground Surface in Dunedin**

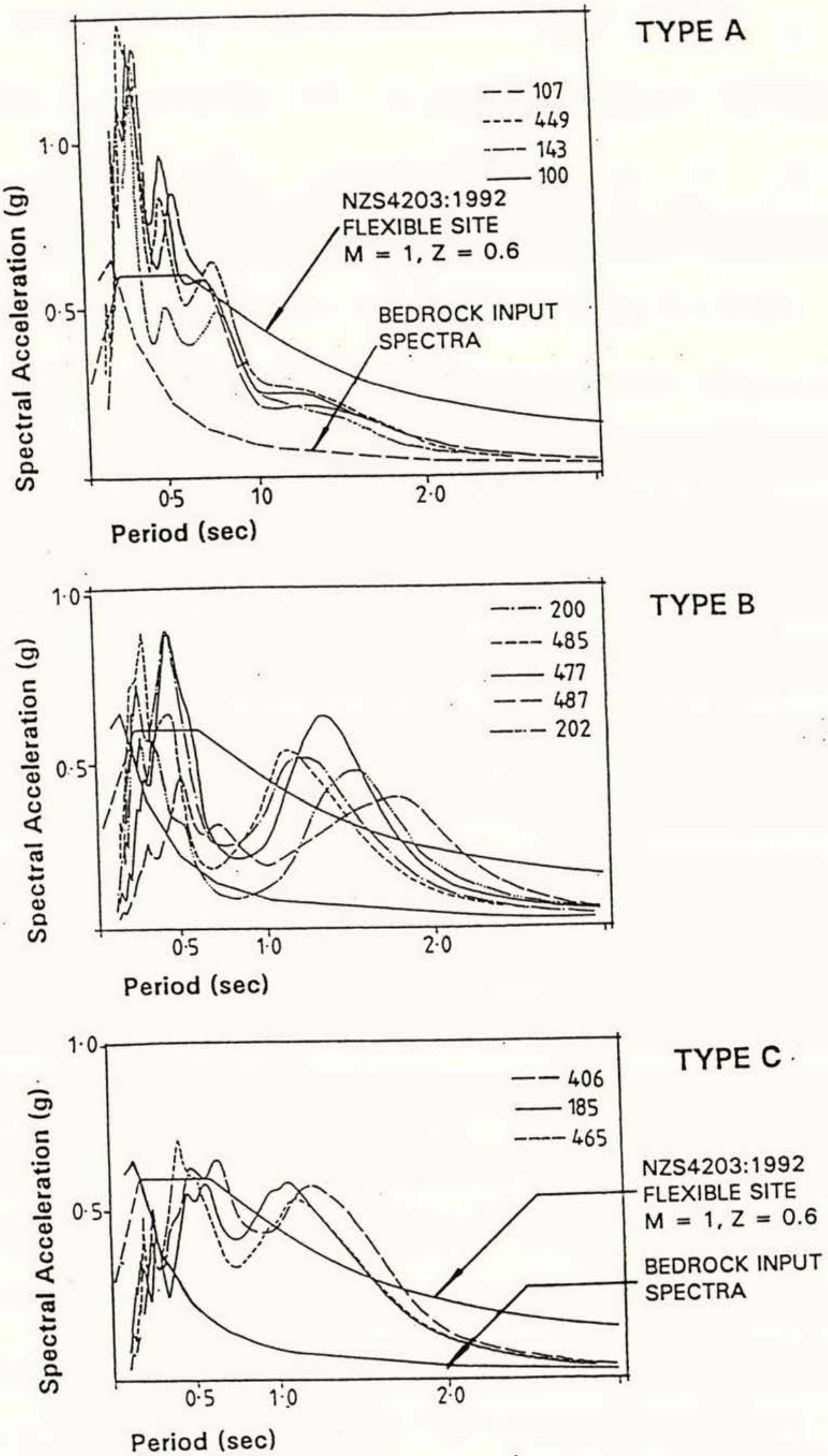



FIGURE 5.13

Response Spectra at Ground Surfac
in Dunedin, Grouped According
to Shape

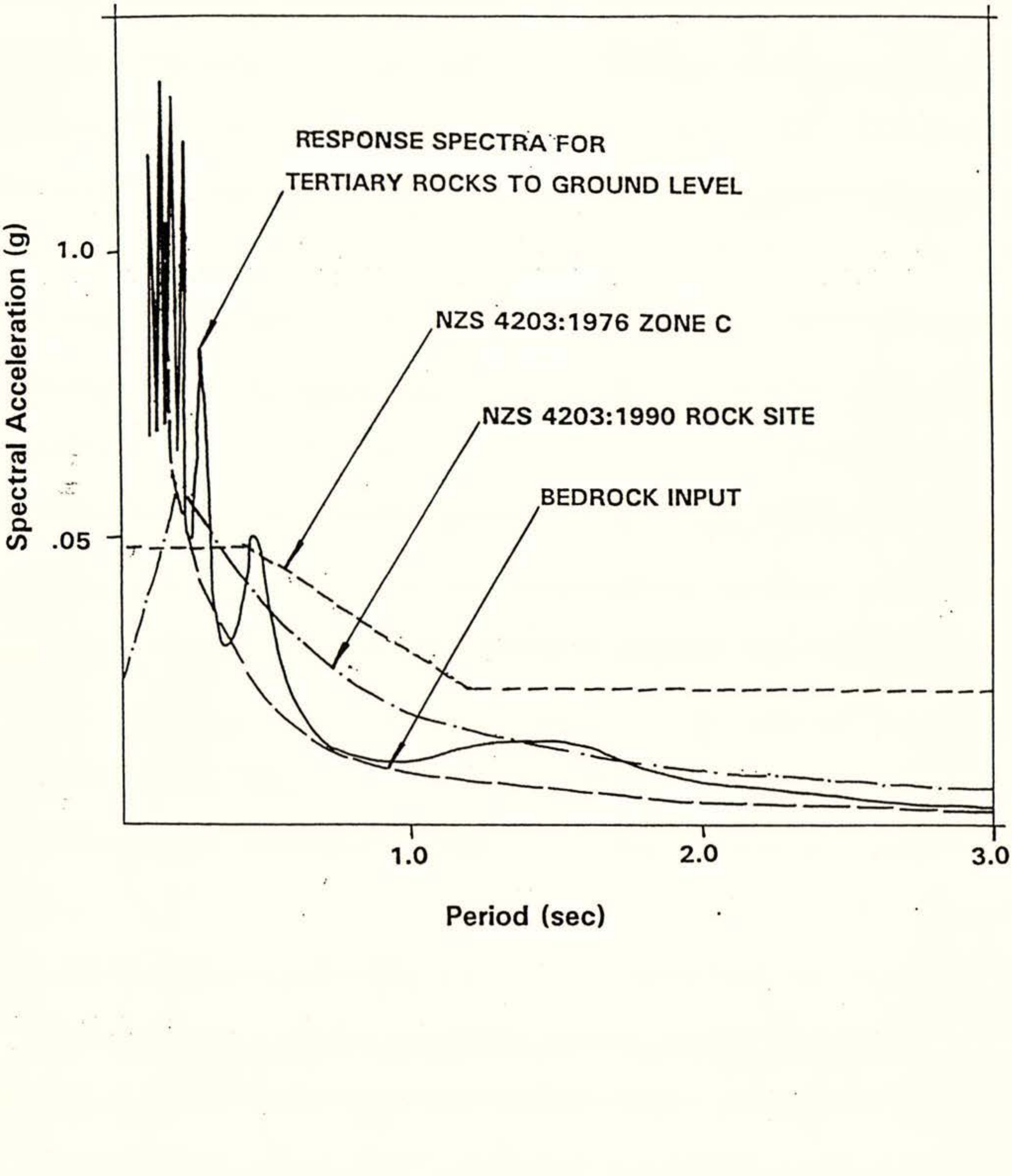



FIGURE 5.14

**Response Spectra for Site on Tertiary Sediments or Volcanics**

The model was also used to determine the site response for a site with Tertiary sediments to ground level, (such as a site on the Dunedin hills), and the spectral response spectra is shown in Figure 5.14. The model produces some very sharp peaks in the 0.1 - 0.3 sec period range. Because of the very narrowness of the peaks, these would probably not effect structures significantly, and the spectra remains inside the upper bound spectra used for NZS 4203:1992.

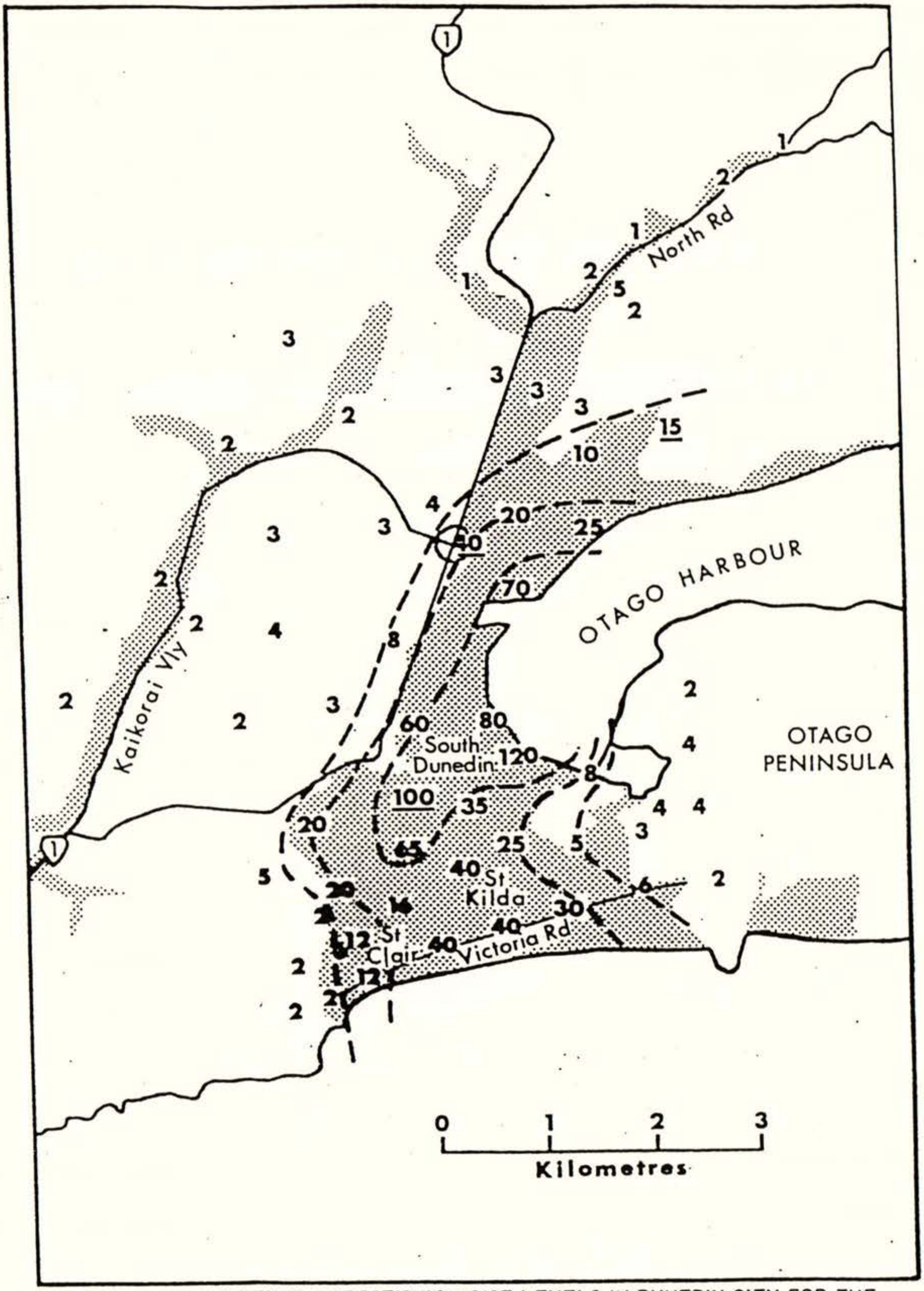
## 5.5 SITE EFFECTS OBSERVED IN 1974 EARTHQUAKE

The 1974 Dunedin earthquake showed strong amplification effects in the southern suburbs on the recent alluvium and reclaimed land. These effects are reported in the 1974 study by Admas & Kean, (Seismological studies), and Bishop, (local effects). Adams & Kean carried out a survey of natural ground noise at 67 sites in June 1974, and the result are shown on Figure 5.15. Bishop made a survey of fallen items in grocery stores, and also plotted the location of damage insurance claims. His results are reproduced here as Figures 5.16 and 5.17.

The ground noise levels show some correlation with depth of softer muds and silts around the head of the harbour, (compare Figure 5.15 with Figure 3 in the Resistivity Survey Report, Appendix B), whereas the fallen items in grocery stores shows a marked peak area in the St Clair - Caversham area which is underlain by the deepest alluvial sediments. The density of damage claims does not show the same pattern other than the area of greatest damage being on the alluvial area in South Dunedin. However, as Bishop notes, this map must be interpreted with caution as the contours are strongly affected locally by factors such as variations in the number of houses per unit area and by regions of older houses.

Both Figures 5.16 and 5.17 also show areas of damage outside the alluvial areas. In particular, in Figure 5.17, two areas of greater than 80 claims per $km^2$ to the east of St Kilda appear to correspond to Musselburgh Rise, and the edge of the Peninsula hill in Andersons Bay. Another distinct concentration occurs from Lookout Point through to Morrington. This latter area is highlighted in the fallen items map, extending north through Roslyn to Wakari. These areas presumably reflect a topographic effect.

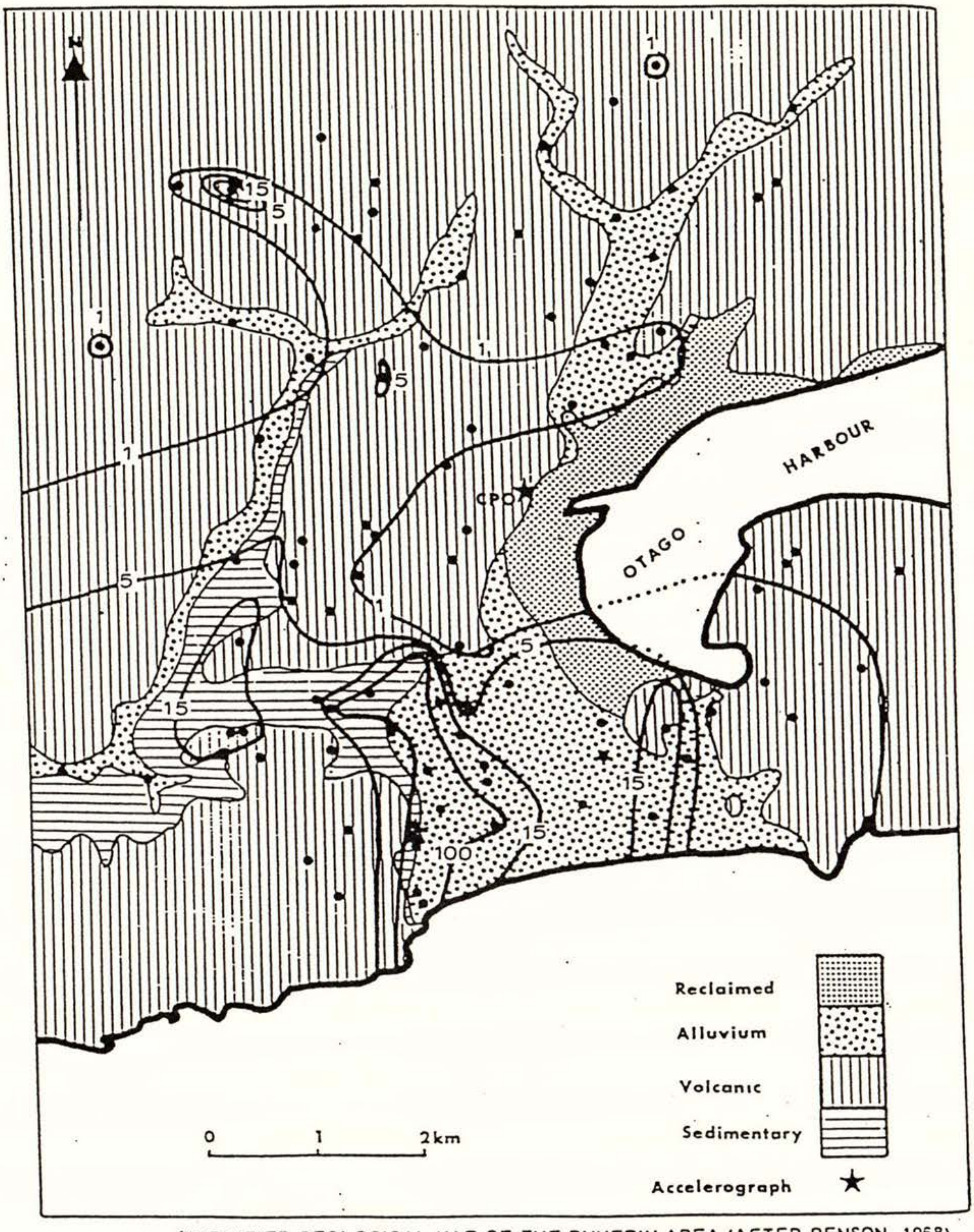
Bishop remarks on the contrast between damage in the southern suburbs with that on the alluvial ground in the northern part of the city. He relates this to three factors, in addition to the distance from the epicentre.




RELATIVE MICROSEISMIC NOISE LEVELS IN DUNEDIN CITY FOR THE FREQUENCY RANGE 5 TO 8 Hz. UNDERLINED VALUES ARE UNCERTAIN BECAUSE OF INTERFERENCE FROM LOCAL NOISE SOURCES. CONTOURS ARE DRAWN AT RELATIVE NOISE LEVELS 6, 20 AND 60. AREAS OF ALLUVIUM AND FILL ARE SHOWN SHADED.

**FIGURE 5.15**

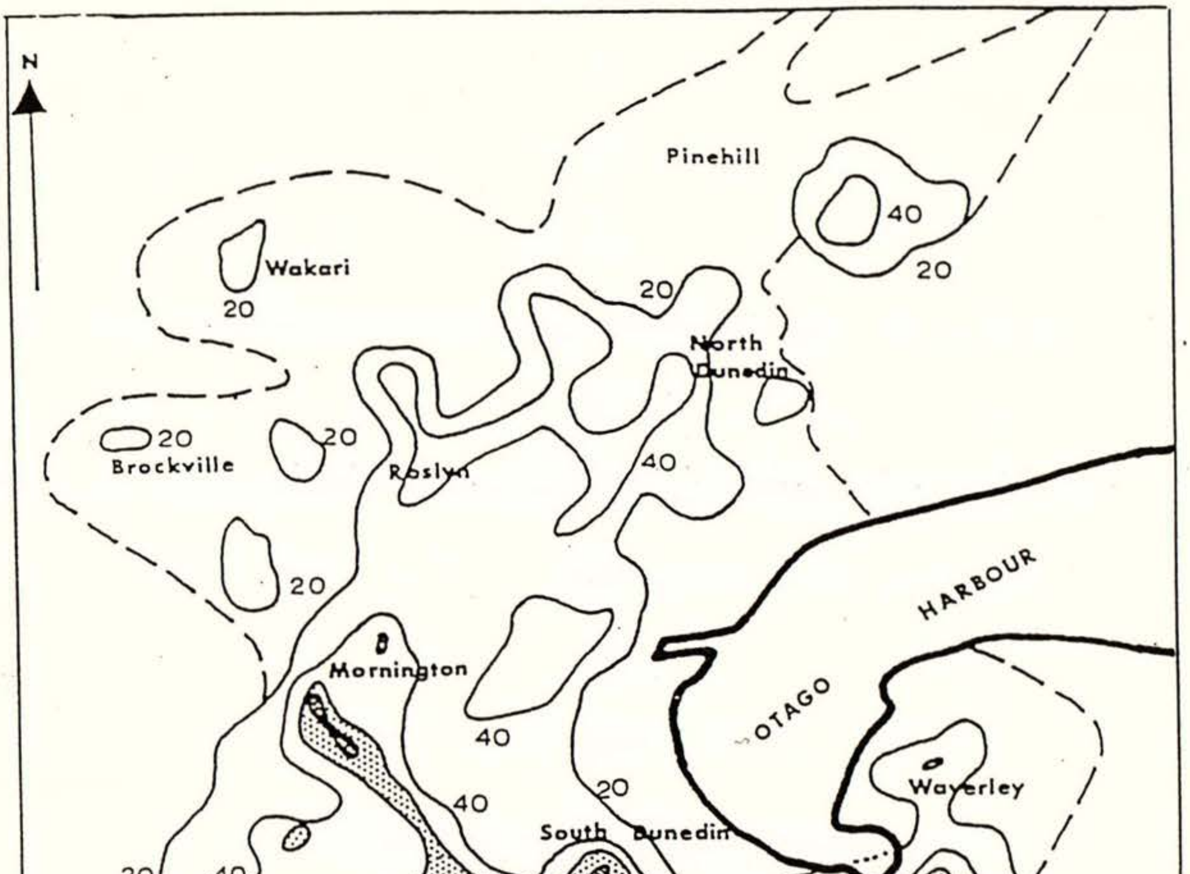
**Relative Microseismic Noise Levels in Dunedin City (After Adams & Kean, 1974)**



SIMPLIFIED GEOLOGICAL MAP OF THE DUNEDIN AREA (AFTER BENSON, 1968) WITH ISOSEISMALS BASED ON THE NUMBER OF FALLEN ITEMS IN GROCERY STORES (HEAVY DOTS). CONTOURS AT 1, 5, 15 AND 100 ITEMS.

**FIGURE 5.16**

**Isoseismal Based on Number of Fallen Items in Grocery Stores Resulting from 1974 Dunedin Earthquake (After Bishop, 1974)**




DENSITY OF DAMAGE CLAIMS RESULTING FROM THE DUNEDIN EARTHQUAKE.
CONTOURS AT 20, 40, 80, 160, 320 CLAIMS PER km$^2$. DASHED LINE INDICATES LIMITS OF BUILT-UP AREA WHERE IT EXTENDS BEYOND THE LOWEST CONTOUR. HEAVY DOTTED LINE INDICATES INNER ZONE OF MORE INTENSE CHIMNEY DAMAGE. UNSHADED RECTANGULAR AREAS ARE LARGE PLAYING FIELDS.

FIGURE 5.17

Density of Damage Claims Resulting From the 1974 Dunedin Earthquake (AFTER BISHOP, 1974)

a) A greater thickness of alluvium in South Dunedin.

b) The composition of the alluvium with fine sands silts and muds predominating in the south, and more gravels and boulder clays in the north.

c) The configuration and possibly the nature of the underlying bedrock.

The study confirms the first two factors and we agree that the third is possible. Bishop notes that both the volcanics and Tertiary sedimentary rocks appeared to have had a similar response during the 1974 earthquake, but he suggests that indications from "on the spot appraisals" indicate that slightly greater amplification was experienced on the sedimentary rocks. The topographic effects that are apparent in Figures 5.16 and 5.17 can also be assumed to be possible in contributing to the damage in the south west area of the alluvial region, where the alluvium/rock interface is steeply dipping.

## 5.6 SUMMARY

Information on the geological profile beneath the alluvial areas of Dunedin City have been compiled, and augmented by a resistivity survey. The limited available information is insufficient to clearly delineate the profile in three dimensions, but does confirm that the subsurface stratigraphy is complex.

A deep soil response model was used to predict approximate acceleration response spectrum at a number of sites. Three typical spectral shapes were identified, depending on the relative firmness of the alluvial soils, and impendence contracts within the soil profile. Present soils information does not allow any generalisation of response shape for Dunedin on an areal basis. It is clear that pronounced amplification effects will occur on the alluvial soils, but the provisions of the current loadings code NZS 4203:1992 are probably adequate, provided design spectrum for a deep soil or flexible site is used.

Effects observed from the 1974 Dunedin earthquake are generally consistent with the information compiled for this study and with the site response predictions.

# CHAPTER 6 : GROUND INSTABILITY: LIQUEFACTION

## 6.1 INTRODUCTION

Liquefaction is a common consequence of moderate to large earthquakes. It can be defined as (Youd, 1973).

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

If the liquefaction is severe and extensive enough, ground failure occurs which results in the damage of any structures supported on the ground. The liquefied soil will cause bearing failure and buildings and structures will settle and tip, or if they are buoyant may float upward. Liquefaction of a subsurface layer can cause both vertical and lateral displacement of surficial blocks of soil. If the area is on a gentle slope, or close to a free face such as an incised river channel or open drain, lateral spread of several metres can occur.

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

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 M6.3 was smaller, but unusually shallow).

The largest earthquake Dunedin has experienced is the 1974 Dunedin earthquake, of magnitude 4.9 a maximum intensity MMVII. There was no report of liquefaction.

A detailed study of the liquefaction potential in Dunedin has not yet been carried out. This study has reviewed the areas underlain with soil types potentially susceptible to liquefaction, and analysed the hazard for a few sites where sufficient geotechnical information is available.

Generally, the prediction of liquefaction involves two steps:

(a) Evaluation of liquefaction susceptibility, which is identifying those areas or layers which have the characteristics of a liquefiable soil (ie loose, saturated, cohesionless and above 15 m depth).

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

The liquefaction susceptibility and opportunity are considered together to determine the liquefaction potential or the relatively likelihood that liquefaction will occur. The liquefaction opportunity is based on the preceding sections of this report. The liquefaction susceptibility is considered in the next section.

## 6.2 LIQUEFACTION SUSCEPTIBILITY

Liquefaction usually occurs in water saturated, cohesionless, granular sediment, in a relatively loose state and at depths of less than 10 to 15 metres. The potential susceptibility varies according to its deposition history and age. Table 6.1 summarises this information for the type of materials found in the Dunedin area.

**TABLE 6.1 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 | | |
|---|---|---|---|---|
| | | <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)

Much of South Dunedin, the industrial area around the head of the harbour, and areas in the Kaikorai Valley and estuary are sited over Holocene age deposits. Areas of St Clair and St Kilda close to the south coast were probably deposited in a high energy beach environment and may well have a low susceptibility, but much of the area further north are probably estuarine or lagoonal deposits.

Figure 6.1 shows the extent of sands and silts in the city area. These areas are approximate only, and will include silts which are too fine to liquefy easily. Some particle size data is available, and the bulk of these samples show a soil gradation which falls within the envelope for easily liquefiable soils, as shown in figure 6.2. However the samples tested may not be representative of the overall soil types present.

The water table is generally high throughout the alluvial parts of the city. The water table was recorded on 41 borelogs, and of these 40% recorded at less than 1.0 m depth, 30% at between 1.0 and 2.0 m and 30% between 2 and 3.5 m depth. The lower water tables were all close to the hill through the central city or in North Dunedin. It can be concluded that the more susceptible soil types are saturated to within 1 - 2 m of the surface.

There is very little information available on the in-situ density of the soils. What data there is indicates that the softest soils are also the finest soils, with a reduced or low susceptibility to liquefaction, and the sands are either denser, or at a depth where soils above the layer would effectively confine the effects of any liquefaction. This is discussed in more detail in the following section.

## 6.3 LIQUEFACTION ANALYSIS

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

- Case histories, where earthquake magnitude is related to distance from the epicentre.
- In situ soil data, using empirical relationships between test data (usually SPT 'N' values) and depth.
- 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.
- Excess pore water pressure generation relating pore water pressure to dissipated energy, and hence earthquake magnitude and distance.

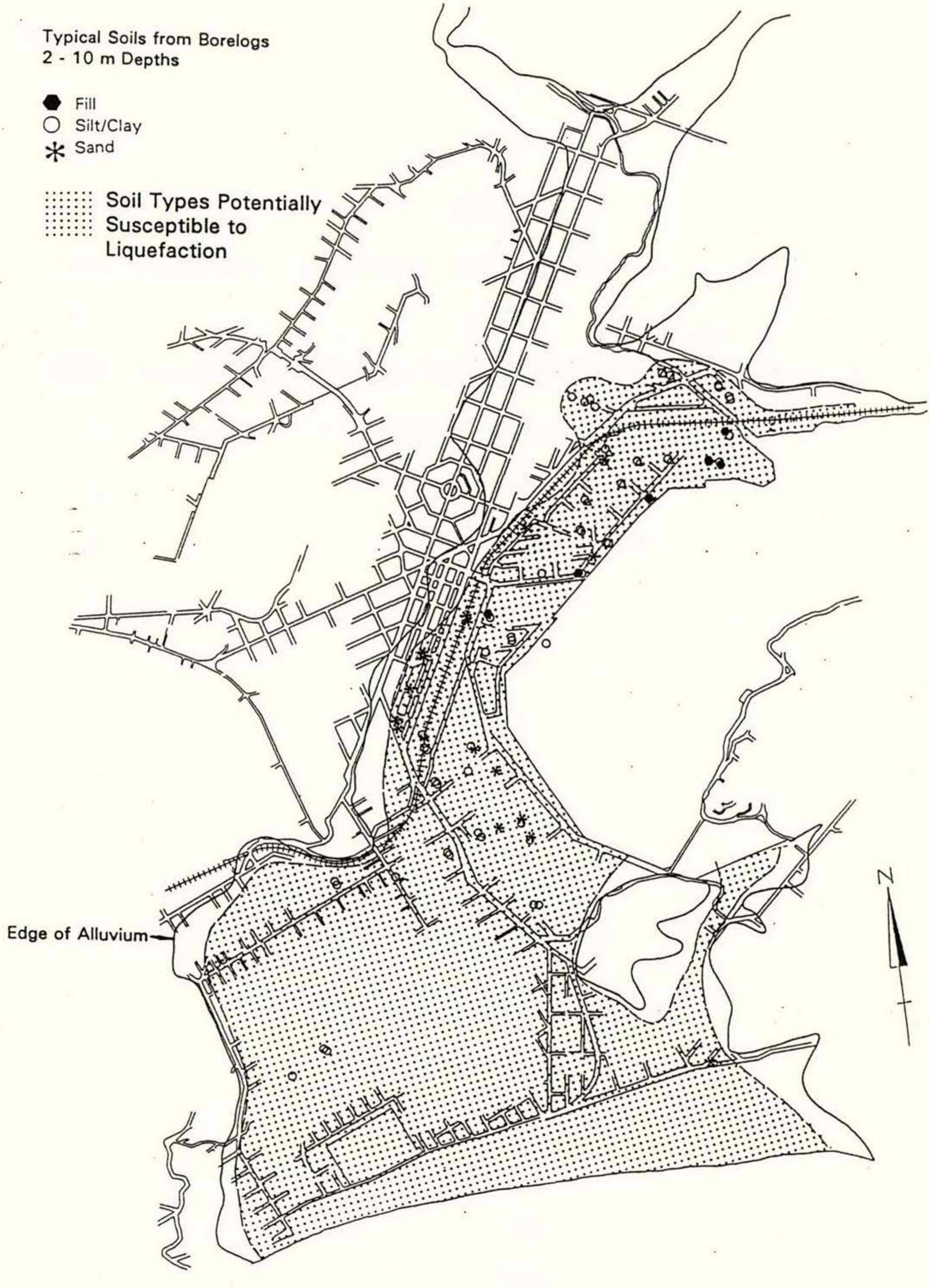
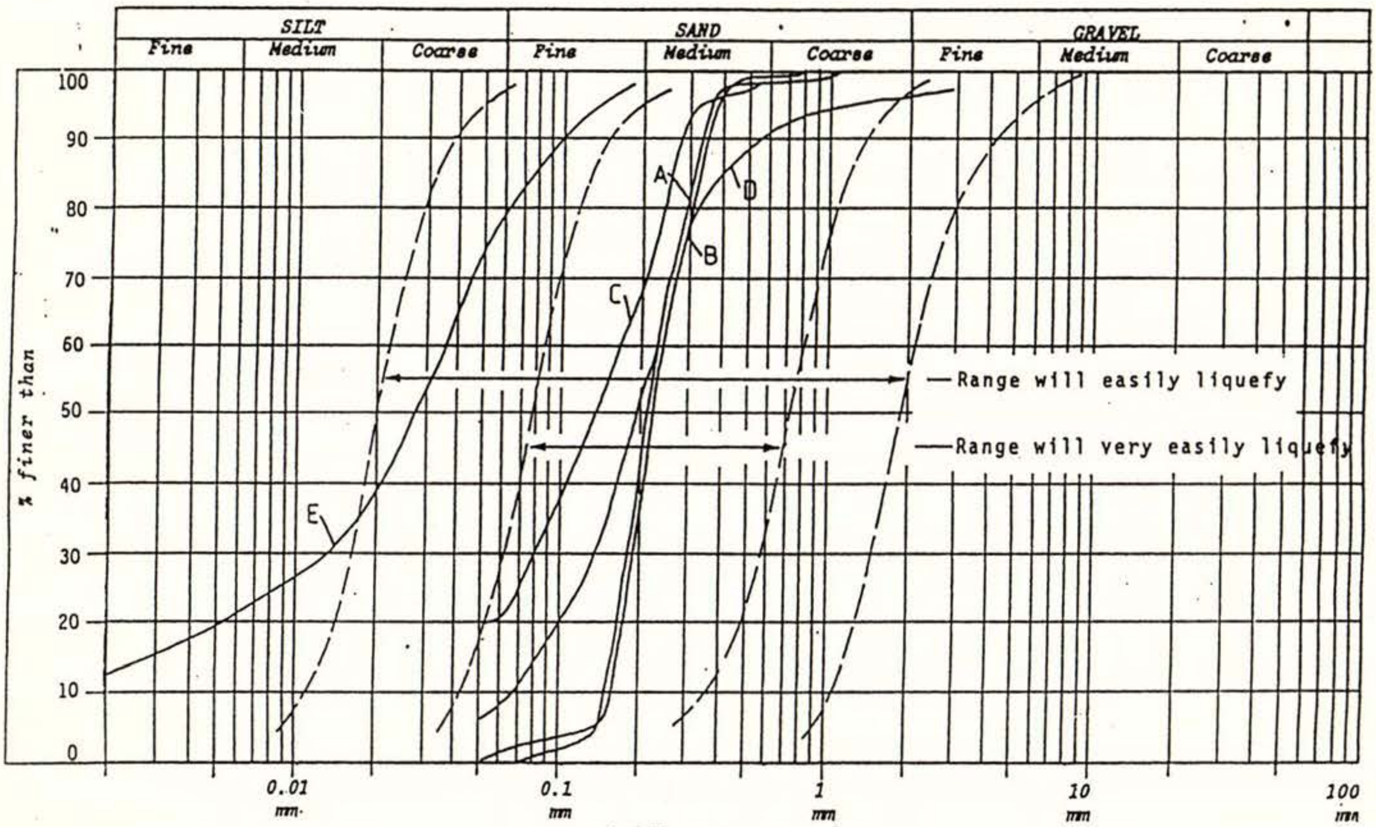



FIGURE 6.1

Map of Dunedin City Showing Areas of Soil Types Susceptible to Liquefactior




| Sample | Site | Depth (m) |
|---|---|---|
| A | Victoria Rd | 3.1 |
| B | Victoria Rd | 15.5 |
| C | Bay View Rd | 9.0 |
| D | Bay View Rd | 9.0 |
| E | Mouth of Water of Leith | 12.4 |

**FIGURE 6.2**

**Typical Particle Size Distribution Curves for Soils from Dunedin Sites and Liquefaction Susceptibility**

The first approach is exemplified by the work of Kuribayashi & Tatsuoka (1975) where Japanese liquefaction observations were plotted on earthquake magnitude vs epicentral distance. The line defining the distance from the epicentre to the farthest site of liquefaction was given as

$$\log_{10} R_{max} = 0.77M - 3.6$$

New Zealand data has also been plotted (Fairless & Berrill, 1984) and found to follow this rule. This gives some confidence in applying this relationship to New Zealand, and the line defining maximum distance for liquefaction is shown in Figure 6.3 which is from the earlier Figure 2.4, showing probable earthquake magnitudes for the known active faults in the Otago Region.

This indicates that liquefaction is likely to occur only with large earthquakes close to Dunedin on the closest faults (Akatore & Titri), or maximum credible earthquakes on the same faults further away, and on the Clifton and Dunstan faults.

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.

$$N_{crit} = N_0 o \; [1 + 0.125 \; (dx - 3) - 0.05 \; (dw - 2)]$$

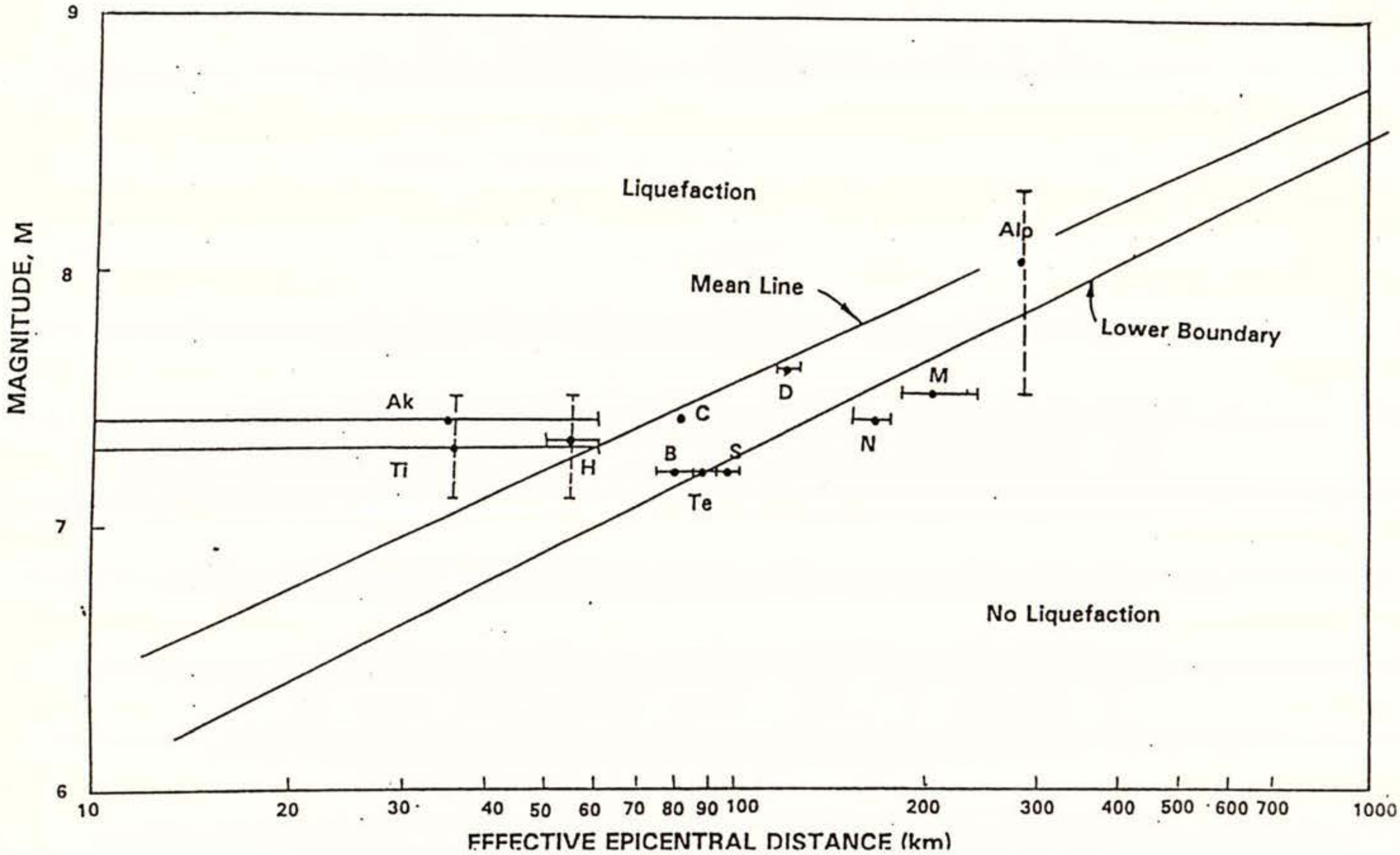
in which dx equals the depth to the layer being considered in metres, dw is the depth to the water table, and $N_o$ is a function of shaking intensity.

The third approach compares dynamic shear strength of the soil and earthquake induced stress. The method derived by Seed & Idriss (1983) uses the cyclic stress ratio, which is the ratio of the average cyclic shear stress $\tau_n$ developed as a result of the earthquake loading, to the initial vertical effective stress $\sigma_o$, and which can be computed from

$$\frac{\tau_n}{\sigma_o} = 0.65 \; \frac{A_{max}}{g} \cdot \frac{\sigma_0}{\sigma'_0} . r_d$$

where $A_{max}$ = maximum acceleration at the ground surface, $\sigma_o$ = total overburden pressure on the layer under consideration, $\sigma'_o$ = initial effective overburden pressure, 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.

Correlation with field and test data, using SPT N values as the measure of in situ density, gives a curve dividing cases of liquefaction from no liquefaction. This curve can be scaled to reflect the different number of cycles of stress induced by earthquakes of different magnitudes.




KEY

| | |
|---|---|
| Ak | Akatore Fault |
| Alp | Alpine Fault |
| B | Blue Mountain No. 1 Fault |
| C | Clifton Fault |
| D | Dunstan Fault |
| H | Hyde Fault |
| M | Moonlight Fault |
| N | Nevis - Cardrona Fault |
| S | Spylaw Fault |
| Te | Teviot Fault |
| Ti | Titri Fault |

**FIGURE 6.3**

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

The fourth method, rather than using a ground acceleration and calculated shear stress is based on pore pressure increase being proportional to the density of dissipated seismic energy (Berrill et al, 1988). The soil density is characterised by the SPT N value, and the seismic energy by the value of $[r(\sigma'_o)^{3/2}\ 10^{1.5M}]$ where r is the distance from the earthquake source, $\sigma'_o$ is the initial effective overburden stress $\sigma'_o$ and M is the earthquake magnitude. Liquefiable and non-liquefiable cases are separated by the line

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

This equation can be rewritten in the form

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

where the right hand side is determined by the seismic event and the left hand side is derived from the specific site data, which can be plotted as a function of depth at the site. Values of r and M can be selected for a range of seismic intensities, from the seismicity model for Dunedin. The right hand side can therefore be plotted as vertical lines on the $N^2\ (\sigma'_o)^{3/2}$ v.depth profile, representing liquefaction thresholds for different shaking intensities. These thresholds can be assigned return periods from the seismicity model.

For this study, three representative sites for which adequate borelog and insitu test data was available were selected in areas of susceptible soils and analysed for liquefaction potential. The results are shown on Figures 6.4 - 6.6, and summarised here.

**TABLE 6.2 Analysis of Liquefaction Potential for Three Sites**

| | Analysis Method | | |
|---|---|---|---|
| Site | Chinese | Cyclic Stress | Energy |
| Timaru Street | MMVII at depth 5 - 9 m | Liquefiable | Not Liquefiable |
| Victoria Road | MMVIII 12 - 14 m | Liquefiable | Not Liquefiable |
| Leith Stm Mouth | MMVIII but soil may be too fine | Possibly some Liquefaction at depth | Probably not Liquefiable 7.5 - 9.0 m for Tr = 100 year |

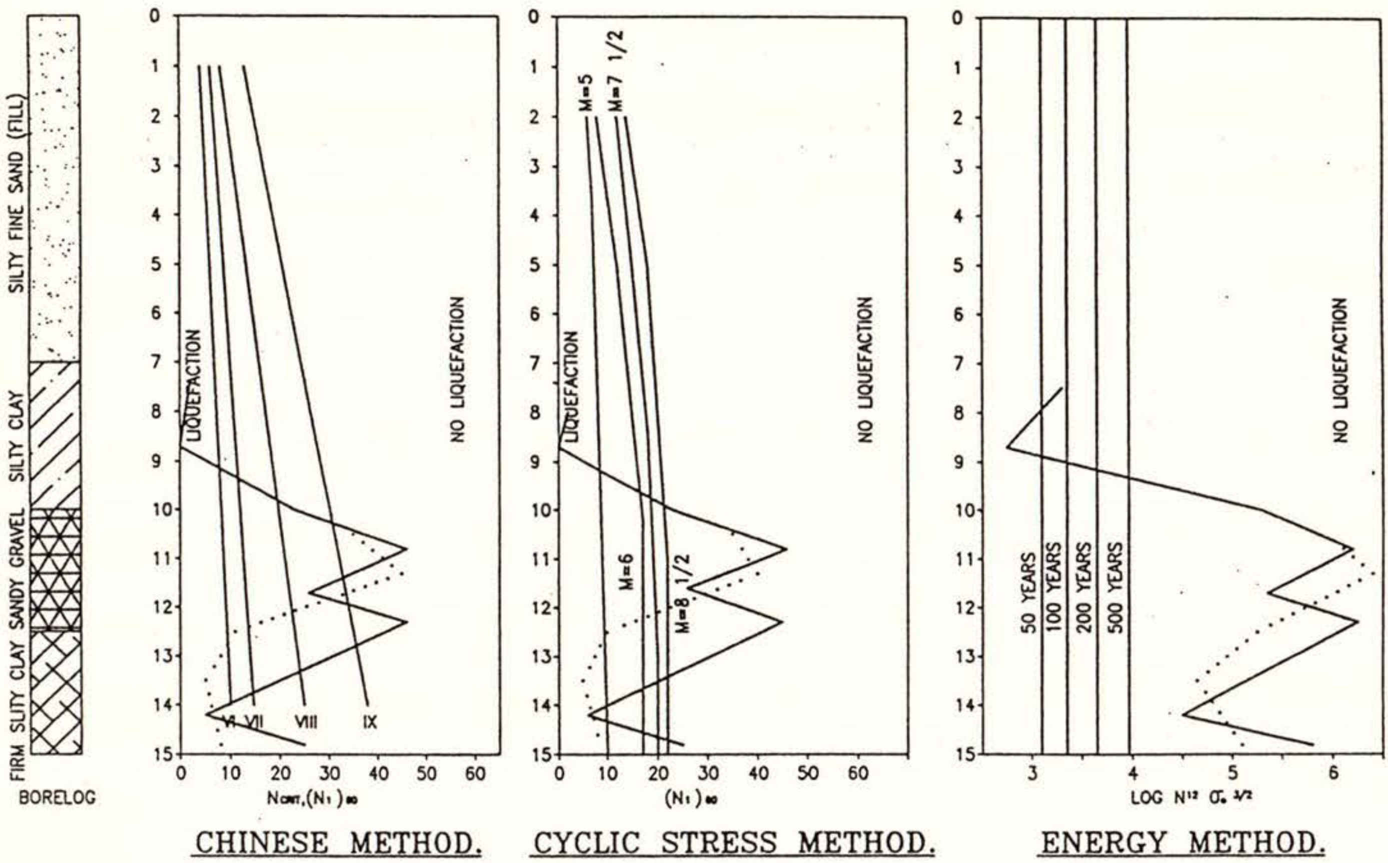



**FIGURE 6.4**

**Liquefaction Analysis**
**Site No. 200, North Dunedin Reclamation**

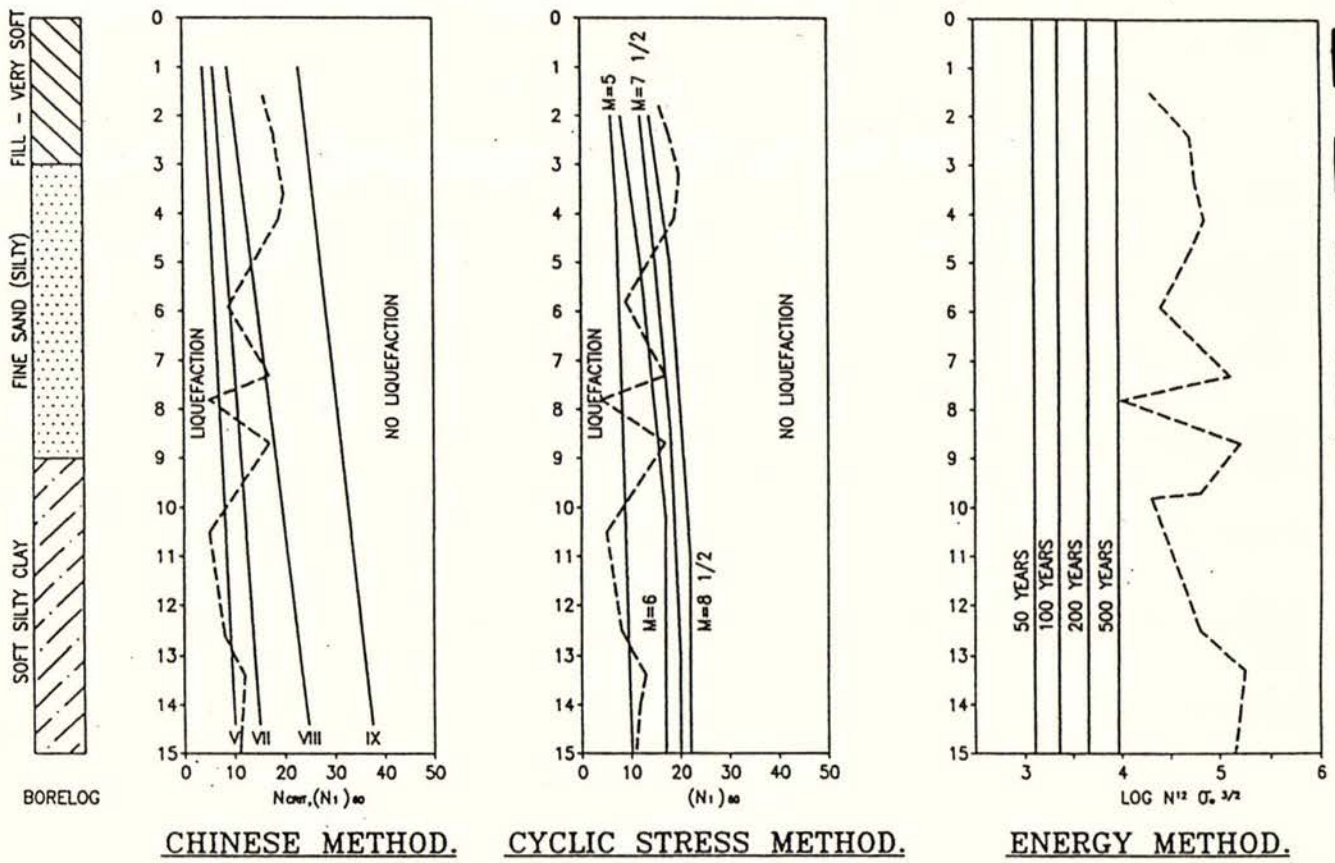



**FIGURE 6.5**

**Liquefaction Analysis**
**Site No. 158, South Dunedin Reclamation**

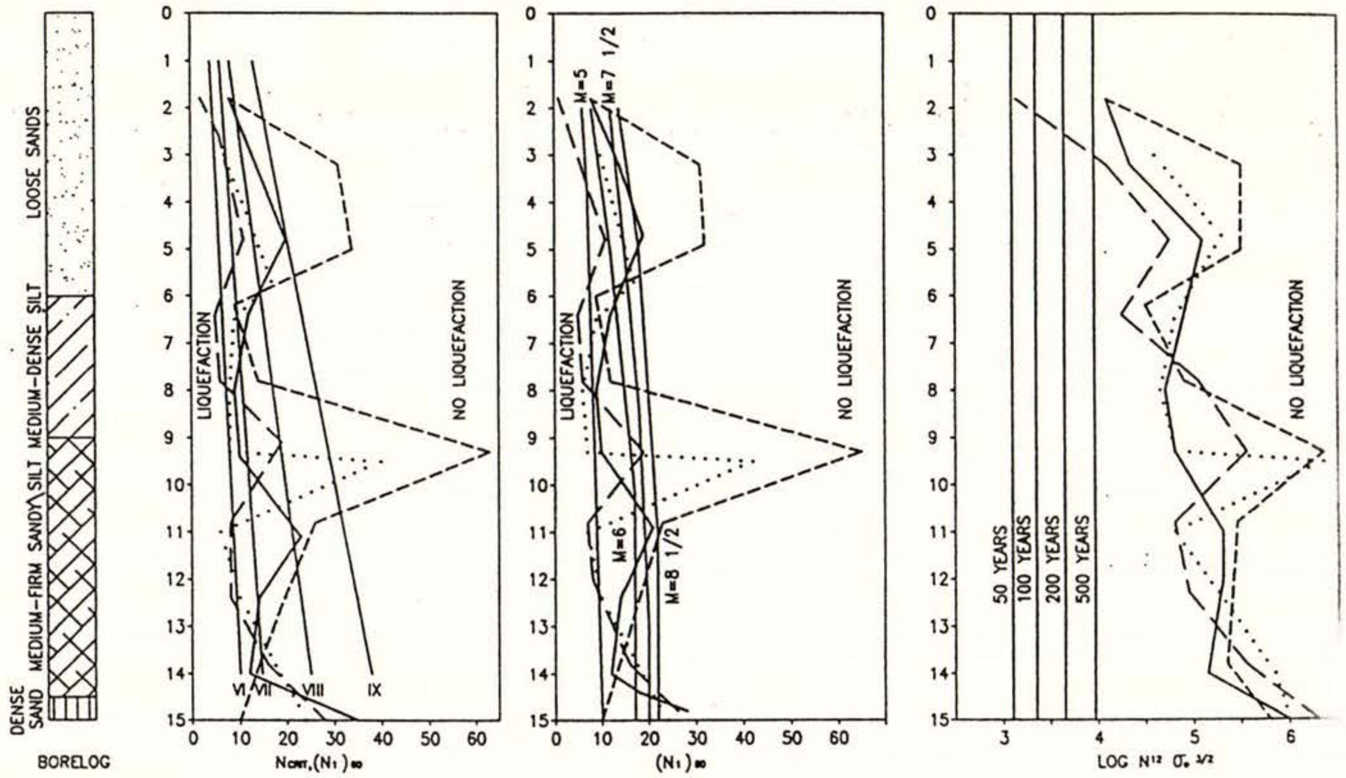



**FIGURE 6.6**

**Liquefaction Analysis**
**Site No. 143, Victoria Road, St Kilda**

The Chinese method indicates liquefaction at intensities of MMVIII or greater, with a return period of about 450 years. The energy methods indicates virtually no liquefaction even for 1000 year return period earthquakes.

The Cyclic Stress method indicates some liquefaction for a peak ground acceleration of 0.2g. Taking the peak ground acceleration as 40% of the peak spectral acceleration, 0.2 g peak ground acceleration has a return period of about 125 years.

There is therefore some variation in the estimated likelihood of liquefaction between the methods. This variation is common, and not related to any specific conditions in Dunedin. One reason for this variation is probably due to the effects of shaking duration in liquefaction as duration is not explicitly included in the analysis methods. A peak ground acceleration in excess of 0.2 g may not induce liquefaction unless the shaking is of sufficient duration. The tentative conclusion is that liquefaction may occur on some sites, but probably only during large earthquakes with long return periods.

The analysis carried out above is specific for those sites, and any major development within the areas outlined in Figure 6.1 should include an assessment of liquefaction potential in the site investigation.

## 6.4 SUMMARY

Large parts of South Dunedin are underlain with saturated sands and silt soils which are potentially susceptible to liquefaction. Information necessary to analyse liquefaction hazard such as particle size distribution and in-situ density is limited to a very few sites. Liquefaction analysis carried out on three such sites suggests that liquefaction is unlikely to be widespread, but may occur in some areas during large earthquakes with correspondingly long return periods.

# CHAPTER 7: SLOPE STABILITY AND EARTHQUAKES IN DUNEDIN

## 7.1 INTRODUCTION

Dunedin has a significant number of large deep seated landslides which have developed particularly in the Tertiary aged clay rich sediments of South Dunedin. The best known of these is the Abbotsford Landslide which failed in 1978 causing considerable damage. In addition there are a number of smaller slides which have formed in or on the volcanic materials underlying the central and western city, for example the Albany Street slide just north of the central business district of Dunedin.

Earthquakes can trigger most forms of mass movement. In a comprehensive study by Keefer (1984) seismic triggered mass movement was divided into 3 categories and the relative frequency compared. Categories defined by Keefer in order of relative frequency include Disrupted Slides and Falls where chaotic masses of small soil or rock blocks move down slope (this includes shallow rock and soil slides and falls and soil avalanches).

The second category which was less frequent on a numerical basis but which generally involved movement of larger total areas he classed as Coherent Slides (soil and rock slumps, soils and rock block slides, slow earthflows). Abbotsford and many of the other larger mass movement features in Dunedin belong to this category.

The least frequent group of seismic triggered mass movement were the Lateral Spreads and Flows involving liquefaction and fluid like flow on relatively low gradients.

In the Christchurch project (Elder et al, 1991) the most significant potential hazard belonged to the first of Keefer's categories and was considered to be landslides triggered in the relatively thick loessial soils common on Banks Peninsula. Rock roll and rock fall was a significant additional hazard in the eastern suburbs and belong to this same category. Given the small areas of the typical slide and fall types and their widely scattered distribution no attempt was made to map or zone the Christchurch hill areas for seismic triggered mass movement at that time.

The critical importance of pre existing moisture levels in shallow fine grained hill soils subject to pronounced capillary suction was demonstrated in the Christchurch project. Loess slopes in the region were normally only vulnerable to major earthquake triggered failure for 2 - 3 months of the year. This effectively reduced the likely damage return periods significantly.

In Dunedin the deeper seated nature of most of the landslides and the generally higher rainfall results in less dramatic watertable fluctuations and a correspondingly increased vulnerability for a given triggering acceleration. The most significant risk is associated with the Coherent Slide category. This category is directly linked to the underlying geology and the risk extends over broad areas making the areas of most risk relatively easy to define.

## 7.2 RATIONALES FOR SEISMIC MASS MOVEMENT HAZARD ZONING

Mass movement is used here as a general term which includes all the various forms of slope failure induced by gravity (slides, falls, topples, flows etc). While an earthquake can trigger mass movement there must be pre existing conditions which result in the slope being at or near equilibrium prior to any earthquake induced ground acceleration. These conditions include some combination of slope angle, soil or rock shear strength and groundwater conditions.

Normally these conditions by themselves are sufficient to result in at least occasional static (non earthquake induced) slope failures, generally as groundwater conditions reach their highest levels in particularly wet cycles. It therefore follows that, in the absence of documented local records of actual earthquake induced mass movement, the best guide to likely vulnerable areas is provided by the distribution of static mass movement features.

There are factors which complicate this general relationship. In the case of existing landslides which have already moved a significant distance (i.e greater than around 300 mm) the basal shear zone has gone through a modification as the soil particles realign and the most efficient and least resistance failure surface is established. The resulting shear strength is referred to as the residual strength and this strength is unlikely to reduce much further despite continued displacements.

An existing landslide at residual strength subjected to a large enough earthquake induced ground acceleration will move in response to this acceleration but will stop moving once the acceleration reduces below the movement threshold. The resulting earthquake induced displacement may actually be quite small with corresponding damage confined to the slide perimeters where the relative movement is concentrated.

In contrast a slope near equilibrium which commences to move for the first time during an earthquake may not stop after the earthquake accelerations drop below the threshold required to initiate the movement because in commencing to move the shear strength has been reduced to residual levels. The slope displacements may be large and the internal disturbance correspondingly high thus the most serious and substantial damage from earthquake induced mass movement often occurs in first time slides.

As noted above while existing landslides may not accelerate to complete failure in an earthquake displacement concentrated at the slide perimeters is likely. In addition the new first time failures frequently develop near or adjacent to the existing landslides reflecting the unfavourable pre existing conditions in the area. Therefore the rationale that existing slides which have been active under static conditions can be used to indicate areas which are most vulnerable to seismic triggered sliding generally holds.

For rock falls, rock roll and topple failures the link between static and seismic triggered events is slightly different. In many instances these processes only commonly occur during earthquakes which in effect clean off the weathered and relaxed material which has accumulated since the last significant earthquake event. In these cases properties located at cliff edges or in the run out zone of these processes may have no history of previous problems but still be vulnerable to future seismic damage. In these cases a general study such as this can only consider all properties located immediately above or below high rocky slopes to be potentially at risk.

## 7.3 ZONING DUNEDIN CITY

Figure 7.1 is a map of Dunedin at 1:50,000 showing three levels of seismic mass movement hazard. Considering each zone in turn and commencing with the highest risk class:

**Zone 3. Moderate - High Risk**

This includes the areas underlain by less stable Tertiary rocks plus the known landslides, and areas immediately adjacent to these known landslides, which are underlain by volcanic rocks and soils.

In the Tertiary rocks the less stable units include the Abbotsford Formation, the Burnside Mudstone and the Saddle Hill Siltstone (stratigraphic nomenclature after McKellar, 1990). In addition the interbedded Green Island Sand and the Steel Greensand have been included here as potentially high risk. This reflects the stratigraphic location of the Green Island Sand above the Abbotsford Formation such that block slides develop within the Abbotsford Formation which carry the Green Island Sand down dip. The Abbotsford Landslide of 1978 was one such failure and the damaged houses were all built on Green Island Sand.

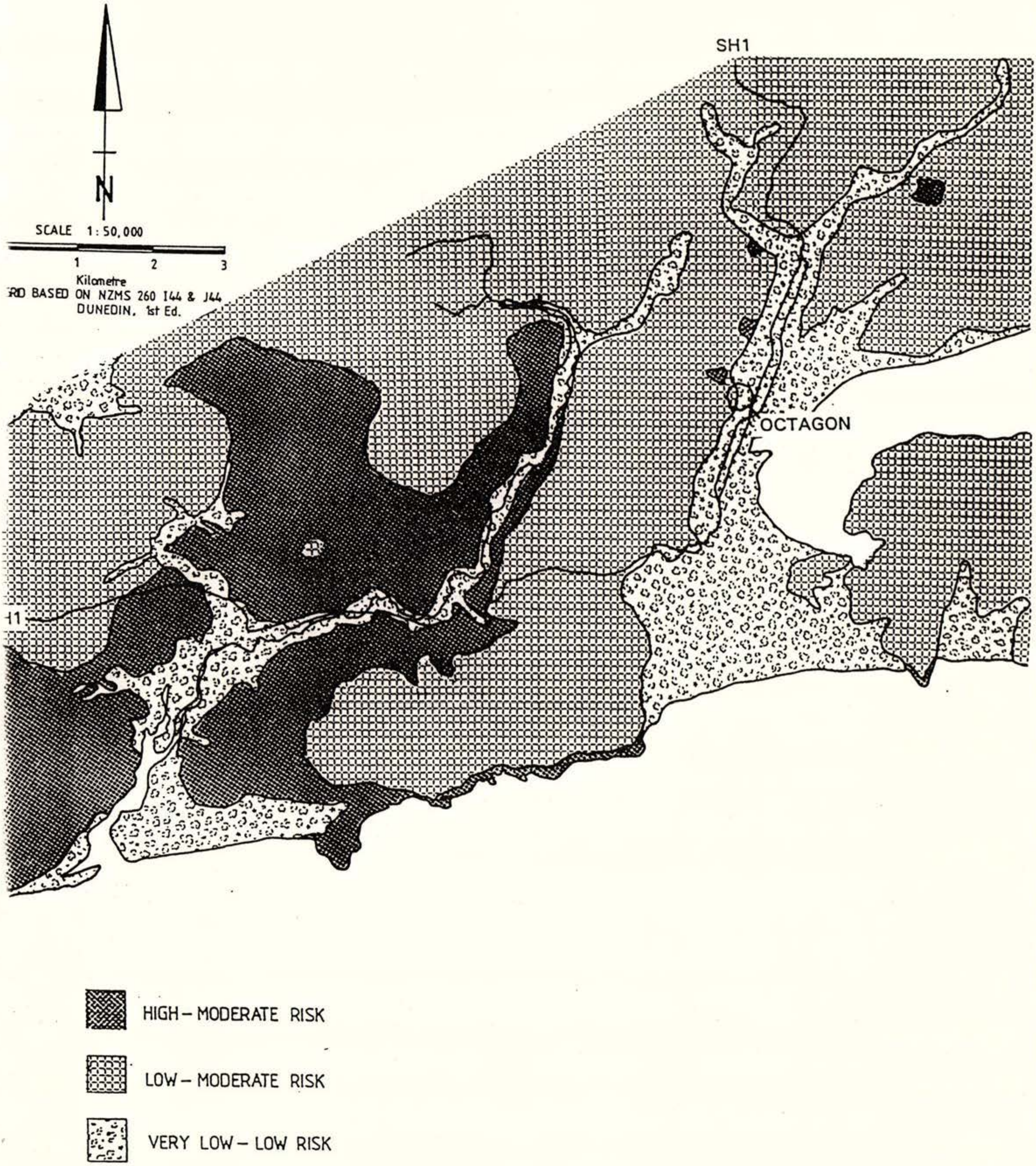



FIGURE 7.1

**Seismic Triggered Mass Movement Hazard in Dunedin**

The Steel Greensand at Saddle Hill is also destabilised by the underlying Saddle Hill Siltstone and includes various earthflows which originate above it in the Abbotsford Formation.

The lower areas of the Caversham Sandstone are vulnerable to movement originating deeper in the Burnside Mudstone and are included here within the higher risk category.

Further to the north in the central and northern city area, several existing landslides have been built over which have developed in the volcanic rocks and soils. These slides and their immediate environs have been included in zone 3 in addition to areas located immediately above high cliffs or in the runout zone from these cliffs or bluffs.

**Zone 2. Low - Moderate Risk**

This zone includes the bulk of the hill areas of moderate to steep gradient underlain by volcanic materials. Specific slope studies carried at a detailed scale (e.g the work carried out by Leslie, 1974, for the southern part of the Otago Peninsula) indicates considerable variation in stability within these materials with pyroclastic rocks and tuffaceous trachyte much more prone to failure due to deep weathering and abundant clay minerals. Leslie also noted 97% of the slides he encountered were on moderate slopes (12 - 28 degrees) rather than steeper slopes where presumably harder volcanic flow rocks dominated (1% of failures).

Given the scale of mapping involved in this project no attempt has been made to further subdivide this low - moderate risk category.

**Zone 1. Very Low - Low Risk**

This zone includes the flat and near flat areas of the city underlain by alluvium or volcanics. In south Dunedin it also includes gently sloping valley floors with gradients less than 3 degrees (the minimum known slide gradient in the Abbotsford Formation, McKellar, 1990).

## 7.4 DISCUSSION

A significant area of South Dunedin is vulnerable to possible deep seated seismically triggered landsliding in the event of a major earthquake. The absence of large landslides developing or reactivating in the 1974 earthquake suggests the triggering event must be of greater intensity or duration. Although the 1974 earthquake was considered as a potential trigger for the Abbotsford Landslide by the Commission of Enquiry there was no direct causal link established. The corresponding return period for an event of equal or greater intensity is about 100 years.

Away from South Dunedin there are local areas where reactivation of existing landslides is possible. New failures may develop near these existing slides but in general smaller areas are vulnerable in comparison to South Dunedin.

Pre-existing groundwater conditions are less important in the moderate to high risk areas of Dunedin than they are, for example, in Christchurch. However, there would be a significantly greater chance of major failures developing if the earthquake were to occur in the late winter and early spring when groundwater levels are generally highest.

## 7.5 SUMMARY

Large earthquakes in Dunedin could trigger slope movements. Existing landslides would probably move with only small displacements with damage confined to the slide perimeters. New landslides may be initiated if other conditions such as slope angle, soil strength, and groundwater pressures already result in the slope being at or near equilibrium. Such areas are difficult to determine, and the best guide is provided by the distribution of static mass movement features. On this basis, the Dunedin areas has been zoned into three risk categories. A significant area of South Dunedin is potentially vulnerable, but probably only during earthquakes of a return period in excess of 150 - 200 years.

# CHAPTER 8: POTENTIAL DAMAGE:

## 8.1 INTRODUCTION

This chapter suggests the type and extent of damage that could result from a major earthquake affecting Dunedin. Any such damage scenario is conjecture, but can be realistically based on earthquake damage experienced elsewhere in New Zealand and the world, combined with knowledge of the local conditions and the type of ground motion that can be expected. Local conditions apply not only to geological and topographical features of Dunedin, 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, particularly for areas of alluvium. The area of most intense shaking was largely on the Rangitaiki Plains with a considerable depth of alluvial material overlying bedrock, and similarities in the response spectra, and amplification, can be expected with the response possible in the alluvial areas of Dunedin. The Edgecumbe earthquake of magnitude 6.3 was only of moderate size, but maximum intensities of up to MM X were recorded at Edgecumbe. Some of the damage descriptions below are 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 Dunedin. This would be an event of about 450 year return period, allowing for an increase of one intensity unit in parts of Dunedin underlain with soft alluvium. The probability of occurrence of such an event within any 50 year period is about 10%.

## 8.2 STRUCTURES

Few buildings from before 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. Dunedin has a higher proportion of older building stock than most other New Zealand cities, many of them of masonry or brick construction, and considerable damage is likely with strong shaking.

Seismic provisions for buildings were introduced in 1935 building bylaws, and subsequently upgraded in 1955 (NZSS 95), 1965 (Chap. 8, NZSS 1900), 1976 (NZS 4203), and 1992 (NZS 4203).

Structures built to the 1976 Loadings Code can be expected to survive a large earthquake with limited damage. The response accelerations that the buildings would be subjected to are not significantly greater than those allowed for in the codes, even on the alluvial areas of Dunedin (refer Figure 5.13). For the firmer alluvial sites, the spectral accelerations between 0.2 and 0.5 second periods may be up to twice the code spectral values, but the commentary to the 1992 Loadings Code (NZS 4203) states:

> "The majority of New Zealand buildings have a fundamental period less than 0.45 s. For many of these structures particularly in domestic construction, the dynamic characteristics, which depend on soil response and soil-structure interaction, cannot be readily established. Experience also indicates that these structures tend to have sources of both strength and deformation, not readily quantified in a simplified analysis, which increase their ability to survive major earthquakes.
>
> Furthermore, the spectral displacements at short periods are rather small (in the order of 10 mm for a period of 0.2 sec). The effects of such small displacements are such that collapse is unlikely in the region of maximum acceleration response for normally proportioned members, so reduced strength is admissible. For these reasons, the spectra for rock and intermediate soils have been truncated at a period of 0.45 sec" (and 0.6 sec for flexible soil sites).

Therefore, provided the buildings on the alluvium areas have been designed for the code spectra for flexible soil sites, they can be expected to perform well. The other difference between the predicted spectra and the code spectra is for the more flexible soil sites with secondary spectral acceleration peaks in excess of 1.0 second period. Only long period structures (eg buildings more than 10 - 12 stories high) would be affected by these secondary peaks, and it is unlikely that there are many on the soft sites in Dunedin.

Variations in the amount of damage can be expected to occur across Dunedin, as experienced in the 1974 Dunedin earthquake (refer section 5.6 and Figures 5.15 - 5.17). This is likely both from topographic focusing effects, and from the different shaking that could be expected on the deeper or softer alluvial areas. While the modelled response accelerations do not differ significantly from loading code spectra, the amplifications at the longer periods are likely to cause greater velocities and displacements, and inertial damage could be more severe.

The 1987 Edgecumbe earthquake probably gives a good indication of the type and extent of damage to domestic buildings likely for felt intensities up to MMX. 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 for where MM X isoseismal exists) |

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. These damage ratios for MM > VI are significantly lower than previous studies. This may reflect the probable attenuation effects of the deep alluvium of the Rangitaiki Plains for the period range less than 0.1 - 0.2 sec that most affect New Zealand houses. Another likely reason is the age and construction type, and as Dunedin, and particularly South Dunedin, has an older housing stock, higher damage ratios can be expected for Dunedin.

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. Less than 50 houses suffered substantial structural damage that made them uninhabitable, out of a total of 5,300 dwellings in MM VII 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, as 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 5, but is likely to occur in parts of Dunedin. 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 the South Dunedin area.

Chimneys were particularly vulnerable, as demonstrated before in previous New Zealand earthquakes, including the 1974 Dunedin earthquake, and several free-standing stoves and fireboxes were dislodged due to insufficient fixing during the Edgecumbe earthquake. Similar damage can be expected in Dunedin for equivalent felt intensities, and if this occurred during winter, many house fires could result.

## 8.3 WATER SUPPLY AND RETICULATION

The Dunedin Water Supply is a relatively complex system with six supply sources, three bulk storage reservoirs, five treatment plants, seven pumping stations, fifteen service reservoirs, and over 600 km of water mains. The main supply sources are the Ross Creek Catchment within 3 km north of the Octagon, the Waitaki - Leith Catchment feeding Sullivans Dam 7.5 km north of the Octagon, Silverstream Catchment with a pipeline and race 30 km long, Deep Creek Catchment supplying a 64 km long pipeline, Deep Stream Catchment with a 58 km long pipeline, and the Taieri Bores near Outram, 20 km west of Dunedin.

A major earthquake affecting Dunedin is likely to affect at least some of the supply sources, especially given the long supply routes into the city. Over half of the total supply crosses the Taieri River on one large span pipe bridge which could conceivably lose an abutment during a large earthquake. The Taieri aquifers may be damaged, with variations in well yield, and turbidity, and well pumps are vulnerable to any power surges that may occur as equipment in the electrical supply is damaged. Power supply to the pumping stations may be cut, and the pipelines could be damaged by slips. The structural strength of the dams and reservoirs is not known, but some damage could occur, particularly to the older structures (Ross Creek Reservoir was commissioned in 1867. Sullivan Dam dates from 1916, and the Southern Reservoir was completed in 1923).

However the number of different supply sources, the interconnections between them, and the general redundancy in the supply system means that it is unlikely that the city would lose more than a proportion of its water supply.

The water reticulation is likely to be significantly damaged in the softer alluvial areas. During the 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 South Dunedin. 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 Dunedin, 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 unlikely as no active fault traces are known within the city area.

Pipe damage in the 1989 San Francisco (Loma Prieta) Earthquake was much lower relatively than in Edgecumbe. This lower of level would still mean about 20 to 30 breaks to the water mains on the alluvial areas. The damage in South Dunedin/St Kilda area is likely to be enough for the system to lose pressure and water. It would be possible for the service reservoirs in the area to empty through the pipe breaks and hence many mains would need to be turned off to prevent the water loss. Either way, water availability for fire-fighting could be severely limited, in an area of older housing on narrow sections.

## 8.4 SEWERAGE RETICULATION AND TREATMENT

All development within the city boundaries is provided with piped sewerage except for isolated housing within the rural areas. In the harbour area, there are water pollution control plants and outfalls at Sawyers Bay, St Leonards, Company Bay, and Broad Bay. The main urban area is served by a gravity system converging on the Musselburgh Pumping Station, where the sewage is pumped to the treatment plant in Tahuna Road, with a gravity outfall at Lawyers Head.

A separate system serves Green Island, Fairfield, Wingatui and industry at Mosgiel via a 9 km main pipeline with five pumping stations, treatment plant and outfall near Waldronville.

Both the Tahuna Road treatment plant which was commissioned in 1981, and the Green Island system, commissioned between 1976 and 1990, are likely to survive a major earthquake with only moderate damage.

Of more concern is the reticulation system. It is estimated that 70% of the sewers in the city area are more than 50 years old, and construction is likely to be ceramic or brick, which may not perform well in an earthquake. The type of damage is likely to be similar to the water reticulation, but older brick sewers, and the larger diameter trunk sewers through South Dunedin could suffer more serious damage, requiring a long time for full reinstatement. The main intercepting sewer is about 2 m in dia, laid in the first decades of this century and in soft soils several metres below mean seal level. Even a short length of failure would require major repair efforts and could cause substantial surface sewage ponding for some days.

Liquefaction may occur in places within South Dunedin, and if this occurred around a large diameter sewer, or manholes, then gross distortion can be expected. Floatation of half full pipes and manholes is probable in these circumstances, resulting in breakage of joints and lateral connections, and the raising of sewer manholes through the ground surface.

## 8.5 ELECTRICAL SUPPLY AND TELECOMMUNICATIONS

The electrical supply to Dunedin City comes through the two major Transpower substations at Halfway Bush and Portsmouth Drive. 33 kV lines feed radically from these nodes to about 20 city substations. The city substations are interconnected, but at 6.6 kV and 11 kV limiting maximum interconnection capacity at about one third of the normal capacity.

Although substation equipment has been designed for earthquake resistance in recent years, damage at Edgecumbe in 1987 indicates that failures will still occur. This is particularly likely at the older substations, and substations sited on flexible soils where inertial forces on heavy equipment such as transformers can be severe. The Portsmouth Drive substation is on soft reclaimed land, and damage here could cut power to a large proportion of the city. (During the relatively small 1974 Dunedin earthquake, shaking tripped high tension switches in this substation, interrupting power supplies to the Corstophine area for 45 minutes). Other older city substations at Ward Street, Andersons Bay and Hillside could be vulnerable to damage.

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.

Telecommunications are unlikely to be badly damaged. The main exchange in the central city is on a rock site, and the South Dunedin exchange was built in the late 1980's to high seismic specifications. Other exchanges are on hill or firm alluvial sites. Telephone cabling could be damaged, particularly in soft soil areas, but fibre optic cables are probably not vulnerable. Telecommunication links out of the city are by microwave and fibre optic cables, and it can be expected that at least one of these links would remain operative.

## 8.6 TRANSPORT ROUTES

Considerable damage to transportation routes can be expected in a severe earthquake, with cracking and settlement affecting road surfaces. In the southern 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 6.1 show areas of soil types potentially 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.

Embankments on soft soils would be prone to slumping and fissuring. Potentially vulnerable embankments include the railway south of Andersons Bay Road and along the harbour towards Port Chalmers, and the Andersons Bay causeway.

Settlement and slumping of bridge approaches can be expected to be widespread, but most bridges will have at least some seismic resistance and will probably remain in a useable state.

Slips and rockfalls from steep cut batters could cause short term access problems.

## 8.7 TSUNAMIS

Fifteen tsunamis are reported to have been detected on the New Zealand coastline since 1848, with twelve of these affecting the east coast. Any major earthquake in the Pacific Ocean region is capable of generating tsunamis which could cause damage to New Zealand. 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 are less likely to be a hazard to the Dunedin coastline with the low seismicity of the eastern Otago area. It is conceivable that a tsunami could be generated by a large earthquake on the Akatore fault.

The effects of a tsunami on Dunedin are probably not great. The narrow harbour entrance, and the narrow channels at Portobello are likely to restrict any effects in the harbour to relatively small waves or surges. Vulnerable areas on the south coast would be St Clair, and low lying areas around the entrance to Tomahawk Lagoon and the Kaikorai Lagoon.

## 8.8 SUMMARY

Strong seismic shaking in Dunedin can be expected to produce widespread damage to the city. Structures built to recent Loading Codes will probably survive well, even on the alluvial areas provided allowance has been made for the flexible nature of the sites.

Services will be affected, but the nature of the supply and reticulation networks may make Dunedin services somewhat less vulnerable than other major cities. The length of water supply lines increases their vulnerability, but the number of supply sources and interconnectedness will reduce the exposure of the overall system. Damage to watermains in the South Dunedin/St Kilda area could limit water supply for fire-fighting purposes. Damage to the older interception sewers may cause public health problems in the first few weeks after a major earthquake.

A full engineering lifelines study for the city is recommended.

# CHAPTER 9: RECOMMENDATIONS

## 9.1 RECOMMENDATIONS

Although this study has confirmed that Dunedin has a low seismic hazard relative to much of the rest of New Zealand, there is still the probability of a large earthquake seriously damaging the city. Further action should be taken to enable the city to be better prepared for such an event.

1. All structures built on alluvial soils should be designed for the deep soil spectrum of the loadings code.

2. A review of development and planning control on areas of potential mass movement should be undertaken.

3. A full Engineering Lifelines Study for Dunedin should be carried out, considering in particular the impact of a major earthquake on essential services, ie

    - Water Supply and Reticulation
    - Sewerage System
    - Electrical Supply
    - Telecommunications
    - Transport

4. Carry out site specific studies of the likely seismic performance of <u>critical and/or high risk facilities</u> from an economic, public health, safety and environmental perspective. The following should be included:

    - Major Water Supply Dams and Reservoirs
    - Critical Water Supply and Sewerage Pumping Stations
    - Electrical Substations
    - Hospitals
    - Civil Defence Facilities
    - Port Facilities

5. Undertake a study of the likely costs of seismic damage to the city.

6. Consider the likely impact of a substantial earthquake on the regional economy.

## 9.2 FUTURE WORK

The following areas of further research would be of benefit.

(a) Detailed geological evaluation of the active faults in East Otago, in particular the local Akatore and Titri Faults.

(b) Continued compilation of subsurface information on the alluvial areas of the city to more accurately define the depths of alluvium and the soil stratigraphy.

(c) Research should continue to derive a reliable attenuation model for seismic accelerations for New Zealand conditions.

(d) Research should continue on the validity or otherwise of the various intensity attenuation models used in New Zealand.

(e) Continued compilation of subsurface information, particularly in-situ density, to more accurately define the stratigraphy and liquefaction hazard in Dunedin.

# CHAPTER 10: CONCLUSIONS

## 10.1 HAZARD MODEL FOR DUNEDIN

Eastern Otago is one of the least seismically active areas in New Zealand, but there are a number of active faults close to Dunedin capable of producing moderately large earthquakes. The more seismically active Fiordland area is remote from the city, but a very large magnitude earthquake in this region would cause damaging shaking in Dunedin. Overall, Dunedin is shaken most often by larger distant earthquakes, but with an intensity insufficient to cause damage. The potentially very damaging earthquakes are the less frequent events on the nearby faults.

To assess the probability of future earthquake shaking in Dunedin, 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). The seismicity regions close to Dunedin were reviewed, and some boundary changes made, making a larger Otago region. The Canterbury seismicity model of Elder et al (1991) was incorporated for the central South Island.

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

The average return periods for various intensities of shaking for average ground conditions in Dunedin have been calculated in this study as:

| | | |
|---|---|---|
| Intensity | MMVI | 30 years |
| | MMVII | 100 years |
| | MMVIII | 450 years |
| | MMIX | In excess of 2,500 years |
| | MMX | Very small probability |

These return periods are comparable with the predictions of the earlier study by Smith & Berryman (1983).

Some parts of the city, such as the reclaimed areas around the head of the harbour and the alluvial soil area of South Dunedin, are likely to experience felt intensities of up to one intensity unit higher than for "average" ground.

The attenuation model for seismic acceleration used in this study is the Kawashima model. This was adopted after comparison of the return periods predicted by using the Katayama model which has been widely used previously in New Zealand. The Kawashima model is considered to more closely model the attenuation expected in the lower South Island. Return periods for a range in peak spectral response accelerations have been estimated, and a 450 year return period uniform risk response spectra developed.

The uniform risk response spectra has been used as input into a site response model. A number of typical soil profiles have been modelled to obtain response spectra at the ground surface of the alluvial areas of the city. These show pronounced amplification effects will occur. Three typical spectral shapes were identified, depending on the relative stiffness of the soils and impedence contrasts within the soil profile. Provision of the current loadings code is probably adequate, provided design spectrum for a deep soil or flexible site is used.

Information of the geological profile beneath the alluvial areas of the city has been compiled and augmented by a resistivity survey commissioned as part of this study. The limited amount of information precludes any generalisation of response shape on an area basis.

Effects observed from the 1974 Dunedin earthquake are consistent with the general predictions from this study.

## 10.2 POTENTIAL CONSEQUENCES AND HAZARDS

Much of South Dunedin, the industrial area around the head of the harbour, and areas in the Kaikorai Valley and estuary are sited over Holocene age deposits of sands and silts which are potentially susceptible to liquefaction. There is very little good data on in-situ densities and particle size gradings, and analysis of three sites for which data was available indicates that liquefaction is unlikely to be widespread. The loosest soils tend to be soils or clays with a low susceptibility, and the sands tend to be denser or at a depth where liquefaction would be effectively confirmed. Any liquefaction would be likely to occur only with large earthquakes on the closest faults to Dunedin, or maximum credible earthquakes on more distant faults.

Dunedin has a significant number of large landslides and there is a risk of movement of either these existing landslides, or new ones, being triggered by earthquake shaking. Coherent slides is the most significant risk category and are directly linked to the underlying geology. Areas that could be subject to new movement triggered by an earthquake can be indicated by the distribution of existing mass movement features. On this basis, the Dunedin area has been zoned into three risk categories. A significant area of South Dunedin is potentially vulnerable, but probably only during strong shaking with a return period in excess of 150 - 200 years.

## 10.3 POTENTIAL DAMAGE

While Dunedin has a lower probability of experiencing an earthquake than most of New Zealand, it is likely to experience strong and very damaging seismic shaking at intervals of an average 400 - 500 years.

Amplification effects on the alluvial soil areas, and topographic focusing will cause local areas of high damage, particularly for older buildings. Structures on alluvial areas if designed to recent loading code requirements for flexible sites should perform reasonably well.

Services are likely to be affected, particularly water and sewer reticulation on the soft alluvial soil ares. The length of water supply lines increases their vulnerability, but the number of supply sources and interconnectedness will reduce the exposure of the overall system. The sewerage system is probably most vulnerable to damage of older sewers in the alluvial and reclaimed areas of the city. An engineering lifelines study is recommended to assess the type and extent of potential damage.

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# APPENDIX A: MODIFIED MERCALLI SCALE OF INTENSITY OF EARTHQUAKE SHAKING (NZ VERSIONS OF 1965 AND 1991)

Ref: Bulletin of NZ National Society for Earthquake Engineering
Vol 25, No. 4, December 1992

## MODIFIED MERCALLI INTENSITY SCALE - NZ 1991

| NZ 1965 | | NZ 1991 Proposed | |
|---|---|---|---|
| MM 1 | 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.<br><br>Branches of trees, chandeliers, doors, and other suspended systems of long natural period may be seen to move slowly.<br><br>Water in ponds, lakes, reservoirs, etc., may be set into seiche oscillation. | MM 1 | *People*<br><br>Not felt except by a very few people under exceptionally favourable circumstances. |
| MM 2 | Felt by a few persons at rest indoors, especially by those on upper floors or otherwise favourably placed.<br><br>The long-period effects listed under MM1 may be more noticeable. | MM 2 | *People*<br><br>Felt by persons at rest, on upper floors or favourably placed. |
| MM 3 | Felt indoors, but not identified as an earthquake by everyone. Vibration may be likened to the passing of light traffic.<br><br>It may be possible to estimate the duration, but not the direction. Hanging objects may swing slightly. Standing motorcars may rock slightly. | MM 3 | *People*<br><br>Felt indoors; hanging objects may swing, vibration similar to passing of light trucks, duration may be estimated, may not be recognised as an earthquake. |

COMMENTS

MM1. (1) "Reported mainly from ..." defines one favourable circumstance.

(2) "Birds and animals disturbed" and "systems of long natural period may ... move slowly". These phenomena may be observed at <u>any</u> intensity and are thus not definitive of any particular intensity.

MM2. The reference to an increase in long-period effects is tautological: it will be true of <u>all</u> intensities.

MM3. (1) The use of "trucks" rather than "traffic" is considered clearer.

(2) The NZ 1965 qualification "but not direction" is redundant - see MM 5.

(3) The reference to motorcars rocking slightly, but with the suggestion that their occupants could be unaware of the motion is considered doubtful (cf MM 4).

**NZ 1965**

MM4 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 rattle.
Liquids in open vessels may be slightly disturbed.

Standing motorcars may rock, and the shock can be felt by their occupants.

**NZ 1991 Proposed**

MM4 *People*

Generally noticed indoors but not outside. Light sleepers may be awakened. Vibration may be likened to the passing of heavy traffic or to the jolt of a heavy object falling or striking the building.

*Fittings*

Doors and windows rattle. Glassware and crockery rattle. Liquids in open vessels may be slightly disturbed. Standing motorcars may rock.

*Structures*

Walls and frame of buildings, and partitions and suspended ceilings in commercial buildings, may be heard to creak.

**COMMENTS**

MM4. (1) "Very" qualification for light sleepers superfluous.

(2) If standing motorcars rock, their occupants are likely to feel the movement.

(3) Creaking of walls is not general at this level.

**NZ 1965**

MM5 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.

**NZ 1991 Proposed**

MM5 *People*

Generally felt outside, and by almost everyone indoors.
Most sleepers awakened.
A few people alarmed.
Direction of motion can be estimated.

*Fittings*

Small unstable objects are displaced or upset. Some glassware and crockery may be broken.
Hanging pictures knock against the wall.
Open doors may swing.
Cupboard doors secured by magnetic catches may open.
Pendulum clocks stop, start, or change rate (H*).

*Structures*

Some Windows Type I* cracked.
A few earthenware toilet fixtures cracked (H).

COMMENTS

MM5. (1) "Alarmed" for "frightened". This is consistent with higher intensities, but there is some feeling that "fright" rather than "alarm" may generally be better.

(2) Pictures "knock" rather than swing at this intensity.

(3) Inclusion of "shutters" with doors is doubtful; few New Zealand houses have them, and most are secured.

* See Appendix.

| NZ 1965 | NZ 1991 Proposed |
|---|---|
| MM 6 Felt by all.<br>People and animals alarmed.<br>Many run outside.<br>difficulty experienced in walking steadily.<br><br>Slight damage to Masonry D.<br>Some plaster cracks or falls.<br>Isolated cases of chimney damage.<br>Windows, glassware, and crockery broken.<br>Objects fall from shelves, and pictures from walls.<br>Heavy furniture moved.<br>Unstable furniture overturned.<br>Small church and school bells ring.<br><br>Trees and bushes shake, or are heard to rustle.<br>Loose material may be dislodged from existing slips, talus slopes, or shingle slides. | MM 6 *People*<br><br>Felt by all.<br>People and animals alarmed.<br>Many run outside.<br>Difficulty experienced in walking steadily.<br><br>*Fittings*<br><br>Objects fall from shelves.<br>Pictures fall from walls (H*).<br>Some furniture moved on smooth floors.<br>Some unsecured free-standing fireplaces moved.<br>Glassware and crockery broken.<br>Unstable furniture overturned.<br>Small church and school bells ring (H).<br>Appliances move on bench or table tops.<br>Filing cabinets or "easy glide" drawers may open (or shut).<br><br>*Structures*<br><br>Slight damage to Buildings Type I*.<br>Some stucco or cement plaster falls.<br>Suspended ceilings damaged.<br>Windows Type I* broken.<br>A few cases of chimney damage.<br><br>*Environment*<br><br>Trees and bushes shake, or are heard to rustle.<br>Loose material may be dislodged from sloping ground, e.g. existing slides, talus slopes, shingle slides. |

COMMENTS

MM6. (1) Pictures secured with modern pinned picture hooks unlikely to fall at this intensity.

(2) "Some" (rather than "Heavy") furniture moved <u>on smooth floors</u>. Furniture on carpet unlikely to move at this intensity.

(3) "Plaster" falls - ambiguous. "Stucco" rather than interior plaster is intended.

(4) Cracking to unreinforced chimneys is common.

* See Appendix

| NZ 1965 | NZ 1991 Proposed |
|---|---|
| MM 7 General alarm.<br>Difficulty experienced in standing.<br>Noticed by drivers of motorcars.<br><br>Trees and bushes strongly shaken.<br>Large bells ring.<br>Masonry D cracked and damaged.<br>A few instances of damage to Masonry C.<br>Loose brickwork and tiles dislodged.<br>Unbraced parapets and architectural ornaments may fall.<br>Stone walls cracked.<br>Weak chimneys broken, usually at the roof-line.<br>Domestic water tanks burst.<br>Concrete irrigation ditches damaged.<br><br>Waves seen on ponds and lakes.<br>Water made turbid by stirred-up mud.<br>Small slips, and caving-in of sand and gravel banks. | MM 7 *People*<br><br>General alarm.<br>Difficulty experienced in standing.<br>Noticed by motorcar drivers who may stop.<br><br>*Fittings*<br><br>Large bells ring.<br>Furniture moves on smooth floors, may move on carpeted floors.<br><br>*Structures*<br><br>Unreinforced stone and brick walls cracked.<br>Buildings Type I cracked and damaged.<br>A few instances of damage to Buildings Type II.<br>Unbraced parapets and architectural ornaments fall.<br>Roofing tiles, especially ridge tiles may be dislodged.<br>Many unreinforced domestic chimneys broken.<br>Water tanks Type I* burst.<br>A few instances of damage to brick veneers and plaster or cement-based linings.<br>Unrestrained water cylinders (Water Tanks Type II*) may move and leak.<br>Some Windows Type II* cracked.<br><br>*Environment*<br><br>Water made turbid by stirred up mud.<br>Small slides such as falls of sand and gravel banks.<br>Instances of differential settlement on poor or wet or unconsolidated ground.<br>Some fine cracks appear in sloping ground.<br>A few instances of liquefaction. |

COMMENTS

MM7. (1) "Noticed by motorcar drivers who may stop." Modern cars transmit the shaking to the occupants more effectively than old ones. This effect may commence at a lower intensity.

(2) "Trees and bushes strongly shaken" is too subjective to be of much use.

(3) Buildings types replace Masonry types.

(4) "Concrete irrigation ditches" doubtfully damaged at this intensity.

(5) Commencement of damage in a number of areas at this intensity in brick veneers, wall linings, ordinary windows, perhaps liquefaction under most favourable conditions.

(6) "Waves seen" not useful and omitted.

(7) "Slides" rather than "slips" is consistent with international use.

(8) Care must be taken to ensure that ground cracking was due to shaking and not shrinkage, etc.

| | NZ 1965 | | NZ 1991 Proposed |
|---|---|---|---|
| MM 8 | Alarm may approach panic.<br><br>Steering of motorcars affected.<br><br>Masonry C damaged, with partial collapse.<br>Masonry B damaged in some cases.<br>Masonry A undamaged.<br><br>Chimneys, factory stacks, monuments, towers, and elevated tanks twisted or brought down.<br>Panel walls thrown out of frame structures.<br>Some brick veneers damaged.<br>Decayed wooden piles broken.<br>Frame houses not secured to the foundation may move.<br>Cracks appear on steep slopes and in wet ground.<br>Landslips in roadside cuttings and unsupported excavations.<br>Some tree branches may be broken off.<br>Changes in the flow or temperature of springs and wells may occur.<br>Small earthquake fountains. | MM 8 | *People*<br><br>Alarm may approach panic.<br>Steering of motorcars greatly affected.<br><br>*Structures*<br><br>Buildings Type II damaged, some seriously.<br>Buildings Type III damaged in some cases.<br>Monuments and elevated tanks twisted or brought down.<br>Some pre-1965 infill masonry panels damaged.<br>A few post-1980 brick veneers damaged.<br>Weak piles damaged.<br>Houses not secured to foundations may move.<br><br>*Environment*<br><br>Cracks appear on steep slopes and in wet ground.<br>Slides in roadside cuttings and unsupported excavations.<br>Small earthquake fountains and other manifestations of liquefaction. |

## COMMENTS

MM8. (1) Steering of motorcars is likely to be so affected that drivers will have to stop.

(2) Changes to building damage consistent with changes to Building types.

(3) "Weak Piles" covers a wider range than "Decayed wooden piles".

(4) "Tree branches broken off" is too likely to depend on the state (i.e. rottenness) of the branch.

(5) "Manifestations of liquefaction" - these are likely to be general at this intensity in susceptible ground.

(6) "Changes in the flow or temperature of springs and wells may occur" - no New Zealand data to support inclusion at this intensity. Springs and wells are affected by stress changes before and after a shock.

| NZ 1965 | | NZ 1991 Proposed | |
|---|---|---|---|
| MM 9 | General panic.<br><br>Masonry D destroyed.<br>Masonry C heavily damaged, sometimes collapsing completely.<br>Masonry B seriously damaged.<br><br>Frame structures racked and distorted.<br>Damage to foundations general.<br>Frame houses not secured to the foundations shifted off.<br>Brick veneers fall and expose frames.<br>Cracking of the ground conspicuous.<br>Minor damage to paths and roadways.<br>Sand and mud ejected in alluviated areas, with the formation of earthquake fountains and sand craters.<br>Underground pipes broken.<br>Serious damage to reservoirs. | MM 9 | *Structures*<br><br>Very poor quality unreinforced masonry destroyed. Buildings Type II heavily damaged, some collapsing.<br>Buildings Type III damaged, some seriously.<br>Damage or permanent distortion to some Buildings and Bridges Type IV.<br>Houses not secured to foundations shifted off.<br>Brick veneers fall and expose frames.<br><br>*Environment*<br><br>Cracking of ground conspicuous.<br>Landsliding general on steep slopes.<br>Liquefaction effects intensified, with large earthquake fountains and sand craters. |

COMMENTS

MM9. (1) "Sand and mud ejected" - an intensification of MM8 effects.

(2) "Serious damage to reservoirs" - not at this intensity without qualification about the construction of the reservoir.

(3) "Minor damage to paths" and "underground pipes broken" - very doubtfully by <u>shaking</u> at this intensity.

(4) "Landsliding general" - the area of widespread landslides has approximately corresponded to the MM 9 isoseismal in several historical events.

**NZ 1965**

MM 10 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 banks.

**NZ 1991 Proposed**

MM 10 *Structures*

Most unreinforced masonry structures destroyed.
Many Buildings Type II destroyed.
Many Buildings Type III (and bridges of equivalent design) seriously damaged.
Many Buildings and Bridges Type IV have moderate damage or permanent distortion.

COMMENTS

MM10. (1) Very few clear examples of MM 10 in the recent past.

(2) Damage that could arise from static compression or dilatations of the ground ("bent railway lines", "cracked pavements") is omitted.

(3) "Large landslides" occur at lower intensities under favourable (i.e. saturated) conditions.

(4) Liquefaction effects here represent a subjective intensification.

(5) "Water thrown up on banks" may occur at lower intensities.

| | NZ 1965 | NZ 1991 Proposed |
|---|---|---|
| MM 11 | Wooden frame structures destroyed.<br>Great damage to railway lines and underground pipes. | |
| MM 12 | Damage virtually total. Practically all works of construction destroyed or greatly damaged.<br>Large rock masses displaced.<br>Lines of sight and level distorted.<br>Visible wave-motion of the ground surface reported.<br>Objects thrown upwards into the air. | |

COMMENTS

MM11. Great damage to underground pipes and railway lines not unambiguously caused by shaking - observed at MM 9 at Edgecumbe.

"Wooden frame structures destroyed" did not appear in pre-1965 versions of the scale.

MM12. "Large rock masses" have undoubtedly been displaced at lower intensities (e.g. 1929).

"Lines of sight and level distorted" and "visible wave motion reported" undoubtedly occur at lower intensities.

"Objects thrown upwards", when general, indicates a vertical acceleration of more than 1.0 g. Where this has been reported it has been in an area of generally lower intensity.

## Appendix

### NZ 1965
### Categories of non-Wooden Construction

Masonry A

Structure designed to resist lateral forces of about 0.1 g, such as those satisfying the New Zealand Model Building Bylaw, 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 workman–ship, 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 standard 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 MM5 are usually either large display windows, or windows tightly fitted to metal frames.

### Water Tanks

The "domestic water tanks" listed under MM7 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.

### NZ 1991 Proposed
### Categories of Construction

Buildings
Type I:

Weak materials such as mud brick and rammed earth; poor mortar; low standards of workmanship (Masonry D in other MM scales).

Buildings
Type II:

Average to good workmanship and materials, some including reinforcement, but not designed to resist earthquakes (Masonry B and C in other MM scales).

Buildings
Type III:

Buildings designed and built to resist earthquakes to normal use standards, i.e. no special damage limiting measures taken (mid-1930's to c. 1970 for concrete and to c. 1980 for other materials).

Buildings and Bridges
Type IV:

Since c. 1970 for concrete and c. 1980 for other materials, the loadings and materials codes have combined to ensure fewer collapses and less damage than in earlier structures. This arises from features such as: (i) "capacity design" procedure, (ii) use of elements (such as improved bracing or structural walls) which reduce racking (i.e. drift), (iii) high ductility, (iv) higher strength.

### Windows

Type I - Large display windows, especially shop windows.

Type II - Ordinary sash or casement windows.

### Water Tanks

Type I - External, stand mounted, corrugated iron water tanks.

Type II - Domestic hot-water cylinders unrestrained except by supply and delivery pipes.

H - (Historical). Important for historical events. Current application only to older houses, etc.

### General Comment

"Some" or "a few" indicates that the threshold of a particular effect has just been reached at that intensity.

# APPENDIX B: THICKNESS OF SOFT HOLOCENE SEDIMENTS IN THE DUNEDIN AREA REPORT BY C PEARSON

# THICKNESS OF SOFT HOLOCENE SEDIMENTS IN THE DUNEDIN AREA

CHRISTOPHER PEARSON

REPORT TO
Soils and Foundations Ltd.
Christchurch

## TABLE OF CONTENTS

## INTRODUCTION

The purpose of this project is to determine the thickness of poorly lithified sediments which underlie portions of Dunedin City using geophysical techniques. Because the central city is well covered by existing site investigations, the scope of this study was restricted to St Kilda, South Dunedin the flat low lying portions of St. Clair and North Dunedin. In this report I will concentrate my discussion on the results from South Dunedin, St Kilda, and St. Clair where most of the soundings were conducted. A total of 15 soundings were conducted in this area. The locations are shown in figure 1. A further 6 soundings were conducted in North Dunedin. The results from North Dunedin are presented as a series of spot determinations in the appendix.

The study was undertaken using D.C. resistivity soundings as the primary method. This method was selected because it can be carried out with little impact on the surroundings.

## GEOLOGICAL MODEL

During the last glacial epoch, sea level was about 100 m lower than at present and what is currently Otago Harbor was occupied by two stream valleys, one through Taiaroa Heads and the other through South Dunedin (Benson and Raeside 1963). The watershed seperating the two drainages ran between Portobello and Port Chalmers. At this time South Dunedin was occupied by a broad, low valley which may have been partly filled by river borne gravels. As sea level rose, marine sediments began to be deposited in the valley and in time they completely blocked the channel through South Dunedin. The floor of the valley was probably weathered volcanic rock, although the lower portions might have been Tertiary Marine Rocks if the stream had completely eroded through the volcanics. The nature of the sediments in South Dunedin have not been much studied, but Scott and Landis (1975) show that for the Aramoana area, in the northern end of the harbor, sand sourced from the Clutha River is the predominant type of sediment. Silt and mud in Aramoana make up only a small percentage of the sediments.

Thus I would expect South Dunedin to be underlain by poorly lithified Holocene sand, with limited amounts of mud and silt which is in turn underlain by a basement made up of volcanics or possibly Tertiary Marine Rocks. Between the Holocene sand and the basement there may be a layer of river borne gravels and shingle in places. The thickness of the Holocene sand is likely to be variable depending on the profile of the original stream valley.

## SUBSURFACE INFORMATION

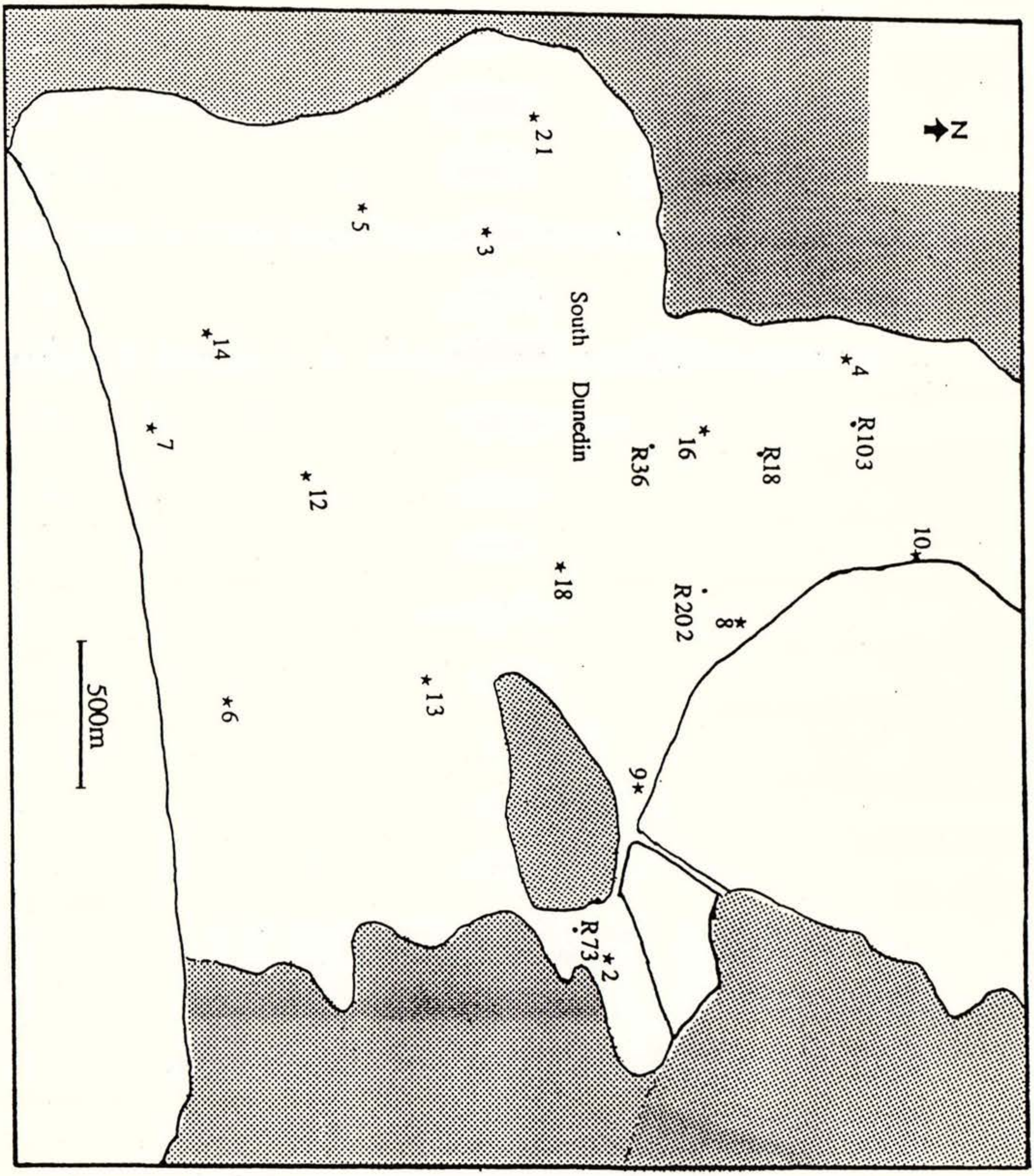
Some well logs from foundation investigations exist for South Dunedin. Unfortunately, it was never possible to conduct a sounding adjacent to a logged well, because, by their nature, these wells are always located on developed sites, which are not suitable for soundings. They still provide valuable constraints on the interpretation of the soundings because the wells are usually located within a few hundred meters of one or more soundings so it is possible to correlate between the wells and soundings with confidence.

The well logs show that the sediments in South Dunedin are a mixture of Marine sands mud and silt. Silt and mud are reported the northern part of this area, particularly at shallow levels, while farther south, near St. Kilda Beach, sands predominate. Where silt and mud are present, they tend to be found near the surface, and are often underlain by sands. At the base of the Marine sequence, the logs often describe a thick sequence of volcanic boulders or cobbles, usually in a clay matrix. This could represent some sort of river borne gravels or beach cobbles, but I suspect that they often represent weathered volcanic rocks. In any event, the few cores that penetrate the cobble layer encounter volcanic basement immediately under this layer, with no evidence of further Holocene marine sediments (Foundation Report #18 from Railroad engine yards and #202 from the Otaki-Midland St. substations). It is possible that Tertiary Marine Rocks make up Basement over part of South Dunedin. Foundation Report #18 from Railroad engine yards reports material that resembles the Tertiary Marine Rocks at a depth of 46 m. If these rocks continue at this depth under South Dunedin, they could immediately underlie the Holocene marine sediments under much of South Dunedin. The report of a test piling at the city gas works (Foundation Report 36) mentions a competent sand layer at 76' or 23 m. this could represent Tertiary Marine Rocks. Tertiary Marine Rocks could be expected to have a very similar resistivity to weathered volcanic rocks so it may not be possible to distinguish between the two types of basement using resistivity.

Locations of wells from site investigation reports from South Dunedin, that I have used in this report are shown in figure 1. Figure 2 shows the interpreted depth to basement for this data as well as for the soundings discussed below. When estimating depth to basement I have used the first reference to volcanic boulders in the logs as representing basement. In the case of foundation report #36 I have used the depth to a competent sand layer as representing basement, because, as discussed above, I think that this may represent Tertiary Marine Rocks.

## RESISTIVITIES OF EARTH MATERIALS

In cases of sediments and sedimentary rocks which are saturated with relatively pure water, which is commonly the case for surface sediments, except for coastal salt water intrusion, the bulk resistivity of the material is controlled by surface conduction due to exchangable ions on grain or particle boundaries. As a result the resistivities are inversely related to the average particle size with sediments such as gravels (with large average particle size and high porosity) having high resistivities while sand and silt (smaller particle sizes) having lower resistivities. The presence of certain clay minerals can cause disproportionately low resistivities in sediments that contain them because they contain disproportionately high numbers of exchangable ions. Most solid igneous or metamorphic rocks also have very high resistivities because they have very low surface area to volume ratios. Weathered volcanic rocks may have a substantial clay fraction, however, and the resistivities of these rocks may be much less than fresh volcanic rocks. Foundation reports from North and South Dunedin make it clear that volcanic boulders are often found with large amounts of interstitial clay, and the bulk resistivity of the mixture will probably be much less than for clay alone.




1 Locations of soundings undertaken in South Dunedin, St Kilda and St. Clair as part of this project. Locations of the soundings are indicated by a star and are labeled by number. Locations of logged wells or test pilings which are discussed in the text are indicated by a dot and are labeled by report number from the Dunedin City Council files (Report numbers start with 'R').

If sediments are saturated by sea water, the bulk resistivity of the material is controlled by ionic transfer in the pore water. Surface conduction still occurs but it is overwhelmed by the amount of current transferred by the dissolved salts in the seawater. In this case, the bulk resistivity is a function of the porosity, and can be calculated by Archie's equation (Schlumberger 1972 p2).

Clearly, a knowledge of the probable resistivities of formations can assist in interpreting . resistivity data. The best estimates of resistivities are from field measurements in areas where the geology is clear and the problem of equivalence can be minimized. The estimated resistivity ranges from table 1 were mostly determined from soundings conducted as part of this study. Lithologic identification was made from well the logs located nearby. An exception is resistivity of Tertiary Sediments which comes from a series of soundings undertaken in North Otago as part of a program of hydrologic investigations. The resistivities of salt water saturated sands can calculated from Archie's Equation (Schlumberger 1972 p2 eq 1-2a) if the porosity of the rock is known. I used this procedure to see if the resistivities that I inferred for salt-water saturated sands are associated with reasonable values of porosity.

TABLE 1
RESISTIVITIES OF ROCK UNITS

| DESCRIPTION | RESISTIVITY Ω-m | REFERENCE |
|---|---|---|
| SOIL | 7-200 | All soundings |
| FINE GRAINED TERTIARY SEDIMENTS | 30-70 | Pearson 1989 |
| DRY SAND | 1000-2000 | S6&7 |
| SATURATED SAND (Fresh Water) | 100-200 | S6&7 |
| SATURATED SAND (Salt Water) | 4-6* | S13&14 |
| SATURATED SAND-SILT (Fresh Water) | 8-20 | S3&5 |
| SILT & MUD | <3 | S8,9&18 |
| FRESH VOLCANICS | >200 | Caldwell et al. 1986 |
| WEATHERED VOLCANICS | 25-50 | S2,4,15 & 21 |

*Values of porosity from Archie's equation (sand formula Schlumberger 1972 p2 eq 1-2a) range between 15% and 20% assuming a water resistivity of 0.2 Ω-m (Telford et al. 1976 pg. 452).

When interpreting the results of soundings from South Dunedin, I have assumed that resistivities less than 2.5 Ω-m represent silt & mud rather than salt water saturated sands. These layers usually occur at shallow levels in the Northern part of the study area. Since the tops of these layers are usually within a few meters of the surface, salt water saturation is unlikely to be the cause. In addition, site investigation reports from this region tend to mention layers of mud or silt in the depth range where the very low resistivity material is found. If the sediments are saturated by fresh water, then they probably represent marine mud rather than silt because these very low resistivities imply a relatively high clay content as discussed earlier in this section.

The resistivities of volcanic rocks in the Dunedin area usually range between 25-50 Ω-m. These values are from soundings 2,4,15 and 21, which are located close to outcrops of volcanic rocks. These values are much lower than fresh volcanic rocks implying that the near surface volcanic rocks are highly weathered in the study area.

## GEOELECTRICAL MODEL

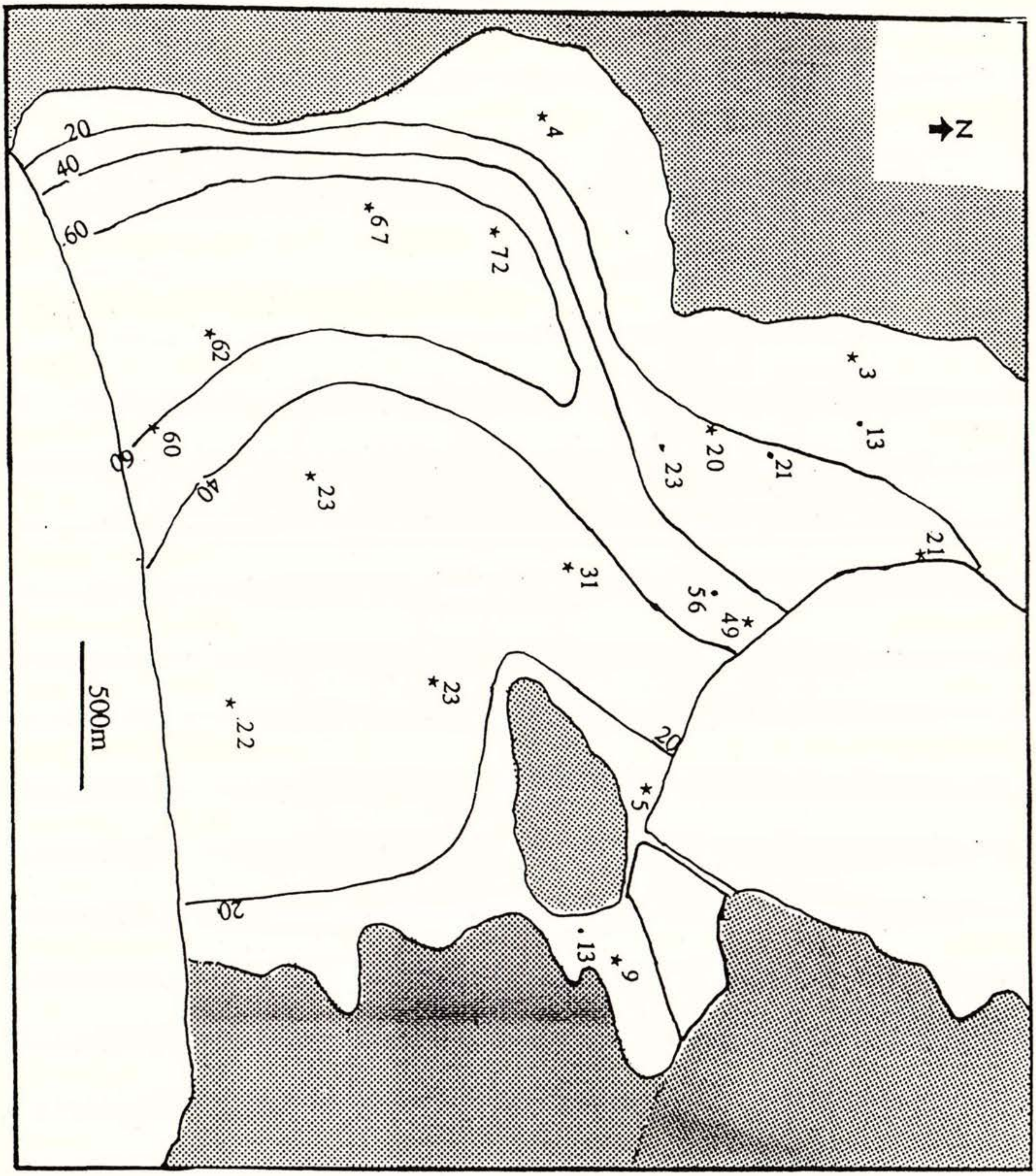
Combining the information in table 1 with the simplified geological model outlined above produces a simple resistivity model of the South Dunedin area. In all parts of the area which are underlain by a near surface layer of silt, the top few meters will show very low resistivities (> 2.5 Ω-m) which may be underlain by a layer of sand saturated with sea water with slightly higher resistivities (~5 Ω-m). Basement could either be Tertiary sediments or Volcanics possibly overlain by a layer of river born boulders and beach cobbles. In either case the resistivity of the basement is probably on the order of 30 Ω-m to 50 Ω-m. Higher basement resistivities (>80 Ω-m) are possible if the weathered layer is thin or absent.

In areas where no significant silt layer is present, the top layer may have fairly high (200 Ω-m to 20 Ω-m) resistivity which represents fresh water saturated sands. The range in the resistivities probably represents variations in the average particle size of the sand layer. Under this may occur a low resistivity (4Ω-m to 6 Ω-m) layer representing salt water saturated sand which is in turn underlain by basement at 30 Ω-m to 50 Ω-m. In the dunes along the south coast, the top layer is unsaturated sands with a resistivity of 1800-2000 Ω-m.

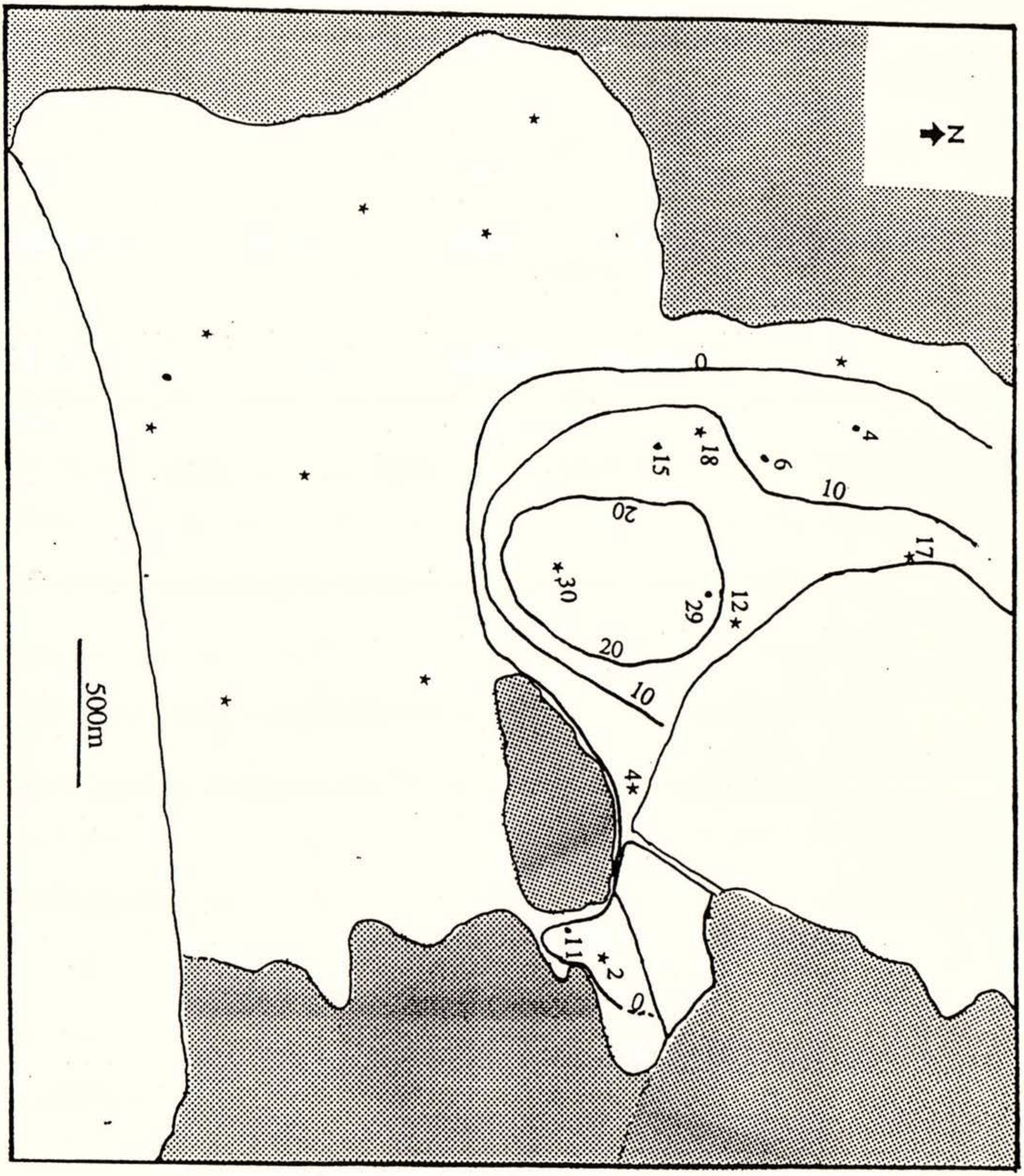
## DEPTH TO BASEMENT IN SOUTH DUNEDIN

The inferred depth to basement in South Dunedin is shown in figure 2. The greatest depth is 72 m at Bathgate Park. In general the depth appears to be greatest in the south-west of the study area in St. Clair and the western part of St. Kilda. It appears that during the last ice age, a stream channel may have passed under the central part of Portsmouth Drive, then flowed west-southwest until it hit the western side of the valley and then traveled south leaving the study area around St. Clair Beach.

With the exception of the north-eastern part of the study area, the material overlying basement appears to be predominantly sand. Over most of the study, the sand appears to be saturated with fresh water to within a meter or so of the surface, however, behind the dunes along St. Kilda beach, a surface layer of unsaturated sand up to 6 m thick was identified. The two soundings that detected the unsaturated layer were located on slightly elevated land adjacent to the dunes so these unsaturated sands are probably restricted to a narrow zone a few hundred meters wide along St Clair and St. Kilda beaches. In the north-east of the study area, a layer of marine silt and mud appears to overlie the sand. This layer covers the area between Portsmouth Drive and Andersons Bay Road. The extent and inferred thickness of this layer is shown in figure 3.




2 Inferred depth to basement from soundings and site investigation reports. Depths are in m. Locations of the soundings are indicated by a star and locations of logged wells or test pilings are indicated by a dot. If no depth is given, then no near surface marine mud and silt layer was detected at the site.




3 Inferred thickness of near surface marine mud and silt. Depths are in m. Locations of the soundings are indicated by a star and locations of logged wells or test pilings are indicated by a dot. If no depth is given, then no near surface marine mud and silt layer was detected at the site.

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## APPENDIX 1 FIELD METHODS AND RESULTS

### A-1.1) FIELD METHODS

All the soundings in this survey were conducted using the Schlumberger electrode configuration. A description of the field procedures employed during Schlumberger soundings is given in Koefoed (1979 pg. 1-3). In this array, four collinear electrodes (two current and two potential) are used with the potential electrodes (which are closely spaced) located midway between the current electrodes. Customarily, the separation between the potential electrodes is referred to as (mn) while the distance between the center of the array and the current electrode is referred to as AB/2. The geometry of the array is illustrated in fig. A1. During a sounding, only the current electrodes are shifted between each measurement, while the mn distance is increased only when necessary to increase signal strength. It is good practice to repeat measurements for several (at least two) stations immediately before a pot shift with both the new and the old pot positions so that any shifts in the resistivity curve due to inhomogeneous resistivity structure near the pots can be identified and corrected for.

I used the University of Otago's resistivity gear in the 21 soundings that I conducted in this project. The equipment consists of a Redfern Radio (mark 2) power supply modified and repackaged by GENZL and a multicore pot cable which allows all of the potential electrodes to be used during a sounding to be set out in the beginning. The operator can readily choose between them using a double pole switch. Potentials were conducted using a Fluke multimeter which has a reading precision of 0.1mv. Fortunately pot stability was very good throughout the survey with drift rates less than 0.1mv/min so signals of less than 1mv could be reliably estimated. The equipment used in a typical sounding is shown in plate 1.

All the soundings were conducted at 10 points per decade. With the multicore pot cable we were able to record two mn positions for every current electrode position with an AB/2 greater than 5m. unless this was not possible due to low signal strength. This procedure was valuable not only for accurately determining mn shifts but also because it provides two independent measurements for each point.

In all during this project I conducted 21 soundings, locations of which are shown in figure 1 of the main report.

### A-1.2) INTERPRETATION

Interpretation of resistivity data involves finding a subsurface distribution of resistivities that match the measurements. Before the currents and voltages are converted to apparent resistivities. Resistivity data are always converted to apparent resistivity before being interpreted because this allows the effect to changes in the array geometry and input current to be be removed before the sounding is interpreted. The equations used in this procedure are given by Koefoed (page 41 eq. 4.2.1). I interpreted the curves using a standard curve matching procedure (see Koefoed pg. 103-106). In order to calculate the model curves which are shown in appendix A3, I used a standard program which is described by Koefoed (1979 pg.98-99). The version I use has been modified for 10 point per decade liner filter and runs on the University of Otago's VAX computer. Of course the results of this interpretation are not necessarily unique, a problem which is discussed below.

### A-1.3) EQUIVALENCE AND SUPPRESSION

Interpretation of sounding curves is hampered by the problem of nonuniqueness which occurs because , in general, there are a range of models with different thicknesses and resistivities that will do an equally good job of matching an apparent resistivity curve giving us no way of distinguishing between them. Koefoed (1979) identifies two types of nonuniqueness. The first (equivalence) occurs at a local maximum or minimum in the apparent resistivity curves which correspond to cases where the true resistivity of a layer is greater than the layers on either side (for a local maxima) or less than the layers on either side (for a local minimum). In either case by simultaneously adjusting the thickness and resistivity, a range of models can be generated. The way that resistivities and thicknesses trade off is different for local maximum and local minimum. In the case of a local maximum the resistivity and thickness must be adjusted so that the product of the resistivity and thickness is constant while for local minimum the quotient must be constant. Note that, as shown by Koefoed(1976), these equivalence rules have only a limited range of applicability the width of which depends on the curve in question. The equivalence range is dramatically limited for curves with very broad flat topped maxima because this indicated that the layer is thick enough that the apparent resistivity is approaching the true resistivity. Fortunately this situation is quite common in the Ohau data.

The second type of type of nonuniqueness in resistivity curves is called suppression. This problem arises in ascending (or descending) sequences (i.e. when the true resistivities of three layers increase (or decrease) sequentially with depth). In this case, the danger is that even if the resistivities of the top and bottom layer are correctly estimated from the curve, intermediate layers may be missed altogether. If the intermediate layer has a thickness of a third that of the top layer or less, a satisfactory match could be made by neglecting the intermediate layer and using a slightly thicker upper layer.

## A-1.4 SOUNDING INTERPRETATION

In this appendix I present the interpretations for all of the soundings together with a brief description of the site and the reasons that I have interpreted them as I have. The headings are defined as follows. STA # is the sounding number from fig.1 of the main report. Resistivity and Thickness are the resistivity of the thickness of the layers of the model, listed from the surface down. Depth is the calculated depth to basement. Interpretation is the geological interpretation of each layer.

### SOUTH DUNEDIN SOUNDINGS

| Sounding # | Elevation m | Resistivity Ω-m | Thickness (m) | Depth (m) | Interpretation |
|---|---|---|---|---|---|
| 2 | 2 | 25 | 2 | 9 | Soil-Fill |
| | | 1.8 | 2 | | Marine silt |
| | | 5 | 5 | | Sea water sat. sands |
| | | 50 | ∞ | | Basement |

This sounding was conducted on the grounds of Bayfield High School. There is quite a lot of equivalence in this sounding, and it can be interpreted in two quite different ways. The sounding can be interpreted with a 3 layer, which gives a total depth of about 5 m or I can have the 4 layer model as shown above. I prefer the 4 layer model because the log of the nearby Bayfield High School bore suggests 4 layers exist on the site.

| Sounding # | Elevation m | Resistivity Ω-m | Thickness (m) | Depth (m) | Interpretation |
|---|---|---|---|---|---|
| 3 | 2 | 14 | 3.6 | 72.1 | fresh water sat. silty sand |
| | | 33 | 3.5 | | Sat. sand with less silt |
| | | 12 | 65 | | fresh water sat. silty sand |
| | | 45 | ∞ | | Basement |

This sounding was conducted in Bathgate Park parallel to Donaghy's Ropes and Twynes. The curve is dominated by a thick 12 Ω-m layer (layer 3). The resistivity of the 14 Ω-m top layer is also well constrained. The resistivities and thicknesses of the other two layers are subject to equivalence. I have used 33 Ω-m for layer 2 (which is near the bottom of the equivalence range) because it is a reasonable value for water saturated sand. As found in sounding 6 and 7, much higher values are possible, depending on how much silt is contained with the sand. Taking a higher value would reduce the thickness of this layer, but only by 1 m or 2 m, since the layer is already thin. Thus the total thickness of the sediments at this site is not very much effected by this layer. The resistivity of the basement is also not well constrained by this sounding. I chose 45 Ω-m by comparison with other soundings where it is well constrained (i.e 2, 4 and 15).

Changing the resistivity of this layer has no effect on the total thickness.

| Sounding # | Elevation | Resistivity | Thickness | Depth | Interpretation |
|---|---|---|---|---|---|
| | m | Ω-m | (m) | (m) | |
| 4 | 3 | 7 | 3.4 | 3.4 | fresh water sat. silty sand |
| | | 35 | ∞ | | Basement |

This sounding was conducted in the Oval. This is a simple two layer curve and there is very little room for equivalence.

| Sounding # | Elevation | Resistivity | Thickness | Depth | Interpretation |
|---|---|---|---|---|---|
| | m | Ω-m | (m) | (m) | |
| 5 | 2 | 14 | 3.7 | 66.7 | Soil |
| | | 21 | 63 | | fresh water sat. sand |
| | | 80 | ∞ | | Basement |

This sounding was conducted Tonga Park on a playing field. The curve has very little room for equivalence except for the basement resistivity. The value of the basement resistivity has very little effect on the inferred depth to basement however.

| Sounding # | Elevation | Resistivity | Thickness | Depth | Interpretation |
|---|---|---|---|---|---|
| | m | Ω-m | (m) | (m) | |
| 6 | 4 | 1100 | 2.16 | 21.16 | Dry Sand |
| | | 22 | 10 | | fresh water sat. sand |
| | | 2.1 | 9 | | sea water sat. sands |
| | | 30 | ∞ | | Basement |

This sounding was conducted Ketttle Park on a playing field inland from the high sand dune. The soil was extremely sandy right to the surface. Since the sea was located at least 200m from the sounding a direct coastal influence on the sounding seems unlikely. All of the layers are strongly effected by equivalence except for the basement resistivity.

| Sounding # | Elevation | Resistivity | Thickness | Depth | Interpretation |
|---|---|---|---|---|---|
| | m | Ω-m | (m) | (m) | |
| 7 | 7 | 1800 | 5.98 | 60.38 | Dry dune sand |
| | | 200 | 20.8 | | fresh water sand aquifer |
| | | 4 | 33.6 | | salt water sand aquifer |
| | | 32.5 | ∞ | | basement |

This sounding was conducted Ketttle Park on a playing field inland from the high sand dune. The soil was extremely sandy right to the surface. Since the sea was located 200 m from the sounding a direct coastal influence on the sounding seems unlikely. I have interpreted the high resistivity near surface layer as unsaturated sand. The elevation of the site is about 6 or 7 m. The 200 Ω-m layer probably represents a fresh water aquifer in sand or gravel. The 4 Ω-m layer represents sand saturated with saline ground water. There is very little room for equivalence in determining the resistivity and thickness of layer 1. Other resistivities are possible for layer 2 but these would have littie effect on the depth to basement. A wide range of models are also possible for layer 3, and in this case, the equivalence does have an important effect on the depth to basement I chose 4Ω-m for this resistivity by comparison with sounding 14, which is located nearby.

| Sounding # | Elevation | Resistivity | Thickness | Depth | Interpretation |
|---|---|---|---|---|---|
| | m | Ω-m | (m) | (m) | |
| 8 | 3 | 85 | 2 | 49 | Soil - Fill |
| | | 2 | 12 | | Marine Silt |
| | | 6 | 35 | | salt water sand aquifer |
| | | 40 | ∞ | | Basement |

This sounding was conducted on Portsmouth Drive opposite the substation. Not surprisingly, this station was very noisy electrically. Since the sea was located 20 m from the sounding a direct coastal influence is possible. The sounding was conducted at low tide however, and the effect is probably not all that great. I have interpreted the 2 Ω-m layer as marine silt. The 6 Ω-m layer represents sand saturated with saline ground water. There is room for equivalence in determining the resistivity and thickness of layer 2 &3. Other resistivities are possible for layer 2 and in this case, the equivalence does have an important effect on the depth to basement I chose 2 Ω-m for this layer (which is one of the highest values possible within the equivalence range because the bore log for the Otahi-Midland St. substation describes silt and mud with shell fragments, which might be expected to increase the resistivity above what it would be for pure mud. I chose 6 Ω-m for layer 3, by comparison with sounding 13, which is located nearby.

| Sounding # | Elevation | Resistivity | Thickness | Depth | Interpretation |
|---|---|---|---|---|---|
| | m | Ω-m | (m) | (m) | |
| 9 | 3 | 85 | 1.26 | 5.42 | soil-fill |
| | | 0.6 | 4.16 | | marine mud |
| | | 35 | ∞ | | basement |

This sounding was conducted on Portsmouth Drive near the junction with Portobello Road. Since the sea was located 20 m from the sounding a direct coastal influence is possible. The sounding was conducted at low tide however, and the effect is probably not all that great. I have interpreted the 0.6 Ω-m layer as marine mud. The resistivity of this layer is strongly effected by equivalence, but this is about the maximum value which will allow a good match. If the resistivity was lower the layer would be thinner so 5.4 m is probably an upper bound for the thickness of this layer.

| Sounding # | Elevation | Resistivity | Thickness | Depth | Interpretation |
|---|---|---|---|---|---|
| | m | Ω-m | (m) | (m) | |
| 10 | 3 | 35 | 3.36 | 20.36 | Soil - Fill |
| | | 0.9 | 17 | | Marine mud |
| | | 35 | ∞ | | Basement |

This sounding was conducted on Kitchiner St off Portsmouth Drive Since the sea was located 20 m from the sounding a direct coastal influence is possible. The sounding was conducted at low tide however, and the effect is probably not all that great. I have interpreted the 0.9 Ω-m layer as marine mud. Other resistivities are possible for layer 2 and in this case, the equivalence does have an important effect on the depth to basement I chose 0.9 Ω-m for this layer (which is one of the highest values possible within the equivalence range) because this is as close as possible to values determined for S18, were the mud resistivity is well determined.

| Sounding # | Elevation | Resistivity | Thickness | Depth | Interpretation |
|---|---|---|---|---|---|
| | m | Ω-m | (m) | (m) | |
| 12 | 2 | 9.5 | 23 | 23 | Silty sand |
| | | 95 | ∞ | | Basement |

This sounding was conducted on Carle Park. It is a simple two layer curve. The basement resistivity is not to well constrained but this will not have too much effect on the depth to basement.

| Sounding # | Elevation | Resistivity | Thickness | Depth | Interpretation |
|---|---|---|---|---|---|
| | m | Ω-m | (m) | (m) | |
| 13 | 2 | 44 | 1.5 | 23.5 | Soil - Fill |
| | | 6 | 22 | | salt water sat. sand |
| | | 20 | ∞ | | Basement |

This sounding was conducted on a wide road verge along Royal Cres. The minimum associated with the 6 Ω-m layer (layer 2) is so broad that resistivity (and thus thickness) for this layer is well constrained. There is room for equivalence in both the 44 Ω-m layer (layer 1) and the 20 Ω-m layer (layer 3) but changing these will not have much effect on the depth to basement.

| Sounding # | Elevation | Resistivity | Thickness | Depth | Interpretation |
|---|---|---|---|---|---|
| | m | Ω-m | (m) | (m) | |
| 14 | 2 | 8.5 | 12.8 | 62.4 | peat |
| | | 60 | 3.6 | | fresh water sat. sand |
| | | 4 | 46 | | salt water sat. sand |
| | | 60 | ∞ | | Basement |

This sounding was conducted on a paddock in the center of the Forbury Park Racecourse. The paddock was quite swampy and peat was found when we dug potholes. The minimum associated with the 4 Ω-m layer (layer 3) is so broad that resistivity (and thus thickness) for this layer is well constrained. The same is true for layer 1 where the resistivities at small AB/2 are constant. I have interpreted this layer as peat, based on surface observations, but I am rather surprised that the resistivity of the layer is as low as 8.5 Ω-m since peat saturated with fresh water usually has quite a high resistivity so it may also represent sandy-silt. There is room for equivalence in both layer 2 and layer 4 (both 60 Ω-m) but changing these will not have much effect on the depth to basement.

| Sounding # | Elevation | Resistivity | Thickness | Depth | Interpretation |
|---|---|---|---|---|---|
| | m | Ω-m | (m) | (m) | |
| 16 | | 13 | 1.8 | 19.8 | soil-fill |
| | | 1.3 | 18 | | marine mud |
| | | 40 | ∞ | | basement |

This sounding was conducted on the playing field in the center of the track in the Caladonian Ground All of the layers are subject to equivalence. The minimum associated with the 1.3 Ω-m layer (layer 2) has a particularly broad equivalence range. This is the only parameter that has much effect depth to basement. I have selected 1.3 Ω-m which is near the top of the equivalence range because it is as close as possible to the mud resistivity from sounding 18, where the resistivity is well determined. Similar values were chosen in soundings # 2, 8, 9 and 10.

| Sounding # | Elevation | Resistivity | Thickness | Depth | Interpretation |
|---|---|---|---|---|---|
| | m | Ω-m | (m) | (m) | |
| 18 | 2 | 32 | 1.5 | 31.5 | soil-fill |
| | | 1.5 | 30 | | marine mud |
| | | 40 | | | basement |

This sounding was conducted on the median strip of Andersons Bay Road. The minimum associated with the 1.5 Ω-m layer (layer 2) is so broad that resistivity (and thus thickness) for this layer is well constrained. This is the only parameter that has much effect depth to basement.

| Sounding # | Elevation | Resistivity | Thickness | Depth | Interpretation |
|---|---|---|---|---|---|
| | m | Ω-m | (m) | (m) | |
| 21 | 3 | 7 | 0.88 | 3.88 | soil-fill |
| | | 70 | 3 | | slightly altered volcanics? |
| | | 25 | | | basement |

This sounding was conducted on the grounds of Fulton rest home (ex. Parkside Hospital). There is only a very thin layer of low resistivity sediments at this site suggesting that basement is very near to the surface. I have interpreted the layer to have a resistivity of 70 Ω-m although due to equivalence, any higher value would work. If a higher value is chosen the thickness would be lower so 3 m can be interpreted as an upper bound on the thickness. Basement resistivity is well constrained in this case. I have interpreted the 70 Ω-m layer as part of the volcanic basement although it could represent a clean sand aquifer directly overlying the Volcanic Basement. In any event, depth of basement cannot be greater than 4 m at this site.

NORTH DUNEDIN SOUNDINGS

Locations of the North Dunedin soundings are shown in figure 4.

| Sounding # | Elevation m | Resistivity Ω-m | Thickness (m) | Depth (m) | Interpretation |
|---|---|---|---|---|---|
| 1 | 18 | 37 | 3.0 | 3.0 | soil-fill |
| | | 100 | 22.4 | | Volcanic Boulders |
| | | 20 | ∞ | | Altered Volcanics ? |

This sounding was conducted on on North Ground near the Chemistry Building. There is only a very thin layer of low resistivity sediments at this site suggesting that basement is very near to the surface. I have interpreted the 74 Ω-m layer as a layer of hard volcaniç boulders by comparison with the the log of drill holes at the Chemistry Dept. University of Otago (site report # 107) which is located about 100 m away. As a result the basement is very shallow in this site, as it is at the Chemistry Dept. The depth to basement is not much effected by equivalence.

| Sounding # | Elevation m | Resistivity Ω-m | Thickness (m) | Depth (m) | Interpretation |
|---|---|---|---|---|---|
| 11 | 26 | 200 | 1.2 | 1.2 | soil-fill |
| | | 60 | 14 | | Volcanic Boulders |
| | | 24 | ∞ | | Altered Volcanics |

This sounding was conducted on park between North Rd., Opoho Rd. and SH1, across from the Botanic Gardens . The 200 Ω-m layer probably represents a soil layer, which is is sandy, in comparison to the clay rich soil at North Ground. I have interpreted the 60 Ω-m layer as a layer of hard volcanic boulders by comparison with sounding 1 above and the 24 Ω-m layer probably represents volcanic basement. There is no equivalence in interpreting this sounding except for the 200 Ω-m layer. The value that I have chosen is about the lowest value possible, so other possible models will have a higher resistivity and a lower thickness.

| Sounding # | Elevation m | Resistivity Ω-m | Thickness (m) | Depth (m) | Interpretation |
|---|---|---|---|---|---|
| 15 | 40 | 140 | 8 | 8 | soil Volcanic Boulders? |
| | | 42 | | | Altered Volcanics |

This sounding was conducted on the cricket pitch in Chingford Park. Since the sounding was located against the south-east boundary of the park, it is in the approximate center of the flat bottom of North East Valley at this point. There is no equivalence on either resistivity.

| Sounding # | Elevation m | Resistivity Ω-m | Thickness (m) | Depth (m) | Interpretation |
|---|---|---|---|---|---|
| 17 | 2 | 200 | 1.65 | 3.6 | soil-fill |
| | | 1 | 1.95 | | marine mud |
| | | 40 | ∞ | | basement |

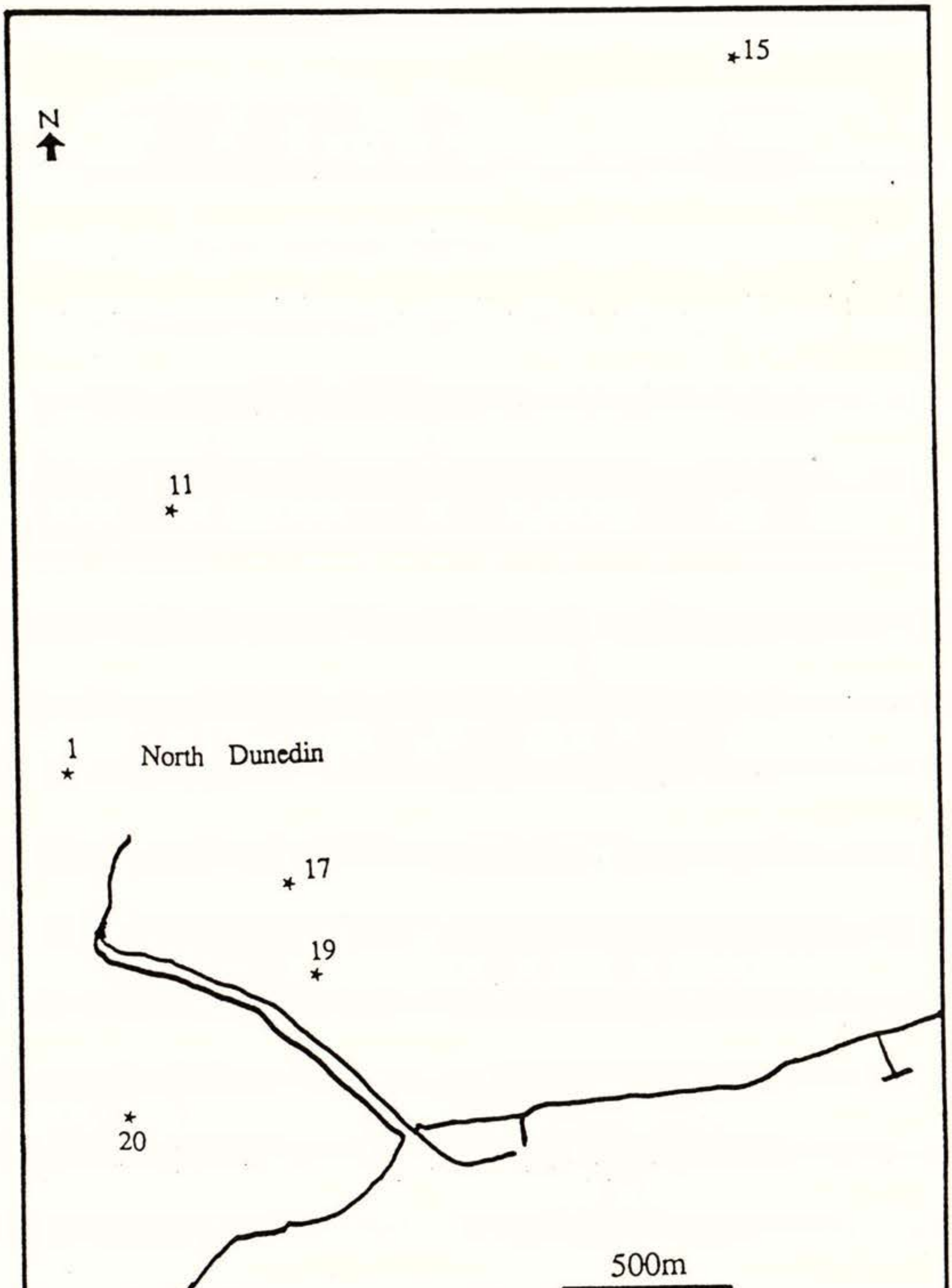
This sounding was conducted on Logan Park, just above the Field Hockey Stadium. There is a considerable room for equivalence in all layers. I have constrained the resistivities on the basis of S 19 which is located on the other side of the park, near some well control. While there is room for equivalence, 6 m is a maximum depth to basement at this site.

| Sounding # | Elevation m | Resistivity Ω-m | Thickness (m) | Depth (m) | Interpretation |
|---|---|---|---|---|---|
| 19 | 2 | 340 | 3.06 | 7.2 | soil-fill |
| | | 1 | 4.14 | | marine mud |
| | | 40 | | | basement |

This sounding was conducted on Logan Park, just opposite the Teachers College. There is considerable room for equivalence in interpreting the sounding so I used the logs from Report 109 to help constrain the interpretation.

| Sounding # | Elevation m | Resistivity Ω-m | Thickness (m) | Depth (m) | Interpretation |
|---|---|---|---|---|---|
| 20 | 2 | 120 | 5.7 | 5.7 | soil |
| | | 17 | | | Boulders? Volcanics? |

This sounding was conducted along Anzac Ave by the Railroad Overbridge. The site was quite noisy so there is considerable scatter in the data however there is very little room for equivalence in the interpretation.




4 Locations of soundings undertaken in North Dunedin as part of this project. Locations of the soundings are indicated by a star and are labeled by number.