METHODOLOGY FOR EARTHQUAKE DAMAGE PREDICTION

Report Prepared for
Earthquake Commission

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DRAFT REPORT

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SUMMARY

The objective of the project was to develop a methodology for providing earthquake risk information for buildings in urban areas using computer databases that can be conveniently used for insurance or planning purposes.

A database of hazard estimates was prepared for general areas within a study region using digital cadastral information, identified geotechnical and geological hazards and regional seismicity predictions. Buildings within the study area were classified according to their risk of damage from strong ground shaking and damage ratios assigned for each of the building classifications. By using both the geotechnical hazard data and the building vulnerability information, annual costs for earthquake damage and the costs arising from a maximum credible event were estimated.

For the study area that contained a large number of older buildings founded on weak soils, the annual earthquake damage cost was found to be very high at about 1% of the replacement cost of the buildings and houses. The damage from a maximum credible event was found to be about 70% of the replacement costs.

CONTENTS

1. INTRODUCTION

1.1 Project Objectives

The risk of damage to buildings during an earthquake depends on a number of factors including, the location of the building relative to the earthquake epicentre, site geology, soil strength and details of the foundation and structural design. The risk can vary widely within a city or urban region and for earthquake insurance purposes or land use planning it is useful to be able to quantify the risk.

Using digital cadastral information it is possible to provide a seismic hazard estimate for building sites or larger geographic areas based on the regional seismicity and the site geotechnical information. This database can then be combined with a similar database of building information to derive seismic damage factors for each building. The building database needs to include classifications of buildings according to their damage potential and relationships between ground shaking intensity and building damage for each classification.

The objective of the study outlined in this report was to develop a methodology for providing earthquake risk information for buildings in urban areas within New Zealand using a computer based system that can be directly accessed for insurance or planning purposes. Using both geotechnical and building structural input parameters, annual costs due to earthquake damage were produced for a study area. The method also provided damage costs from the maximum credible earthquake.

The method developed by the study should enable local authorities and earthquake insurance agencies to obtain quantative assessments of earthquake risk and use this information for reducing future losses in major earthquakes.

1.2 Method of Investigation

The former Petone Borough was used as a study area to develop the methodology.

A computer database of seismic hazard information developed previously for the Petone Borough Council was available for the study. Included in the database was information on the best estimate location of the surface trace of the Wellington Fault and details of areas susceptible to liquefaction during strong ground shaking. A limited amount of soils engineering information was available from records of investigation drillholes.

As part of the present project additional studies were carried out to determine the most appropriate methods of evaluating the ground shaking and liquefaction hazards in the study area.

A review of previously published earthquake damage ratios for buildings was carried out and comparisons made between damage ratios suggested by researchers in USA and New Zealand. In particular, the relevance of the building damage ratios published by

Applied Technology Council, California (ATC Report ATC-13, 1985) to New Zealand buildings was investigated.

A building classification system that took into account the variation in expected performance between different types of structures was adapted from previous work carried out by the New Zealand National Society for Earthquake Engineering. The classification considered such factors as age, materials, load resisting system, foundation type, dimensions, and damage prone details. A damage ratio/ground shaking intensity relationship was assigned to each building classification.

The buildings in the study area, including both commercial buildings and domestic dwellings, were classified by a "street front" field inspection of a many sites and the use of inspection records obtained previously by Petone Borough Council in a project directed at identifying earthquake risk buildings.

The final phase of the work involved integration of the databases of seismic hazard and building information. Computer analyses were undertaken to provide annual direct damage costs and expected costs to the buildings in the study area from a maximum credible earthquake event.

1.3 Study Area

The study area was initially taken as the area of Petone Borough on the eastern side of State Highway 2 as shown in Figure 1. Geotechnical and seismic hazard information was gathered for this major part of the Borough.

To simplify the processing of building data for the evaluation of earthquake damage costs, a reduced study area in the southern part of Petone was used (Figure 1). This reduced area is bounded by the waterfront on the south side; Nevis Street and the Hutt Road on the west; Udy Street on the north and William Street on the east.

The majority of the buildings in the present commercial centre of Petone were constructed prior to 1940 (Spencer, Holmes-Millar & Jackson (SHMJ), 1983) and many are of one or two storey unreinforced masonry construction. Many of these buildings can therefore be expected to be seriously damaged in moderate to large earthquakes. A number of light industrial areas surround the main commercial centre. These buildings include a wide range of construction styles, many have been constructed since the introduction of earthquake design standards in 1936. The number of buildings in the reduced study area totals about 530.

Most of the 1100 residential houses in the reduced study area are of timber frame construction with timber weatherboard exterior cladding and galvanised steel roofs. The majority of the houses were constructed between about 1910 and 1950.

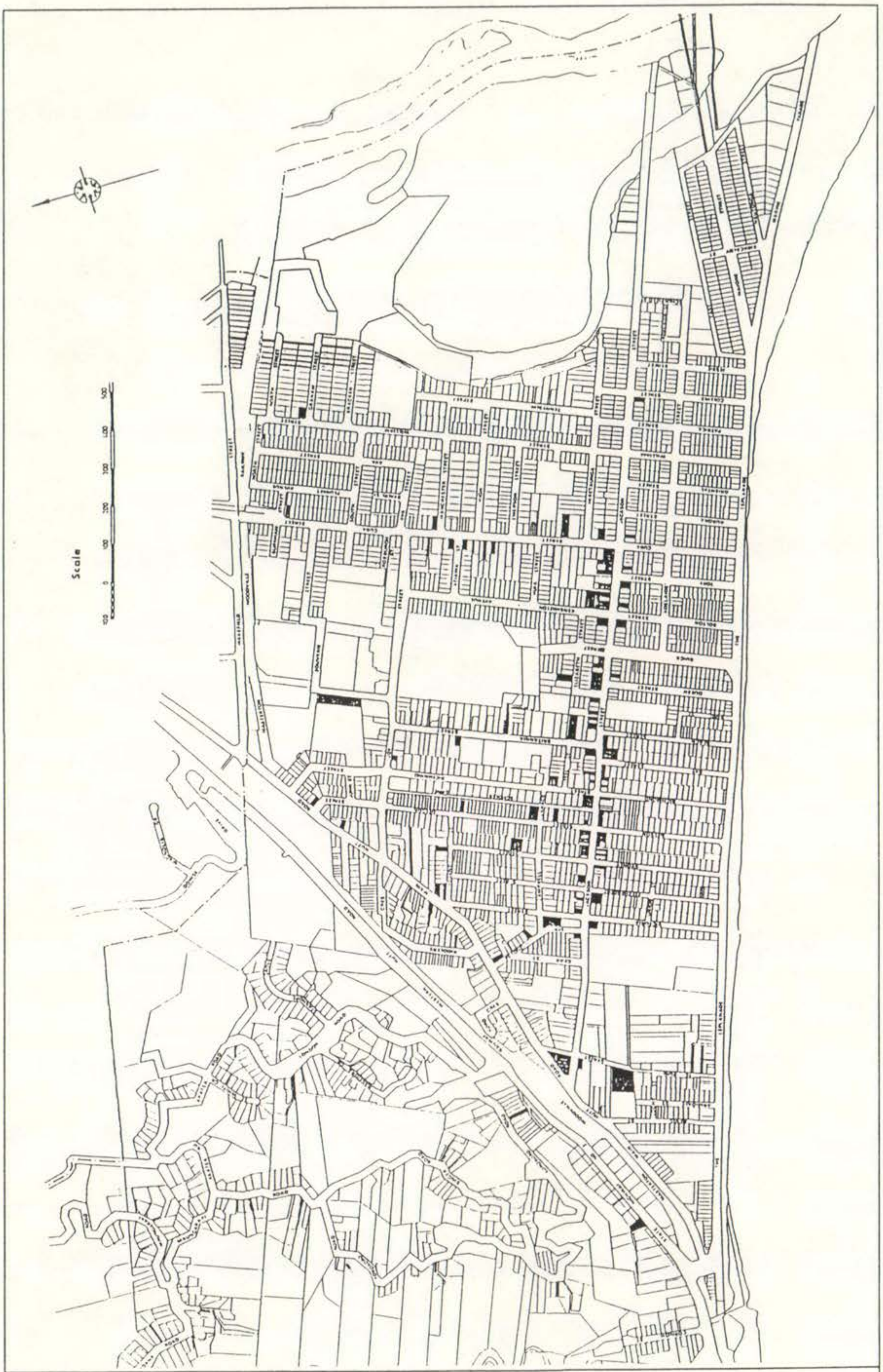

Figure 1. Petone Borough study area.

2. GEOLOGICAL SETTING

2.1 Geomorphology

The Petone landsurface has developed in response to Holocene alluvial and coastal marine processes, and regional uplift. The landsurface is generally of very low gradient, and is crossed by several E-W trending stranded shoreline features (beach ridges). The youngest of these was uplifted c.2 m during the 1855 Wairarapa earthquake, and is now c.200 m back from the present shoreline.

The east facing active scarp of the Wellington Fault runs SW through Petone, generally 1.5 m in height. Three fans occur on the west side of Petone, situated adjacent to gullies at the base of the Western Hutt Hills. The Western Hutt Hills form an abrupt western wall to the valley, structurally controlled by the Wellington Fault (i.e., fault line scarp).

2.2 Wellington Fault

The Wellington Fault passes through the study area and earthquakes from this source dominate the earthquake hazard. The other major seismic sources that may cause significant ground shaking in the study area are mentioned in Section 4.2.2 below. For these other sources, the minimum distances to the study area are of the order of 10 to 30 km and the annual probabilities of rupture are significantly lower.

Van Dissen and Berryman (1991) and Van Dissen et al (in press) have recently published data on the timing, size and recurrence intervals of prehistoric earthquakes in the Wellington region. They provide the following information for the Wellington Fault:

Horizontal Slip Rate	6.0 - 7.6 mm/year
Single Event Horizontal Slip	3.2 - 4.7 m
Recurrence Interval (600 year mean)	420 - 780 years
Elapsed Time Since Last Event	300 - 450 years
Earthquake Magnitude	M_S 7.1 - 7.8

The 75 km long section of the Wellington Fault that extends from the fault's termination in Cook Strait to the 2 km wide right side-step at Kaitoke comprises a single segment. Based on a comparison with surface ruptures throughout the world, and assuming either a 75 km long rupture length or a 4.7 m lateral displacement, a rupture of this Wellington-Hutt Valley segment is expected to be associated with a M_S 7.1 - 7.8 earthquake (Berryman 1990).

The Wellington-Hutt Valley segment is thought to behave in a characteristic rather than a random fashion. This implies a time dependent accumulation of energy between major earthquakes with the repeat times described by a probability curve of approximately log-normal shape (Van Dissen and Berryman, 1991). The probability of an earthquake occurring is a function of the average recurrence interval, the coefficient of variation of the interval and the time since the most recent event.

Using less recent information than given above, Beetham et al (1987) estimated the probability of rupture of the Wellington Fault. Using a preferred best estimate of the recurrence interval of 560 years they gave probabilities of rupture within a 150 year period of 29% and 58% for elapsed times from the last rupture of 300 and 700 years respectively. For design purposes, McVerry and Zhao (1991) interpreted this information to give a present annual probability of rupture of the Wellington Fault as 0.34%.

2.3 Historical Seismicity

The Wellington Region has been shaken by at least three large earthquakes since Polynesian and European settlement began. The 1855, estimated M_s 8.1 Wairarapa Earthquake, and 1848 Marlborough earthquake (estimated M_s 7.1 and MMI VIII; Fairless 1984) caused considerable disruption to early settlement, and localised liquefaction occurred in Petone. Additionally, Maori legend suggests that a large earthquake occurred around 1460 AD, known as the Hau Whenua earthquake (Stevens 1973). Institute of Geological and Nuclear Sciences (IGNS) records show that approximately 12 earthquakes of M_s 5 or greater have occurred within a 50 km radius of Petone since 1840. M_s 5 to 6 earthquakes have occurred most frequently in and around Cook Strait in that time period.

3. QUATERNARY SEDIMENTS

3.1 Stratigraphy

The Petone area is underlain by approximately 300 m of Quaternary sediments that have been deposited in an actively deepening fault angle depression. Sediments are dominantly non-marine, derived from the uplifting greywacke ranges. Stevens (1956) named the suite of sediments Hutt Formation, and subdivided them into the following members:

Moera Basal Gravels: c.200 m of weathered gravels, with yellow coarse sand at the base, and resting on weathered greywacke. The sediments were probably reworked from Tertiary sediments (Haywards Gravels) during the early stages of Quaternary uplift. The member is a significant aquifer, as both greywacke basement and overlying sediments effectively confine groundwater.

Wilford Shellbed: A 1.5 m thick shellbed, usually found at c. 90 m depth, and deposited during the first major incursion of seawater into the lower Hutt Valley in the Late Quaternary.

Waiwhetu Artesian Gravels: This is a c.60 m thickness of dominantly coarse grained sediments, comprising 1-15 m thick beds of dense clayey to sandy gravels and gravelly clays. Groundwater is confined in this member by the fine grained marine sediments above and below the member.

Petone Marine Beds: This member generally comprises fine grained marine sediments, comprising 2-5 m thick beds of silty sands, clays and clayey gravels. The member has been largely deposited in the period 4-7,000 years (Stevens 1956).

Melling Peat: This non marine member comprises c.1-5 m thick beds of carbonaceous silty sands, and is present beneath Central Lower Hutt, to the north of Petone. Radiocarbon dates of 4,350 and 4,275 years B.P. have been determined for the member (Stevens 1956).

Taita Alluvium: A c.12 m thickness of non marine and coastal marine sediments, generally comprising 1-2 m thick beds of silts and silty sands beneath Petone.

3.2 Surface Soils

Figure 2 shows the distribution of geological materials at the ground surface in the study area (Read et al, 1991).

The surface soils at Petone are loose to medium dense, and are correlated with Taita Alluvium (Stevens 1956). The soils have been deposited by coastal marine processes (longshore drift and deposition of nearshore sands), and alluvial processes (Hutt River sedimentation, and build-up of fan deposits adjacent to gullies in the Western Hutt Hills).

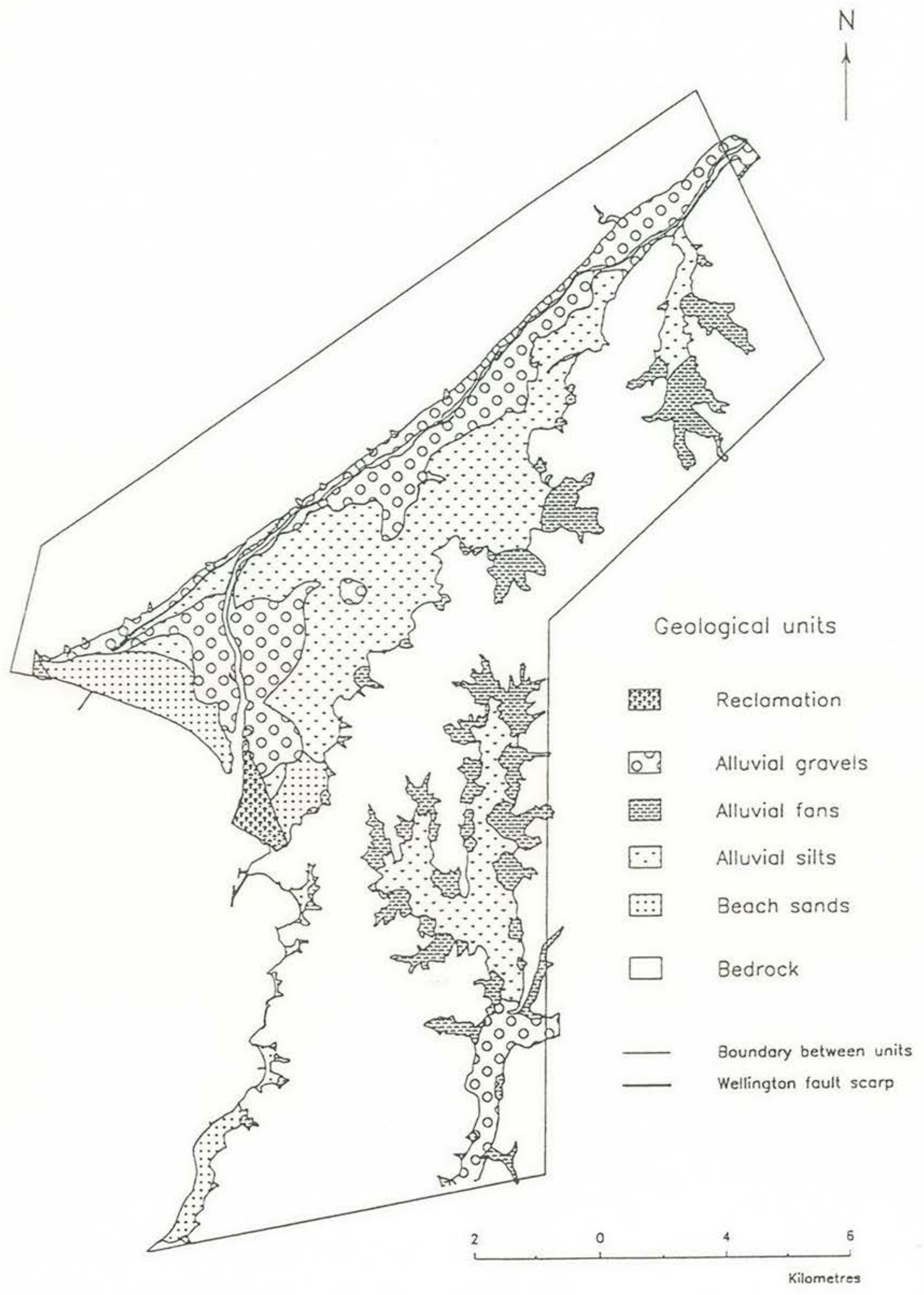

Figure 2. Surface geological map

Drillholes in the Petone area generally show c. 5 m of bedded silts and silty sands. Site investigations have revealed deposits of peat in several areas (Silcock, 1990), which is consistent with much of the Petone area being swampy prior to the 1855 earthquake uplift.

3.3 Subsurface Soils

Structure contours on the base of the soft sediments in the study area and on top of bedrock are shown in Figure 3 and 4 respectively (Read et al, 1991).

The Holocene soils of the Hutt Formation (Taita Alluvium and Petone Marine Beds) are generally less than 30 m thick beneath Petone. They reach their thickest immediately east of the Wellington Fault (c.25 m), thin to the west across the fault (20 m maximum thickness on the upthrown side), and thin gradually to the east. They also thin gradually upvalley. The soft soils of Petone are everywhere underlain by dense Waiwhetu Artesian Gravels.

3.4 Soil Test Data

Very loose to medium dense materials occur in the upper 2-5 m of Taita Alluvium soils (SPT N values typically 1 to 20), while the underlying sandy gravels of the member are typically medium dense to dense (SPT N values typically 14 to 30). These soils overlie silts of the Petone Marine Beds, which generally have SPT N values of 20 to 30 (medium dense to dense).

3.5 Variations Affecting Earthquake Hazard

Three geological zones in the Petone area can be distinguished in terms of earthquake hazards. They are the Wellington Fault zone, the general area west of Wellington Fault, and area east of the fault. The Wellington Fault zone is clearly defined by a scarp through most of Petone (Petone Avenue to Wakefield Street). To the south of Petone Avenue, the fault zone is probably broader, as the fault appears to bend or sidestep towards the west. The soils along the Wellington Fault zone are likely to be disrupted by past fault displacements, and peat deposits are likely to be common along the downthrown eastern side of the fault.

Soft soils (Taita Alluvium) and near surface groundwater levels present a significant earthquake hazard to the zone east of the Wellington Fault. Strong ground shaking, surface deformation and liquefaction would be expected in this area during a large local earthquake. The area could also be downdropped around 0.3-0.4 m during a Wellington Fault displacement event. The zone to the west of the Wellington Fault would have slightly deeper groundwater levels than to the east of the fault, so liquefaction might not be as extensive during an earthquake event.

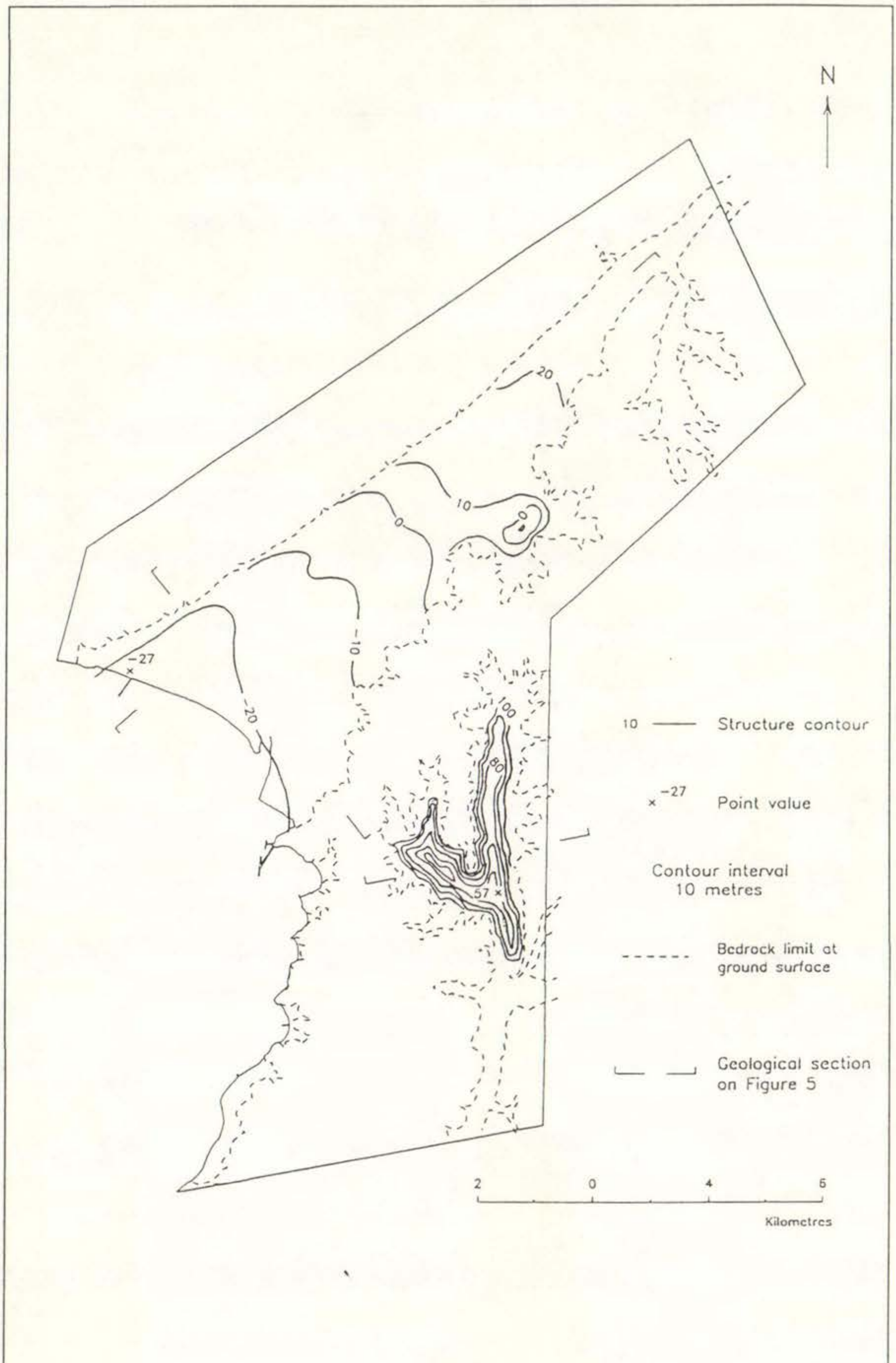

Figure 3. Structure contours on base of soft sediments.

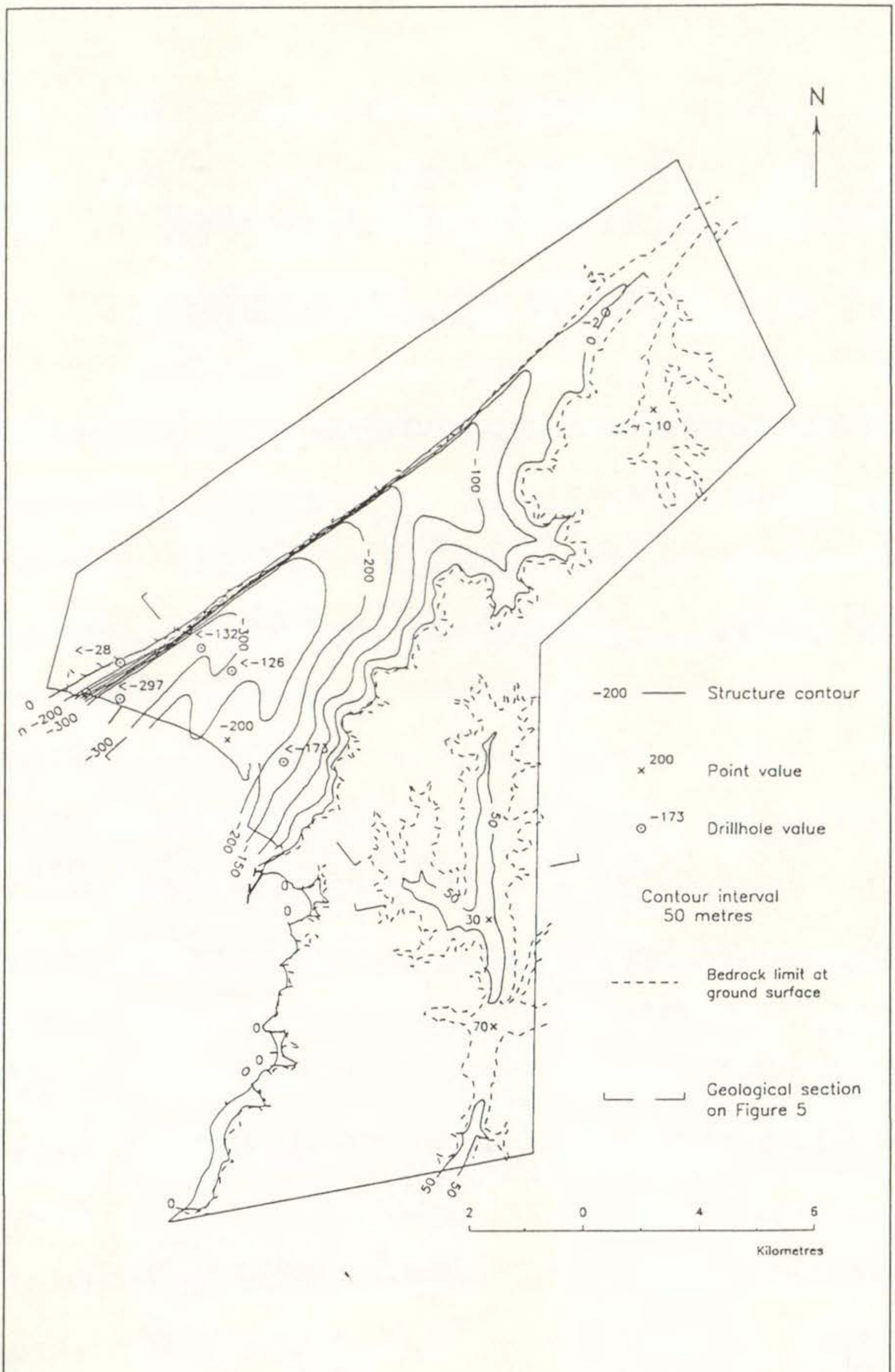

Figure 4. Structure contours on top of bedrock.

4. GROUND SHAKING HAZARD

4.1 Ground Shaking Hazard Zones

As part of the Wellington Regional Council's "Regional Natural Disaster Reduction Plan", the former Department of Scientific and Industrial Research (DSIR) completed a study that identified and quantified the variation in ground shaking expected in the Lower Hutt and Porirua areas from damaging earthquakes (Hastie and Grindell, 1991). Hazard Zones, graded from 1 to 5, are defined that are based on geological, microseismic and small earthquake inputs. The shaking intensity is expected to increase with increasing Zone number.

The Lower Hutt ground shaking Hazard Map, prepared in the DSIR study, is shown in Figure 5. The Petone study area is mainly within Zone 5 but also includes a small area of Zone 2-3 and borders on Zone 2 on the western extremity. Descriptions of the geological materials that typify each hazard zone were summarised in the study (Van Dissen, 1991) as follows:

Zone 1

Greywacke bedrock, including areas overlain by less than 10 m of deeply weathered gravel and loess, or well engineered fill.

Zone 2

Alluvial gravel and fan alluvium; fine to coarse gravel, up to 200 m thick, with some beds and lenses of finer grained sediment (sand, silt, clay and peat) usually less than 5 m thick. The coarser grained sediments typically have SPT N values of 20 to 60.

Zone 3-4

Up to 15 m of fine grained sediment (fine sand, silt clay and peat) within the top 20 m of alluvial gravel, underlain by up to 250 m of alluvial gravels and finer grained sediment. Near-surface fine grained sediments have SPT N values of less than 20. Coarser consolidated sediments have SPT N values of 20 to 60.

Zone 5

Soft sediment (fine sand, silt, clay and peat) up to 10 - 30 m deep near the surface, underlain by bedrock or a variable thickness of gravel and other finer grained sediment.

The shear wave velocity of the soft sediment in Lower Hutt is of the order of 175 m/s.

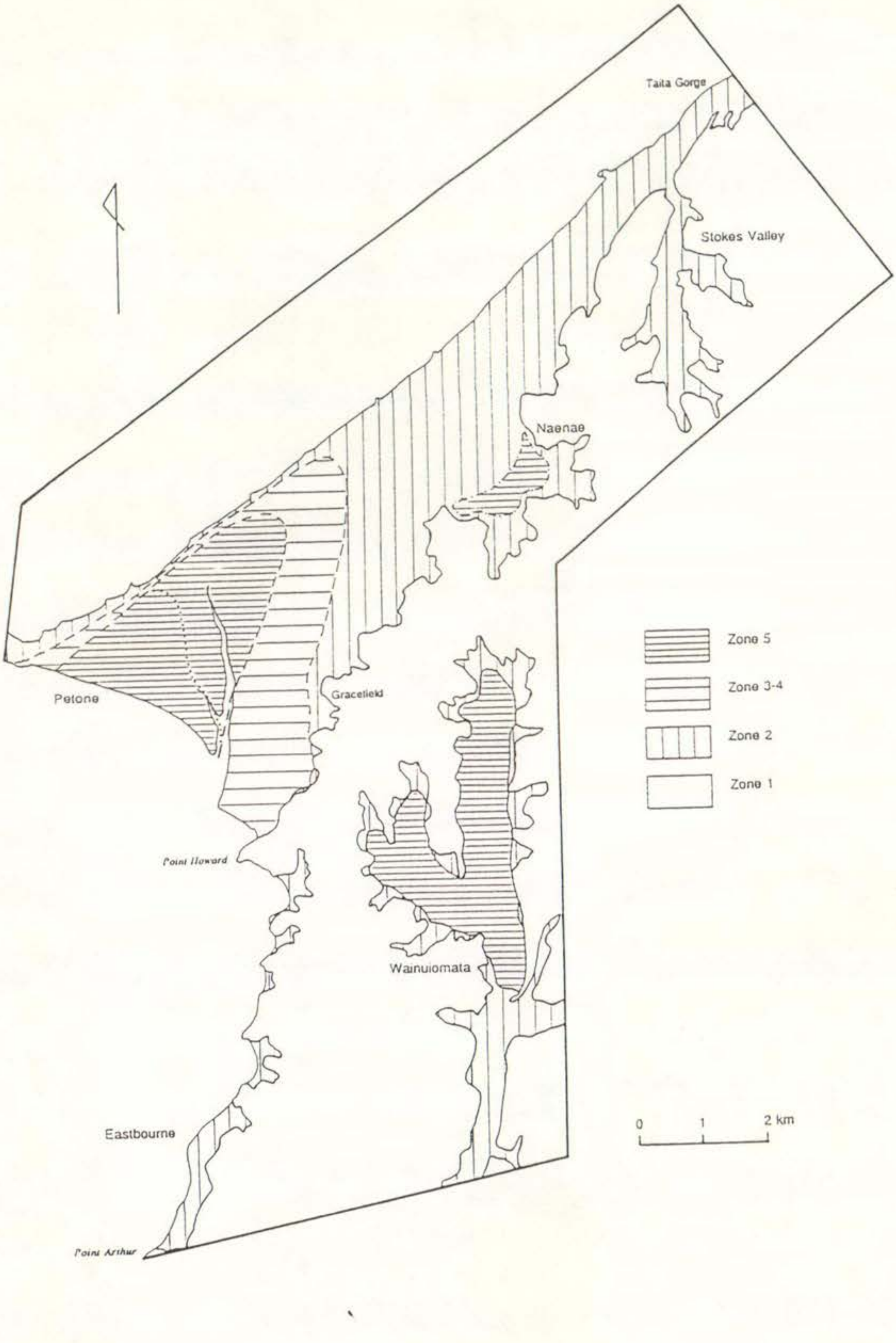

Figure 5. Ground shaking hazard map.

The response of each zone was assessed for two earthquakes. Scenario 1 represented a large distant earthquake and scenario 2 an earthquake on the Wellington Fault.

The Scenario 1 earthquake was assumed to be distant and shallow (<60 km) and to produce Modified Mercalli intensities (MMI) V-VI in bedrock over the Wellington region. The return period of this intensity of shaking is about 20 years. An example given for such an event was an M_S 7 earthquake centred at about 100 km from the study area.

Rupture of the 75 km long Wellington-Hutt Valley segment of the Wellington Fault that extends from Cook Strait through Wellington and the Hutt Valley to Kaitoke was assumed to produce the Scenario 2 earthquake. This earthquake is expected to have a focal depth of less than 30 km with a M_S of about 7.5. The Wellington Fault rupture will produce up to 4.7 m right-lateral and 0.4 m vertical displacement at the ground surface. The probability of this event occurring in the next 30 years is estimated to be of the order of 10%.

A summary of the estimated ground shaking parameters for the Lower Hutt Hazard Zones from each earthquake is given in Table 1 (from Van Dissen, 1991).

In Zone 5, which covers most of the Petone study area, the Scenario 1 earthquake was estimated to produce MMI VIII to IX and the Scenario 2 event MMI X to XI. The peak ground accelerations were estimated as 0.05 to 0.1 g and 0.6 to 0.8 g for the Scenario 1 and 2 earthquakes respectively.

4.2 Intensity of Shaking in Study Area

Most published information on earthquake damage to structures is related to ground shaking intensity expressed in terms of MMI. To assess the risk of structural damage to the buildings in the study area it is therefore necessary to know the frequency of occurrence of strong ground shaking in terms of MMI.

The main limitation of using MMI is that the records from past earthquakes tend to average out the influence of differing ground conditions. The published values generally refer to average ground conditions over areas of up to about 100 km^2. To estimate the effects of local geology on the damaging potential of strong shaking it is necessary to make rather arbitrary adjustments to the MMI to allow for amplification effects from soft soils and possible deamplification if the site is on rock. Smith (1977) has suggested that on poor soil the return period for the next lower intensity should be used.

Because of the loose nature of the surface soils in most of the Petone study area, the MMI's will be higher than indicated by regional average values.

An advantage of using MMI to quantify shaking intensity is that there is a large data base of MMI records from both recent and historical earthquakes.

TABLE 1

Ground motion parameters for Ground Shaking Hazard Zones (From Van Dissen, 1991.)

SCENARIO 1

<table>
<tr><th colspan="2">ZONES</th><th>MM Intensity</th><th>Peak ground acceleration (g)</th><th>Duration</th><th colspan="2">Amplification of ground motion (Fsr) and frequency</th><th>Amplification of peak of acceleration response spectra (5% damped)</th><th>SA max* (g) and period</th><th>SV max* (m/s) and period</th></tr>
<tr><td colspan="2">1</td><td>V-VI</td><td>0.02-0.06</td><td><5 sec</td><td colspan="2">1-3 × @ >1 Hz</td><td></td><td>0.1 @ 0.2-0.3 sec</td><td>0.06 @ 0.3-0.4 sec</td></tr>
<tr><td colspan="2">2</td><td>VI</td><td>0.02-0.1</td><td>2-3 ×</td><td colspan="2">2-5 × @ 0.5-5 Hz</td><td><5×</td><td>0.4 @ 0.3-0.4 sec</td><td>0.3 @ 0.4 sec</td></tr>
<tr><td colspan="2">3-4</td><td>VI-VII</td><td>0.02-0.1</td><td>2.3 ×</td><td colspan="2">5-10 × @ 0.5-3 Hz</td><td>2-5×</td><td>0.5 @ 0.4 sec</td><td>0.4 @ 1.0 sec
0.3 @ 0.4 sec</td></tr>
<tr><td rowspan="5">5</td><td>Porirua</td><td rowspan="5">VIII-IX</td><td rowspan="4"><0.3 generally between 0.1-0.2</td><td rowspan="5">>3 ×</td><td rowspan="5">10-20×</td><td>@ 2 Hz</td><td rowspan="3">>4 ×</td><td>not calculated</td><td>not calculated</td></tr>
<tr><td>Naenae</td><td>@ 1 Hz</td><td>0.4 @ 0.8 sec</td><td>0.6 @ 0.9 sec</td></tr>
<tr><td>Wainuiomata (shallow)</td><td>@ 2 Hz</td><td>1.0 @ 0.5 sec & 0.7 @ 0.3 sec</td><td>0.7 @ 0.5 sec</td></tr>
<tr><td>Wainuiomata (deep)</td><td>0.5-3 Hz, with a peak at 1 Hz</td><td>>5 ×</td><td>0.9 @ 0.3 sec & 0.7 @ 0.4 sec</td><td>0.7 @ 1.1 sec</td></tr>
<tr><td>Lower Hutt</td><td><0.2 generally around 0.05-0.1</td><td><3 Hz, down to at least 0.5 Hz</td><td>2-5 ×</td><td>0.3 with several peaks between 0.5 - 1.5 sec</td><td>0.4-0.5 with several peaks between 0.5-1.8 sec</td></tr>
</table>

* These values were estimated for specific sites within a Hazard Zone, and may not be general enough to characterise the response of the Zone as a whole

SCENARIO 2

<table>
<tr><th colspan="2">ZONE</th><th>MM Intensity</th><th>Peak ground acceleration (g)</th><th>Duration</th><th>Amplification of peak of acceleration response spectra (5% damped)</th></tr>
<tr><td rowspan="2">1</td><td>near fault</td><td>IX</td><td>0.5-0.8</td><td rowspan="2">15-40 sec</td><td rowspan="2"></td></tr>
<tr><td>Porirua & Wainuiomata</td><td>VIII</td><td>0.3-0.6</td></tr>
<tr><td rowspan="2">2</td><td>near fault</td><td>IX-X</td><td>0.5-0.8</td><td rowspan="2">1-2 ×</td><td rowspan="2"><5 ×</td></tr>
<tr><td>Porirua & Wainuiomata</td><td>VIII-IX</td><td>0.3-0.6</td></tr>
<tr><td colspan="2">3-4</td><td>IX-X</td><td>0.5-0.8</td><td>1-2 ×</td><td>2-5 ×</td></tr>
<tr><td rowspan="2">5</td><td>near fault</td><td rowspan="2">X-XI</td><td>0.6-0.8</td><td rowspan="2">>2 ×</td><td rowspan="2">>5 ×</td></tr>
<tr><td>Porirua & Wainuiomata</td><td>0.5-0.8</td></tr>
</table>

In contrast, there have been only a few strong motion accelerograph records obtained within 50 km of the epicentres of damaging New Zealand earthquakes. A further advantage is that MMI is a direct measure of the damaging effects on structures and therefore tends to integrate the influence of both the severity and duration of the ground shaking. In contrast, other simple methods of describing the ground motion intensity, such as peak ground or spectral acceleration, ignore the duration of strong shaking and therefore do not necessarily provide a good measure of the damage potential of the earthquake.

4.2.1 Seismicity Model Predictions of MMI

The frequency of occurrence of MMI intensity levels for any location in New Zealand can be obtained from the seismic hazard analysis work of Smith and Berryman (Smith, 1977; Smith and Berryman, 1983 and 1986; Smith, 1990, and Smith and Berryman, 1992). These studies developed attenuation expressions relating the MMI to the earthquake source, magnitude and distance from the source and combined this information with a seismicity model which relates the earthquake magnitude and frequency of occurrence for a number of specific seismo-tectonic regions. The analytical procedure is summarised below.

An intensity formula is derived from the recorded data and inverted using an iterative computer program to give:

$$M = M(p, d, h, I) \qquad (1)$$

Where
- M = magnitude
- p = location of observer
- d = distance of observer from epicentre
- h = focal depth
- I = MM intensity

From the seismicity model of a particular region, the annual frequency, per 1,000 km^2, of earthquakes of magnitude M or greater is given by:

$$N(M) = a_4[10^{b(4-M)} - 10^{b(4-Mmax)}] \qquad (2)$$

Where a_4 and b are seismic frequency parameters that vary from region to region and M_{max} is the maximum magnitude assumed for a region. The parameter a_4 is in fact the annual number of earthquakes of magnitude 4 or greater in an area of 1,000 km^2.

The annual frequency of occurrence of intensity I at a particular site is given by the following area integration:

$$N(I) = \int N(M)\, da \qquad (3)$$

Where N(M) is given by Equation 2 and M by Equation 1. The integration is taken over all source regions in which earthquakes of sufficient magnitude occur to cause intensity I or greater at the site.

The attenuation relationship derived for the Wellington region (including Hutt Valley) is shown in Figure 6. The effective epicentral distance shown on the plots is intended to take into account the elliptical shape of the isoseismals. In the Wellington area, the major axis is orientated at about N 40 E with the effective epicentral distance the actual distance in this direction. Distances on the minor axis are reduced by a factor of about 0.8.

Figure 7 shows the relationship between MM intensity and annual frequency of occurrence for Wellington and the Hutt Valley derived from the Smith and Berryman (1992) study. For the soft soil in the Petone study area, MM IX and X can be expected to occur with mean return periods of 100 and 300 years respectively.

4.2.2 Source Model Predictions of MMI

An alternative approach for estimating the frequency of occurrence of strong shaking is to consider the main seismic sources in the region and to estimate the occurrence of strong shaking at the site caused by each source. As for the uniform seismicity model approach, an attenuation relationship that gives either peak acceleration or MMI intensity in terms of the distance from the fault or seismic source is required. The source model approach is strictly speaking more precise than the uniform seismicity model method which produces regional averages of intensity and does not account for faults or other sources that might be very close to the site under investigation. However, it has limitations in that the total length or area of faults and other sources may not be well defined and the location of epicentres along known faults is likely to occur randomly.

Because the Wellington Fault runs through the Petone study area, the Smith and Berryman uniform seismicity model will underestimate to some extent the frequency of occurrence of the higher MMI values. (Intensities of perhaps IX and greater). From Beetham et al, (1987) the probability of rupture of the Wellington Fault over the next 150 years, based on the mean recurrence interval of 560 years and the time since the last rupture, estimated to be between 300 to 700 years, is between 30 to 60%.

The length of the Wellington Fault segment between Kaitoke and the south coast of Wellington is about 75 km, with the Petone area located at 25 km from the southern extremity. From the attenuation relationship of Smith and Berryman (1983) shown in Figure 6, an M_L 7.5 earthquake on the Wellington Fault will produce intensities equal to or greater than MMI IX (average soil conditions) within an effective epicentral distance of about 42 km. For the soft surface soils in Petone, the MMI is likely to equal or exceed MMI X within this epicentral distance. An epicentre located at a point on the 75 km length of the fault segment will lie within the 42 km epicentral distance over about 90% of the length of the fault segment. Therefore, the probability that an earthquake associated with the fault rupture will result in ground shaking with MMI X or greater in the Petone area is about 0.9. Similarly, the probability that the Wellington Fault rupture will result in MMI of XI or greater in the Petone soft soils is about 0.6.

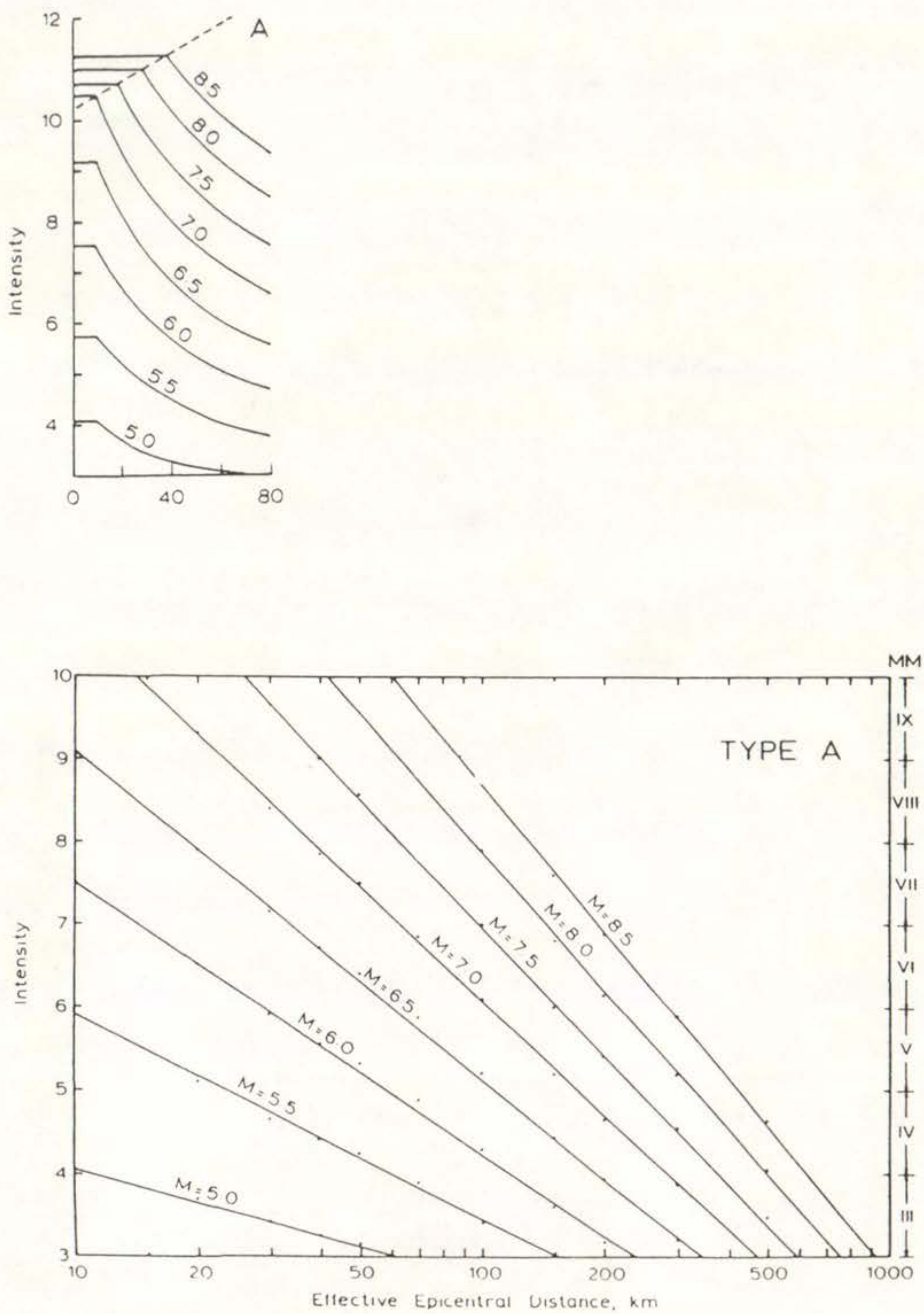

Figure 6. MMI attenuation with distance. (From Smith, 1977).

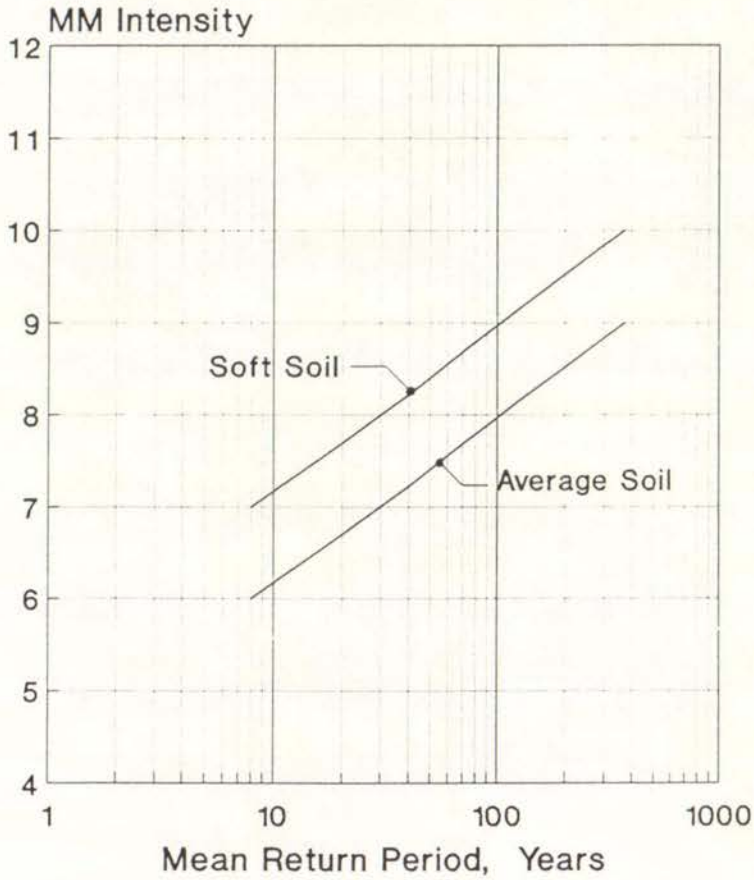

Figure 7. Return periods for MMI levels. (From Smith and Berryman, 1992)

Considering the rupture of the Wellington Fault alone and taking the annual probability of rupture as 0.34% (McVerry and Zhao, 1991), the return periods of MMI's X and XI are therefore about 330 and 500 years respectively. This estimate makes no allowance for the uncertainty in the estimated magnitude of the characteristic earthquake or the uncertainty in the attenuation relationship.

Other major sources in the Wellington region that are likely to result in strong ground shaking in the Petone area (Beetham et al, 1987; Wellington Regional Council, 1990) are listed in Table 2 which also gives estimates of the source recurrence intervals and the overall annual probability for each source of producing MMI X in Petone. The location of the fault sources in relation to the study area is shown in Figure 8. All the sources given in Table 2 are capable of producing MMI X on soft soil, but for some sources, the epicentre may occur either within or outside the maximum effective epicentral distance required to produce shaking of this intensity. Earthquakes on the West Wairarapa Fault and within the subduction zone could produce MMI's equal to or exceeding XI on soft soils in the study area if their epicentres occur within about 40 km of Petone. However, the probability of MM XI is low because of the relatively long recurrence intervals and the fact that many of the events on these sources would occur at distances greater than 40 km from the study area.

If the other known nearby sources are considered together with the Wellington Fault, the mean return period for MMI's X and XI on soft soil in the study area are about 250 and 400 years respectively. These compare with the Smith and Berryman regional mean return periods of about 250 and 1,000 years (soft soils). The Smith and Berryman regional seismicity model therefore appears to give satisfactory predictions of return periods in the study area for MMI's of X or less.

4.3 Peak Ground Acceleration in Study Area

Most methods of assessing the earthquake risk of soil liquefaction and slope stability are related to peak ground acceleration (PGA). For earthquake hazard studies it is therefore convenient to have ground shaking expressed in terms of PGA as well as MMI. As for the case of MMI, the peak ground acceleration can be obtained from either a regional seismicity or a source model approach.

4.3.1 Seismicity Model Predictions of PGA

Because of the limitations of the New Zealand strong motion data base, the most satisfactory method for estimating the frequency of occurrence of PGA is to use MMI values from the Smith and Berryman studies and convert these to PGA using a standard correlation relationship.

TABLE 2

Probability estimate for equalling or exceeding MMI X on firm soil.

Earthquake Source	Charact Magn. M	Min d to Site km	Max Dist for MMI X km	Prob >= MMI X	Annual Prob of Rupture	Combine Prob
Wairau	7.5	30	28.00	0.00	0.0011	0.00000
Shepherds Gully	7.5	15	28.00	0.30	0.0003	0.00009
Ohariu - Por. Seg	7.3	9	20.00	0.60	0.0005	0.00030
Ohariu - Kahao Seg	7.3	25	20.00	0.15	0.0008	0.00012
Wellington	7.5	1	28.00	0.60	0.0034	0.00204
West Wairarapa	8.1	17	42.00	0.20	0.0002	0.00004
Subduction Zone	8.4	30	58.00	0.30	0.0003	0.00009

0.00268
373

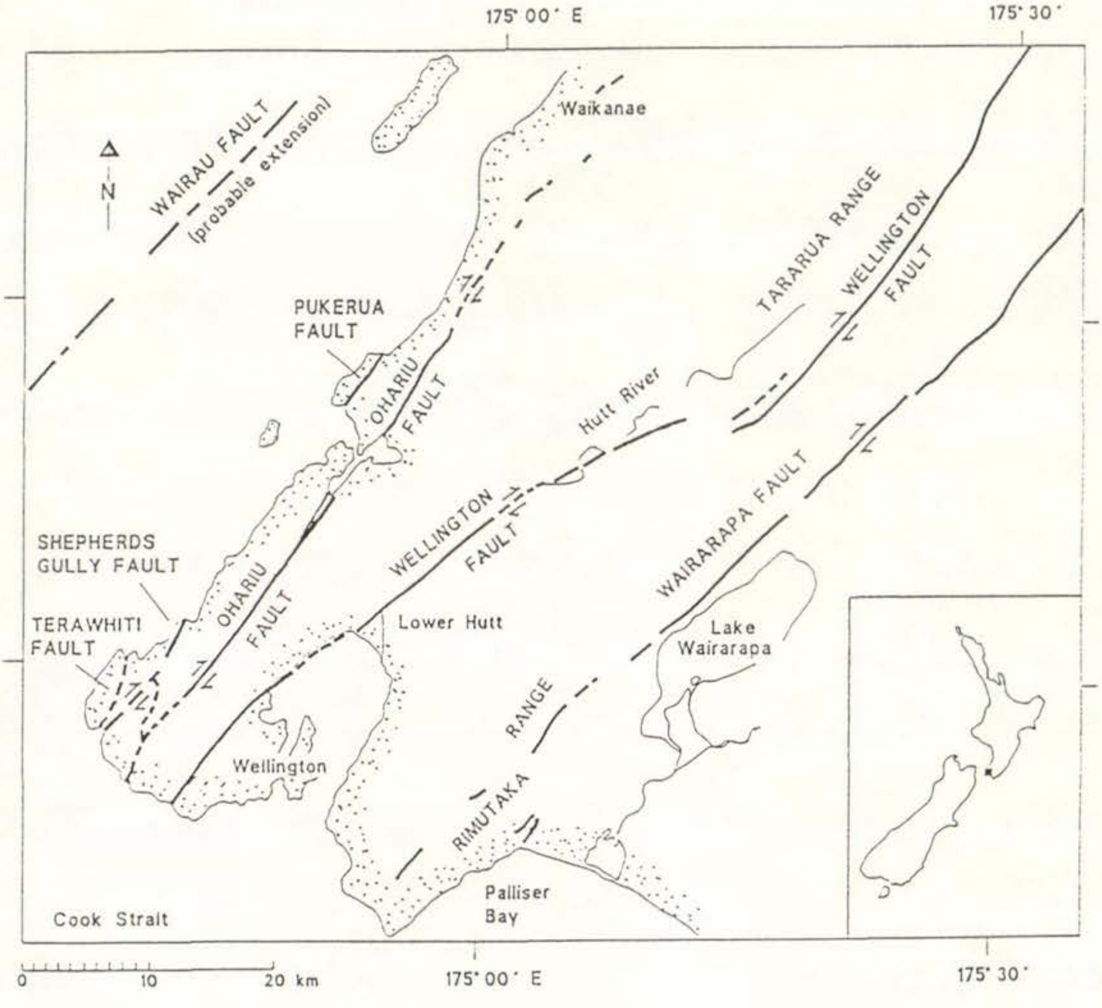

Figure 8. Active faults in the Wellington Region.

A comprehensive study of the correlation between MMI and PGA has been carried out Krinitzsky and Chang (1988). A world-wide set of 679 accelerograms of horizontal motion was used in the study and the data was grouped into near and far field and into hard or soft site conditions. The correlation relationship for soft soil sites and near-field ground motions is the most appropriate for the study area and was given as:

$$\log a = 1.320 + 0.138\, I_{MM} \qquad (4)$$

Where

a = peak ground acceleration in cm/s^2
I_{MM} = Modified Mercalli intensity

Equation 4 gives PGA values of 0.37, 0.51 and 0.70 g for MMI values of IX, X and XI respectively.

Equivalent expressions are given by Krinitzsky and Chang (1988) for hard soil sites and far-field motions.

The PGA occurrence frequencies were estimated for the study area using the Smith and Berryman uniform seismicity model and Equation 4. The results of this analysis are compared in Figure 9 with the return periods obtained by the acceleration attenuation/uniform seismicity model approach described below.

An alternative approach is to use the Smith and Berryman uniform seismicity model relationships between earthquake magnitude and frequency of occurrence, together with an appropriate attenuation relationship expressed in terms of PGA rather than MMI. (The Smith and Berryman attenuation relationship is not directly suitable because it is expressed in terms of MMI). The Joyner and Boore (1981) attenuation relationship developed from an analysis of western North America strong motion records has gained wide acceptance and is in a convenient form for earthquake hazard analysis work. The relationship for PGA is:

$$\log a = 0.49 + 0.23(M-6) - \log r - 0.0027r \qquad (5)$$

Where

a = peak ground acceleration
r = $\sqrt{(d^2 + 64)}$
d = shortest distance between site and vertical projection of fault rupture on the surface
M = earthquake moment magnitude

Expression 5 is for the larger of the two horizontal components. Joyner and Boore also give a similar expression to Expression 5 for the randomly orientated PGA. In this case, the leading coefficient of 0.49 reduces to 0.43 with the other terms remaining unaltered.

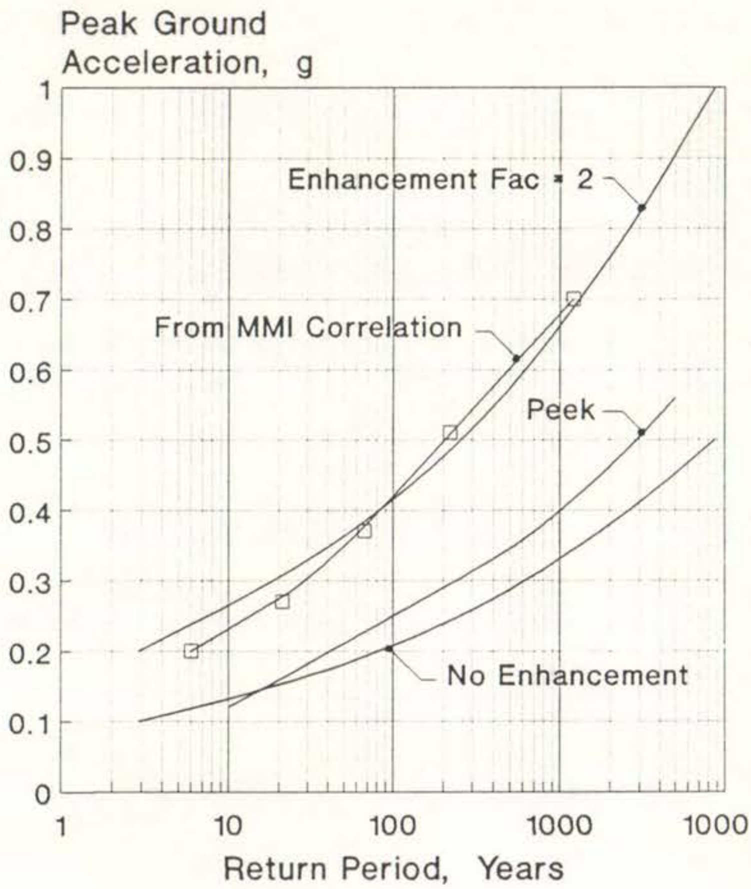

Figure 9. Return periods for peak ground accelerations.

For a M 7.5 earthquake on the Wellington Fault, Equation 5 gives a peak ground acceleration at zero distance of 0.81 g. At distances of 5 and 10 km this acceleration attenuates to 0.68 and 0.49 g respectively. Equation 5 is not bounded by recorded data at distances less than about 10 km to the source and will not necessarily give reliable predictions at distances within 10 km.

Similar attenuation expressions to the Joyner and Boore relationships have also be derived by Campbell (1981) and Krinitzsky et al (1988). Campbell used a similar data base but Krinitzsky et al used a more extensive world-wide set of strong-motion accelerograms.

The Katayama (1982) attenuation relationship has been used in past New Zealand hazard studies but does not appear to be a good fit to the limited amount of recorded New Zealand data. It gives a slower decay rate and lower PGA near the source than the Joyner and Boore expression. There are no New Zealand near-source data for verification of attenuation relationships but the limited more distant records show attenuation that fits the Joyner and Boore expression better than Katayama. McVerry (1986) analysed scratch plate peak accelerations from the Inangahua earthquake and found that PGA were approximately proportional to $d^{-1.1}$ for distances greater than 20 km. He reported that other New Zealand data showed that the decay exponent might be about -2.0 instead of -1.1. At distances greater than 20 km from the epicentre, the Joyner and Boore relationship has an equivalent decay exponent of between -1.1 and -1.5 and thus seems in reasonable agreement with the limited New Zealand data.

A recent analysis by Fukushima and Tanaka (1990) of both Japanese and western United States strong motion records resulted in the development of an attenuation relationship very similar to that of Joyner and Boore. The Fukushima and Tanaka expression gives near-source PGA that are about 7% higher than given by Joyner and Boore. This difference increases to about 30% at 20 km from the source and increases to a maximum value of about 43% at greater distances. Most of the difference between the two expressions, particularly at epicentral distances greater than 20 km, was attributed to the lower accelerations evident in the United States records. (No Japanese records at epicentral distances less than 16 km were available for analysis).

The Fukushima and Tanaka rate of attenuation of PGA with epicentral distance was significantly greater than given by Katayama and it appears that there where shortcomings in the statistical correlation procedure used by Katayama.

The Joyner and Boore analysis showed no significant statistical difference between PGA on rock or soil. However, the Fukushima and Tanaka study indicated that PGA were about 40% less on rock sites and about 40% greater on soft soil than given by their attenuation expression which was initially derived using all records without recognition of the different site conditions. The attenuation expression was found to give a good approximation to the mean PGA for hard and medium soil sites.

The frequency of occurrence of PGA was estimated for the Petone study area using the

Smith and Berryman seismicity model. The procedure used was similar to the method adopted by Smith and Berryman (1983) but with the Joyner and Boore attenuation given by Equation 5 used instead of the Smith and Berryman plotted relationships (Figure 6). The numerical evaluation was carried out on a spreadsheet employing the following steps.

(a) Select PGA Range

A PGA range of 0.1 to 0.7 g divided into six 0.1 g intervals was adopted. The objective of the analysis was to determine the probability of equalling or exceeding each acceleration level.

(b) Source Areas

The earthquake source areas were assumed to be circular annulii centred on the Petone study area with the maximum radius taken as 80 km. Each annulus had a width of 1 km.

In general, the circular annulii intersected a number of seismo-tectonic regions with differing a_4 and b values as shown in Figure 10. For these cases, the areas were subdivided into areas within each region.

(c) Earthquake Magnitudes to Exceed Acceleration Levels

For each source region, the attenuation expression (Equation 5) was used to calculate the smallest magnitude earthquake, M_o necessary to produce each of the 7 PGA levels at the site.

(d) Annual Probability of Exceedance of Accelerations

The probability of the selected site acceleration levels being equalled or exceeded by a M_o or greater magnitude earthquake in each source area was calculated using Equation 2 (Smith and Berryman model). The total annual probability of exceedance for each acceleration level was then found by summing the individual probabilities for each source area.

The mean return period for each acceleration level was calculated by taking the reciprocal of the annual probability of exceedance.

(e) Enhancement Factor

Peek (1980) has shown that if a deterministic attenuation expression is used it is necessary to apply an enhancement factor to either the PGA or the return periods to allow for the statistical uncertainty in the attenuation relationship. Methods of calculating the enhancement factor are given by Peek (1980) and McVerry (1986).

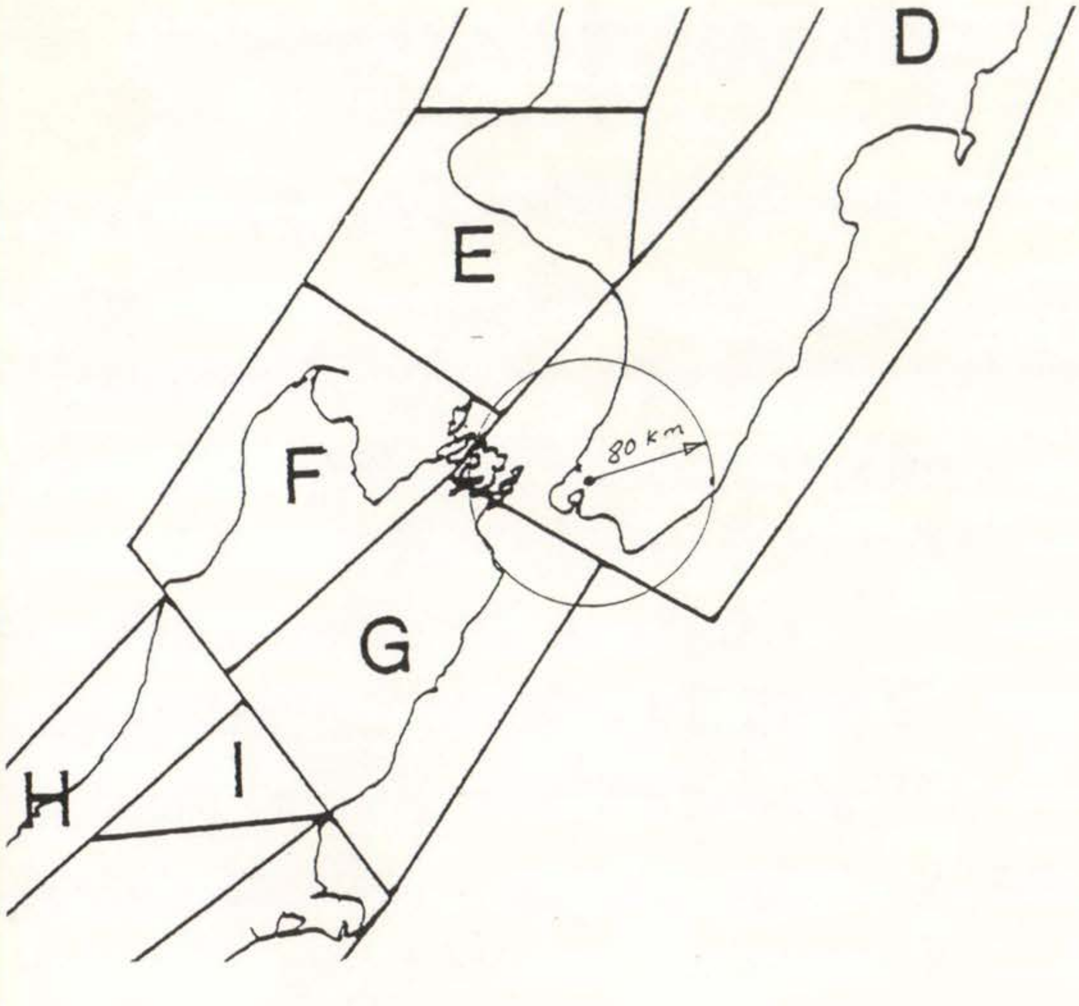

Figure 10. Seismo-tectonic regions. (From Smith and Berryman, 1992).

In this study, an enhancement factor of 2.0 was applied to the PGA. It is thought that this value, although satisfactory for higher accelerations, may be too large for accelerations below 0.3 g.

The calculated return periods for each of the selected PGA levels are shown in Figure 9. The results are compared with a similar study carried out by Peek (1980) for the Haywards site which is about 10 km north of the Petone study area. Peek used a different seismicity model and the results shown in Figure 9 were obtained using the Katayama attenuation relationship for a rock site (or a shallow depth of firm soil). The plotted results suggest return periods of, 100 and 700 years for PGA of 0.4 and 0.6 respectively. These results are in good agreement with the return periods obtained by correlation of the PGA with MMI from the Smith and Berryman study.

4.3.2 Source Model Predictions of PGA

In a recent study, McVerry and Zhao (1991) computed PGA's for the Wellington city area at return periods between 500 and 2,000 years using a source model consisting of the nearby active faults and the subduction zone with an acceleration attenuation expression proposed by Idriss (1985) for rock sites. The distribution of the peak ground accelerations at the site expected from each source was assumed to be log normal, with the standard deviation of the natural logarithm of the peak ground acceleration taken as 0.35. They did not include the Wairau Fault source and considered the Ohariu Fault to be a single source with a larger recurrence interval than given in Table 2 for the individual segments. The PGA for deep soil sites were estimated from the rock accelerations using the Idriss relationship between rock and soil PGA shown in Figure 11. The attenuation relationship of Idriss is similar to the Joyner and Boore expression and was apparently developed from a similar data base.

Following the method of McVerry and Zhao, return periods were estimated for a range of PGA in the Petone study area. The sources considered and a summary of the procedure for a PGA of 0.6 g is shown in Table 3. The Joyner and Boore rather than the Idriss attenuation relationship was used. Joyner and Boore give the logarithm (base 10) of the standard deviation of the PGA as 0.28. The results from the analysis are shown in Figure 12 and indicate return periods of 420 and 620 years respectively for PGA of 0.6 and 0.8 g respectively. These return periods are significantly lower than indicated by the uniform seismicity model results shown in Figure 9.

From Table 3, the probability that a mean PGA value of 0.6 g will be equalled or exceeded in the study area by a rupture on the Wellington Fault is approximately 58%. If this probability is combined with an annual probability of rupture of the Wellington Fault of 0.34%, the overall annual probability of exceedance of a PGA of 0.6 g is 0.197%. This probability is equivalent to a return period of about 500 years. A Wellington Fault event, producing a PGA of 0.6 g, can therefore be taken for the study area to represent the 500 year return period earthquake.

TABLE 3

Probability estimate for equalling or exceeding PGA of 0.6g.

Earthquake Source	Charact Magn. M	Min d to Site km	Mean PGA at Site g	z = log(PGA/M) /0.28	Pexc.	Annual Prob of Rupture	Combine Prob
Wairau	7.5	30	0.18	1.85	0.03	0.0011	0.00004
Shepherds Gully	7.5	15	0.36	0.78	0.22	0.0003	0.00007
Ohariu - Por. Seg	7.3	9	0.47	0.37	0.36	0.0005	0.00018
Ohariu - Kahao Seg	7.3	20	0.25	1.36	0.09	0.0008	0.00007
Wellington	7.5	5	0.68	-0.20	0.58	0.0034	0.00197
West Wairarapa	8.1	17	0.45	0.46	0.32	0.0002	0.00006
Subduction Zone	8.4	30	0.29	1.11	0.13	0.0003	0.00004

Total Ann. Prob. = 0.00242

Return Period = 413

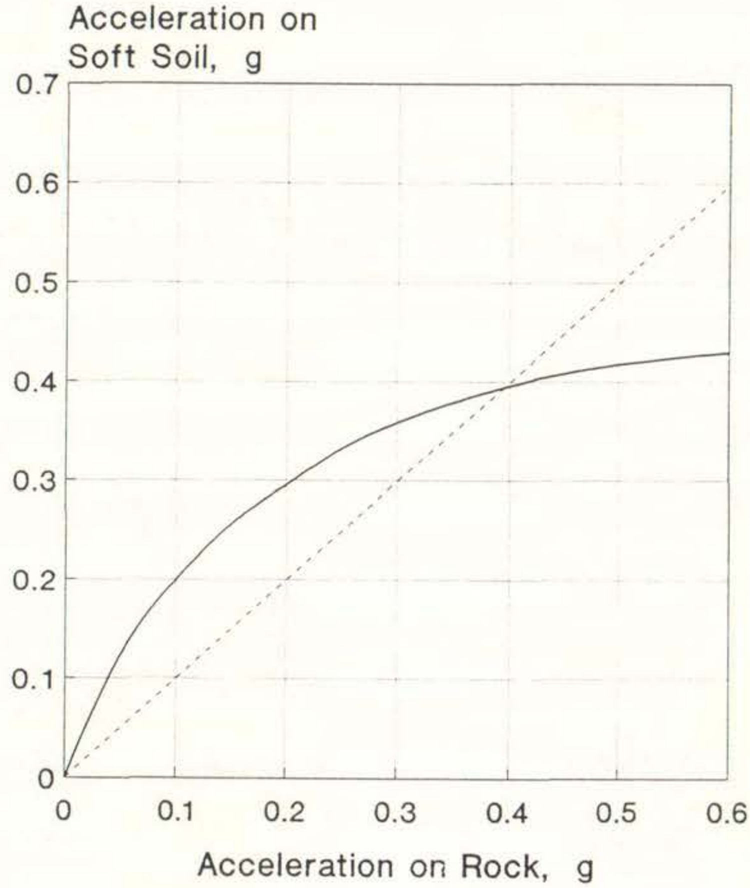

Figure 11. Accelerations of soft sites versus rock sites. (From Idriss, 1990).

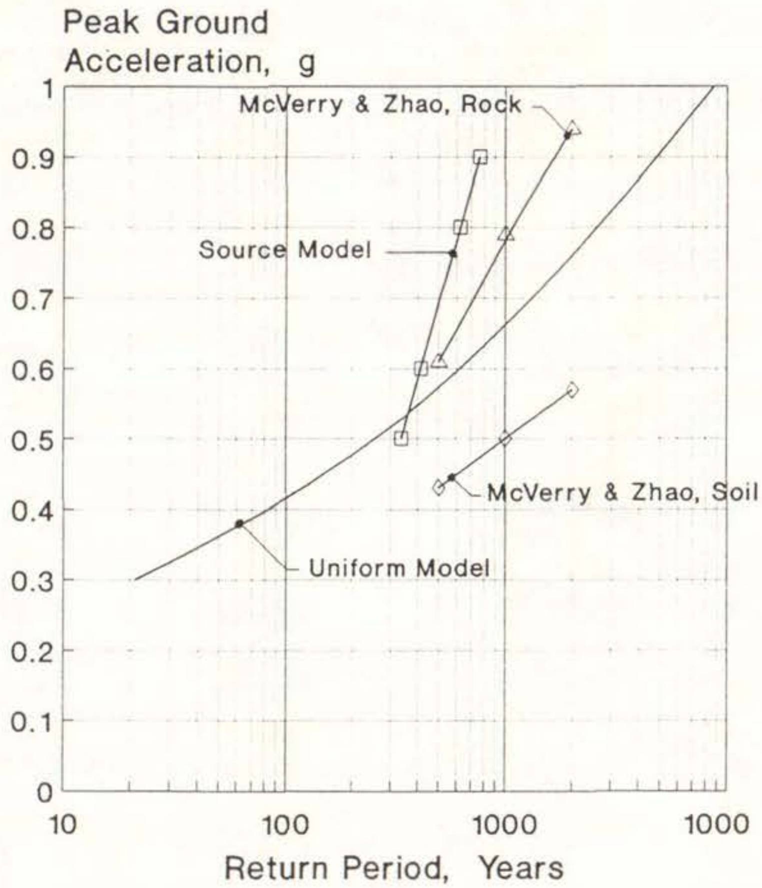

Figure 12. Return periods for PGA from source model.

4.3.3 Uncertainty in PGA

The following factors give rise to a large range uncertainty in the mean predicted PGA values.

(a) Uncertainty in the distance between the centre of earthquake energy release and the distance to site.

(b) There is a wide range of scatter in recorded PGA with the prediction methods giving a best fit to the mean value. For example, a variation of one standard deviation either side of the mean value of 0.6 g results in a range of PGA from about 0.31 to 1.1 g (Joyner and Boore).

(c) The magnitude of the characteristic earthquake on the fault can only be estimated and may vary between successive events. The Van Dissen et al estimate is for a magnitude M_s between 7.1 and 7.8. Corresponding PGA at the site, from the Joyner and Boore prediction, are 0.48 and 0.7 g respectively.

Only the Uncertainty due to (b) has been considered in the estimates of probability and return period given above.

4.4 Soil Amplification Effects

Figures 7 and 9 indicate that MMI IX corresponds to about a PGA of 0.5 g for average soil conditions. At lower MM intensities and PGA, soft soil sites will tend to amplify the peak ground accelerations and at higher intensities the nonlinear behaviour of soft layers may reduce the PGA as shown in Figure 11.

In terms of damage to buildings, it is usually the spectral acceleration response produced by the site ground motion rather than the PGA that is important. This is because most building structures have periods of vibration of the lowest horizontal mode in the range of 0.3 to 5 sec. That is, significantly greater than zero. In this case the earthquake forces on the building are approximately proportional to response spectrum ordinate corresponding to the first mode period of vibration.

The Joyner and Fumal (1985) studies of strong motion records from sites in western North America resulted in statistical estimates of response spectra ordinates related to magnitude and distance from source as well as the PGA discussed previously. These analyses produced spectra for both soil and rock sites and investigated the influence of soil stiffness by estimating the shear wave velocity of the soil at the recording site. The site shear wave velocity was taken to be the average velocity over the depth of a quarter wave length of a 1 s period wave. Plots of the ratio of the soil spectral ordinates over the rock spectral ordinates for three soil conditions are shown in Figure 13. The soil cases plotted are for the average of all records at soil sites and for sites with average shear wave velocities of 300 and 500 m/s. It is thought that the 300 m/s shear wave velocity is representative of the average soil properties over a quarter wave length depth in the Petone area.

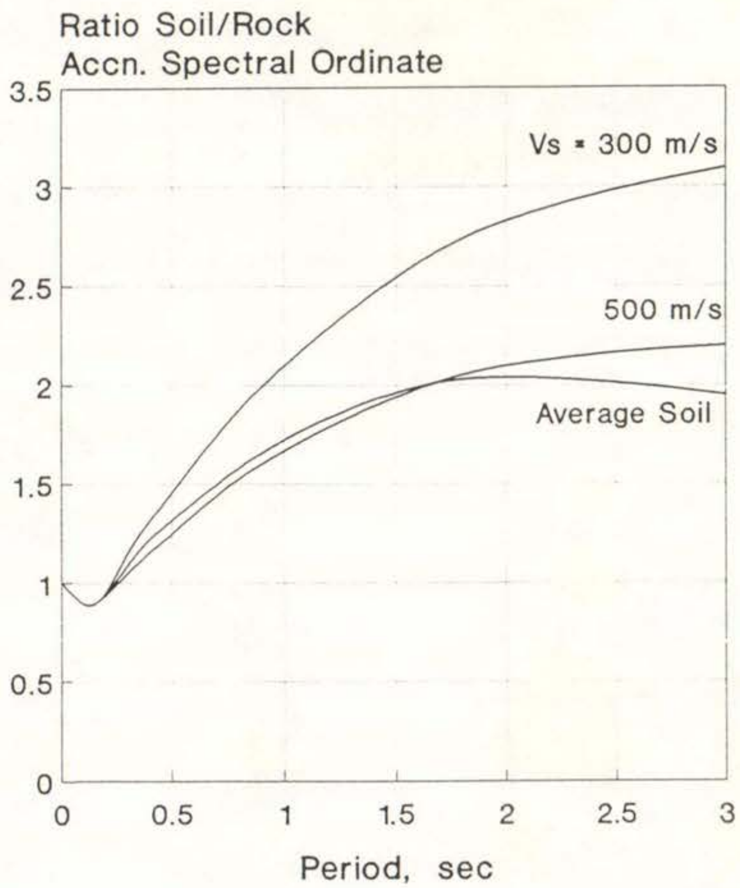

Figure 13. Amplification of spectral ordinates by soft soils.

From Figure 13 it is apparent that amplifications of spectral accelerations of up to 3 can be expected in the longer period range on soft soil sites. The amplification of spectral accelerations becomes significant at periods of vibration greater than about 0.4 s. Most of the buildings in the Study area are one or two stories and most will therefore have horizontal periods of vibration of less than 0.5 s. For these buildings, the effects of soil amplification of the ground accelerations are probably small and it is likely that ground failure and settlement of the soft soils will be more critical than amplification.

4.5 Summary

From the investigations described above, typical values of return periods for both MMI and PGA have been derived. The values in Table 4 have been adopted for estimating the damage to buildings on the soft soils of the study area.

TABLE 4
Return periods for ground shaking levels.

MMI	Return Period Years
VII	6
VIII	20
IX	70
X	220
XI	400

PGA g	Return Period Years
0.3	50
0.4	100
0.5	250
0.6	500
0.8	1000

The recommended method for estimating the frequency of occurrence of MMI is to use the Smith and Berryman uniform seismicity model with modifications from the Smith and Berryman attenuation curves if the site is very close to known active earthquake sources. A satisfactory method of estimating PGA is to work from the Smith and Berryman MMI values and the Krinitzsky and Chang MMI/PGA correlation equation.

5. LIQUEFACTION HAZARD

During an earthquake, shear stresses induced in the soil layers by ground shaking may cause the granular particles in saturated soil to re-arrange leading to the generation of excess pore water pressure. Liquefaction occurs if the pore water pressures increase sufficiently for the granular soil to transform from a solid to a liquid state. The following four types of ground failure commonly result from liquefaction of soil layers during earthquakes (Youd and Perkins, 1978):

(a) Lateral spread
(b) Flow failure
(c) Ground oscillation
(d) Bearing strength failure

Lateral spreads involve the lateral displacement of surfical blocks of soil as a result of liquefaction in a subsurface layer. Gravitational and inertia forces from the ground accelerations may cause the block to move downslope or towards a cut or natural slope face. Lateral spreads have been recorded on gentle slopes that range between 0.3° to 3.0° and displacements of several meters or more may occur. This mode of failure may cause heavy damage to pipelines, services and structures with shallow foundations.

Flow failures are similar to lateral spreads but usually develop on slopes greater than about 3° with displacements in the order of tens of meters.

Ground oscillation takes place if liquefaction occurs at some depth and if the slopes are insufficient to result in lateral spreading. The overlying blocks of soil tend to oscillate on the liquefied substrate. Ground settlement, soil fissures and sand boils may accompany the oscillations.

Loss of bearing strength occurs when the soil layers under the foundation of a structure liquefy. Large settlements and tilting of the structure may occur. Buoyant structures, such as tanks and pipes may float upwards.

Mapping of liquefaction hazards has been carried out in a number of regions of the world, including many parts of United States and Japan. Unfortunately no extensive hazard mapping has been carried out in New Zealand. Youd and Perkins (1978) have developed a technique in which two constituent maps showing "liquefaction susceptibility" and "liquefaction "opportunity" are superimposed to produce a "liquefaction potential map".

5.1 Liquefaction Susceptibility

The susceptibility of soils to liquefy depends on a number of factors including (Juang and Elton, 1991):

(a) **Type of Deposit.** River-channel and flood-plain deposits are more susceptible to liquefaction than other deposits such as alluvial fans and residual soils.

(b) **Age of Deposit.** The more recently a sediment has been deposited, the more likely it is to be susceptible to liquefaction.

(c) **Depth of Water Table.** A water table close to the surface increases the risk of liquefaction.

(d) **Soil Density.** Loose soils are more susceptible to liquefaction than dense soils.

(e) **Soil Density and Strength as Indicated by SPT N-value.** The SPT test gives a measure of how densely packed the sediment is.

(f) **Depth of Burial.** Soil layers covered with between 1.5 to 6 m of surfical soil are generally more likely to liquefy than deeper or shallower layers.

(g) **Grain Size.** Sand and silty sand (grain sizes between 1 mm to 0.075 mm) are more likely to liquefy than coarser or finer grain sizes. Gravelly sands are less likely to liquefy and coarse gravels and deposits containing more than 15% clay are not known to liquefy.

(h) **Grain Size Distribution.** Liquefaction is more likely in uniform grain sizes.

(i) **Liquefiable Layer Thickness.** Thicker layers are likely to be more susceptible to liquefaction than thin layers.

(j) **Areal Extent of Liquefiable Deposit.** Liquefaction is more likely to occur in deposits with large areal extent.

(k) **Capping With Low Permeability Layer.** Liquefaction is more likely where the liquefiable layer is covered by a less permeable soil layer.

(l) **Ground Slope.** The severity of spreading and lateral deformation increases with increasing ground slope.

A detailed qualitative assessment of the susceptibility of soils to liquefy can be made using the ratings and weighting factors given for the above factors by Juang and Elton (1991).

Liquefaction susceptibility is primarily dependent on sediment type, relative density and water table depth. From a study of liquefaction occurrences during past earthquakes, Youd and Perkins (1978) developed the criteria listed in Table 5. These qualitative criteria form the basis for most geologic criteria used for mapping liquefaction susceptibility. Reference to Table 5 and considering the distribution of near surface soft soils and shallow groundwater levels, suggests that soils over much of the Petone study area are in the very high to high range of liquefaction susceptibility. In particular, the area to the east of the Wellington Fault (especially near the harbour shoreline) would be prone to liquefaction and lateral spreading during strong ground shaking. Fault displaced soils along the Wellington Fault zone may also be prone to liquefaction and settlement.

The most generally accepted geotechnical method of evaluating the liquefaction resistance of sediments is based on SPT field testing and was developed by Seed and colleagues (Seed et al 1983). It is referred to as "the simplified procedure for evaluating liquefaction potential". If sufficient SPT data is available it is would be possible to use both the geological and geotechnical information to develop a precise liquefaction susceptibility map. As is the case in the Petone study area, the SPT data is often widely spaced and limited to specific sites where construction development or planning has taken place. It is therefore necessary to place considerable reliance on geological data to prepare susceptibility information over extensive study areas. However, site investigations for specific developments often provide the information required for the liquefaction susceptibility of the soils to be reliably predicted over limited areas. A brief description of the Seed "simplified procedure" for estimating liquefaction susceptibility is given below.

The ratio between the earthquake induced cyclic shear stress and the effective vertical stress at some depth below the surface is given by Seed and Idriss (1971) as:

$$\tau_{av}/\sigma_o' = 0.65\,(a_{max}/g)\,(\sigma_o/\sigma_o')\,r \qquad (6)$$

where:

	τ_{av}	=	average cyclic shear stress
	σ_o	=	total overburden pressure on layer
	σ_o'	=	initial effective overburden pressure
	a_{max}	=	maximum acceleration at ground surface
	g	=	acceleration of gravity
	r	=	stress reduction factor (varies from 1 at surface to 0.9 at 9.6 m)

The liquefaction resistance of the soil is determined from empirical charts that relate the standard penetration N-value to the critical cyclic stress ratio at which liquefaction is likely to occur. The charts are usually expressed in terms of N_1, the SPT N-value corrected for overburden pressure. The corrected N_1 value is given by:

$$N_1 = C_N\,N \qquad (7)$$

Where C_N is a function of the overburden pressure as shown in Figure 14.

TABLE 5

Susceptibility of sedimentary deposits to liquefaction. (From Youd and Perkins, 1978.)

Type of deposit (1)	General distribution of cohesionless sediments in deposits (2)	Likelihood that Cohesionless Sediments, When Saturated, Would Be Susceptible to Liquefaction (by Age of Deposit)			
		<500 yr (3)	Holocene (4)	Pleistocene (5)	Pre-pleistocene (6)
(a) Continental Deposits					
River channel	Locally variable	Very high	High	Low	Very low
Flood plain	Locally variable	High	Moderate	Low	Very low
Alluvial fan and plain	Widespread	Moderate	Low	Low	Very low
Marine terraces and plains	Widespread	—	Low	Very low	Very low
Delta and fan-delta	Widespread	High	Moderate	Low	Very low
Lacustrine and playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dunes	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	High	?	?
Residual soils	Rare	Low	Low	Very low	Very low
Sebka	Locally variable	High	Moderate	Low	Very low
(b) Coastal Zone					
Delta	Widespread	Very high	High	Low	Very low
Esturine	Locally variable	High	Moderate	Low	Very low
Beach					
High wave energy	Widespread	Moderate	Low	Very low	Very low
Low wave energy	Widespread	High	Moderate	Low	Very low
Lagoonal	Locally variable	High	Moderate	Low	Very low
Fore shore	Locally variable	High	Moderate	Low	Very low
(c) Artificial					
Uncompacted fill	Variable	Very high	—	—	—
Compacted fill	Variable	Low	—	—	—

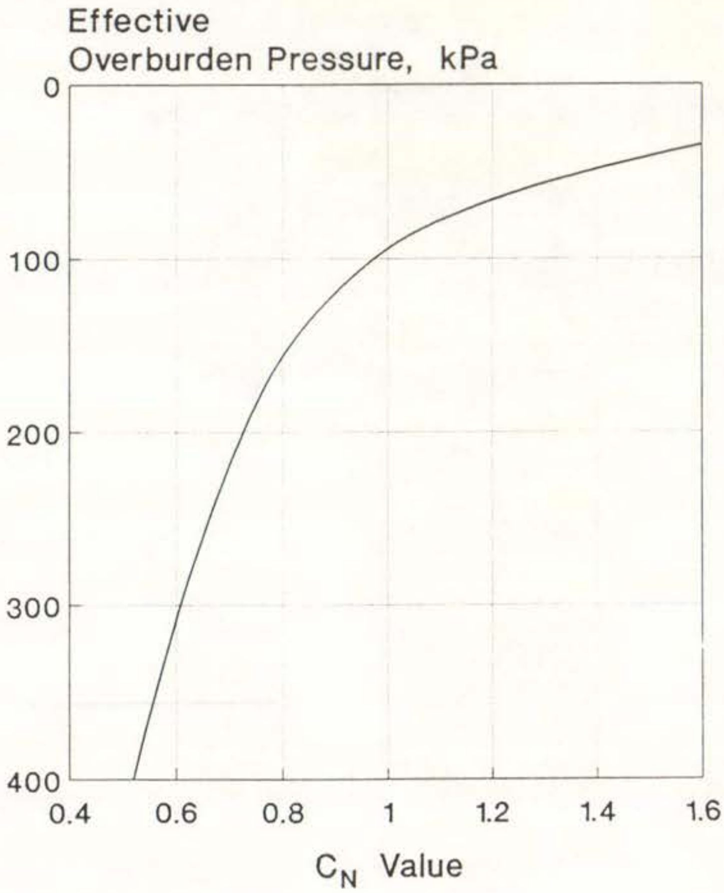

Figure 14. Overburden pressure correction factor. (From Seed et al, 1983).

A further correction needs to be applied to the N-values to allow for the difference between the equipment commonly used in New Zealand and that assumed in the derivation of the Seed empirical charts. The Seed uncorrected N-values are approximately a factor of 1.4 greater than the uncorrected New Zealand equipment N-values.

Figure 15 shows the Seed et al (1983) empirical charts that relate the critical liquefaction stress ratio to the modified penetration resistance N_1-values for various magnitude earthquakes. The earthquake magnitude is an important parameter because the liquefaction susceptibility depends on the number of cycles of shear stress applied to the soil as well as the shear stress amplitude. The curves shown in Figure 15 are for clean sand. The same curves may be used for silty sands, provided the N_1 is increased by 7.5 before entering the chart.

5.2 Liquefaction Opportunity

The major factors affecting liquefaction opportunity are the frequency of earthquake occurrence and the intensity of ground shaking in major events. To develop an opportunity map, an earthquake source model is required that gives locations of earthquake sources and the number and magnitudes of earthquakes occurring in the study area. Also required is an attenuation relationship that defines either the attenuation of ground shaking or the limiting distance to which liquefaction might be generated as a function of earthquake magnitude (Youd, 1991). For example, from observed occurrences of liquefaction, Youd and Perkins (1978) developed the magnitude-distance criterion compared in Figure 16 with a similar relationship suggested by Ambrasseys (1988). These criteria provide a bounding distance for significant liquefaction effects, characterised by ground displacements of about 100 mm or more.

From the magnitude-distance criteria given in Figure 16 it is clear that major earthquake events on all the potential earthquake sources for the study area (Table 2) will provide the opportunity for liquefaction within susceptible soils in the study area.

The occurrence of liquefaction is of less importance than the severity or the capability of liquefaction effects to cause damage to structures. Youd and Perkins (1987) introduced the concept of "liquefaction severity index (LSI)", which is defined as the general maximum lateral spread on liquefiable, gently sloping late-Holocene fluvial and deltaic deposits. The LSI index used is the maximum horizontal spread measured in inches. Anomalously large displacements are neglected. Youd and Perkins plotted LSI values against horizontal distances from the sources for several western United States earthquakes and analysed the data to develop the chart shown in Figure 17.

From the LSI severity chart (Figure 17) it is apparent that a Wellington Fault event would produce an LSI value of at least 100 (2.5 m lateral displacement on gently sloping ground) for susceptible soils in the Petone study area. This level of severity would cause extensive damage to structures located in zones of liquefaction.

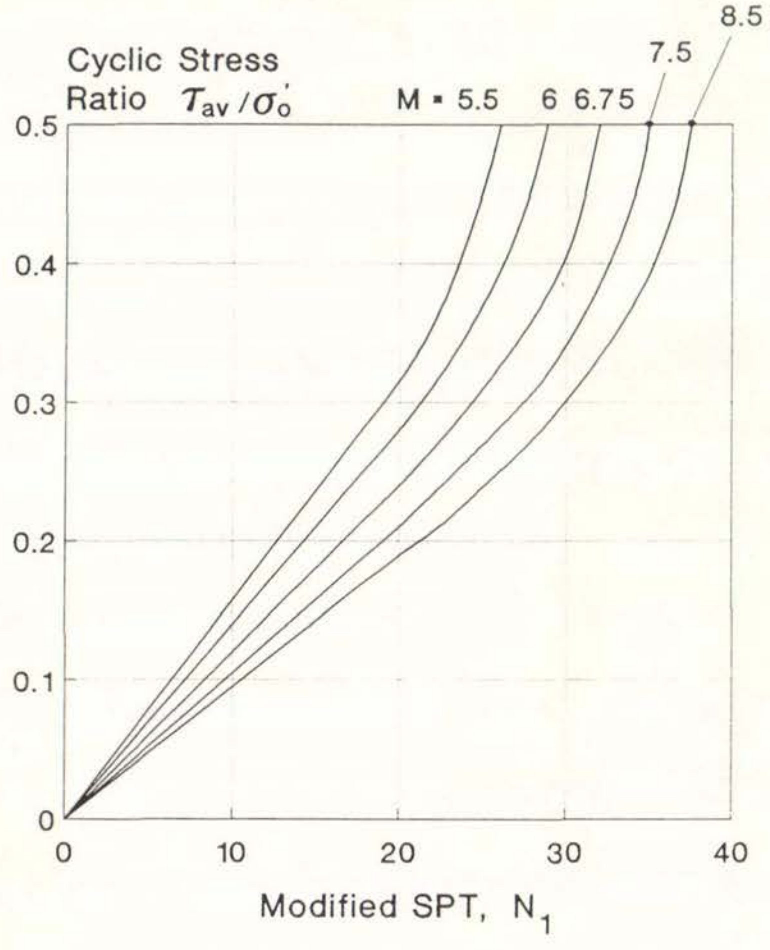

Figure 15. Evaluation of liquefaction susceptibility. (From Seed et al, 1983.)

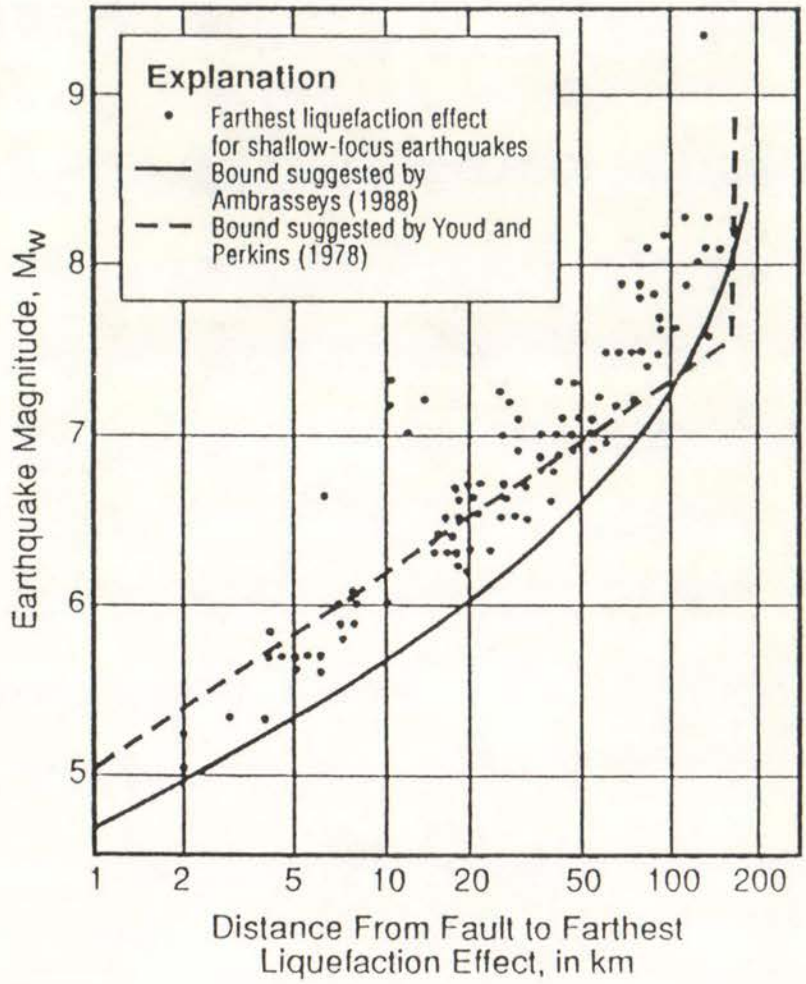

Figure 16. Distance from source to farthest liquefaction site. (From Youd, 1991)

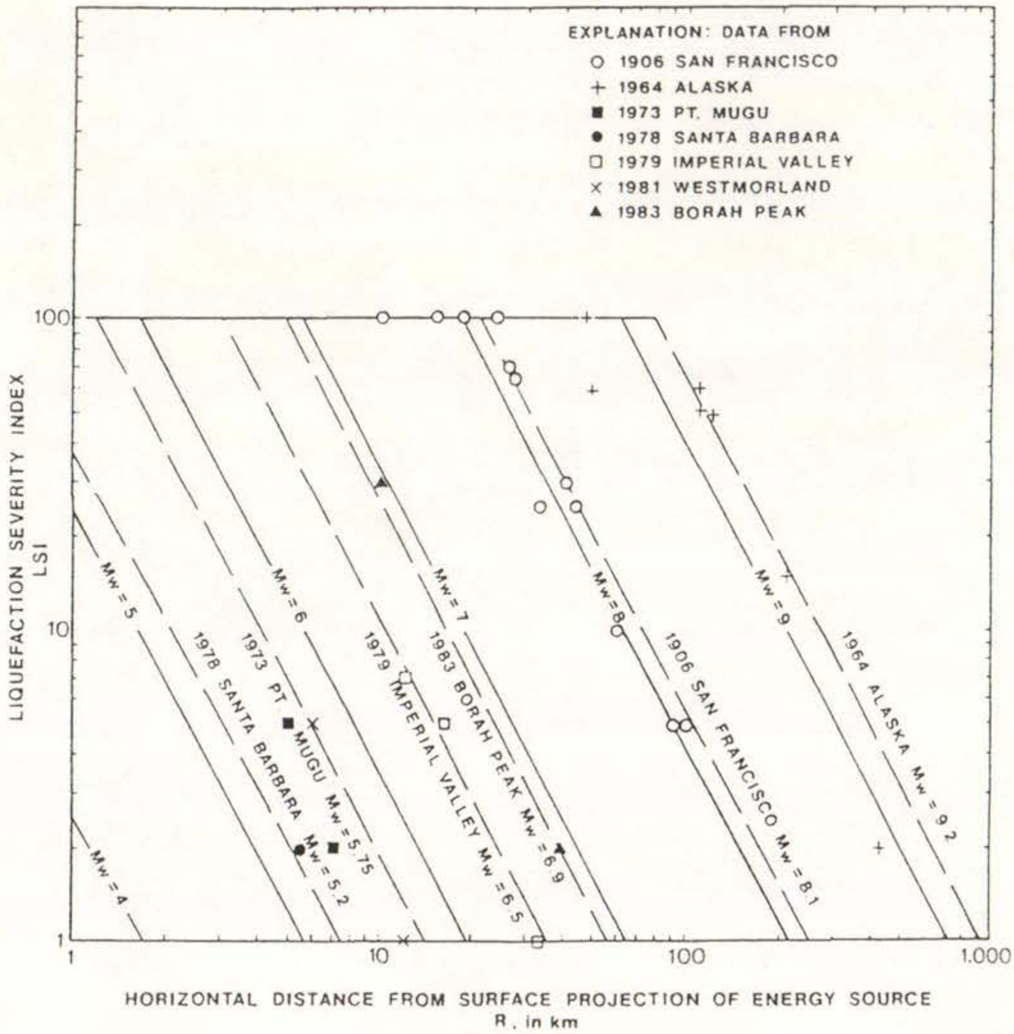

Figure 17. Liquefaction severity index curves. (From Youd, 1991)

Other major sources in the region (for example a M 7.3 event at 30 km from the study area) are likely to produce a LSI of 10 or greater. This indicates lateral spreads of about 250 mm on gently sloping ground.

The liquefaction opportunity in the Petone study area can be assessed by considering the major earthquake sources in the surrounding region (Figure 8). If the liquefaction resistance of a soil layer at a particular site is known in terms on a N_1 value, Equation 6 and Figure 15 may be used to determine the peak ground acceleration required to induce liquefaction for a given magnitude. From acceleration attenuation relationship relationships it is then possible to determine the threshold distance from the site to one of the sources required to initiate liquefaction. Using a threshold locus curve, the fraction of possible earthquake events on a source, for which the site will lie within the threshold distance for the given magnitude, can be calculated. The product of the annual rate of occurrence of the given magnitude and the fraction of possible occurrences gives the annual rate of liquefaction opportunity from the source for the given magnitude. All possible magnitudes from the source can be considered in a similar manner to give the total annual liquefaction opportunity from the source (Tinsley et al, 1985). In the study area region, most sources are thought to have a characteristic magnitude which simplifies this evaluation procedure. Based on the assumption of independent sources, the total liquefaction opportunity rate in the study area can be found by adding the opportunity rates from each source.

The source model procedure outlined above for evaluating the liquefaction opportunity has not been carried out in detail for the Petone study area. An event on the Wellington Fault segment tends to completely dominate the assessment of the annual liquefaction opportunity rate. Thus a simplified source model procedure using only Wellington Fault gives a source model opportunity assessment adequate for most purposes.

A typical site in the study area was investigated for liquefaction opportunity assuming a M 7.5 earthquake on the Wellington Fault segment. Based on the Joyner and Boore attenuation relationship for a randomly orientated horizontal acceleration component, and the Idriss soft soil modification (Figure 11), it was assumed that the peak horizontal acceleration on the study site was 0.45 g. The depth of ground water was assumed to vary between 1 to 3 m below surface and was taken to be at a depth of 3 m at the time of SPT measurement. The soil dry unit weight was assumed to be 16 kN/m^3. Figure 18 shows the results of the analysis for both clean sand and silty sand soils plotted as curves of uncorrected SPT N-value (New Zealand test equipment) versus depth below the surface. If the soil resistance given by measured SPT N-values lies significantly to the left of the curves there is a high probability of liquefaction.

Figure 19 compares the liquefaction opportunity from the Wellington Fault event (M = 7.5, ground acceleration 0.45 g) using N-value curves for clean sand with a similar analysis for an M 7.3 earthquake producing a PGA of 0.22 g at the site. This lower level of ground shaking would be expected from an earthquake on the more distant major fault sources in the region.

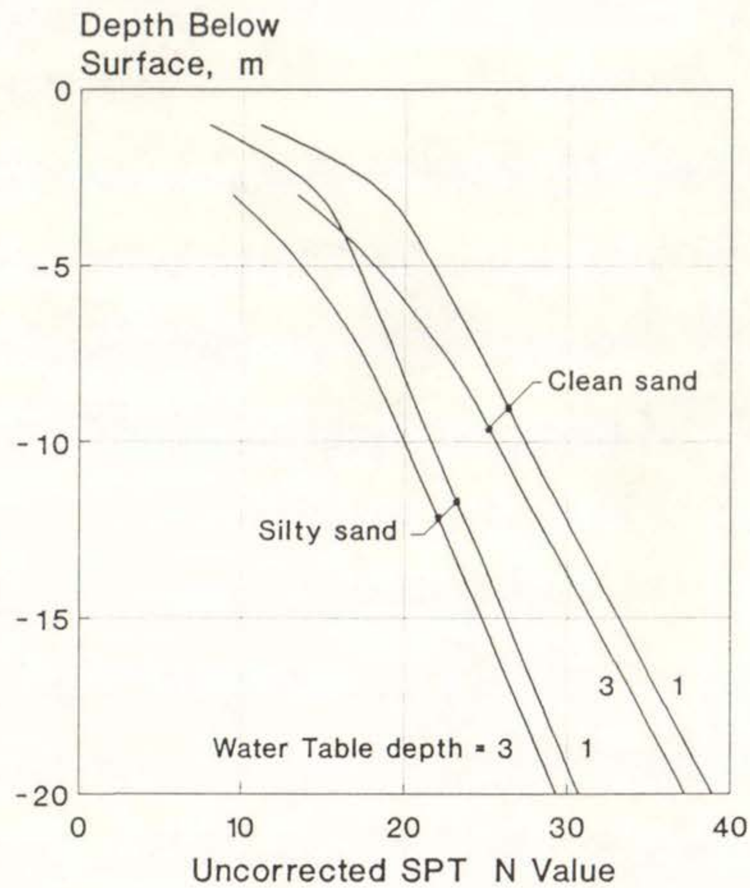

Figure 18. Liquefaction opportunity for Petone area during Wellington fault event.

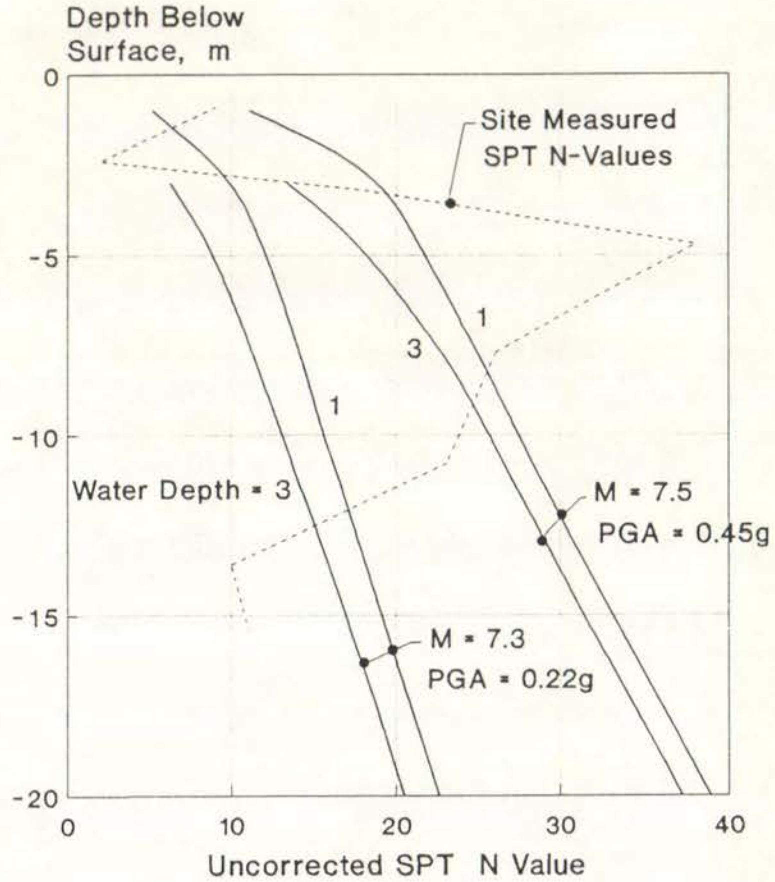

Figure 19. Liquefaction potential for typical Petone site.

Also show in Figure 19 are measured SPT N-values at a site in the study area close to the harbour waterfront on the eastern side of the Wellington Fault. It is clear from a comparison of the measured and opportunity curves that there is a very high risk of liquefaction in soil layers at depths of about 2.5 m and 14 m in moderate to strong ground shaking.

An alternative to the source model approach for evaluating liquefaction opportunity is a method based on a uniform seismicity model for the region. The Smith and Berryman seismicity model can be used together with the Joyner and Boore attenuation relationship in a similar way to the uniform seismicity model method described in Section 4.3.1. for estimating probability of occurrence and return periods for PGA. This approach was used to evaluate the liquefaction opportunity for the Petone study area. A summary of the procedure is as follows:

(a) Select Critical N-value

A critical N-value for the study area was selected with the objective of determining the annual probability (or return period) of liquefaction occurring in a soil with the selected resistance.

(b) Determine Acceleration to Cause Liquefaction

For a selected magnitude of earthquake, the Seed et al (1983) charts were used to determine the peak horizontal acceleration required to induce liquefaction in a soil layer with the selected N-value.

(c) Determine Area of Liquefaction Risk

Using the Joyner and Boore attenuation relationship, the area surrounding the site was determined in which the selected magnitude earthquake produced peak horizontal accelerations that equalled or exceeded the level required to induce liquefaction for the selected N-value. The attenuation relationship was used to determine the peak acceleration for a randomly orientated horizontal component on rock and the Idriss (1990) modification shown in Figure 10 applied to convert the rock acceleration to a surface acceleration on soft soil.

(d) Determine Annual Probability of Earthquake Magnitude

The Smith and Berryman uniform seismicity model was used to determine the annual probability of an earthquake of the selected magnitude occurring within the area determined in (c). The range of possible magnitudes for the region was divided into 0.2 magnitude units with the probability evaluated for the selected magnitude $\pm$ 0.1 unit.

(d) Combine Probabilities for Range of Magnitudes

Steps (b) to (d) were repeated for the range of earthquake magnitudes using 0.2 unit steps and the annual probabilities for each magnitude summed to give the total annual probability of liquefaction for the selected N-value.

Since a deterministic attenuation expression was used it is necessary to apply an enhancement factor to either the PGA or the return periods to allow for the statistical uncertainty in the attenuation relationship. Methods of calculating the enhancement factor are given by Peek (1980). For this study, an enhancement factor of 30 was applied to the calculated return periods.

The results of the uniform seismicity model assessment of liquefaction opportunity for the Petone study area are shown in Figure 20 by a chart that gives the return periods of liquefaction as a function of uncorrected N-values (New Zealand equipment) for a range of overburden depths between 3 to 10 m. The chart is based on a soil with a dry unit weight of 16 kN/m^3 and an assumed ground water depth of 2.0 m.

The uniform seismicity model does not make allowance for the close proximity of the Wellington Fault and the chart will therefore underestimate the liquefaction opportunity for return periods greater than about 300 years. Because of uncertainty in the estimate of the probabilistic enhancement factor, the reliability of the chart at low return periods is also questionable. However, the plotted information provides a useful guide for assessing the liquefaction opportunity in the return period range of about 50 to 300 years.

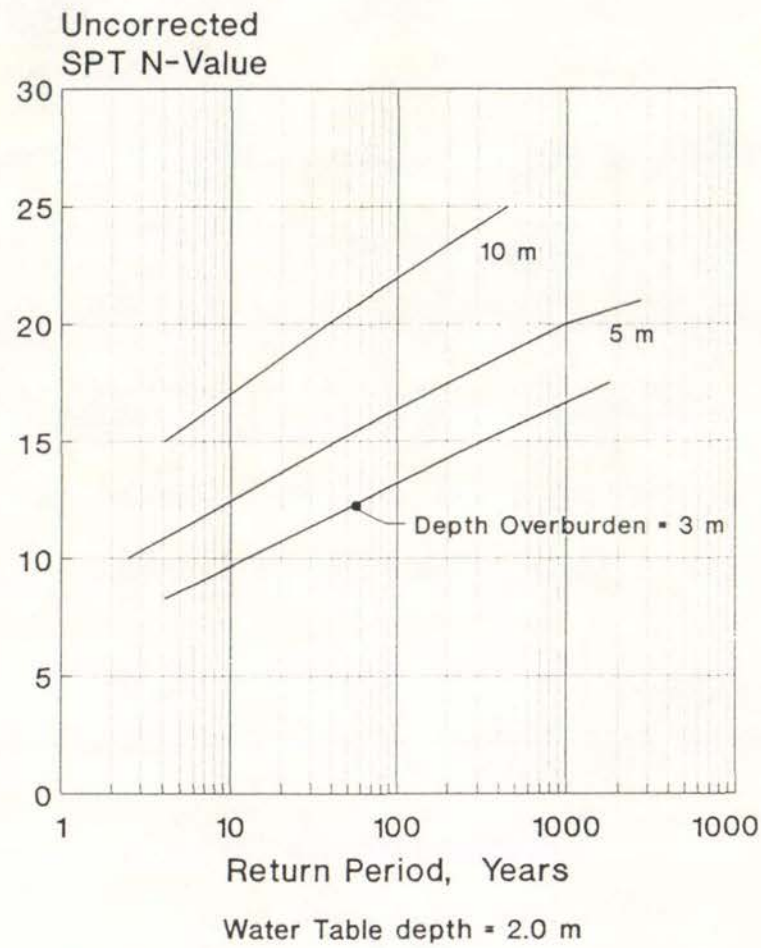

Figure 20. Liquefaction opportunity in Petone based on uniform seismicity model.

6. FAULT DISPLACEMENT HAZARD

The active scarp of the Wellington Fault runs along the east side of the Hutt Road. When a future rupture occurs, the ground beneath buildings straddling the fault could be sheared by up to 4.7 m laterally and 0.4 m vertically in a discrete zone, perhaps no more than 100 m wide. However, in the south-west area of the study area the zone of rupture could be potentially much wider (e.g., 500 m or more) as the fault appears to have a westward bend or sidestep in this area (Ian R. Brown Associates Ltd 1991a).

7. TSUNAMI HAZARD

The Petone coast has been affected by tsunamis in historical times. Water levels rose at least 1.2 m above high water level along the Wellington waterfront, and water level oscillations of at least 3 m amplitude were observed at 20 minute intervals in Port Nicholson during the 1855 earthquake (Wellington Regional Council, 1990).

A future earthquake on the West Wairarapa Fault is the most likely event to cause tsunamis at the Petone shoreline. Wave action at the foreshore would commence after c.20 minutes, and a 1.5 m rise in water level would occur after some 50 minutes. Subduction thrust events at the Hikurangi margin would result in considerable regional uplift and tilting, so could also cause tsunamis at Petone. There has not been a large earthquake from this source in historical time, so the tsunami hazard of subduction earthquakes is poorly understood.

8. BUILDING DAMAGE RATIOS

Building damage records from past earthquakes can be used to estimate the damage that various classes of buildings might experience when subjected to various intensities of ground shaking in future earthquakes. It is convenient to express building damage in terms of a damage factor (DF) defined as the cost of repairing the damaged building divided by the cost of replacing the building. Sometimes the damage factor is expressed in terms of the market value rather than the replacement value; however, the replacement value appears to more commonly used in recent literature. Building damage information is usually presented as curves relating the mean damage factor (MDF) for a particular type or class of building to the ground shaking intensity expressed as MMI values. Curves of this type can be used to gauge the relative performance of different classes of buildings. Together with building inventory information, they can also be used to estimate the total damage costs for a specific study area.

The most widely accepted building damage information is in the form of a series of damage/intensity curves prepared by Algermissen, Steinbrugge and colleagues for a wide range of building classes. (See Rinehart et al, 1976, and Algermissen and Steinbrugge, 1978.) These curves are based on damage records from past earthquakes. More recently, the Applied Technology Council (ATC) in California carried out a major project to determine information on damage to buildings and other engineering structures caused by ground shaking. This information is published in report ATC-13 (1985).

8.1 ATC-13 Study

In the ATC project, estimates of the percent of physical damage caused by ground shaking for 78 different engineering facility classes, expressed in terms of DF versus MMI, were developed through a multiple questionnaire process involving 71 selected earthquake engineering experts. In Round One of the three-round questionnaire process, each expert was asked to provide low, best and high estimates of the DF for selected engineering facility types at MMI levels VI through XII. In addition, each expert was asked to evaluate his experience with the facility class being evaluated and to provide a self-evaluated degree of certainty in the low, best and high estimates. The second and third rounds were carried out in an attempt to reach some consensus on the damage ratio estimates. In these rounds, each expert was provided with graphs showing his answers to the previous questionnaire together with the answers of all other experts responsible for the same facility class. Each expert was then asked to re-evaluate his estimates in light of the responses of the others. A Beta distribution was use to convert the results of the final questionnaire into damage probability matrices (DPM's) that tabulate the percentage of buildings expected to fall into seven different damage states at each MMI level. The damage states and corresponding damage factor ranges used in the project are given in Table 6. A typical DPM for low-rise timber-frame buildings is given in Table 7.

TABLE 6
Earthquake damage states. (From ATC-13.)

Damage State	Damage Factor Range %	Central DF %
1 - None	0	0
2 - Slight	0 - 1	0.5
3 - Light	1 - 10	5
4 - Moderate	10 - 30	20
5 - Heavy	30 - 60	45
6 - Major	60 - 100	80
7 - Destroyed	100	100

TABLE 7
Damage probability matrix for low-rise timber frame (1-3 stories). (From ATC-13.)

Central Damage Factor	Modified Mercalli Intensity VI	VII	VIII	IX	X	XI	XII
0.00	3.7	0.0	0.0	0.0	0.0	0.0	0.0
0.50	68.5	26.8	1.6	0.0	0.0	0.0	0.0
5.00	27.8	73.2	94.9	62.4	11.5	1.8	0.0
20.00	0.0	0.0	3.5	37.6	76.0	75.1	24.8
45.00	0.0	0.0	0.0	0.0	12.5	23.1	73.5
80.00	0.0	0.0	0.0	0.0	0.0	0.0	1.7
100.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Mean DF	1.7	3.8	5.5	10.6	21.4	25.5	39.4
Note: 0.0 = Very small probability							

Damage curves for four of the building engineering facility classes used in the ATC-13 study are shown in Figure 21. The plotted mean damage factors were obtained by taking a weighted summation of the percentages of buildings falling within each damage state of the published DPM's. The facility classes shown in Figure 21 are listed in Table 8.

The four facility classes plotted in Figure 21 were chosen because most of the buildings within the study area are represented by these types. Information on a wide range of other building types is available in the ATC-13 report.

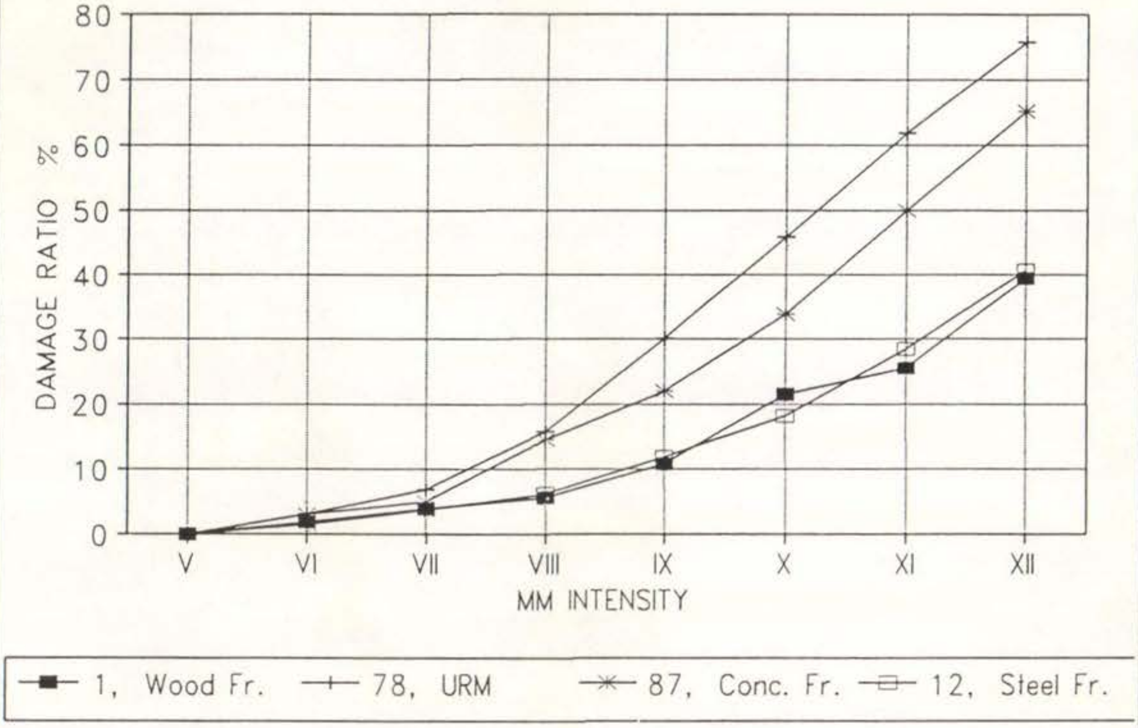

Figure 21. Damage ratio curves from ATC-13.

TABLE 8
Facility classes shown in Figure 21.

Facility No.	Description
1	**Wood Frame**, low rise (1-3 stories)
78	**Unreinforced Masonry**, low rise, (with load bearing frame)
12	**Braced Steel Frame**, low rise
87	**Concrete Frame**, moment-resisting, non-ductile, distributed

The results of the ATC study were compared in the ATC-13 report with data from other sources for a number of different classes of buildings. The comparisons for low-rise timber-frame, unreinforced masonry, reinforced concrete and steel are relevant to this study and are shown in Figures 22 to 25. In the ATC-13 report it is stated that there are at least 10 existing sources of data for low-rise timber-frame buildings, although some of these are updates of earlier publications. Some of the published data are for pre-seismic code buildings and other data are for modern buildings designed to have good earthquake resistance. It is therefore difficult to correlate one source of data with another.

It is clear from the damage curves in Figures 22 to 25 that there is considerable spread in the data presented. However, the ATC results appear to be reasonably consistent with the bulk of the other published information.

One of the stated limitations of the expert opinion approach of the ATC study is that it does not directly take into account damage records from past earthquakes. However, it is questionable whether this is a significant limitation since it seems likely that most experts would make their judgements taking into account the available information from past earthquakes.

The performance of buildings during earthquakes is obviously dependent on how well they are designed and the quality control procedures applied during construction. Many older structures may not have been specifically designed to resist earthquake loads. In the ATC study, the question of design standard and building age was not addressed in any detail. The ATC damage probability matrices are for "standard" construction. Although the meaning of this description is not well defined it appears that the presented damage data are for modern structures designed to normal seismic design standards. The DPM results are based on the structure being founded on firm soil. Collateral causes of damage such as ground failure, fault rupture, inundation and fire were considered separately from the main study which included only the effects of ground shaking.

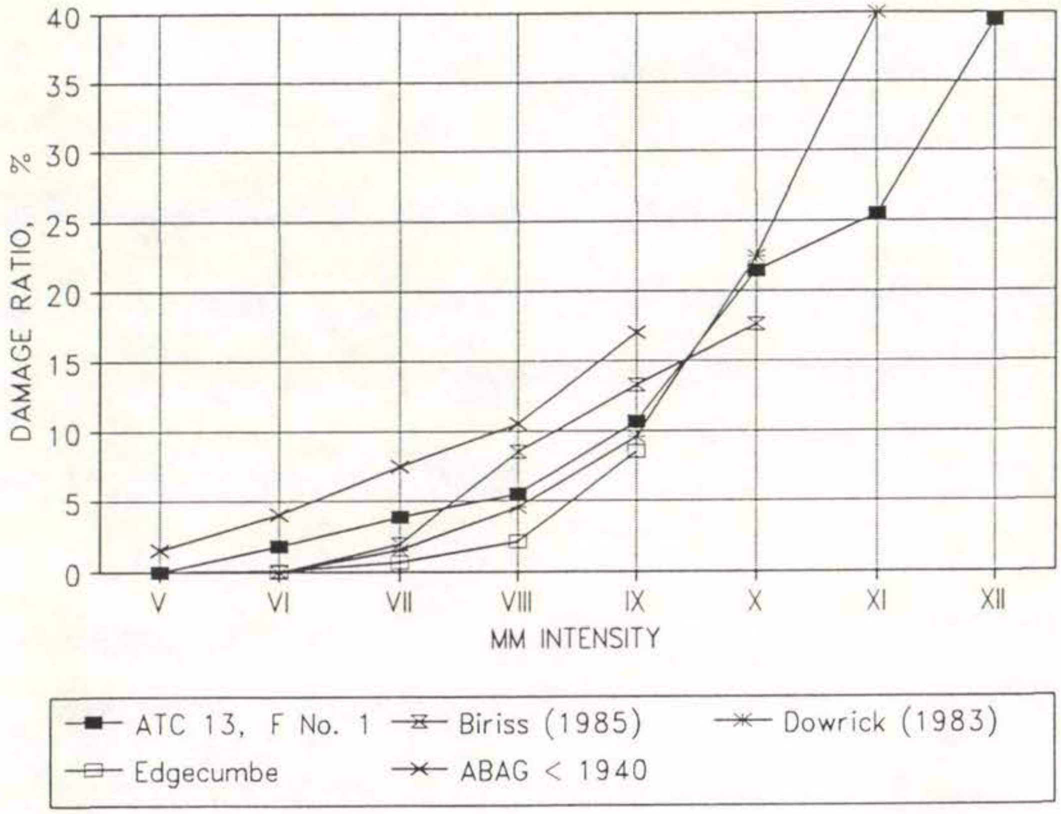

Figure 22. Damage ratio curves for timber-frame dwellings.

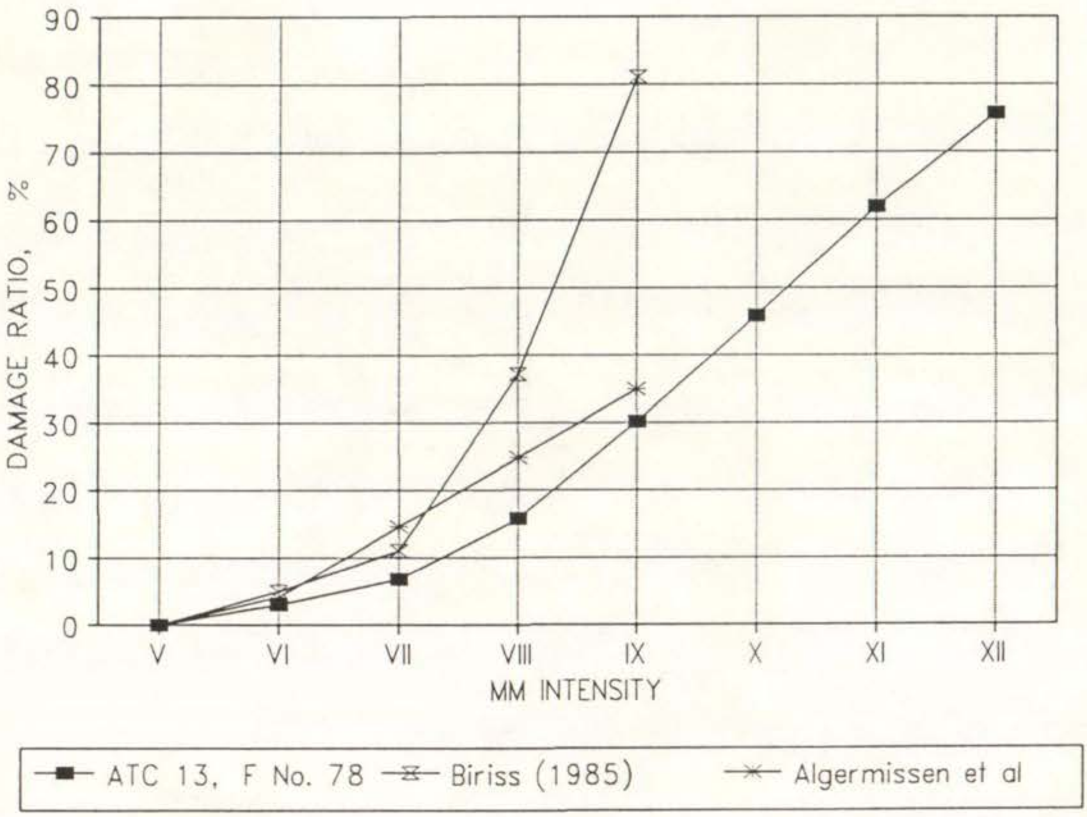

Figure 23. Damage ratio curves for low-rise unreinforced masonry.

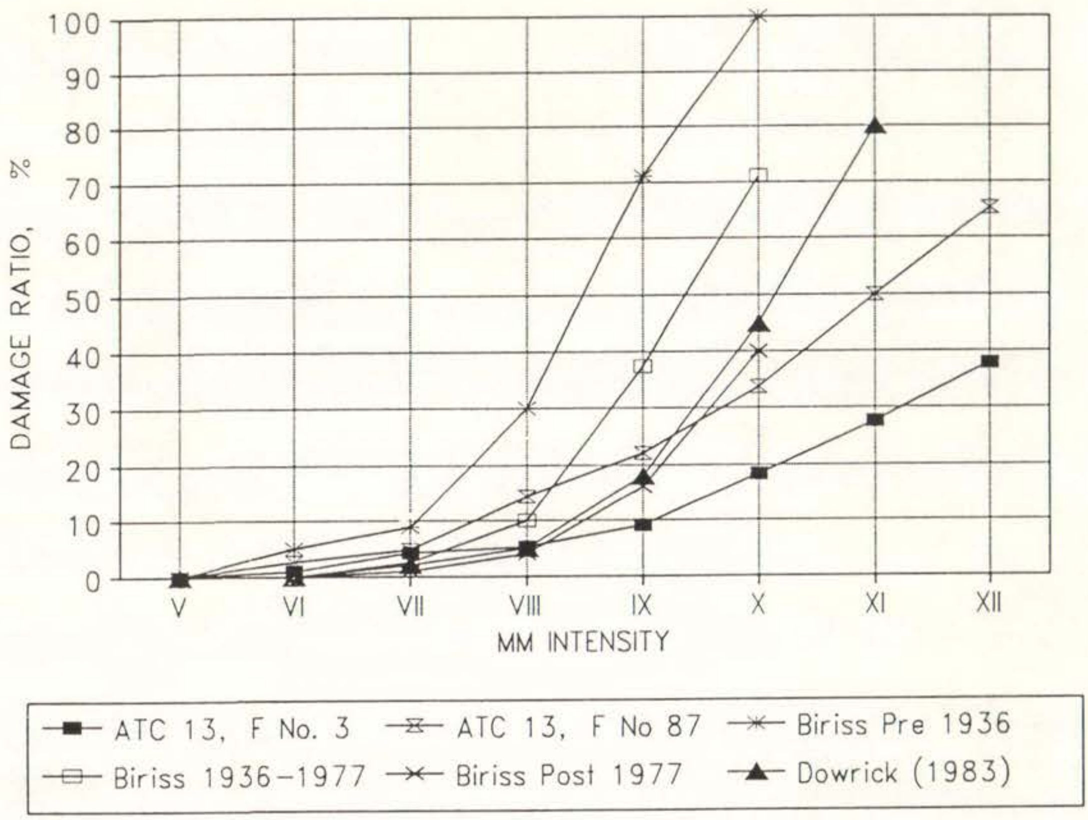

Figure 24. Damage ratio curves of low-rise reinforced concrete.

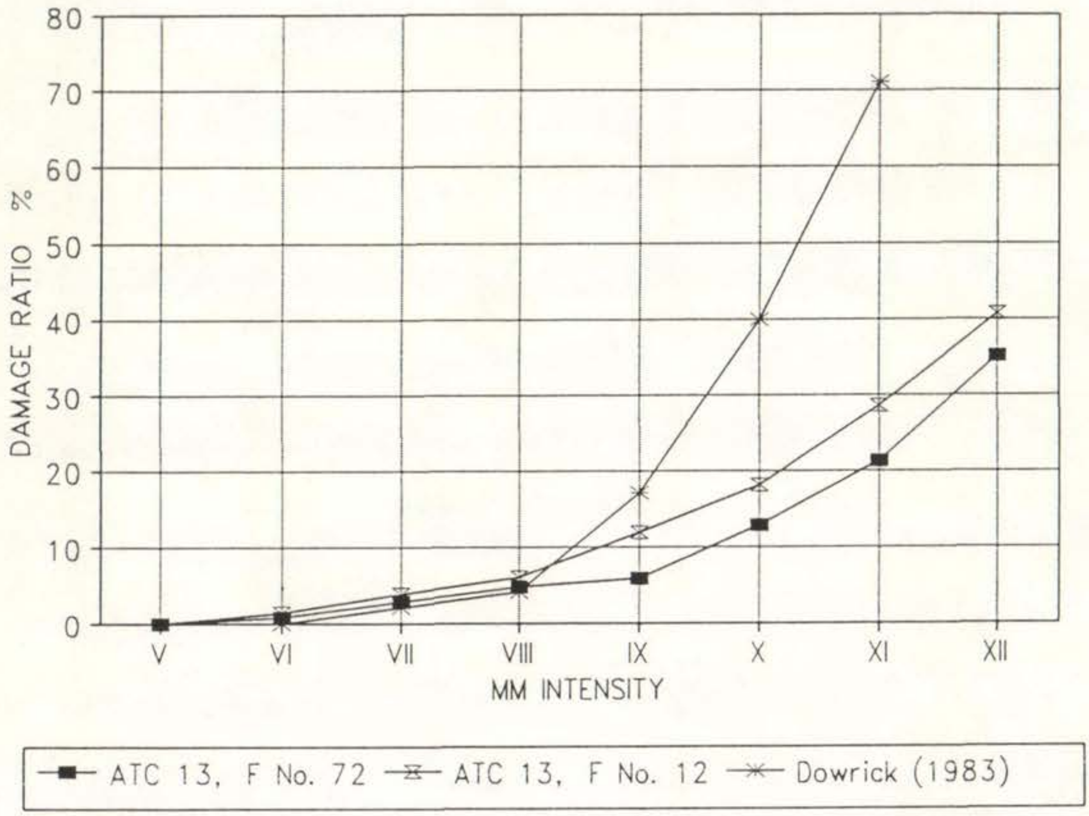

Figure 25. Damage ratio curves for low-rise steel frame.

The ATC-13 report recommends that for buildings of "nonstandard" construction, the damage probabilities listed in the DPM be shifted down two intensity levels. The "nonstandard" category includes those structures that are more susceptible to earthquake damage than standard construction and apparently is intended to include most pre-earthquake code buildings.

8.2 Damage Factors for New Zealand Buildings

Damage factor curves for New Zealand buildings have been published by Birss (1985) and Dowrick (1983, 1990a, 1990b). Birss used data from a variety of sources that analysed damage data for buildings similar to those existing in New Zealand to derive the curves shown in Figures 22 to 24. As shown in Figure 24, Birss divided reinforced concrete structures into three separate classes related to age. The age classifications were related to the introduction and later revision of earthquake design standards. Birss investigated whether the MDF should be based on market value or replacement value. His investigation indicated that the market value could be as low as 40% of the replacement value for older buildings. For correlations with other studies, Birss determined that his curves best fitted MDF's based on market value. Birss also suggested that the costs of repairs could exceed the product of MDF and market value by several-fold. As an example, he stated that a moderate to severely damaged older building would probably be demolished and rebuilt to current standards.

Dowrick (1983) published damage factors based on replacement value for reinforced masonry, reinforced concrete, steel and timber commercial building types. His curves, shown in Figures 22, 24 and 25 were based on a careful comparison of all data available and relevant to New Zealand buildings. Scaling factors were suggested to allow for different design standards. For buildings designed between 1936 to 1965 the standard plotted curves were modified by multiplying the DF's by 1.1. For buildings post 1977 a multiplying factor of 0.9 was recommended.

Damage data from the 1987 Edgecumbe for domestic buildings was published by Dowrick (1990b). Although details of the types of dwellings included in the survey were not presented it seems likely that nearly all were single-storey with timber frames. The damage factor curve from the earthquake data is compared in Figure 22 with data for timber-frame dwellings from other sources. The Edgecumbe data lies below the other curves.

8.3 Comparison of Damage Factor Curves

It is informative to compare the damage curves recommended by Birss and Dowrick with the ATC-13 study results. A comparison for timber-frame dwellings is shown in Figure 22. The curve labelled ABAG < 1940 is from an Association of Bay Area Governments (1986) publication and refers to pre-1940 dwellings. Many of these buildings were supported on cripple studs without a perimeter foundation wall extending up to the floor. In recent earthquakes, this type of timber-frame construction has been observed to perform very poorly. The ABAG < 1940 curve can be expected to represent an upper bound to DR's for timber frame dwellings. The Dowrick and Birss DR's are significantly lower than the ATC-13 values at MMI's of VI and VI. At higher MMI values the comparison with ATC-13 is reasonably good. ?

Figure 23 shows a comparison of the Birss (1985) DR curve for unreinforced masonry with ATC-13 and Algermissen and Steinbrugge (1978). Dowrick did not publish a curve for this type of construction. The ATC curve is for unreinforced masonry with a load bearing frame. The Algermissen and Steinbrugge curve is for their classification 5E which includes buildings with unreinforced solid-unit masonry of clay brick, concrete brick, stone or unreinforced concrete. Although the Birss DR's ratios are higher than the others, part of the difference can be attributed to the fact that the Birss DR's are based on market value rather than replacement value as used for the other curves. The Birss curve may also be based on older or poorer quality construction than assumed for the others.

Damage ratio curves for low-rise concrete construction are shown in Figure 24. The ATC Facility Numbers 3 and 87 refer to shear wall with moment-resisting frame and moment-resisting non-ductile frame structures respectively. The curves show wide scatter which in part is due to the different structural types and the age classifications considered. The DR's given by Birss are higher than the others at MMI values greater than IX. Some of this difference can be attributed to the use of market value by Birss to define his DR's.

Damage ratio curves for low-rise steel structures are shown in Figure 25. The ATC Facility Numbers 12 and 72 refer to braced-frame and moment-resisting frame structures respectively. Birss did not present a damage curve for this class of structure. The Dowrick DR's are significantly higher than ATC-13 at MMI values greater than IX. There is no obvious reason for this difference other than the different approach used to derive the curves.

8.4 Recommended Damage Ratio Curves

For this study it was decided to adopt the ATC-13 curves for damage cost assessment work. The ATC data offers the following advantages:

(a) A consistent and rational derivation method has been used to develop damage data for a wide range of different structural types.

(b) The damage probability matrix has a wider range of applications than single mean damage ratios.

(c) The buildings considered are generally of a similar type to those constructed in New Zealand.

The main limitation of the ATC data is that it is based on standard modern construction and does not apply directly to older buildings. Judgement can be used to apply arbitrary adjustments to allow for building age but this results in a high degree of uncertainty. However, the other published damage data also has limitations and does not enable a more rational approach to be followed.

In this study, for buildings where no particular earthquake risk was identified, the ATC damage ratios were increased by a factor of 1.5 for pre-1936 buildings and a factor of 1.3 for buildings constructed between 1936 and 1977. These classification dates were based on the introduction of the first model building bylaw in 1935 and the major revision of seismic design codes that occurred in the late 1970's. For buildings with parts, such as chimneys, parapets and gables of unreinforced masonry or the main load bearing structure of unreinforced masonry, further factors were applied as discussed in the following section. For these buildings the damage ratio factor applied to the ATC values was between 1.1 to 2.2. For all types of construction, where the main structure was considered to low seismic resistance a damage ratio factor of between 1.5 to 2.0 was applied.

9. BUILDING DESCRIPTIONS AND CLASSIFICATIONS

Descriptions of the buildings in the study area were obtained from a survey of earthquake risk buildings carried out by Spencer, Holmes Miller & Jackson (SHMJ) in 1983. This survey covered all non-residential buildings in Petone and all residential buildings of two or more stories with three or more occupancies. Buildings on the Unilever NZ Ltd and Gear Meat Company sites were not surveyed. The Unilever site was not surveyed as this company was in the process of strengthening and upgrading its buildings. Buildings on the Gear Meat Company site were in the process of demolition.

Since the 1983 SHMJ survey, a number of buildings have been demolished and new ones built. Since the survey date the total number of new buildings constructed in the study area numbers about 25. The total number of buildings surveyed was 793 and thus the total change to the building inventory since the survey date is not very significant. For the purpose of this study, it was considered unnecessary to update the 1983 building inventory; however, because a computer data base of buildings was developed as part of the present study it would be a relatively simple task to update the information.

The present main shopping centre of Petone is located in Jackson Street. Initially the Petone township was located near Queen Street and subdivision of the land in the main commercial centre did not occur until 1880-1890 (SHMJ, 1983). Many of the buildings constructed up to the turn of the century were of all timber construction. A major fire in Wellington resulted in the Petone Borough Council passing a "brick area" bylaw in February 1900. This bylaw required all boundary walls to be constructed of brick and as a consequence buildings with all masonry (unreinforced) walls and timber floors were common until the early 1930's. The majority of the buildings in the commercial centre were constructed prior to 1940 (SHMJ, 1983) and are of one or two storey construction. Many of these buildings can therefore be expected to be seriously damaged in moderate to large earthquakes.

A number of light industrial areas surround the main commercial centre. Although these buildings include a wide range of construction styles, many have been constructed since the introduction of earthquake design standards in 1936.

Most of the residential houses in the study area are of timber frame construction with timber weatherboard exterior cladding and galvanised steel roofs. Many are founded on timber or concrete piles with the floor raised about 500 mm above ground. The majority of the houses were constructed between about 1910 and 1950.

The objective of the SHMJ study was to identify earthquake risk buildings as defined by Amendment 301A of the Municipal Corporations Act. The potential danger to life during major earthquakes of unreinforced masonry or concrete buildings was recognised by Central Government in 1968 when Amendment 301A was enacted. (In 1979 Clause 301A of the Municipal Corporations Act was replaced by Section 624 of the Local Government Act.) Amendment 301A (and Section 624) gave Local Authorities the power, subject to Ministerial approval, to require unreinforced masonry or concrete buildings which had insufficient strength to resist earthquake forces of 50% of those defined by NZS 1900 Chapter 8 (1965) to be removed or strengthened.

The SHMJ survey was primarily concerned with identifying buildings that are hazardous from the point of view of causing injuries and loss of life in earthquakes. In contrast, the present study has been more directed at identifying the likely cost of earthquake damage to buildings and other structures. Although the objectives of the SHMJ study were different from the present study, the data base of building information collected can be applied directly in the earthquake damage assessment work for the study area. The survey procedure used by SHMJ and a summary of the information gathered relevant to the present study is given below.

The SHMJ survey consisted of a document search of the Petone Borough Council's Permit Application records, followed by a site inspection of each of the buildings included in the survey. Each earthquake risk building was classified in accordance with the system recommended in the New Zealand National Society for Earthquake Engineering (NZNSEE) Code of Practice, October 1973. (This classification has since been amended by NZNSEE.) At the time of inspection a standard inspection report form, similar to one recommended by NZNSEE, and shown in Table 9 was completed. The inspection information was then used to find the NZNSEE numerical rating and building classification. The building classification system used by NZNSEE, 1973 is as follows:

Class MA

Masonry buildings which should be evacuated as soon as possible because they constitute a hazard in the event of an earthquake of intensity much lower than moderate. Identification of these buildings can be carried out using the numerical rating system discussed under Class MC below.

Class MB(i)

Masonry buildings which have a particular hazard such as a parapet, gable or chimney which are liable to damage in the event of an earthquake. On the removal of these hazards the buildings should be re-classified into one of the MC class divisions.

In the SHMJ survey all buildings of any construction material (including reinforced concrete, steel and timber) that had unreinforced masonry parapets, gables, chimneys or partition walls were given a MB(i) classification.

Class MB(ii)

Masonry buildings with concrete floors supported on unreinforced masonry walls.

TABLE 9
Building inspection report form. (From SHMJ, 1983.)

PETONE BOROUGH COUNCIL BUILDING SURVEY	Inspected By		Date
BUILDING	Name:	Address:	Date Built: Age:
OWNER		Lessor:	
OCCUPANCY		8/24 hour	5/7 days
STRUCTURE	Drawings:.............No. of Storeys ....Mezz....Basement.... Dimensions: Width.....Length......Height......Area;B........ Foundations: Strip, Raft, Pads, Piles, Caissons. G........ Soil description:.............................. 1........ Frame: None, Concrete, Structural steel. 2........ Exterior walls: Panels, Veneer, Bearing Material:...		

	N	S	E	W
Concrete Bands:.........				
Area of openings (%)				
Height of Floors				

Floors: Material:................. Diaphragm: Eff./Non-eff.
Roof: Pitched/Flat/Other...................................

Roof Frame:	Wood	Steel	Concrete
Rigid frame			
Simply supported truss			
Kneebraced truss			
Other			

Roof diaphragm: Eff./Non-eff: Remarks.................
Roof covering:..
Chimneys: No. of ...... Remarks:...........................

	Where	Material	Height	Width	Breadth
Parapets:					
Gables:					
Heavy Ornaments:					

Lifts: No.......Type:.....................................
Stairs: No......Type:.............Material:................

NON STRUCTURAL	Partitions: Material:.............Condition:............... Ceilings: Material:.............Condition:...............
ADDITIONS	DATES
DAMAGE & STRUCTURAL CONDITION	

DEMERIT POINTS	1. Structure	OCCUPANCY	CLASS
	2. Structural Condition		
	3. No. of Storeys above ground		
	4. Structural Damage		
	5. Walls	LIFE EVALUATION	
	6. Diaphragms		
	7. Foundation	....	
	TOTAL		

Class MC

This class comprises all low-earthquake resistant masonry buildings not qualified for immediate evacuation and not requiring remedial treatment. The class is subdivided into 5 sections numbered from 1 to 5 in order of decreasing risk. The sub-classification is based on the NZNSEE numerical rating system shown in Table 10. The building rating depends on the construction type, structural condition and damage, height and lateral support of external walls, effectiveness of floor diaphragm and nature of the foundation soil.

The NZNSEE classification only applies to earthquake risk buildings of unreinforced masonry or inadequately reinforced concrete. Other buildings were classified in the SHMJ study using a system based on age and construction materials. Building classifications used were as follows:

Type B

Buildings, including any parts, not covered by Section 624 of the Act but which were built prior to the introduction of seismic load design requirements. This type includes buildings constructed prior to 1932 and possibly prior to 1939. (A draft General Earthquake Building Bylaw was introduced in 1931.)

Buildings of this type may have parts constructed in structural steel, reinforced concrete or masonry with reasonable seismic resistance but often with weak connections or some parts which may be hazardous in an earthquake. Buildings of predominantly timber construction were not included as they were considered separately (see Type E).

Type C

Buildings designed and constructed in accordance with NZS 95 Part... (introduced in 1935) or NZS 1900 Chapter 8 (introduced in 1935).

Type D

Buildings designed and constructed to resist seismic forces in accordance with NZS 4203 (1976).

Type E

Buildings predominantly built in timber, that is timber frame exterior and interior walls, timber or galvanised steel exterior cladding, and timber floors.

A sub-classification of 'R' was given to a B, C, D or E type building where the building or any part was thought to present a danger or where the ultimate load capacity was likely to be exceeded in a moderate earthquake. Examples of the use of this classification included cases where a facade appeared to be inadequately connected to the side walls or the length of a laterally unsupported wall appeared to be excessive.

TABLE 10

Numerical rating of earthquake risk buildings. (From NZNSEE, 1985)

RATING	0	1	2	3
1 STRUCTURE	reinforced concrete or reinforced columns and beams with masonry - struct continuity	unreinforced masonry with reinforced bands at roof and floors, some continuity	unreinforced masonry with a band at roof or floor	unreinforced construction
2 CONDITION OF MATERIALS	SOUND materials in sound condition	GOOD minor evidence of deterioration of materials	FAIR deterioration giving reduced strength	POOR considerable deterioration fretting, spalling etc.
3 NUMBER OF STOREYS INCLUDING BASEMENT OVER 50% OF GROUND FLOOR AREA	one	two	three or four	five or more
4 CRACKING OR MOVEMENT	none evident	minor	moderate	severe (See Note 1)
5 WALLS LENGTH AND HEIGHT BETWEEN LATERAL SUPPORT (See Note 2)	L less than 6m H less than 3m	L 6 - 9m H 3 - 4m	L 9 - 12m H 4 - 5m	L greater than 12m H greater than 5m
6 DIAPHRAGM	concrete floor slab adequately connected to walls	timber floor with engineered connections to walls	trusses and/or beams adequately tied	concrete floor or light timber with carpenters connections not tied
7 FOUNDATIONS (Soils)	EXCELLENT (rock)	GOOD	FAIR	POOR
8 GENERAL STANDARD OF MAINTENANCE	EXCELLENT	GOOD	FAIR	POOR
9 APPENDAGES OR STREET FRONTAGE (See Note 3)	walls set back from boundary greater than height of wall	minor amounts of masonry or wall with less than 9m street frontage	moderate amounts of masonry or wall with 9m - 20m frontage	significant amounts of masonry wall over 20m frontage
10 STRUCTURAL OUTLINE (plan or elevation)	symmetrical	minor eccentricities	moderate eccentricities	severe eccentricities soft lower storeys

Note 1 : Severe means a considerable number of cracks or substantial movement giving reduced strength or isolated large cracking.

Note 2 : Height means vertical distance between lateral support or total height of wall if there are not horizontal lateral supports - maximum conditions govern. Wall openings and wall thickness should be assessed in assigning a rating.

Note 3 : Length of frontage applies only to walls which are closer than their height to the street boundary.

The R sub-classification was assigned to only a relatively few buildings with a total of one B Type building and seven E Type buildings been given this sub-classification. Although a number of the C and E Type buildings had hazardous unreinforced masonry components, these buildings were generally classified under the AM(i) type.

9.1 SHMJ Survey Summary

A total of 793 buildings were surveyed. The distribution of the buildings within the various classifications is shown in Figure 26. One building was classified as MA and two as MC1. These buildings are likely to receive heavy damage or be destroyed by a moderate earthquake. A total of 179 buildings were classified within the MC2 to MC5 classification and 8 of the B and E Type buildings were given an R sub-classification. Most of this group of 187 buildings (24% of those inspected) are likely to receive moderate to heavy damage in a strong earthquake. Only 79 (10%) of the buildings, other than those of predominantly timber construction, had been designed to modern seismic design requirements. Only 7 of the E Type buildings appeared to have been designed in accordance with the latest standards adopted in the late 1970's. However, many older timber buildings have reasonably good seismic resistance. The survey was not sufficiently detailed to classify timber buildings in more than one category except that those designed after about 1970 could be identified and buildings thought to be at high risk (7 total) were given an R sub-classification.

The distribution of the buildings within the five main construction material groups are shown in Figure 27. Many buildings are constructed of several materials but the category was based on the predominant material in the main load bearing structural system. Of the total of 134 masonry buildings only 17 were of reinforced masonry construction. Unreinforced concrete buildings were included in the unreinforced masonry (URM) category but it is difficult to differentiate by site inspection between unreinforced concrete and lightly reinforced concrete. Consequently there is some uncertainty regarding the classification of concrete buildings into construction material types. A total of 356 (45%) buildings were identified to have main load bearing structures of reinforced concrete. These buildings range from modern tilt up type construction to old lightly reinforced frame or shear wall buildings constructed prior to the introduction of standards.

Figure 27 identifies the number of earthquake risk buildings within each construction material group. Two categories of risk are shown. Firstly, the buildings with deficient main load bearing structures are identified. These are buildings with a MA, MC1 to MC5, or R classification. The second earthquake risk category are buildings where a component, such as a chimney, parapet, or gable is deficient. These are buildings given a MB(i) of MB(ii) classification. (In fact only one building was classified as MB(ii).)

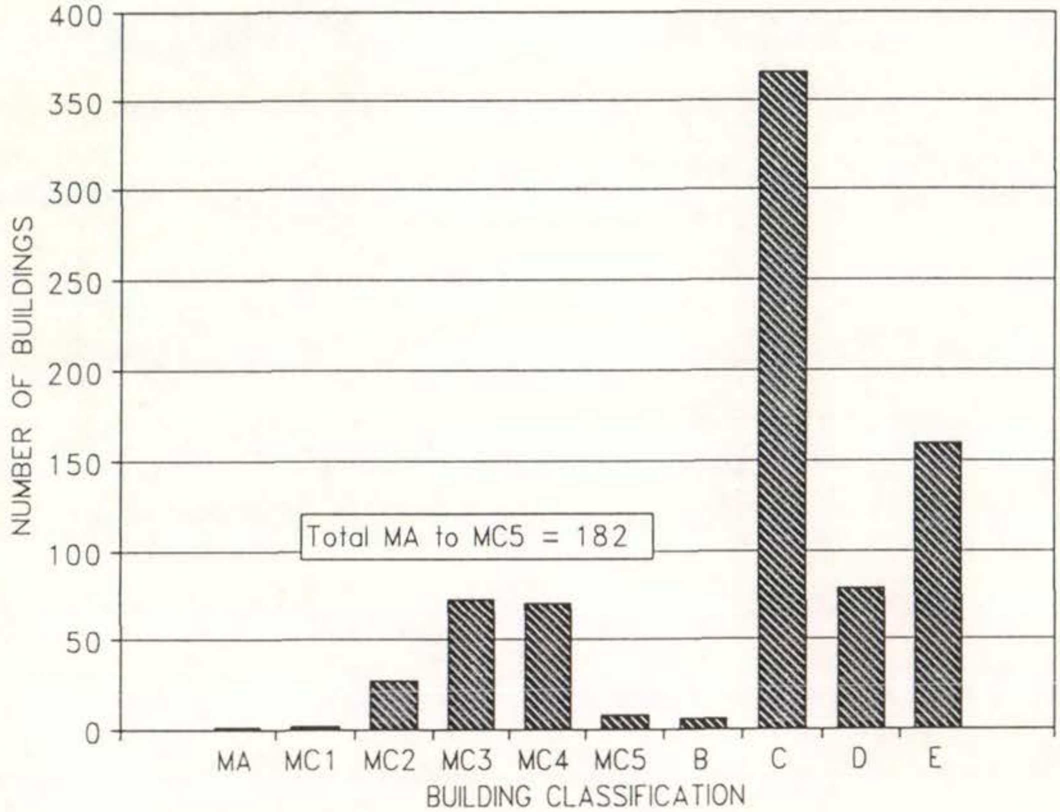

Figure 26. Earthquake classifications for Petone buildings.

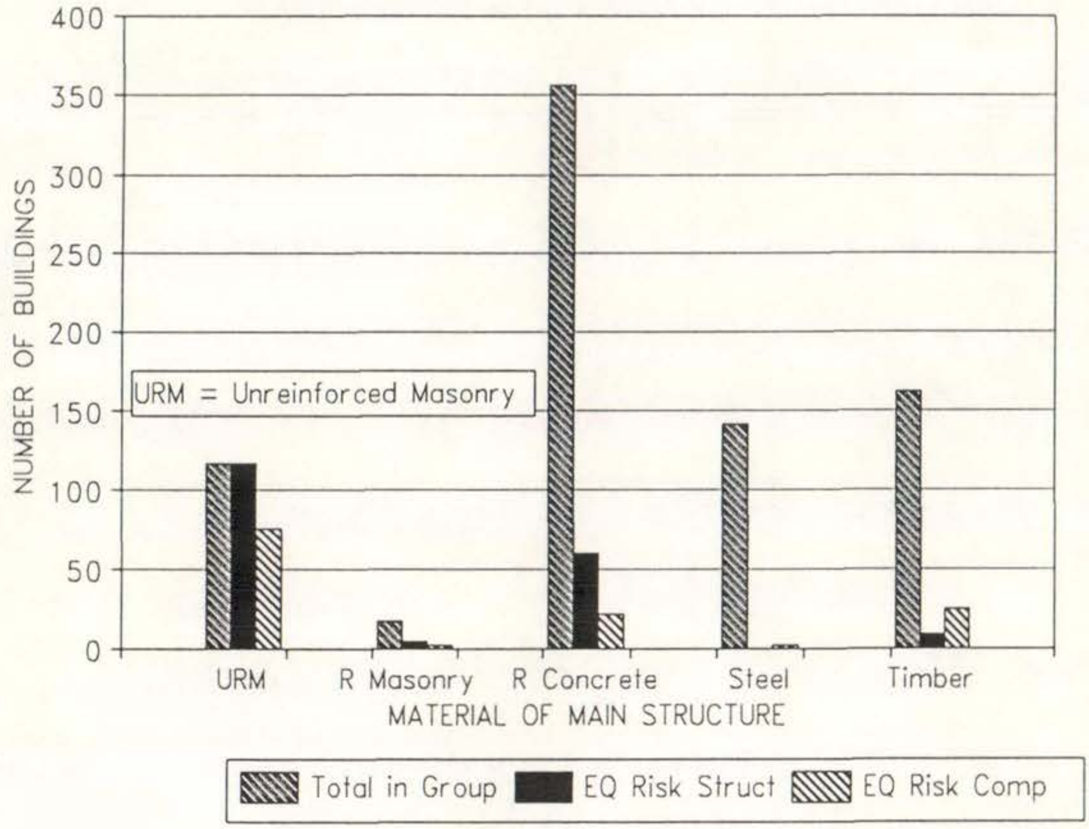

Figure 27. Identification of earthquake risk buildings in material categories.

As indicated in Figure 27 all of the 117 URM buildings were classified as having a deficient structural system and 76 within this group also had one or more hazardous components. Of the 356 reinforced concrete buildings, 60 were found to have deficient main structures and 22 had hazardous components. In most cases the hazardous components were unreinforced chimneys.

10. EARTHQUAKE DAMAGE TO PETONE BUILDINGS

10.1 Study Area

To simplify the evaluation of the likely cost of earthquake damage to the Petone buildings, the study area was restricted to the southern part of Petone as shown in Figure 28. The study area is bounded by the waterfront on the south side; Nevis Street and the Hutt Road on the west; Udy Street on the north and William Street on the east. This section of Petone has an area of 1,380,000 m^2 and included 527 buildings at the time of the SHMJ survey having a total area 242,000 m^2. In addition, the area contains about 1,100 houses of estimated total area of 115,000 m^2 and the Petone Recreation Ground of 81,400 m^2. Buildings in the old Gear Meat Co site were not included in the analysis as they were being demolished at the time of the SHMJ survey. The area of this site is approximately 126,000 m^2.

10.2 Buildings

The reduced study area contained a good representation of the 793 buildings included in the SHMJ survey. The numbers and floor areas of each of the main categories of building construction in the reduced study area are as summarised in Table 11.

TABLE 11
Building types in reduced study area.

Building Construction Material	No of Buildings	Total Area m^2	% of Total Area
Reinforced Concrete	229	112,000	46.4
Reinforced Masonry	10	1,800	0.7
Unreinforced Masonry	101	35,700	14.8
Steel	85	49,700	20.6
Timber	102	42,100	17.5

10.3 Houses

The 1100 houses in the reduced study area appeared to be typical of the overall residential building stock in Petone.

A survey of 200 houses indicated that about 65% had been built before 1940. Only about 4% were more than a single storey. Nearly 80% were of timber weatherboard construction with light weight roofing (usually galvanised steel). About 8% were of unreinforced brick or masonry construction.

Figure 28. Reduced area for building study.

The remaining houses were either clad with stucco or cement based fibre board. About 4% had concrete tile roofs.

Over 95% were on timber or concrete piles and only 7% had concrete perimeter wall foundations.

About 54% of the houses had chimneys and these were mainly of brick construction with concrete being used on the more recently constructed houses.

10.4 Analysis Method for Shaking and Liquefaction Damage

Estimates of the extent of earthquake damage to the study area buildings and houses was made using the ATC-13 damage ratios and information obtained from the SHMJ building survey. The buildings were initially classified according the type of construction materials used for the load bearing structure and assigned one of the ATC-13 damage ratio curves shown in Figure 21. The construction types used were: unreinforced masonry, reinforced concrete, steel and timber. Reinforced masonry was assumed to have the same damage ratio curve as reinforced concrete. The damage ratios adopted for the three material classifications are summarised in Table 12.

TABLE 12
Building damage ratios.

	Damage Ratios % (ATC-13)		
MMI	URM Concrete	Reinforced Timber	Steel &
VI	3.0	2.5	1.0
VII	7.5	5.0	3.0
VIII	16	11	6.5
IX	30	21	11
X	46	34	19
XI	62	50	27
XII	76	65	40

Modifying factors were applied to the ATC-13 damage ratios to allow for the variation in seismic design standards as determined by the age of the building and for the main structure and component deficiencies identified in the building survey. The damage ratios were multiplied by the modification factors which were assumed to be independent of MMI. The modifying factors used are given in Table 13.

TABLE 13
Damage modifying factors.

Unreinforced Masonry

Classification	Modification Factor
MA	2.4
MB(ii)	1.8
MC1	2.1
MC2	1.9
MC3	1.7
MC4	1.6
MC5	1.5

Reinforced Concrete and Reinforced Masonry

Classification	Modification Factor
MC1	2.1
MC2	1.9
MC3	1.7
MC4	1.6
MC5	1.5
B	1.6
B-R	2.0
C	1.3
D	1.0

Steel

Classification	Modification Factor
C	1.3
D	1.0

Timber

E-R	1.8
E	1.0

The above modification factors were increased to allow for the presence of hazardous components including unreinforced chimneys, parapets, gables and bearing walls. The modification was made by adding a coefficient that varied between 0.1 to 0.3 to the modification factor. Generally a coefficient of 0.1 was added for each component.

Multiple chimneys or gables were regarded as a single component for the purpose of this modification. Hazardous internal partition walls were assumed to add a coefficient of 0.2.

The setting of the above modifications for building classification and hazardous components has from necessity been based on judgement as there is little data that can be used to develop a systematic approach. The modification factors and additional coefficients should strictly speaking vary with MMI but in view of the uncertainties involved a more sophisticated procedure was not justified.

The ATC-13 damage ratios for timber and unreinforced masonry were used for the houses as appropriate. No adjustment was made for age. A damage factor of 1.05 was applied to the standard damage ratios to make allowance for the increased damaged expected for about 50% of houses with brick chimneys.

Generally the older houses had weatherboard cladding with relatively small window openings. It was thought that this type of construction would not be significantly more prone to damage than more modern houses. It is possible that the older houses were not as well fixed to piles or as well braced in the sub-floor area as houses built to the more recent codes. However, generally the houses in Petone are on level sites with only short piles cantilevering above ground. This type of foundation can be expected to have satisfactory resistance to earthquake loads.

The above damage factors, ATC-13 damage ratios and floor area of each building was used to calculate the cost of ground shaking damage for each MM intensity from VI to XII. Where the modified damage ratios equalled or exceeded 80% the building was considered to be a complete loss and assigned a damage ratio of 100%. The total damage to the buildings and houses in the study area caused by ground shaking at each MM intensity was obtained by summation of the individual building damage costs. This approach gives directly the damage costs from ground shaking excluding the damage arising from ground failure or liquefaction.

Because detailed soils information was unavailable for specific sites in the study area, two more general methods were used to determine the effects of soil failures and liquefaction. Firstly the ATC-13 method of assuming that the cost of damage to above ground structures on soft ground is approximately five times the cost on firm ground. The total damage ratio can be expressed as:

$$DRT = DRF + DRF \times P(I) \times 4 \qquad (8)$$

where:

DRF = damage ratio on firm soil

P(I) = probability of ground failure at MM intensity I

In the ATC-13 report P(I) is defined for a range of different sedimentary deposits. For the study area, the values of P(I) specified for Holocene alluvium with the water table shallower than 3 m were considered appropriate and are given in Table 14.

TABLE 14
Ground failure probability.

MMI	P(I) in %
VI	2
VII	10
VIII	20
IX	30
X	40
XI	60
XII	80

The multiplying factor of 4 given in Expression (8) is assumed to be independent of structure type. However, it is likely that damage to heavy masonry structures from foundation failures might be greater than to light timber structures such as houses. Obviously the factor can be varied to suit structural type but selection of the appropriate factor needs to be based on judgement as there is no data to support a more rational approach.

The second approach is to assume that MMI on soft ground is related in some simple manner to the MMI on firm or average ground conditions. Smith (1977) recommended that for a given return period, the MMI on soft soil can be obtained by increasing the MMI for average soil conditions by one step.

The probability of exceeding the different levels of MMI intensity in the reduced study area were obtained using the Smith and Berryman (1992) earthquake hazard estimates for Wellington and Lower Hutt. Figure 29 shows a plot of their hazard estimates in terms of the annual exceedance of a specified MMI ranging from VI to IX. In order to extrapolate their estimates to higher and lower intensities a curve fit to their results was investigated. The following expression was found to give a good approximation over the MMI VI to IX range to the Smith and Berryman annual probabilities of exceedance on firm soil:

$$N(M) = 10^{0.5(4-M)} \qquad (9)$$

where

$N(M)$ = the annual probability that MMI will equal or exceed M

For MMI intensities greater than IX the above expression was modified to:

$$N(M) = 0.25 \times 10^{0.36(4-M)} \quad M > 9 \qquad (10)$$

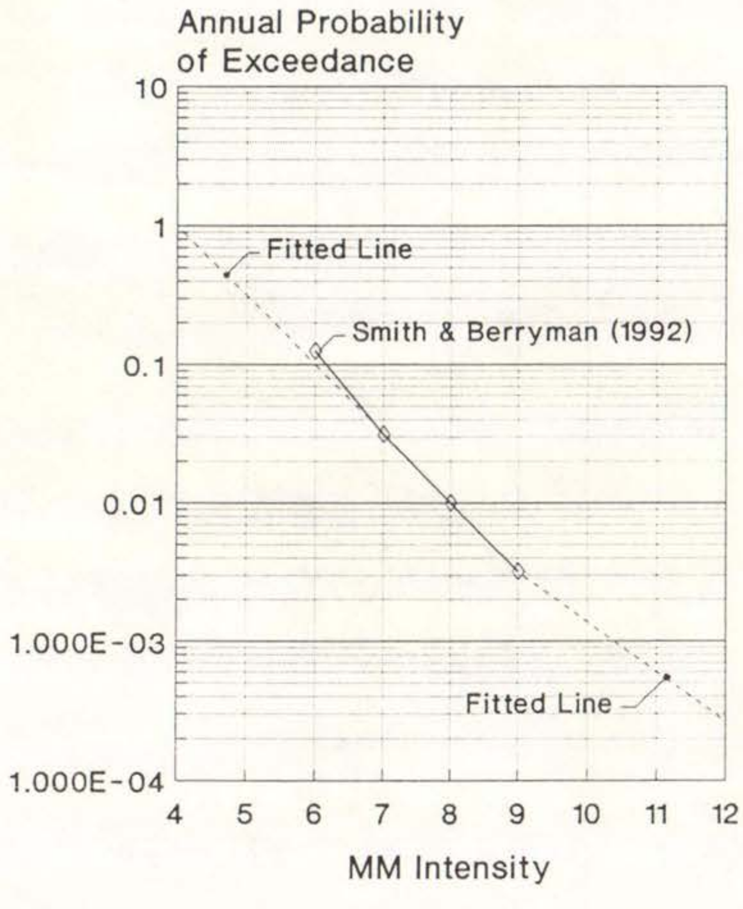

Figure 29. Relationships between MMI and annual probability of exceedance.

This modification was considered necessary to make allowance for the lower return periods of high MMI's expected because of the proximity of the Wellington fault to the study area. (The Smith and Berryman results are based on a uniform seismicity model.)

Expression 10 gives return periods of 630 and 1660 years for MMI's of X and XI respectively. Expressions (9) and (10) are compared in Figure 29 with the Smith and Berryman (1992) hazard estimates.

In order to calculate the damage at the various MMI levels Expressions 9 and 10 were used to calculate annual probabilities of occurrence for MMI ranges of VI to VII, VII to VIII,.....,XI to XII. These probabilities are summarised in Table 15.

TABLE 15
Annual probabilities.

MMI Interval	Ann. Prob. %
VI - VII	6.84
VII - VIII	2.16
VIII - IX	0.68
IX - X	0.18
X - XI	0.08
XI - XII	0.03

The annual damage costs for each MMI interval can simply be found from the total damage costs for each MMI level by multiplying by the annual probabilities of occurrence for each MMI interval. The total annual damage is then obtained from the summation of the MMI interval annual damage costs. The total annual damage in for the study area, expressed in terms of per cent was calculated for a number of different assumptions regarding the effects of soft soil and liquefaction. For the first analysis carried out including soft ground effects, the liquefaction multiplying factor in Expression 8 was assumed to be 4.0 for both buildings and houses. In the second analysis, the multiplying factor was taken as 4.0 for buildings and 2.0 for houses. Finally, both multiplying factors were take as zero and the soft soil conditions taken into account by using the Smith (1977) recommendation of increasing the MMI intensity by one step for a given return period. A summary of the calculated annual damage costs for the study area is given in Table 16.

TABLE 16
Annual damage costs for Petone.

Ground Assumption	Liquefaction Factors		Annual Damage % of Replacement Cost
	Buildings	Houses	
Firm soil	0	0	0.44
Including Liquefaction	4	4	0.66
Including Liquefaction	4	2	0.64
Soft soil MMI Increased	0	0	1.4

From the above analyses it can be concluded that the annual damage expected in the Petone area to buildings and houses from earthquake ground shaking and ground failure is likely to be in the range of 0.6 to 1.4% of replacement cost. On the assumption of an average building and house replacement cost of $800 per square metre, a 1% annual damage rate represents an annual cost of M$2.7 for the 527 buildings and 1100 houses in the reduced study area.

A Wellington fault event is expected to produce MMI X to XI in the study area. Assuming MMI X on firm soil and making the liquefaction modification (Equation 8) using a factor of 4 for buildings and 2 for houses gives a total damage cost to buildings and houses from ground shaking and ground failure in the reduced study area of about M$185. That is a cost of about 68% of the replacement cost of the structures.

It is informative to consider the contributions to the total annual damage from each of the MMI intervals. These contributions are plotted in Figure 30. The results for the firm soil with liquefaction assumption are based on liquefaction multiplying factors of 4.0 for the buildings and 2.0 for the houses. It is apparent that the major contributions to the total annual damage come from the lower intensities (VI to VIII). Although the high intensities of X and XI cause very high damage levels, their very infrequent occurrence results in small contributions to the total annual damage cost.

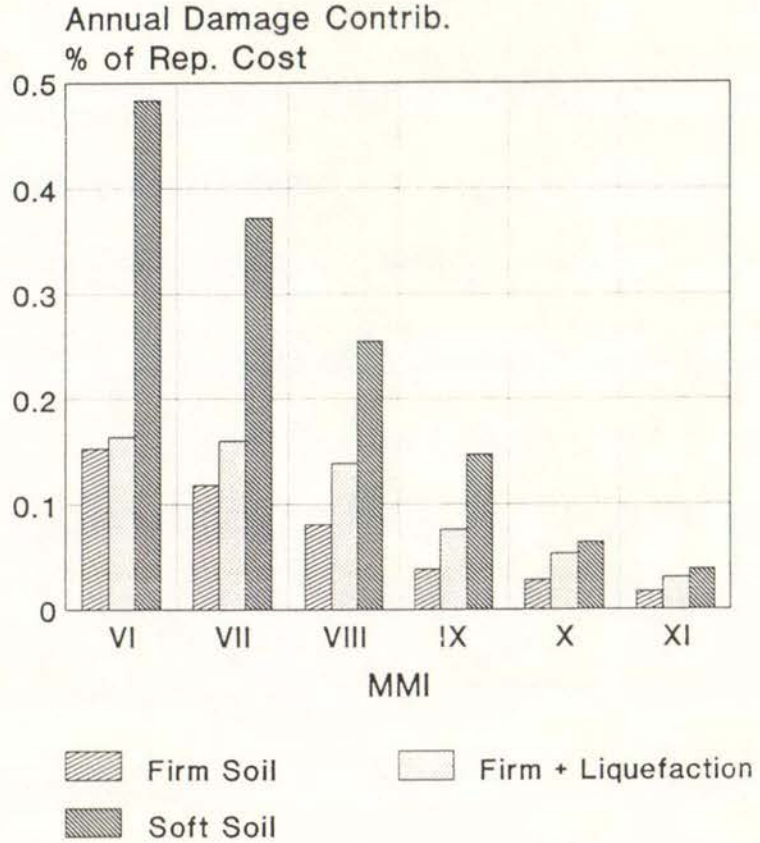

Figure 30. Annual damage contributions from each MMI level.

10.5 Estimate of Fault Displacement Damage

The Wellington Fault runs along the west side of the reduced study area. The location of the most recent fault trace is shown in Figure 31. Within the southern part of the study area, the zone of rupture is poorly defined and because of the apparent eastward bend or sidestep the zone might be up to 500 m in width. In the northern part the location of the trace is reasonably well defined by a scarp on the eastern side of the Hutt Road.

To estimate the damage from fault displacements three zones were defined in the vicinity of the fault as shown in Figure 31. The rupture Zone A was taken to be 30 m wide for the length of the well identified scarp (approximately 500 m long). Zone B representing the fault drag zone encompassed Zone A along the identified scarp and was taken as 100 m on either side of the scarp. Zone C was to the south of the identified scarp length and was assumed to have a maximum width of about 600 m to include the total area likely to be affected by ground displacements over the length where the zone of rupture is poorly defined.

Details of the ground surface and buildings in each of the three zones are given in Table 17.

TABLE 17
Details of fault damage zones.

	Zone A	Zone B	Zone C
Ground area, m^2	15,000	65,000	387,000
Number of buildings	39	45	139
Area of buildings, m^2	15,300	17,200	76,200
Number of houses	9	50	71
Area of houses, m^2	945	5,250	7,455
Fault damage factor, %	100	10	6.1

The damage factors for buildings damaged by fault ground displacement in Zone A and Zone B were assumed to be 100% and 10% respectively. Following the procedure used in ATC-13, these damage factors were assumed to be additional to the ground shaking and ground failure damage factors. The damage factor for Zone C was based on a weighted average value assumed to apply to all buildings in the zone. The weighted average value was based on assuming a fault rupture area 30 m wide with a 100% damage factor and a fault drag area of 200 m wide with a 10% damage factor would occur somewhere within the total area. Outside these two areas the damage factor was taken as 0%.

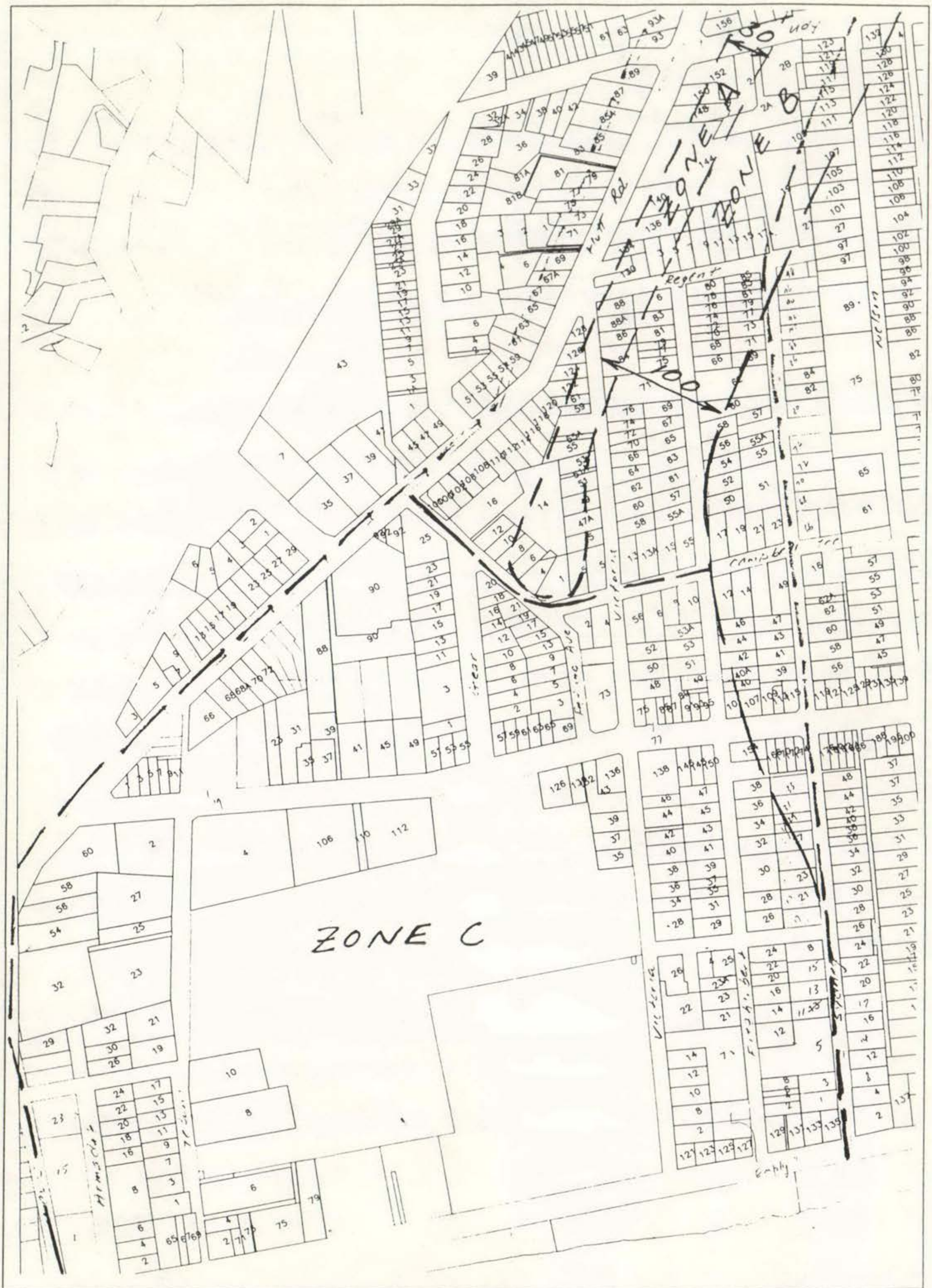

Figure 31. Wellington fault damage zones.

In a Wellington fault rupture event the ground shaking in the vicinity of the fault trace is expected to have an MMI value of X or higher. The analyses indicated that most of the concrete and unreinforced masonry buildings located in the fault damage zones would receive more than 80% damage (percentage of replacement cost) by ground shaking and soil failure alone without the additive effects of fault ground displacement. These buildings would therefore require replacement as a result of ground shaking and ground failure alone and there is essentially no additional damage costs from fault displacement effects. Analyses indicated that most of the steel and timber framed buildings (including houses) would receive between 49 to 64% damage from ground shaking and ground failure alone. For these buildings additional damage from fault displacements results in an increase in the total damage cost.

The additional damage to buildings and houses within the three fault displacement zones caused by ground displacement during a Wellington fault rupture was calculated to be 4.3% of the total replacement value. This damage is equivalent to 1.5% of the total house and building stock replacement value within the study area. These additional damage costs are low because many of the buildings in the fault damage zones would be destroyed or heavily damaged by ground shaking and ground failure.

Based on an annual probability of rupture of 0.34% for the Wellington fault, the additional annual damage cost to the buildings in the study area arising from fault displacement damage is 0.005%.

10.6 Estimate of Tsunami Damage

A future earthquake on the West Wairarapa Fault is the most likely event to cause tsunamis at the Petone shoreline. Subduction thrust events at the Hikurangi margin would result in considerable regional uplift and tilting, so could also cause tsunamis at Petone. These events have estimated annual probabilities of occurrence of about 0.02 and 0.03% respectively.

Studies reported by the Wellington Regional Council, 1990 indicate that the maximum run-up height at Petone foreshore for a West Wairarapa event would be about 1.6 m. If this level of run-up occurred at high tide, water would cover properties at a distance of up to 100 m from the shore line. This would affect mainly houses as there are not a large number of buildings in the study area close to the shore.

Damage from Tsunami inundation is mainly related to depth of water and the water velocity. Within the harbour, surging can be expected rather than the high velocity translating waves that have occurred on open coast-lines. The ATC-13 indicates mean damage factors of 10% for inundation by high velocity water of 1.0 m depth. It would therefore seem likely that the damage factor for Tsunami effects on the Petone foreshore might be significantly less than 10%. Based on low damage factors and taking into account the low annual probabilities of occurrence of the Tsunami generating events, the annual damage costs from Tsunami inundation are very small in relation to damage arising from ground shaking and ground failure.

13 CONCLUSIONS

(a) The study revealed that although analytical procedures for estimating earthquake damage costs are well advanced there are shortcomings in the basic inputs which reduce the reliability of damage estimates. In particular, there was a lack of detailed soils information that prevented a reliable prediction of the extend of liquefaction which was found to have a large impact on the damage costs in the study area. There is also a shortage of reliable information on damage factors for New Zealand buildings. There would be considerable merit in more extensive study of the damage records from past New Zealand earthquakes.

(b) Although estimating damage risks for individual structures is straightforward, collecting the necessary information to allow reliable damage estimates to be made on a regional basis for buildings and houses in urban areas is difficult and time consuming. The records of most local authorities are incomplete and not in a suitable form for convenient application in risk assessment work.

(c) Liquefaction and soil failure in weak soils can result in damage costs being about double the costs expected on firm soil or rock. To make reliable risk estimates it is therefore important to have accurate information on the geology and engineering properties of the foundation soils.

(d) Damage from fault displacements is unlikely to be a large part of the total annual damage costs from earthquakes. Firstly, the recurrence intervals for fault movements are large and although the consequences of movements might be severe, the long time interval between movements results in a low annual damage risk from individual faults. Secondly, when fault rupture occurs the buildings in the vicinity of the fault are likely to be seriously damaged by ground shaking and soil failure so that the additional effects from fault displacement damage are difficult to identify and assess separately. Finally, the zone of high damage from fault rupture displacements is usually less than 100 m wide and thus the area affected by a single event is much less than is the case for strong ground shaking.

(e) As a consequence of the generally weak foundation soils and many old unreinforced masonry buildings in the study area, the annual costs of earthquake damage and the costs of a maximum credible earthquake were found to be exceptionally high.

12. ACKNOWLEDGEMENTS

Ian R Brown and Associates made major contributions to the project by providing the cadastral and geotechnical database and entering the building survey information into a compatible database to enable the computer processing work to be undertaken. They also assisted by writing major parts of Sections 2 and 3 of the report that describe the geology of the study area and by reviewing many aspects of the work.

Spencer Holmes-Millar and Partners provided copies of the building documentation obtained from the survey carried out for the Petone Borough Council.

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