



Institute of
**GEOLOGICAL
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SCIENCES**
Limited

ASSESSMENT OF LIQUEFACTION AND GROUND FAILURE HAZARDS IN THE GREATER WELLINGTON REGION, 1992-1993

STAGE 1

IGNS CONTRACT REPORT No 1992/84

Prepared for

The Wellington Regional Council

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9 October 1992

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16518

SUMMARY

Strong shaking during earthquakes often results in the phenomenon known as liquefaction. In this processes, strong shaking may cause certain soils (mainly saturated, cohesionless, uniform fine sands and coarse silts) to compact, increasing pore water pressure and decreasing shear strength to the point where the soil is transformed to a liquid state. These changes can cause a loss of strength in near-surface materials (generally at depths of up to 20 m below ground level in areas where the water table is within about 5 m of the ground surface), resulting in significant ground deformations.

Ground damage due to the liquefaction process often occurs in the form of compressional ridges, fissures, lateral spreads and flow failures, settlements, along with sand boils and water ejections that are perhaps the most common indication that liquefaction has occurred. Such ground deformation can be particularly damaging to buried underground services, and to poorly designed and constructed buildings. Complete liquefaction of loose, uniform, saturated fine sands and coarse silts, the most susceptible materials, can occur at distances of up to 150 km from large ($\geq M 7.5$) earthquakes, and can cause the most severe ground damage.

Records of historical seismicity show that the greater Wellington area has been affected by strong shaking of MM intensity VI or greater 24 times since 1840. However, it is also clear from these records that only minor ground damage, possibly due to liquefaction phenomena, has occurred only at intensities starting at about MM VIII in the most susceptible areas of soft, saturated recent sediments. Widespread major ground damage and liquefaction effects were reported in low-lying areas after the great 1855 Wairarapa earthquake, which caused MM X intensity shaking in the Wellington Region. It is apparent from this and other historical earthquakes in New Zealand and overseas, that a large earthquake on one of the several major active faults close to large urban centres in the Wellington Region would cause major liquefaction-induced ground damage, and consequent damage to buildings, structures, lifelines and services.

In the five study areas that have been identified in the Wellington Region, the localities most susceptible to liquefaction-induced damage effects are those areas of soft recent sediments classed as Zones 3-4, and 5 in earlier ground shaking hazard reports produced for WRC. In these zones, ground damage and/or liquefaction during a large nearby earthquake is expected to cause major damage to underground services (pipelines, storage tanks), bridge approaches and abutments, embankments for roads, railways, and stop banks, wharves, railway lines, and some buildings. Further studies of the records of the larger historical earthquakes that have affected Wellington Region would be useful to help identify those areas with greatest liquefaction and ground damage hazard.

Previous work that has been carried out for WRC contains much information of direct relevance to this liquefaction hazard assessment. As indicated above, those areas defined as ground shaking hazard Zones 3 to 5 are considered to be most susceptible to ground damage due to liquefaction processes. Further data relevant to the liquefaction hazard assessment are available in reports prepared for various Local and Roding Authorities. Only relatively minor new work is required to complete the assessment of liquefaction hazards. Such work would include CPT probes and field inspections in poorly investigated areas, evaluation and plotting of available sub-surface geotechnical data, review of existing geotechnical records and reports, and the evaluation of the historical earthquake records. It is considered that this additional work will help to more clearly define areas within shaking hazard zones 3 to 5 that are most susceptible to ground damage and liquefaction effects. It would also enable the general type and extent of expected ground damage to be estimated.

For the five study areas it is proposed to present the liquefaction and ground damage hazard assessments on separate maps at the same scales as the existing shaking hazard maps. This will allow direct comparison with earlier hazard zonation work. Data will be presented both as map overlays, and in summary text form. A proposed work programme and estimate of costs to complete the liquefaction hazard assessment is included in the report.

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1 INTRODUCTION

Studies of seismic hazards for the Wellington Regional Council were initiated in 1988 with the aim of improving earthquake safety within the Region. In the past four years, these studies have resulted in the completion of 21 geological and geotechnical reports by DSIR Geology and Geophysics and other DSIR Divisions, as listed in Appendix 1. These reports were prepared on behalf of the Wellington Regional Council (WRC) to provide much of the essential geological and seismic information necessary to define the seismotectonic hazards that exist throughout the Region. Such hazards include surface fault rupture, tilting and warping, ground shaking, and ground failure (e.g., liquefaction effects, landsliding).

This report presents STAGE 1 of an assessment of liquefaction and ground failure hazards in the greater Wellington Region. The work has been undertaken for WRC by the Institute of Geological and Nuclear Sciences Limited (IGNS).

The 1992/93 Annual Plan for the WRC includes the preparation of "reports and maps defining liquefaction and lateral spreading hazards within the urban areas of the Wellington Region".

This work is referred to in this report as Stage 2.

1.1 Authorization, objectives and scope of the report

The scope of Stage 1 of the WRC liquefaction study is defined in an Agreement between WRC and IGNS dated 18 August 1992, which specifies 5 separate study areas (Figure 1), defined as follows:

- Study Area 1 - Wellington City
- Study Area 2 - Hutt Valley
- Study Area 3 - Porirua Basin
- Study Area 4 - Kapiti Coast
- Study Area 5 - Wairarapa

For each of the study areas shown in Figure 1 an assessment of liquefaction potential and resulting ground failure hazards will be required (during Stage 2 studies), with the results to be presented in separate reports and maps for each study area.

Stage 1 of the assessment involves an initial review of existing information and the definition of additional work required in each study area to meet the overall objectives. Specific work that was to be undertaken for Stage 1, and covered by this report is as follows:

- (1) Definition of liquefaction and brief review of important New Zealand and overseas case histories of past occurrences of liquefaction and resulting ground damage that are relevant to an assessment of potential liquefaction and ground damage hazards in the Wellington Region.
- (2) Identification and review of existing and reasonably accessible information on past occurrences, or potential occurrences, of liquefaction and resulting ground damage in the Wellington Region.
- (3) Identification of the most appropriate techniques for completing an assessment of the potential liquefaction and ground damage hazard for the five Study Areas.
- (4) Identification of existing data sources that would be used in an assessment of the potential liquefaction and ground damage hazard in the Study Areas. Include an assessment of the costs of accessing these data.
- (5) Identification and prioritisation of the requirements for additional data collection and analysis to meet the objective of the study (as specified in Clause 3.2. of the WRC/IGNS Agreement).
- (6) Identification of appropriate scales of mapping liquefaction potential and ground damage hazard for each study area.
- (7) Development of a work programme and cost estimates to complete Stage 2 of the study, to the time deadlines specified in Clause 3.2 of the WRC/IGNS Agreement. Alternative time deadlines for draft reports and maps can be provided, with the exception of Study Area 3, provided that all drafts are completed by 2 April 1993, and all final reports and maps are supplied to the Client by 7 May 1993.

1.2 Timing and reporting of Stage 1 work

The results of the Stage 1 assessment are to be presented to the WRC in a draft report by 1 October, 1992. Any comments and suggestions on the draft report are to be provided within three working days by WRC, and 5 copies of the final report are to be delivered to the WRC by 9 October, 1992.

1.3 Methodology

This report has been prepared from existing information available from a variety of sources. The main sources of relevant local data are the 21 reports prepared for the WRC (Appendix 1). In addition extensive use has been made of published case histories of ground failure and liquefaction effects observed during earthquakes, both local and overseas, and also IGNS records and files.

2 THE LIQUEFACTION PROCESS

2.1 Introduction

The 1964 Niigata, Japan earthquake (M 7.5) generated widespread and spectacular building and ground damage as a consequence of soil liquefaction. Loss of bearing strength of soils caused buildings to settle and tip, including several 4-storey apartment blocks in the Kawagishicho complex that were otherwise structurally undamaged by shaking effects of the earthquake. Widely publicised photographs of the tilted buildings dramatically focused world attention on the liquefaction process and associated hazards. During the Niigata earthquake, liquefaction-induced lateral-spreading displacements of up to 10 m tore apart buildings, sheared piles, severed pipelines, compressed or collapsed bridges, and caused general destruction to the effected areas. Overall, liquefaction induced ground-failures caused severe damage to tens of square kilometres of Niigata and its environs (Hamada et al., 1986).

In the same year (1964) the great Alaska earthquake (M 8.3) triggered large flow failures that demolished port facilities in Valdez, Seward and Whittier and carried large parts of those towns into the sea. Earthquake shaking and flow slides in turn generated seiches in surrounding bays, some of which over-ran coastal areas, creating additional damage and havoc, and causing many deaths. The earthquake also caused numerous lateral spread failures that severely damaged the Alaska highway and railway systems, and damaged 266 bridges, many beyond repair. That destruction prevented use of much of the highway and rail systems for months following the earthquake, greatly adding to the havoc caused by the event.

Similar severe effects have been reported during other large earthquakes. Some well documented cases (by no means a complete list) of damage due to liquefaction effects during recent historical earthquakes are as follows:

- destruction of sea walls and port facilities during the 1960 earthquake in Chile (Duke and Leeds, 1963)
- development of large landslides on gentle slopes, and partial failures and significant damage to earth dams during the 1971 San Fernando earthquake in California (Seed, 1972)
- damage to buildings, roads, bridges, pipelines, and farmland, during the 1976 Tangshan earthquake in China (Yong et al., 1988)
- widespread damage to roads, pavements, marina, harbour, and airport facilities, buildings, utilities, and services, with associated ground damage during the 1989 Loma Prieta earthquake (Housner et al., 1990; Seed, 1990)
- considerable damage to buildings, bridges, roads and pipelines due to failure of liquefaction-affected soils adjacent to rivers and coastal areas, and on loose sandy fill during the 1990 Philippines earthquake (Hopkins et al., 1991)

The numerous publications and well documented records of damage, intensity and liquefaction effects in localised areas of soft soils, and numerous strong motion recordings from the Loma Prieta earthquake (e.g. as described by Housner et al., 1990) are a valuable record of the event which are particularly relevant to the Wellington regional liquefaction assessment.

In New Zealand, the most widespread occurrences of liquefaction since 1840 were caused by the 1848 Marlborough, 1855 Wairarapa, 1931 Napier and 1987 Edgecumbe earthquakes. All these earthquake events occurred in coastal regions having plentiful fine-grained, recent alluvial deposits (Fairless and Berrill, 1984). Apart from Edgecumbe, a smaller (M 6.3) but unusually shallow event, liquefaction in New Zealand has been reported for most earthquakes of magnitude 6.9 or greater. Instances of liquefaction in New Zealand are discussed in more detail in Section 3.

The influence of soils on the behaviour of structures during earthquakes received very little attention from the geotechnical community before the catastrophic failures that occurred in Niigata and Alaska during 1964. In addition, the remarkable relationship between the intensity of structural damage and local soil conditions in Caracas, Venezuela during the 1967 Caracas earthquake stimulated great interest in the general field of earthquake soil dynamics. Fostered by public concern for the safety of nuclear power plants, the safety of dams, and the USA National Science Foundation programme on Earthquake Hazards Mitigation, liquefaction studies and research grew dramatically until they now represent a significant part of the work of many geotechnical engineering companies and research organisations.

2.2 Definition of liquefaction and related terms

Many definitions of the liquefaction phenomenon and related effects caused by earthquakes have appeared in geotechnical publications, both in New Zealand and overseas. Liquefaction has been defined by Youd (1973) as "the transformation of a granular material from a solid state into a liquified state as a consequence of increased pore pressures". Alternatively, Ziony (1985) defined liquefaction as "the process by which water-saturated sediment temporarily loses strength, usually because of strong shaking, and behaves as a liquid".

Perhaps the most widely accepted definitions of terms related to seismic liquefaction are those recommended by the American Society of Civil Engineers (ASCE Committee, 1978), and these definitions have been adopted for this study.

The main terms that are relevant to this study are defined as follows:

Liquefaction - The act or process of transforming cohesionless soils from a solid state to a liquified state as a consequence of increased pore pressure and reduced effective stress.

Comments:

- (1) Liquefaction is usually associated with and initiated by strong shaking during earthquakes, which causes certain soils (mainly cohesionless, uniformly-graded fine sands and coarse silts) to compact, increasing pore water pressure and decreasing shear strength. The term is strictly defined as a changing of state that is independent of the initiating disturbance that could be a static, vibratory, sea wave, or shock loading, or a change of ground water pressure. The definition is also independent of deformation or ground failure movements that might follow the transformation to a liquid state. The liquefaction process always produces a transient loss of shear resistance, but not always a longer-term loss of shear strength.
- (2) Liquefaction is most likely to occur in saturated, relatively uniform, cohesionless, fine sands, silty sands, or coarse silts of low relative density (loose), generally at depths of up to 15 to 20 m below ground level, in areas where the water table is within 5 m of the ground surface. Such materials have relatively low permeability and dissipate increased pore-water pressures (drain) slowly. Although liquefaction effects are observed only in loose soils, dense sands and silts may show initial liquefaction (strain softening) effects, these are rapidly inhibited by the dilatancy characteristics of such soils.

Cyclic strain softening - this process is defined in relation to liquefaction as a stress-strain behaviour under cyclic loading conditions in which the ratio of strains to differential shear stresses increases with each stress or strain cycle. In saturated cohesionless soils cyclic strain softening is caused by increased pore-water pressure (see also Section 2.4).

Comments:

- (1) Continued cyclic loading (as experienced during strong earthquake shaking) usually leads to increasing axial strains and increasing pore-water pressures, but does not necessarily lead to loss of ultimate shear strength if the material is dilative.
- (2) Cyclic strain softening occurs in cohesionless loose soils as a part of the liquefaction processes. However, in cohesive soils (mud and clayey soils) cyclic strain softening effects (increased pore-pressures and decreased shear strength) can occur, resulting in some ground deformation or damage (collapse and settlement due to decreased bearing capacity), but complete liquefaction does not occur.

Ground failure - A term related to the field behaviour of soil and rock masses, and defined as a permanent differential ground movement capable of damaging or seriously endangering a structure. Related terms include:

(a) *Lateral spread* - distributed lateral extensional movements in a fractured soil or rock mass, in which extension of the ground results from liquefaction or plastic flow of the materials. Lateral spreading commonly develops along the banks of rivers and streams, and man-made water courses (canals). Sand and water ejections are often associated with lateral spread fissures.

(b) *Flow failure* - Flow failures (slides) are a form of slope movement involving the transport of earth materials in a fluid-like manner over relatively long distances, at least tens of metres.

(c) *Sand boil* - An ejection of sand and water from cracks or fissures, and caused by piping from a zone of excess pore pressure within a soil mass. Sand boils commonly form during or immediately after earthquakes as pressures are relieved from liquefied zones, or zones of excess pore pressures in subsurface saturated cohesionless soils. Sand boils are the most common and unambiguous indicator that liquefaction has occurred.

From a soil mechanics point of view, it is believed that the basic cause of liquefaction in saturated cohesionless soils during earthquakes is the buildup of excess pore pressure due to the application of cyclic shear stresses induced by the earthquake ground motions. These stresses are generally considered to be due primarily to upward propagation of shear waves in a soil deposit, although other wave motions may also be influential. If the liquefaction effect is sufficiently severe and extensive, loss of ground strength may result in damage to any structures located in the affected area. Bearing capacity failure will cause buildings or superficial structures to settle and tilt, and buried structures, such as underground pipes and tanks, may float upwards. Liquefaction of a confined sub-surface layer can cause large vertical and lateral displacements of the ground surface, or possibly only minor effects such as sand boils and water ejections, on otherwise unaffected alluvial (gravel) surfaces. If the area is on a gentle slope, or close to a free face such as an incised river channel or open drain, then lateral spreading failures can occur.

It has long been recognised that the intensity of ground shaking during earthquakes and the associated damage to buildings are influenced by local site conditions, particularly soil and rock types, and depth to the water table. However it has only been in the last 35 years that strong motion instrumental records have been obtained at a number of locations in the same general area to show the major effects of variations in local soil conditions on the characteristics of strong ground motions. Although liquefaction is a most damaging consequence of large and moderate earthquakes, other damage effects on 'soft soil' sites are also important and are commented on in the report.

2.3 Assessment of liquefaction potential

Generally the assessment of liquefaction potential involves two steps:

- (1) Evaluation of liquefaction **susceptibility**. This involves the identification of those areas or layers which have the characteristics of liquefiable soil.
- (2) Evaluation of liquefaction **opportunity**. This involves determination of the occurrence of earthquake shaking strong enough to generate liquefaction in susceptible materials.

Liquefaction potential and hazard are site dependent. Certain soils are more liquefiable than others. Liquefaction is most likely to occur in saturated, relatively uniform fine sands or coarse silts in a loose state, at depths less than 20 m, where the groundwater level is within about 5 m of the ground surface.

For particular sites or regions, information on liquefaction susceptibility and opportunity are assessed and considered together to determine the liquefaction **potential**, and gives an indication of the relative likelihood that liquefaction will occur.

The assessment process may be summarised as follows:

LIQUEFACTION SUSCEPTIBILITY		LIQUEFACTION OPPORTUNITY		
a function of the soil's ability to resist liquefaction. This depends on: - soil type - relative density - water table depth - depth of soil layer	+	a function of the intensity of seismic shaking. This depends on: - magnitude and proximity of the earthquake (at least MM VII) - frequency of events $\geq$ MM VII - duration of shaking	=	LIQUEFACTION POTENTIAL
NOTE: LIQUEFACTION HAZARD = LIQUEFACTION POTENTIAL + GROUND FAILURE POTENTIAL (see Table 1).				

2.4 Cyclic strain softening

Most saturated soils which are subjected to rapid undrained cyclic shearing (as is the case during an earthquake) will experience a change in their pore water pressure. Due to their ability to rapidly dissipate any change, only very permeable soils such as coarse sands and gravels are unlikely to experience much change in pore water pressure. Dense sands may experience a decrease in pore water pressure due to dilation during shearing. However, the soils of greatest interest to the engineer are those in which seismic excitation generates an increase in pore water pressure and a consequential decrease in strength. Such soils are most susceptible to ground damage by strong earthquake shaking.

Laboratory studies show that both cohesive and non-cohesive soils can experience an increase in pore water pressure during cyclic loading. During the application of cyclic loading to a soil, slip occurs at the grain-grain contacts, causing a volume change. For a saturated, undrained soil the tendency for volume change results in the transfer of some of the intergranular stresses to the pore water. If the soil is contractive, a pore water pressure increase results, and if the soil is dilative, a decrease in pore water pressure occurs. This increase in pore water pressure is thought to occur at the major peak seismic stresses, or during the unloading part of the stress cycle. However for sands at least, there appears to be a critical shear stress level, below which no pore water pressure increase occurs.

Some researchers (eg. Tsatsanifos, 1982) consider that the (lower) shear stresses induced by the initial part of a seismic motion record may contribute more to the development of pore water pressure within a layer than the higher stresses in the main part of the record. This is due to a decrease in effective strength resulting from the pore water pressure rise in the early part of the motion, which means *a reduced ability for the soil to transmit the higher shear stresses*. Consequently these later higher shear stresses may have a reduced effect upon the pore water pressure development in a soft soil, but will lead to greater displacements and non-linear behaviour in the weakened soil. A number of researchers have found that the duration of large shear stresses during seismic motion is a major factor governing pore water pressure development.

In undrained cyclic triaxial tests on a fully remoulded silty clay, Ogawa et al., (1977) found that even for a large number of stress cycles, the pore water pressure in the sample would not quite reach the full confining pressure to give complete liquefaction. Moreover it was found that even after cessation of the cyclic loading, the pore water pressure continued to rise for some minutes to almost reach the confining pressure. This result raises the possibility of post-earthquake shear strength failures. In such a case, actual failure may be induced by an aftershock. During cyclic triaxial testing of clay, as the pore water pressure increases, the strain amplitude also steadily increases until eventually a yield point is reached, after which the *axial strain increases* and the deviatoric stress decreases rapidly as the number of cycles increases.

This type of plastic, non-linear soil behaviour can lead to large permanent displacements at the surface of a soil mass during a strong earthquake, such as experienced in the epicentral area of the 1987 Edgumbe earthquake, which may be damaging to buried services (pipelines, etc) paths, roads, rail lines and poorly designed structures (see Section 3.3)

In summary, therefore, it can be seen that the development of excess pore water pressures leading to the liquefaction of a soil mass during earthquakes depends on:-

- soil grain size, permeability, and thickness
- soil layer up to 20 m below ground surface
- depth to water table of about ≤ 5 m (possibly 10 m) below ground surface
- duration and intensity of strong shaking (at least MM VII, or usually MM VIII for liquefaction)
- hydraulic boundary conditions and confining pressures
- level of stress and amplitudes of strain in soil mass

The situation where the pore water pressure generated during an earthquake becomes equal to the overburden pressure (ie a situation of zero effective stress exists in the soil) has been called "true" or "complete" liquefaction, and available evidence indicates that "true" liquefaction occurs commonly in non-cohesive, uniformly graded fine sands and coarse silts. [Note that the ASCE Committee (1978) discourages the use of such qualifiers in relation to liquefaction as they can create confusion.]

Liquefaction generally occurs at strong levels of ground shaking. On the Modified Mercalli intensity scale (Appendix 2), liquefaction effects become evident at MM VIII, at distances of 100-150 km from the epicentre for earthquakes of magnitude 7 to 7.5, and up to 400 km from the epicentres of great (M 8-9) earthquakes (Ambraseys, 1988).

The condition where cyclic stresses induce increased pore water pressures, reduced soil strength, and plastic or non-linear behaviour, leading to permanent ground deformations (**cyclic strain softening** or **cyclic mobility**), can occur in both saturated cohesive and non-cohesive soils. Field evidence indicates that cyclic strain softening occurs most commonly in "soft" soil sites (where the resulting ground damage is most apparent), and at higher intensity shaking (MM VIII or greater) generally close to the epicentral areas of large-magnitude earthquakes.

It is clear that "true" liquefaction can lead to significantly greater ground damage and is potentially more destructive than ground damage associated with cyclic strain softening. However, unless there is detailed and specific sub-surface information available on the nature of the soils at a particular site, it often cannot be readily determined which has caused the resulting ground damage. Hence in many cases they are not differentiated and both are often included under the general banner of "liquefaction".

2.5 Ground damage due to earthquakes

It is well documented in geotechnical literature that ground failures during earthquakes usually are of two main types - (a) landsliding on steep slopes, and (b) failures in flat, low-lying areas. Both types are described by many authors. For example, the former by Keefer (1984), Wilson and Keefer (1985), and Hansen and Franks (1991); the latter by Seed (1968), Youd and Perkins (1978), Seed and Idriss (1982), and Youd (1991). Failures on gentle slopes or flat-lying areas are usually attributed to the process of soil "liquefaction". This process is independent of any ground failure movements that might follow the transformation to a liquefied state. The four main types of ground failures and typical ground damage effects due to liquefaction are summarised in Table 1.

In the past fifty years more efforts seem to have been made to understand and predict liquefaction potential and ground failures and hazards caused by liquefaction, rather than earthquake-induced landsliding. Hansen and Franks (1991) suggest that this is because large areas within large densely populated cities are underlain by potentially liquefiable sediments, and these areas are perceived to be more vulnerable. This is the case in the Wellington Region where ground failures due to liquefaction are likely to be serious because of the essential facilities and structures that are at risk in urban centres, and the greater damage likely to be experienced in areas underlain by potentially liquefiable sediments. These issues are discussed further in Section 4.

LIQUEFACTION FAILURE MODE	TYPICAL GROUND DAMAGE AND EFFECTS
1. LATERAL SPREADS	Small to large lateral displacements of surficial blocks of sediments, on gentle slopes (< 3°). Movements, commonly of several metres to tens of metres, are usually toward a free face, particularly an incised stream channel, canal, or open cut. Particularly damaging to pipelines, bridge, structures with shallow foundations, particularly on flood plains adjacent to river channels.
2. FLOW FAILURES	Flow failures, the most catastrophic mode of liquefaction failure, are usually developed on slopes greater than 3°, with movements ranging from tens of metres to several km, at very rapid velocities. Such flows can be very large, and are highly damaging to all structures located on them, or in their paths.
3. GROUND OSCILLATION	Occurs when liquefaction occurs at depth, on slopes that are too gentle for lateral displacement, or are confined. Produces visible ground oscillation waves, ground settlements, opening and closing of fissures, ejections of sand and water from cracks and fissures (sand "boils"). Overlying and subsurface (pipes, tanks, etc.) structures often damaged, usually relatively minor compared to other failure modes.
4. LOSS OF BEARING STRENGTH	Strength loss caused by liquefaction can cause ground collapse and settlements; structures may settle and topple, and buried structures (pipelines, septic tanks, etc.) may rise to the surface. Spreading and collapse of embankment fills often occurs due to liquefaction of foundation soils.

Table 1: Typical ground failures and damage caused by soil liquefaction (*after Tinsley et al., 1985*)

3 LIQUEFACTION CASE HISTORIES

Some recent earthquakes that have produced significant damage due to liquefaction have been well studied and serve to illustrate points that are relevant to this study. Three of these events are briefly discussed below.

3.1 The 1964 Niigata earthquake

Although the epicentre of the Niigata earthquake ($M \approx 7.5$) was located some distance (about 56 km) from Niigata, and the maximum ground accelerations recorded in the city were only about 0.16 g, the earthquake induced extensive liquefaction in the low-lying areas of the city. Water began to flow out of the ground from boils and cracks during and immediately after the earthquake. This liquefaction caused widespread damage. Many buildings settled by more than 1 m in the liquefied soil, often accompanied by severe tilting. Thousands of buildings collapsed or suffered major damage as a result of these effects.

The city is built upon a deep deposit of uniformly graded medium sand. Following the earthquake an extensive survey of the distribution of the damaged structures was made. It was found that buildings in the coastal dune area (Zone A) suffered practically no damage. The major damage and liquefaction were concentrated in the lowland areas, where two distinct zones (B and C) could be clearly recognised. In Zone B the damage was relatively light, but in Zone C the damage and liquefaction effects were most extensive. Since all zones contained similar types of structures, the differences in the extent of damage could be attributed to differences in the subsoil and foundation behaviour.

Studies were subsequently carried out to determine the differences in soil conditions in the three zones. These showed that the difference in behaviour in Zone A from that in Zones B and C could readily be attributed to two major differences in soil characteristics. Although all zones were underlain by sandy soils to a depth of approximately 30 m, in Zone A the sands were considerably denser than those in Zones B and C, and the water table was at a much greater depth beneath the ground surface. However, in Zones B and C the general topography and shallow depth to the water table was essentially the same. Hence the difference in extent of damage was considered to be related to the characteristics of the underlying sands. Accordingly considerable effort was made to determine any significant differences in the general soil conditions in those zones.

As the soils involved are sands, subsurface investigations efforts were concentrated on the determination of their relative density by means of standard penetration test (SPT) probing. Although there is considerable scatter in the SPT results from any one zone, a clear trend emerges showing that the sands below 4.5 m depth are slightly denser in Zone B than Zone C. Down to 4.5 m depth both zones are essentially the same and below about 14 m depth the sands in both zones are relatively dense and are unlikely to be involved in liquefaction. It is therefore concluded that the relatively small differences in penetration resistance (of about 5 to 10 blows per 300 mm) reflect the differences in sand density that were responsible for the major difference in liquefaction and foundation behaviour in the two zones.

At Niigata, extensive studies were also made of the influence of foundation type on settlement and tilting. Raft foundations were compared with piles, and piles of various length were studied (Seed and Idriss, 1982). The publishing of all these results has made available much information from the extremely damaging earthquake which is invaluable for others planning and designing to mitigate earthquake hazards.

3.2 The 1964 Great Alaskan earthquake

The $M \approx 8.3$ Alaska earthquake of 29 April 1964 produced major and widespread liquefaction effects, including several massive landslides (spreads and flows) in the cities of Anchorage, Seward, and Valdez, and around the shores of Kenai Lake (Seed 1968). The landslide at Valdez led to a decision to relocate the entire town on stronger foundation materials several kilometres away. Damage in Seward and Anchorage was catastrophic.

The massive landslides occurred on gently sloping ground when extensive cohesionless sandy silt soil layers were sufficiently weakened by earthquake-induced cyclic strain softening and/or liquefaction, resulting in massive translational flow failures.

On relatively flat ground away from the towns, soil liquefaction caused extensive damage to a wide variety of bridge foundations and approaches at locations 80 to 130 km from the epicentral area. Damage included horizontal movement of abutment foundations towards stream channels, spreading and settlement of approach and embankment fills, horizontal displacements and tilting of piers, and severe differential settlements of abutments and piers.

The greatest concentrations of severe damage occurred in places characterised by thick deposits of saturated, cohesionless soils. Ample evidence exists of liquefaction of such materials during the earthquake, which is likely to have played a major role in the development of foundation displacements and observed damage to bridges and other structures. Typical foundations in these areas consisted of piles driven through saturated sands and silts of low to medium relative density (SPT N values of about 20 to 25). Of about 60 tested samples from the heavily damaged areas, two-thirds were uniform fine sands. By contrast, bridges founded on gravels or gravelly sands, regardless of their penetration resistance (SPT) values, generally showed little or no displacements, indicating that such materials have higher resistance to liquefaction during large earthquakes.

3.3 The 1987 Edgecumbe earthquake

On 2 March 1987 a M 6.3 shallow (~8 km) earthquake occurred underneath the Rangitaiki Plains close to the township of Edgecumbe in the Bay of Plenty. The earthquake caused widespread damage throughout the Bay of Plenty area, with much of the damage due to permanent inelastic ground displacements caused by liquefaction of the saturated alluvial deposits of the Rangitaiki Plains. During the earthquake, eyewitness accounts in the epicentral area around Edgecumbe describe the ground as rolling, with surface waves up to 0.75 m in height, so that people were unable to stand. The duration of strong shaking (>0.05 g) for the earthquake was 8-9 seconds, recorded at Matahina Dam 11 km from the epicentre.

The area most significantly affected by the earthquake is a low-lying, swampy flood-plain adjacent to the Tarawera, Rangitaiki and Whakatane rivers. The flood plain has been formed during the Holocene (last 10 000 years) by the deposition of large amounts of volcanic-derived pumice alluvium (gravels, sands and silts), which have buried pre-existing river channels, and incorporated varying amounts of peaty material. Ground damage caused by the earthquake included ground surface fault ruptures up to 2.5 m in height and 7 km long, widespread but mainly minor failures of surficial materials on steep slopes, incipient cracking on ridges, slopes and cuttings, and widespread liquefaction effects and localised ground settlements across the plains (Franks et al., 1989).

Widespread subsurface liquefaction was indicated by the ejection of sand, silt and water from linear cracks and holes in the ground forming sand boils (Figure 2), and more significant ground damage was indicated by well developed fissures, sand ejections, and lateral spreading failures along the banks of river and stream channels, particularly along the banks of the Whakatane and Rangitaiki rivers.

Most of the soils underlying the Rangitaiki Plains are saturated, with the water table ranging from close to the ground surface in the coastal strip, to about 3 m below ground surface further inland (at Te Teko). The area of sand boils was restricted mainly to the immediate epicentral area of the Rangitaiki Plains. Within this area probing indicates that the loose saturated alluvial materials with low penetration resistance are some 10 to 20 m thick, and overlie dense sands.

The main ground damage that resulted from the liquefaction processes was that caused by lateral spreading. Such damage included open tension cracks, sand ejections, and settlements of bridge approach fills and flood-control and roading embankments close to the main rivers (Photos 1-4). There was also widespread minor slumping of stop banks.

Very strong shaking and resulting ground damage in the epicentral area caused widespread damage to buildings, roads, railways and underground services, especially in and around Edgecumbe. Such damage included severely bent and distorted railway lines, compressional ripples and tensional cracking in the asphalt surfacing of roads, compressional ruptures of concrete paving and kerbs, and severe compressional damage to underground reticulation services (Photos 5-14). It is considered that the soft, loosely- compacted sediments around Edgecumbe have experienced inelastic deformations during the main earthquake, producing zones of permanent ground deformation and damage (Franks et al., 1989). It was apparent that modern, well constructed dwellings built on good foundations such as a monolithic, well reinforced concrete slab, suffered virtually no structural damage in the event, and were not influenced by the ground damage.

Localised elongate ground depressions or areas of subsidence, up to 0.3 m in depth and up to 100 m wide, were common across the plains. These depressions appear to coincide with near-surface peaty soils, and are considered to be the result of differential settlements associated with earthquake induced compaction of the peat materials.



Photo 1. Settlement and spreading of the bridge abutment fill at Whakatane Bridge. With a duration of strong shaking of 8–9 seconds, the ground damage due to liquefaction effects was relatively slight. For a larger magnitude earthquake the duration of strong shaking would be longer (20–30 s for M 7.5, about 50 s for M 8) and the ground damage far more severe.



Photo 2. Fissures associated with lateral spreading in the river bank at Whakatane.



Photo 3. Aerial view of sand boils near Powell Rd.



Photo 4. Lateral spreading and linear cracks on stop bank close to SH 2 at Greig Road.



Photo 5. Chimney collapse, Kowhai Avenue, Edgecumbe.



Photo 6. House thrown off its piles, south side of High Street Edgecumbe, note compression features in path.



Photo 7. Excavation of high pressure gas pipeline through Edgecumbe fault, note sagging and distortion of pipe.



Photo 8. Railway lines damage at Edgecumbe station.



Photo 9. Railway sleepers 'vibrated' out of stone ballast on embankment opposite Bay Milk Company, Edgumbe.



Photo 10. 67 ton diesel locomotive toppled in Edgumbe.



Photo 11. Branch railway line into Bay Milk Company premises, Edgecumbe.



Photo 12. Compressional features in rail lines east of Edgecumbe.



Photo 13. Compression ripple in asphalt surface of Rata Avenue, Edgecumbe.



Photo 14. Compression flow in asphalt in Rata Avenue, Edgecumbe looking west, note north trending movement of pathway concrete blocks.

4 HISTORICAL EARTHQUAKES AND LIQUEFACTION EFFECTS IN WELLINGTON

4.1 Background

Some 30 definite cases of liquefaction have been reported in New Zealand since 1840 (Fairless and Berrill, 1984). Of the reported liquefaction occurrences, none have been associated with earthquake shaking of less than MM VIII, or possibly MM VII. The most widespread liquefaction effects have been associated with the 1848 Marlborough, 1855 Wairarapa, 1931 Napier, 1942 Masterton, 1968 Inangahua, and 1987 Edgecumbe earthquakes. Surprisingly, Fairless and Berrill (1984) found no reports of liquefaction during the 1929, M 7.8 Murchison earthquake. However, Henderson (1937) reports several cases of what is clearly lateral spreading (areas of open fissures many chains wide aligned parallel to stream channels) in the Murchison area, and also at Greymouth and Westport.

Fairless and Berrill (1984) list the following phenomena that may be used to indicate liquefaction:

- water ejections and sand boils
- landslides on moderate slopes
- foundation failures
- floatation of light structures

Of the above listed phenomena, sand boils and water ejections provide the most common and unambiguous historical evidence of liquefaction in New Zealand, and particularly in the Wellington region. Fairless and Berrill (1984) consider that it is likely that other types of ground damage during past earthquakes, such as settlement and spreading of embankments, may have been caused by liquefaction, but have not been recognised as such. In historical records, more emphasis seems to have been placed on recording the more damaging effects of earthquakes on buildings, although the unusual and often spectacular sand boil or 'volcano' features, so common in the epicentral areas of large earthquakes, were usually noticed and reported on. On alluvial terraces, such features are often the only manifestation of liquefaction (due to increased pore-water pressures within thin layers of fine silty sands at depth) on gravel surfaces that are otherwise undamaged.

Earthquakes since 1840 that have resulted in a Modified Mercalli (MM) felt intensity of MM VII or greater in the Wellington Region, and reported liquefaction effects are listed in Table 2. MM VII was selected as the lower level of earthquake shaking for this study, mainly because it is the intensity at which significant ground damage and landslides are formed, and is the minimum intensity for the appearance of minor liquefaction effects (cracking, sand and water ejections). However, sand boils, water fountains, lateral spreads and flows are generally become apparent at MM VIII or greater (see also Grant Taylor et al., 1974; and the MM Intensity Scale, Appendix 2).

Table 2 shows that liquefaction phenomena have been reported in the Wellington Region during only four historical earthquakes. In most cases the reported ground damage effects have been relatively minor, mainly cracking, sand boils and water ejections, but there are reports of major ground damage (sand boils, lateral spreads and flow failures) after the great 1855 Wairarapa earthquake. However, as discussed above, it is quite possible that other ground failures and landslides in the Wellington area may have been induced by liquefaction, but have not been recognised or reported as such, especially during the early years of colonisation. In studies proposed for Stage 2, reported instances of settlements and collapses in low-lying areas during the historical earthquakes listed in Table 2 will be assessed to determine if liquefaction was the cause.

YEAR	DATE	MM INTENSITY (WGTN)	MAGNITUDE (M)	EPICENTRAL DISTANCE (approx km)	EPICENTRE LOCATION	LIQUEFACTION EFFECTS
1848	16 Oct	VIII	7.1	65	Marlborough	Sand boils (Waikanae); ground cracking and water ejections (Ohau); beach cracking in Wellington
1848 (A/S)	17 Oct 19 Oct 24 Oct	VII VI-VII VII	c. 6.0 " "	65 " "	Marlborough	no reports
1855	23 Jan	X	c. 8.2	30	Wairarapa	large sand boils, fissuring of ground, and subsidence (Lower Hutt valley); gas, sand and water ejections (Ohau); mud and sand ejections (Wellington City); extensive ground damage, fissures, pressure ridges (Wairarapa).
1855 (A/S)	24 Jan 25 Jan	VII VII-VIII	- -	- -	Wairarapa	no reports
1904	9 Aug	VIII	7.5	190	Cape Turnagain	some (unspecified) ground damage at Castlepoint
1914	8 Feb	VII	6.0	30	Cook Strait	no reports
1934	5 Mar	VI-VII	7.5	110	Pahiatua	no reports
1942	24 June	VII	7.0	100	Masterton	sand boils from fissures (Gladstone area); sand and water ejection (Opiki); settlement of wharf fills (Lambton Harbour); settlement of bridge approaches (Petone); sand boil, Aeotea Quay (Wellington).
	2 Aug	VI	7.0	100	Wairarapa	no reports
NOTES: 1. Liquefaction effects after Fairless and Berrill, 1984; earthquake data compiled by Dellow, 1988. 2. A/S denotes aftershocks associated with the 1848 and 1855 earthquakes. 3. Fairless and Berrill (1984) report some 30 definite cases of liquefaction in New Zealand since 1840. In the Wellington area the most widespread effects were associated with the 1848, 1855, and 1942 earthquakes. In New Zealand, historical precedent evidence indicates that at least MM VIII shaking is required for liquefaction, but minor effects may be apparent at MM VII.						

Table 2. Earthquakes of intensity MM VII or greater felt in the Wellington area since 1840, and reported liquefaction effects (after Dellow, 1988, and Fairless and Berrill, 1984) [1]

From studies of historical records it is clear that liquefaction has occurred in New Zealand on many occasions, but most of the damage has been relatively minor. Apart from the 1855 Wairarapa and 1931 Napier earthquakes, there are few records of major ground damage or failures having occurred. However, in Wellington City, Hutt Valley, and Porirua urban areas there is now the potential for major liquefaction-induced damage to occur, especially during a large earthquake on one of the several major active faults in the Wellington region. Such an earthquake (the Scenario 2 Earthquake, as defined in previous reports to WRC, Appendix 1), is likely to cause liquefaction and ground damage effects similar to those caused by the 1855 Wairarapa earthquake.

The probability of major damage to buildings and services in the Wellington Region due to liquefaction effects during a large local earthquake clearly much greater today than it was in 1855. Given the intensive urban development that has occurred in the last 137 years, during a large local earthquake, damage caused by earthquake shaking and liquefaction would be much greater today than it was in 1855, mainly because there are many more large buildings, structures and services that are likely to suffer damage, as highlighted in the CAE Report (1991). Although the accuracy of this statement has yet to be tested, it is prudent to assess what the nature and extent of these effects might be, hence it is these issues that will be addressed in detail in Stage 2 of this study.

It is apparent from Table 2 that the Wellington area has experienced a number of large distant earthquakes of the Scenario 1 type since 1840 (as defined in previous reports to WRC, Appendix 1). Those of most relevance are the two 1942 Wairarapa earthquakes (both $M \approx 7$), for which shaking of about MM VI to VII was felt in the Wellington area.

The June 1942 earthquake was the most recent record of MM VII intensity shaking in Wellington City, but it produced few documented liquefaction effects, and apart from one isolated observation of a sand boil on Aeotea Quay in Wellington City (pers comm. R D Northey), and settlements of wharf fills (Lambton Harbour) and bridge approach fills (Petone). More pronounced effects were recorded close to the epicentre in the Wairarapa. However, as both 1942 earthquakes were magnitude 7 events at a distance of 100 km, they could have been expected to have produce significant shaking and liquefaction effects in the Wellington area, with MM intensities on soft soils significantly higher (possibly 3 to 4 MM units) than on bedrock (Van Dissen et al., 1992 - reference 20, Appendix 20). As illustrated by Table 2, this apparently did not occur, and precedent evidence tends to suggest that the Scenario 1 type event is unlikely to cause significant liquefaction or ground failure damage in the Wellington area. Further evaluation of historical data is required to confirm this conclusion.

It is therefore recommended that in Stage 2 of this work a detailed study be made of the historical records for the two 1942 earthquakes, and also other distant large, shallow earthquakes that have affected Wellington since 1840 so that liquefaction and ground damage effects of a Scenario 1 type earthquake can be assessed. This would involve the compilation, review and evaluation of IGNS and other MM felt intensity data, building damage, and reports of ground damage and slope instability.

Brief summaries of the effects of large historical earthquakes in the Wellington region are presented in Sections 4.2 to 4.5.

4.2 The June 1942 Masterton earthquake

According to Dellow (1988) the shallow (<25 km depth) 24 June 1942 Masterton earthquake caused slope failures and ground damage in the southern North Island inside the MM VI isoseismal. The severest slope failures and ground damage effects were reported in the epicentral area (close to the area of presumed fault rupture) where intensity MM VIII was reported (Figures 3 and 4).

Near the Manawatu River at Opiki in the Horowhenua area, ground damage with sand and water ejection associated with liquefaction occurred, and subsidence of the railway tracks occurred between Paraparaumu and Te Horo. There were three sites of ground damage in Wellington. Reclaimed land along the waterfront, particularly the northern wharf and shed system, subsided and the approaches to two bridges in the Lower Hutt -Petone area subsided slightly (the "Pipe Bridge" settled about 50-150 mm, and there was minor settlement of the Waiwetu bridge in Seaview Road), possibly due to liquefaction. As discussed in Section 4.1, there was also one isolated observation of a sand boil on Aeotea Quay (pers comm. R D Northey).

A slope failure was reported just south of Plimmerton, which covered both railway tracks, with more slope failures between Plimmerton and Paekakariki. Small rock falls were reported along the Western Hutt Road and over the Rimutaka Hill, with fissures on the road near the summit. Small slope failures were also reported on the Mangaroa Hill. On the Wairarapa railway line a small slope failure was reported on the Rimutaka incline, with some "fairly large slips" between Mangamahoe and Mangatainoka. Several railway bridge approaches were reported out.

Many roads in the Wairarapa were damaged, including the Masterton - Longbush and Masterton - Tinui roads. The Mangareia and Tanglewood roads to the west of the fault rupture were found blocked by slips close to the Tauweru - Gladstone line. Road bridges were reported out at Longbush, Gladstone, Kahutara, Eketahuna, Tauweru and Tuhitarata. Slope failures were reported from Longbush, Masterton, Eketahuna, Gladstone and Makuri Gorge. At Tauweru, in the area of fault rupture, Ongley (1943) described scarring of the hills by slips, with fissures and liquefaction phenomena also present. In the south Ongley also reports small slope failures and fissures at the Narrows.

4.3 The August 1942 Masterton earthquake

The August 2, 1942 Masterton earthquake is regarded as the same magnitude ($M \approx 7$) but significantly deeper (55 km) than the earlier June event. There is no recorded surface rupture and the epicentral isoseismal is lower (MM VII rather than MM VIII) than for the June earthquake. Slope failures and ground damage reported for the event are significantly less than for the earthquake only 6 weeks earlier. Minor ground damage and small slope failures are reported within the MM VI isoseismal (Figures 5 and 6).

Slope failures and ground damage are reported from Horowhenua, Wellington and the Wairarapa. In the Horowhenua slope failures and ground damage occurred at Otaki where the roads cracked and there were slope failures on Rahui Road. In Wellington there was slight subsidence of reclaimed harbour land and of the approaches to the Kelburn Viaduct.

In the Wairarapa ground damage was reported on the roads near Eketahuna and on the railway line south of Masterton and Eketahuna. Slope failures were reported on the railway line between Masterton and Eketahuna with another on the Rimutaka Incline.

4.4 The Marlborough 1848 earthquake

This shallow (<10 km), magnitude 7.1 earthquake in the lower Wairau Valley was located some 65 km from Wellington and caused MM VIII intensity shaking in Wellington, Hutt Valley and Porirua ((Figure 7), at a time before there were any significant man-made modifications to the area. Minor ground damage liquefaction effects that were reported from Waikanae and Wellington consisted of small fissures and sand boils (Figure 8). As the population at that time was sparse, some areas of ground damage may have escaped notice, however it is significant that major ground damage is not reported for the event in populated parts of the Wellington area.

4.5 The 1855 Wairarapa Earthquake

The very large earthquake that occurred in the Wairarapa on 23 January 1855, had a focal depth <10 km (judging from the extensive rupture of the West Wairarapa Fault). This event has been assigned a magnitude of ≈ 8 , and is reported to have caused MM X intensity shaking throughout the greater Wellington area (Figure 9). It is the largest earthquake to have affected New Zealand in historic times.

The earthquake caused ground damage at Wanganui, the Wairarapa, Manawatu, Wellington and in Marlborough (Figure 10). When this earthquake occurred in 1855 a further seven years of settlement and expansion had taken place since the 1848 earthquake, hence reports of damage are more widespread. All slope failures and ground damage occurred within the area encompassed by the MM VIII isoseismal.

Inside the MM X isoseismal slope failures and ground damage are widespread and major features. In the areas of the MM VIII and IX isoseismals, the ground damage and slope failures are confined to more susceptible sites such as low-lying plains and steep slopes. Numerous cases of sand boils, fissuring and subsidence are described from low-lying areas of Wellington, Hutt Valley, Porirua, Pauatahanui and it is apparent that liquefaction and ground damage was widespread in these places.

Although the 1855 earthquake occurred some 30 km from Wellington, its effects in the Wellington area (apart from actual fault rupture) are expected to be very similar to the Scenario 2 earthquake - a ~M 7.5 earthquake causing rupture of the Wellington Fault from Cook Strait to a point some 70 km to the north. Hence it merits further detailed study to determine whether any additional information relevant to this study can be obtained.

5 POTENTIAL LIQUEFACTION AND GROUND DAMAGE

Previous reports completed for the WRC (Appendix 1) have outlined in considerable detail the geology, geotechnical properties of soil materials (mainly near-surface, alluvial and marine deposits, and depths to greywacke bedrock), along with past and expected future earthquake shaking levels in the main study areas in the Wellington Region. Information from these studies have important implications for emergency response planning and engineering in terms of specific hazards presented by earthquakes, and possible mitigation measures.

For this liquefaction and ground damage study, much useful and valuable information is presented in the earlier reports. Of particular relevance to this study are the results of the cone penetrometer (CPT), seismic cone penetrometer (SCPT), standard penetration tests (SPT's), drillhole data, strong motion records, seismic wave amplification studies and the geological mapping. A synthesis involving evaluation and assessment of all this data for each of the five study areas is presented in the ground shaking hazard assessment reports (Appendix 1, References 1, 18-21).

In each of the ground shaking hazard assessment report maps are presented dividing the area into five ground shaking hazard zones defined as follows:-

- Zone 1 **Bedrock:** Usually moderately to very strong "greywacke". Bedrock may be overlain by colluvium, deep weathered gravel, loess, or well engineering fill <10 m thick.
- Zone 2 **Stiff Sediment:** Compact to very compact gravels and other finer grained sediments up to 200 m thick. May contain some thin (<5 m) peat layers. Moderate-high SPT values.
- Zone 3-4 **Transition zone of soft-medium, dense-loose, sediments** up to 20 m thick overlying thicker, more dense sediments that may be a few 100 m thick.
- Zone 5 **Soft or Loose Sediment:** (fine sand, silt, clay and peat) up to 30 m thick with low shear wave velocity (<200 m/s). May overly bedrock or thick, denser, sediments.

In geological terms the upper 10-20 m of Zones 3-5 are characterised by very young, recent sediments that have accumulated over the last 10,000 years.

It is clear, from the information presented in the ground shaking hazard assessment reports and some of the other reports (Appendix 1), that in general terms materials in zones 3-5 are susceptible to cyclic strain softening and/or liquefaction. These areas have loose or soft sediments (SPT N values <20, V_s <200 m/s), with low relative density and a high water table close to ground surface. However their relatively good performance during historic earthquakes, such as the 1942 Masterton events, indicates that they are not highly susceptible to "liquefaction" from large distant earthquakes. Rather the evidence from the historical earthquake records suggests that it will take a large, nearby earthquake, such as the 1855 Wairarapa event (or rupture of the Wellington, Ohariu, Wairau extension or Subduction Zone faults) to cause large scale liquefaction and ground damage effects in Wellington, Hutt Valley, Porirua and Kapiti Coast. The June 1942 Masterton earthquake did cause significant liquefaction and ground damage in the epicentral area close to Masterton.

5.1 Future work

The geotechnical information required for a general "liquefaction" hazard assessment appears to be mostly available for Wellington City, Hutt Valley and Wainuiomata, Porirua and Paremata, and the Kapiti Coast, but is sparse in the Wairarapa area. It is apparent that very detailed "site" investigations, that are beyond the scope of this assessment, would be required to ascertain whether a particular area would undergo "complete" liquefaction or cyclic strain softening, and at what level of excitation.

In order to complete a general liquefaction hazard assessment, the relevant sub-surface geotechnical information available in the study areas, with particular emphasis on Zones 3-5, should be evaluated and plotted. (For example numerous SPT values are available for Wellington City which require evaluation and plotting). Once this is done, it may be possible to identify areas within shaking hazard Zones 3-5 that are more susceptible to liquefaction and ground damage than others. The results of this work can then be compared with the reports of ground damage and liquefaction from the great 1855 Wairarapa earthquake, which caused much ground damage in the region, and other less intensely felt earthquakes which have caused some instances of minor ground damage.

In order to properly investigate and document reports of ground damage during the historic earthquakes, particularly for the 1855 Wairarapa and the two 1942 Masterton earthquakes, it will be necessary to access files held at the IGNS Seismological Observatory, at Victoria University, Works Consultancy Services (Head Office), and possibly the National Archives and Turnbull Library. These files contain felt intensity reports, historical records and reports from journals and letters, press reports, and reports of structural and other damage.

Other data which has been obtained in the Wellington City area is in the form SPT tests in site investigation drillholes. IGNS records indicate that some 300 drillholes in Wellington City include SPT test records. This large and potentially valuable data set requires evaluation and plotting.

6 EXISTING DATA SOURCES

As previously indicated, much valuable information relevant to this liquefaction hazard study is available from existing reports. These include:-

- reports prepared and completed for the WRC (Appendix 1)
- a seismic hazard assessment of Lower Hutt City, by Ian R Brown Associates Ltd
- a geotechnical and structural appraisal of existing flood protection works, compiled by Tonkin and Taylor Ltd, as part of the Hutt River flood protection scheme
- an assessment of liquefaction potential in the Wellington railway yards during earthquake shaking, by Dellow and Perrin (DSIR Contract Report, 1991), for NZ Rail
- site investigation reports relating to the Wellington Urban Motorway extension, Terrace Tunnel to Mt Victoria, and geotechnical and geological assessments by WORKS Central Labs and WORKS Consultancy Services
- an evaluation of sediment properties in the Lower Hutt and Porirua areas by means of cone and seismic cone penetration tests - paper (in preparation) by W R Stephenson and P Barker, for the Bulletin of the NZ National Society for Earthquake Engineering
- investigation drillhole records for the Wellington Motorway Thorndon Overbridge, held by Works Consultancy Services
- drilling records of building sites in the Wellington, Hutt Valley and Porirua areas

The best sub-surface geological and geotechnical records are available from those areas that have the most intensive commercial development. Hence the most refined liquefaction and ground damage assessments can generally be made for these areas of greatest risk.

6.1 Data available for the Lower Hutt area

A summary of subsurface geological and geotechnical data for the Lower Hutt-Petone area is presented in the Seismic hazard assessment of Lower Hutt City by Ian R Brown Associates Ltd (1991). The report utilizes the SPT data from 102 drillholes in the Lower Hutt valley, and makes a generalised assessment of liquefaction potential on the basis of this and other geological information. Much of Lower Hutt, Petone, and Wainuiomata areas are underlain by loose or soft recent sediments which have low SPT N values (mainly <20, and many <10), and have a shallow groundwater table. Hence there is considerable potential for liquefaction and related ground damage affects during a large local earthquake. Such effects were described during the 1855 Wairarapa earthquake. However, the historical evidence (e.g., the 1942 Masterton earthquake) indicates virtually no significant ground damage or liquefaction for a large distant earthquake causing shaking intensity of MM VII - VIII.

The report by Brown (1991) does not contain the data that may now be available from the appraisal of existing Hutt River flood protection works (Tonkin and Taylor report), or data from recent site investigations. Furthermore, Brown (1991) observed that sub-surface data, such as drill hole records from building sites, have not always been lodged with the Hutt City Council in the past, but it may be possible to obtain some of these from original sources, if they can be located.

6.2 Costs of accessing data

The costs of accessing data held by other organisations and colleagues has yet to be established by IGNS. However, it is felt that such costs, if any, are unlikely to be significant as it is likely that "*quid pro quo*" arrangements can be made to obtain access to relevant data.

7 REQUIRED ADDITIONAL DATA

As indicated earlier, the objective of this liquefaction hazard assessment is to broadly define places within the five study areas that have the greatest liquefaction and ground damage potential. As sedimentary deposits typically show marked lateral and vertical variation in extent, thickness and sediment type or grain size, detailed investigations will still be an essential requirement at individual sites to adequately determine the types of sediments present and their geotechnical properties. Once this is done the site-structure response during earthquakes can be assessed and a suitable structure can be designed to withstand the expected loadings and conditions. Such work has been properly carried out to a high standard during the planning and design of the new National Museum on the Wellington waterfront at Lambton Harbour.

There remains however, some quite extensive areas where the sub-surface materials and their liquefaction susceptibility are poorly defined and known. In these places it would be useful and practicable to obtain further sub-surface information from CPT probing and field inspections.

Proposed sites for further probing and other work in the 5 study areas is as follows:

Study Area 1 - Wellington City

Seatoun; Zone 5 materials	- 1 probe to 25 m
Kilbirnie-Miramar Zone 5 materials	- 2 probes to 25 m
Wellington Hospital area Zone 3-4 materials	- 1 probe to 25 m
Evaluation and plotting of existing drillhole and geotechnical data	

Study Area 2 - Hutt Valley

No additional probing work proposed at this stage
Evaluation and plotting of existing drillhole and geotechnical data

Study Area 3 - Porirua Basin

No additional probing work proposed at this stage
Evaluation and plotting of existing drillhole and geotechnical data
Search files

Study Area 4 - Kapiti Coast

Existing probing indicates that some 5-10 m of loose/soft sediments overly dense materials. It is suspected that the dense materials are wave compacted sands. Further probing is required to assess this.
Zone 3-4 materials - 2 probes to 15 m
Evaluation and plotting of existing drillhole and geotechnical data

Study Area 5 - Wairarapa

The first requirement for the Wairarapa Study Area is to carry out more detailed geological mapping of the Zone 2-4 areas to determine a finer zoning (if possible). Records of ground damage from historical earthquakes should be plotted onto these maps. Once this has been done an assessment can be made of probing investigations needed.

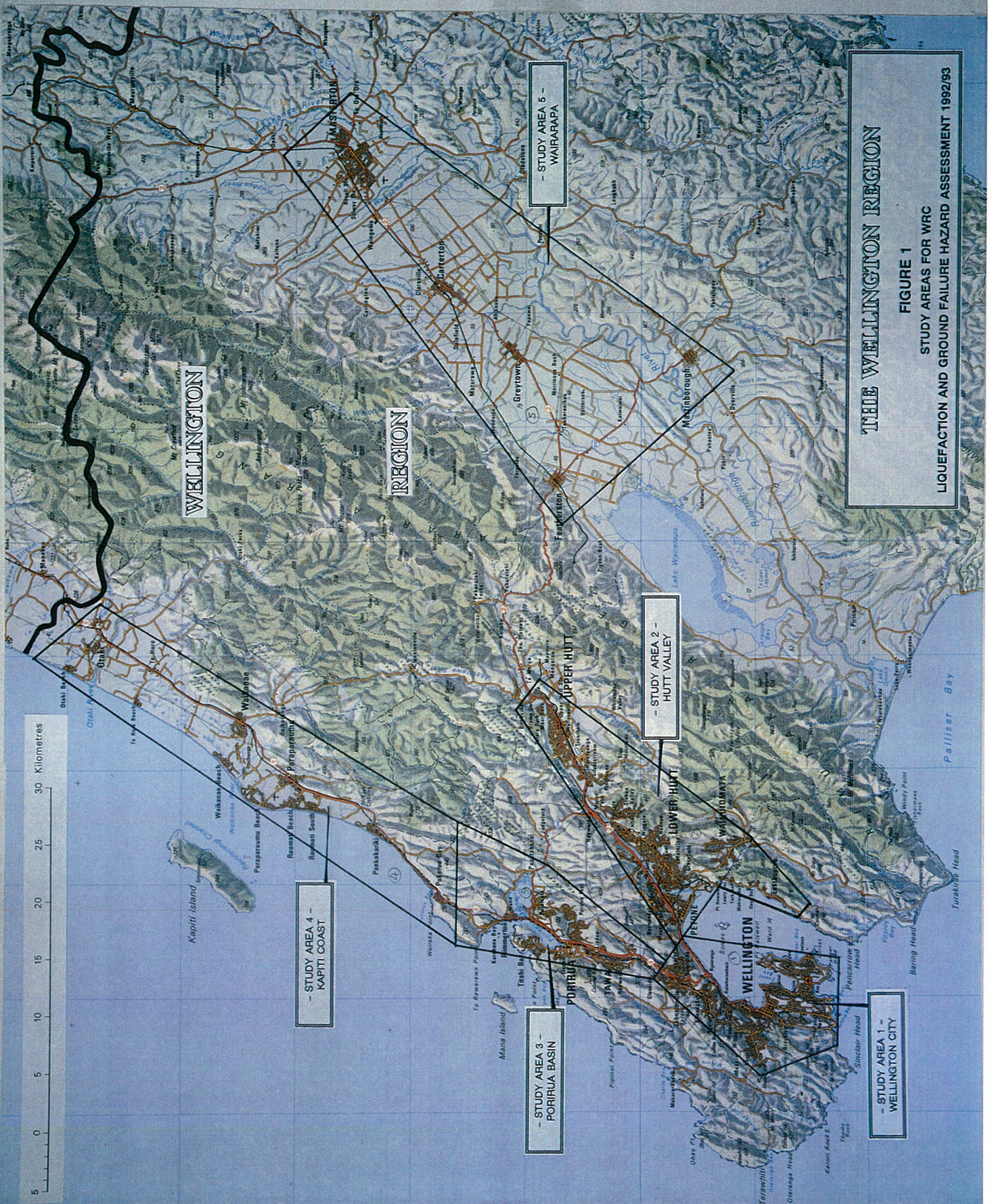
8 MAPPING SCALES

At this stage it appears that liquefaction susceptibility, potential and ground damage hazard can most appropriately be presented on maps at the *same scale* as the ground shaking hazard maps that have already been presented for each of the study areas. This will allow easy comparisons to be made between the sets of maps and the data they present and will allow the varying data to be presented as separate overlays upon a single base-map.

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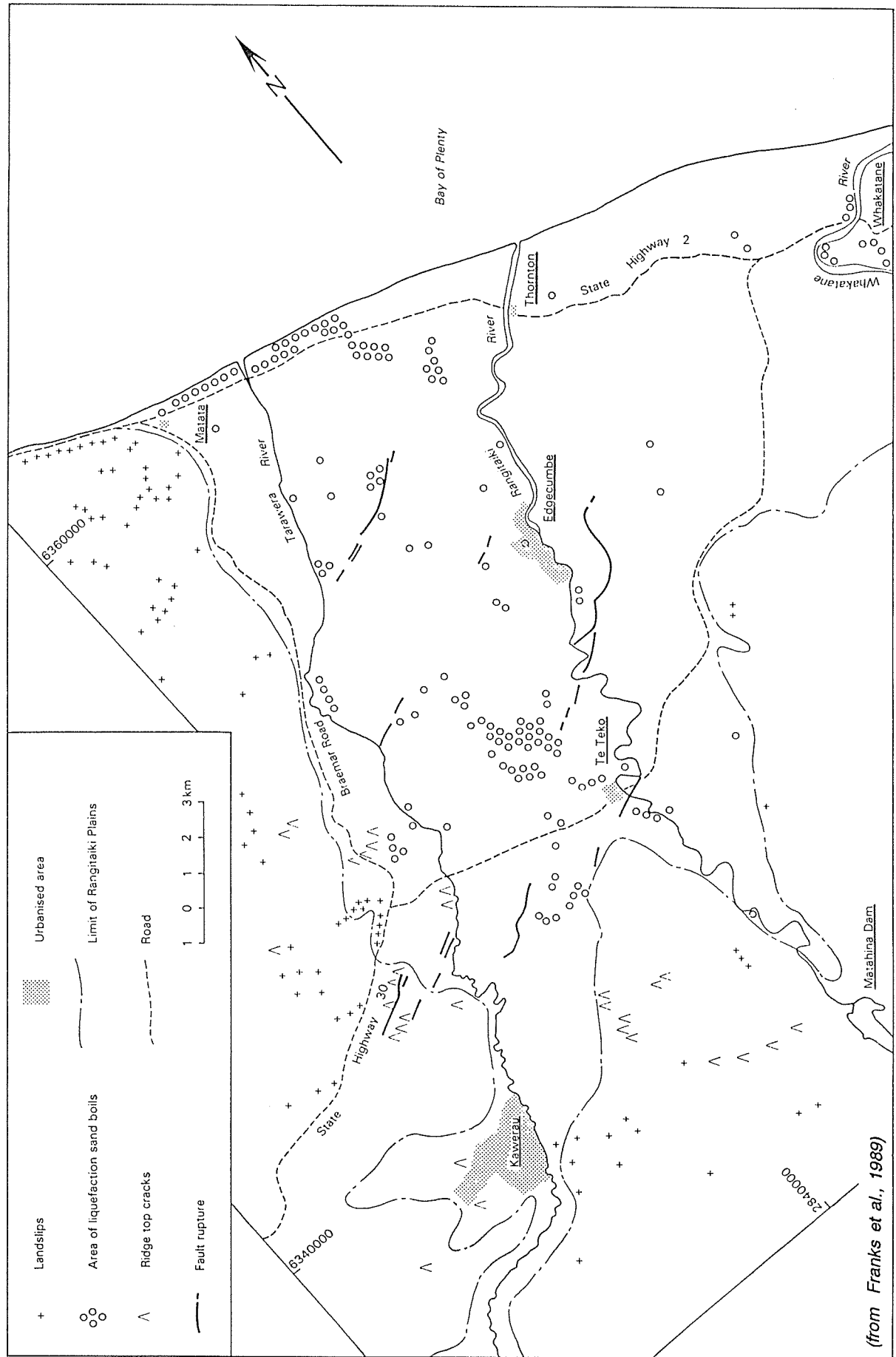
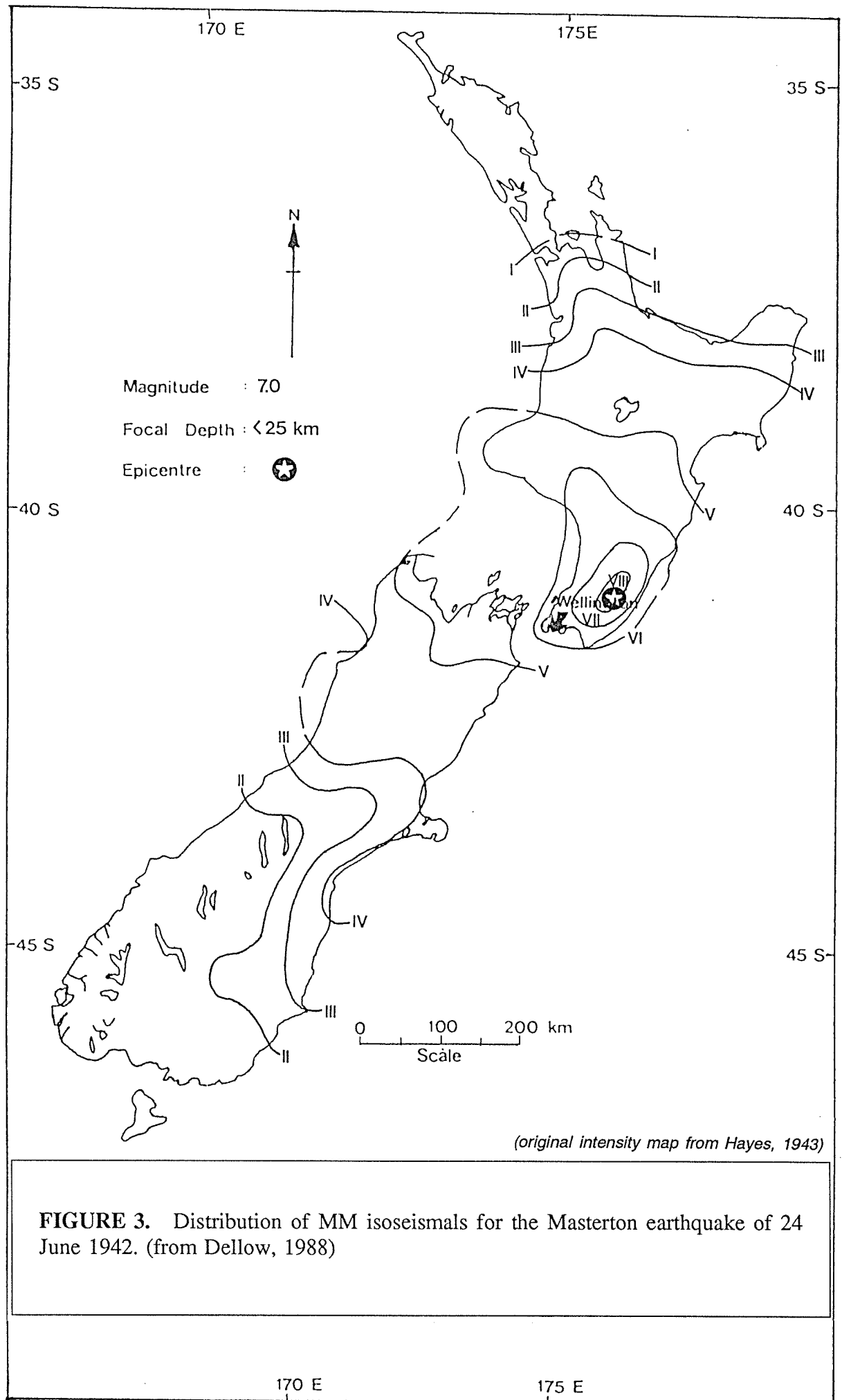


FIGURE 2: Distribution of ground damage across the Rangitaiki Plains after the 1987 Edgecumbe earthquake.



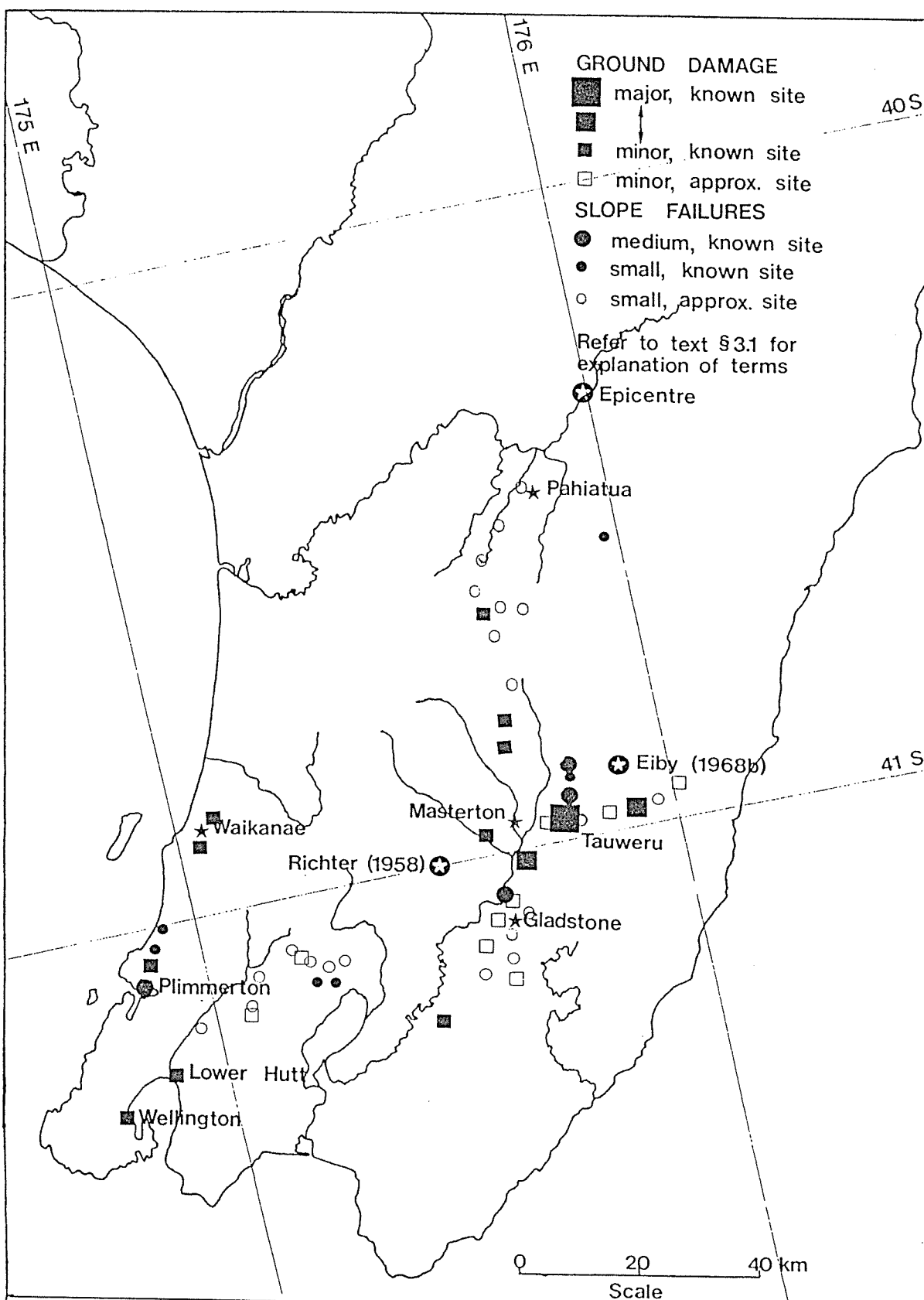
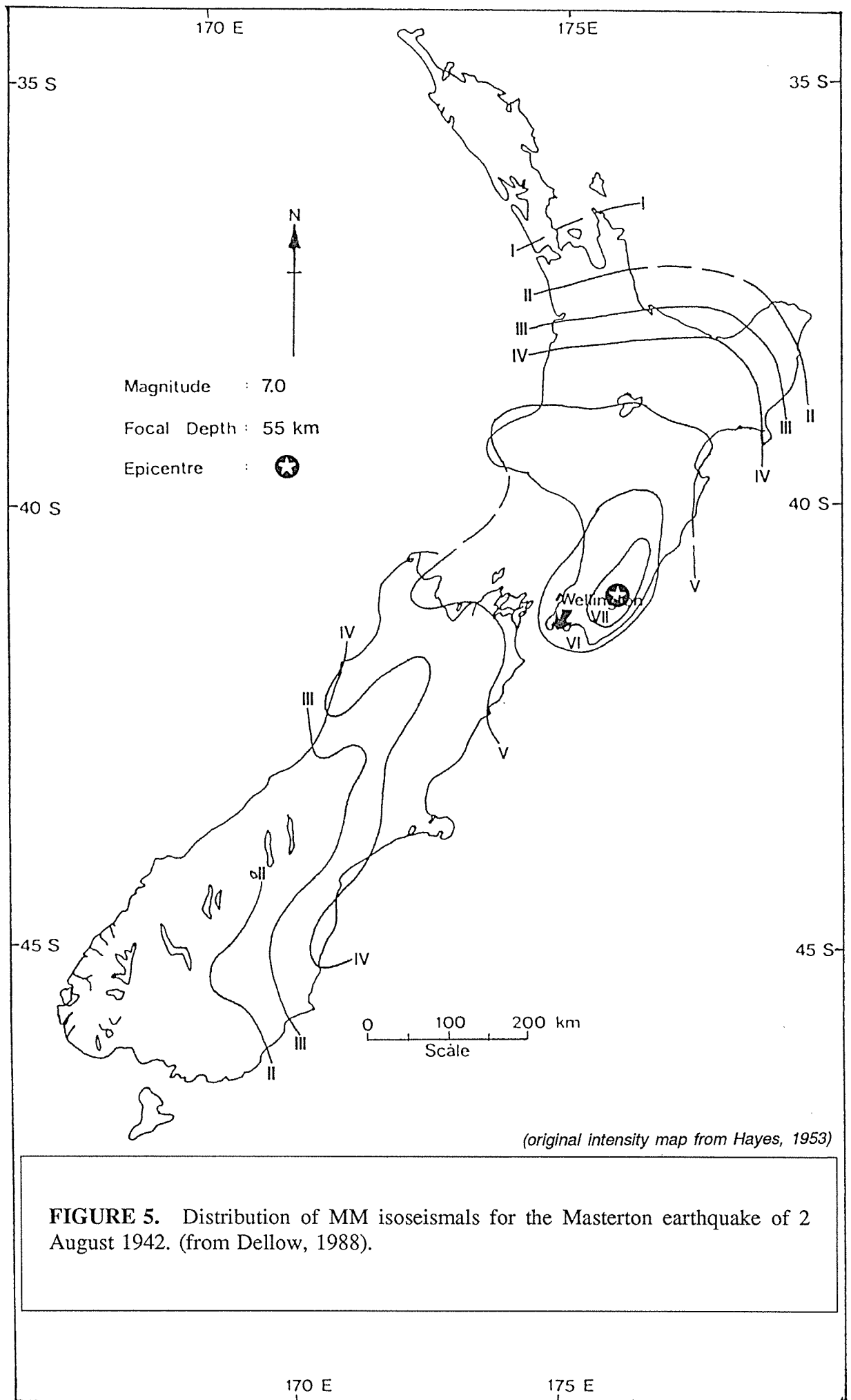


FIGURE 4. Slope failures and ground damage as reported from the Masterton earthquake of 24 June 1942. (from Dellow, 1988).



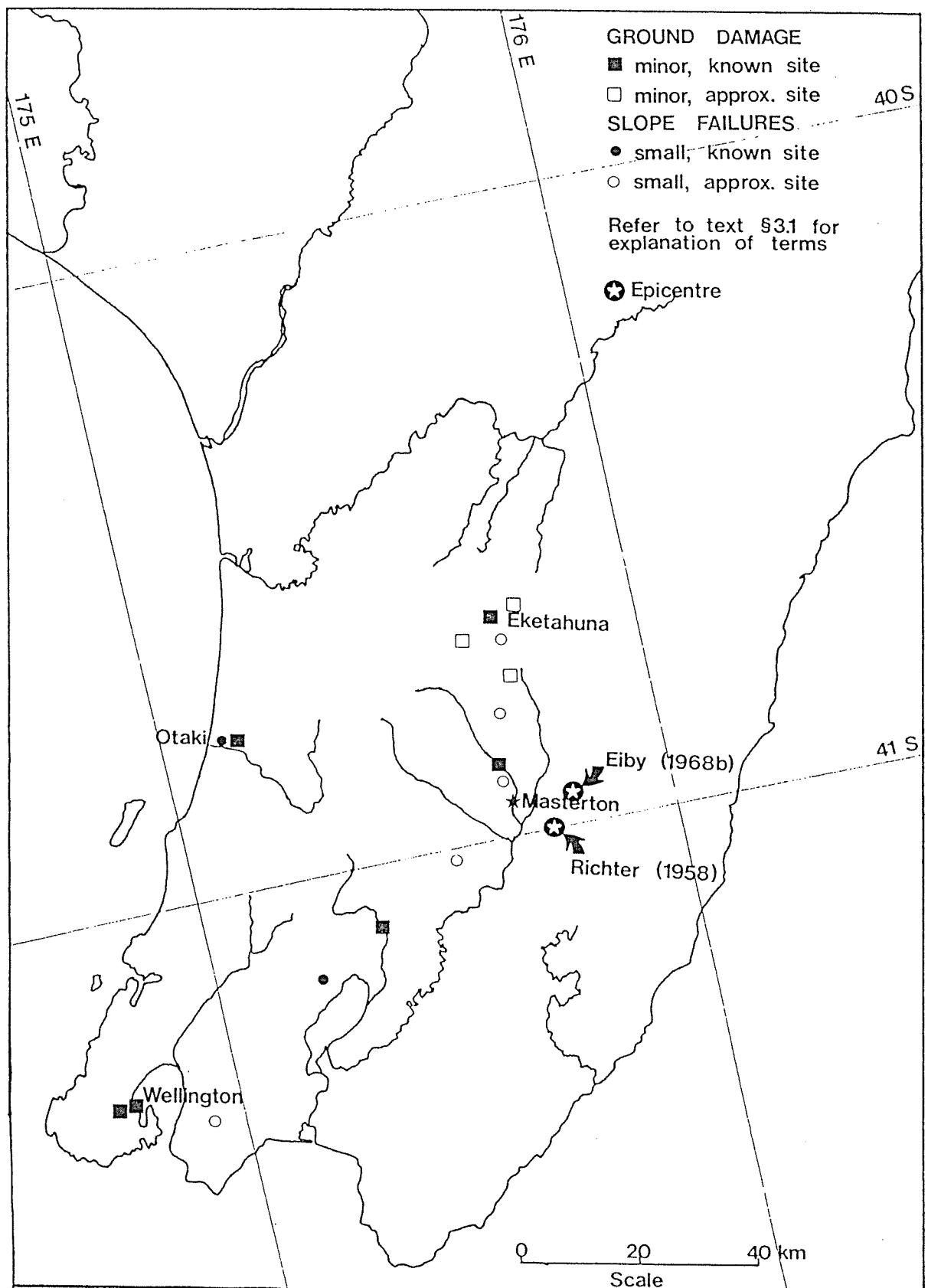
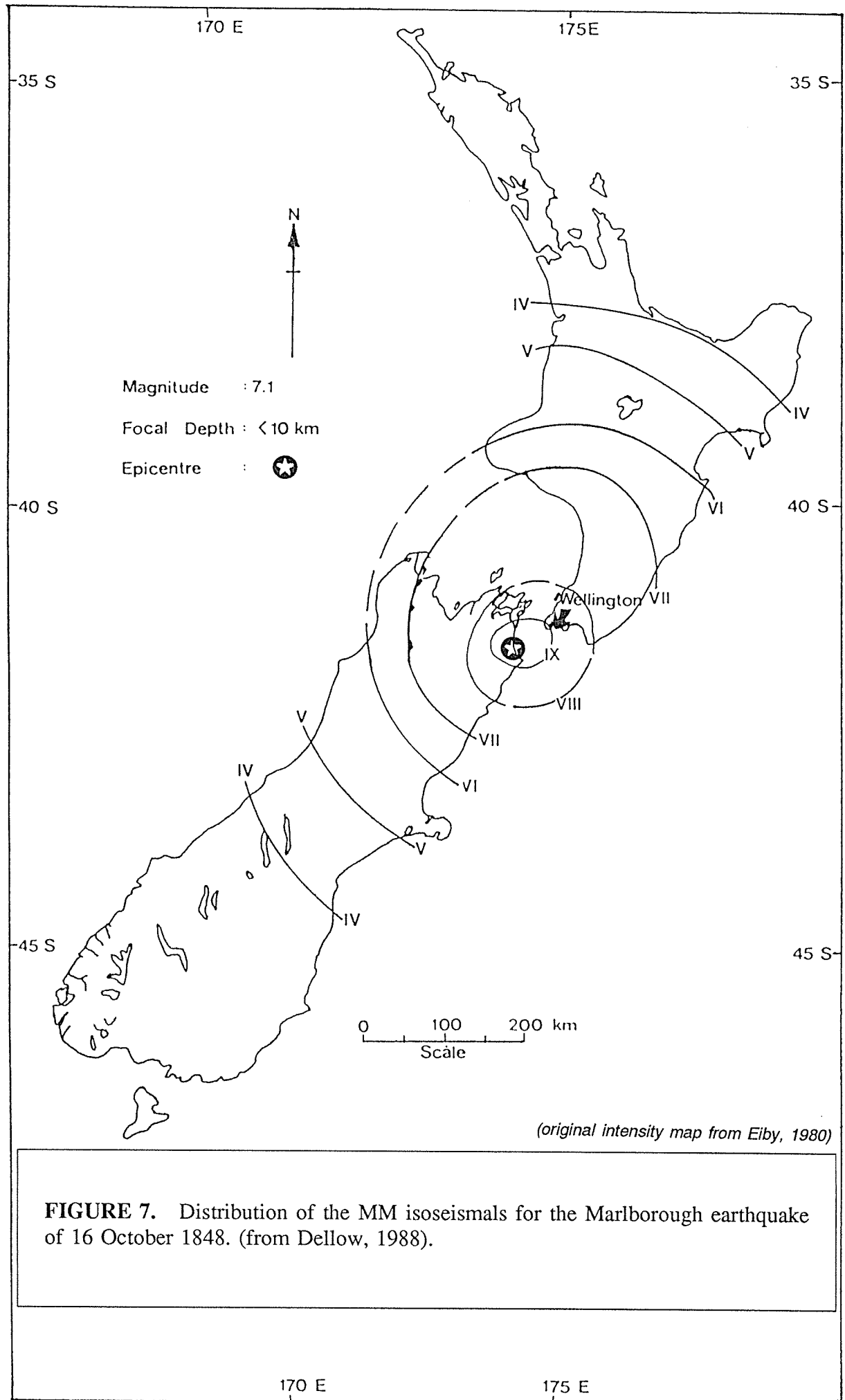


FIGURE 6. Slope failures and ground damage as reported from the Masterton earthquake of 2 August 1942. (from Dellow, 1988).



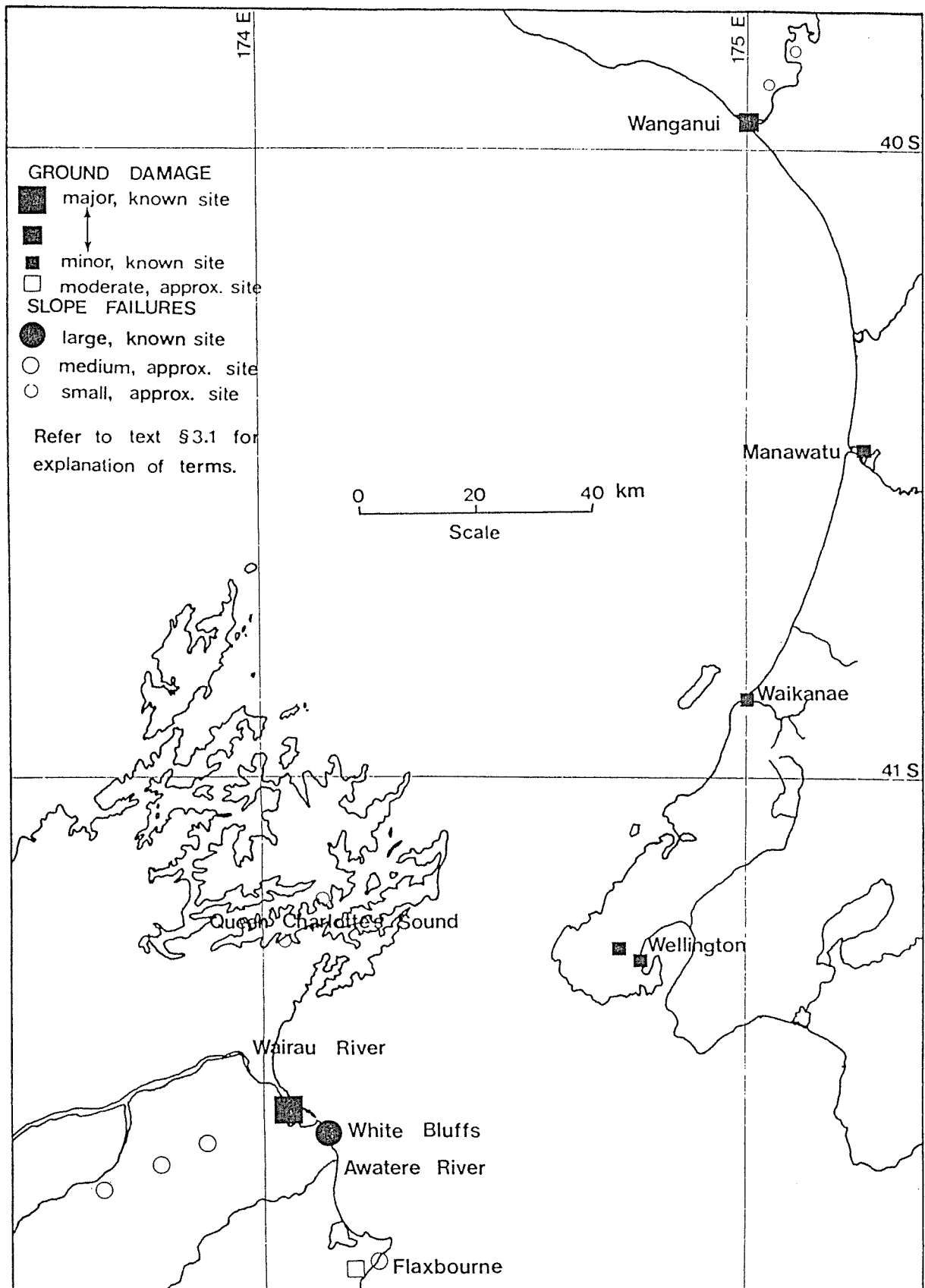
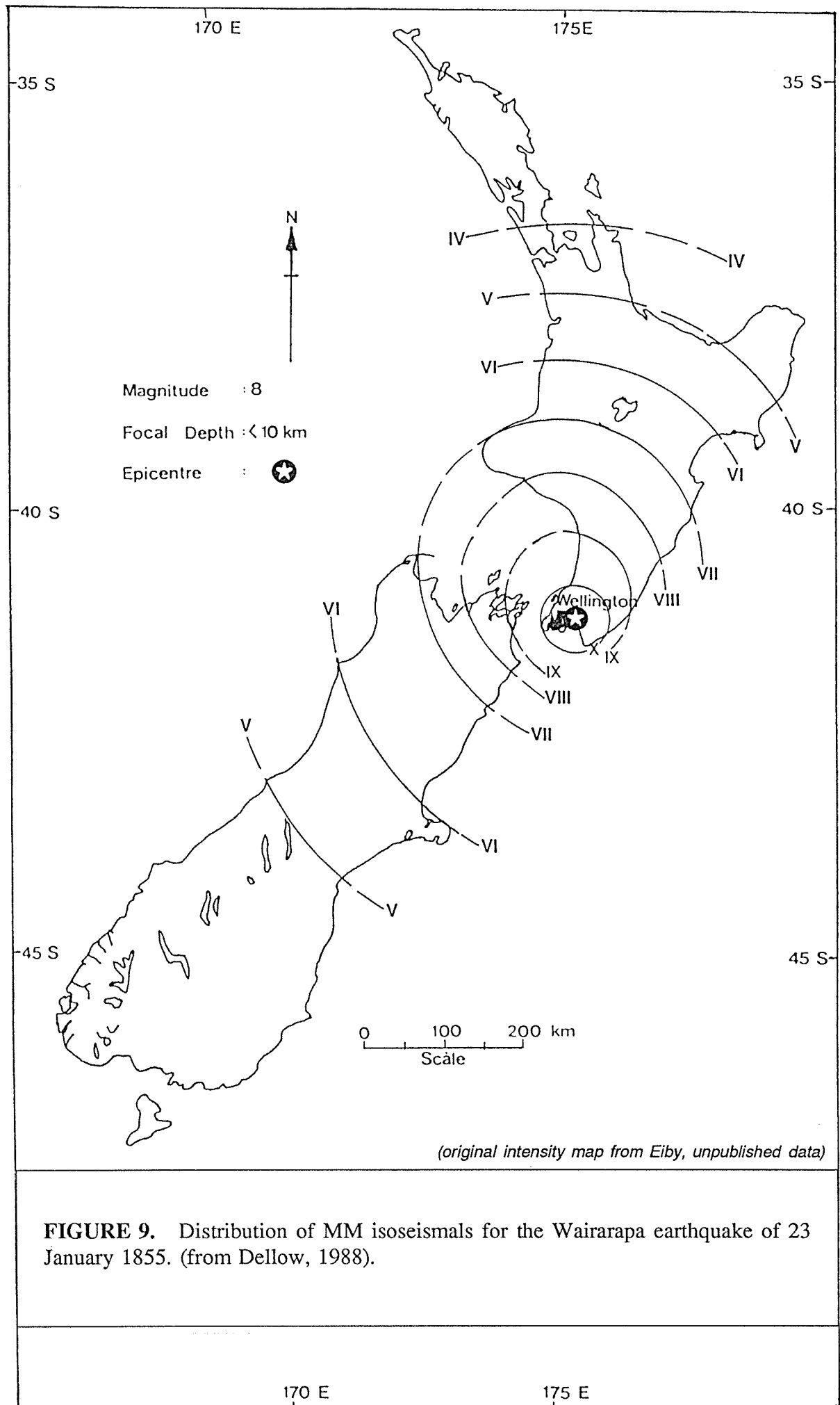


FIGURE 8. Slope failures and ground damage as reported from the Marlborough earthquake of 16 October 1848. (from Dellow, 1988).



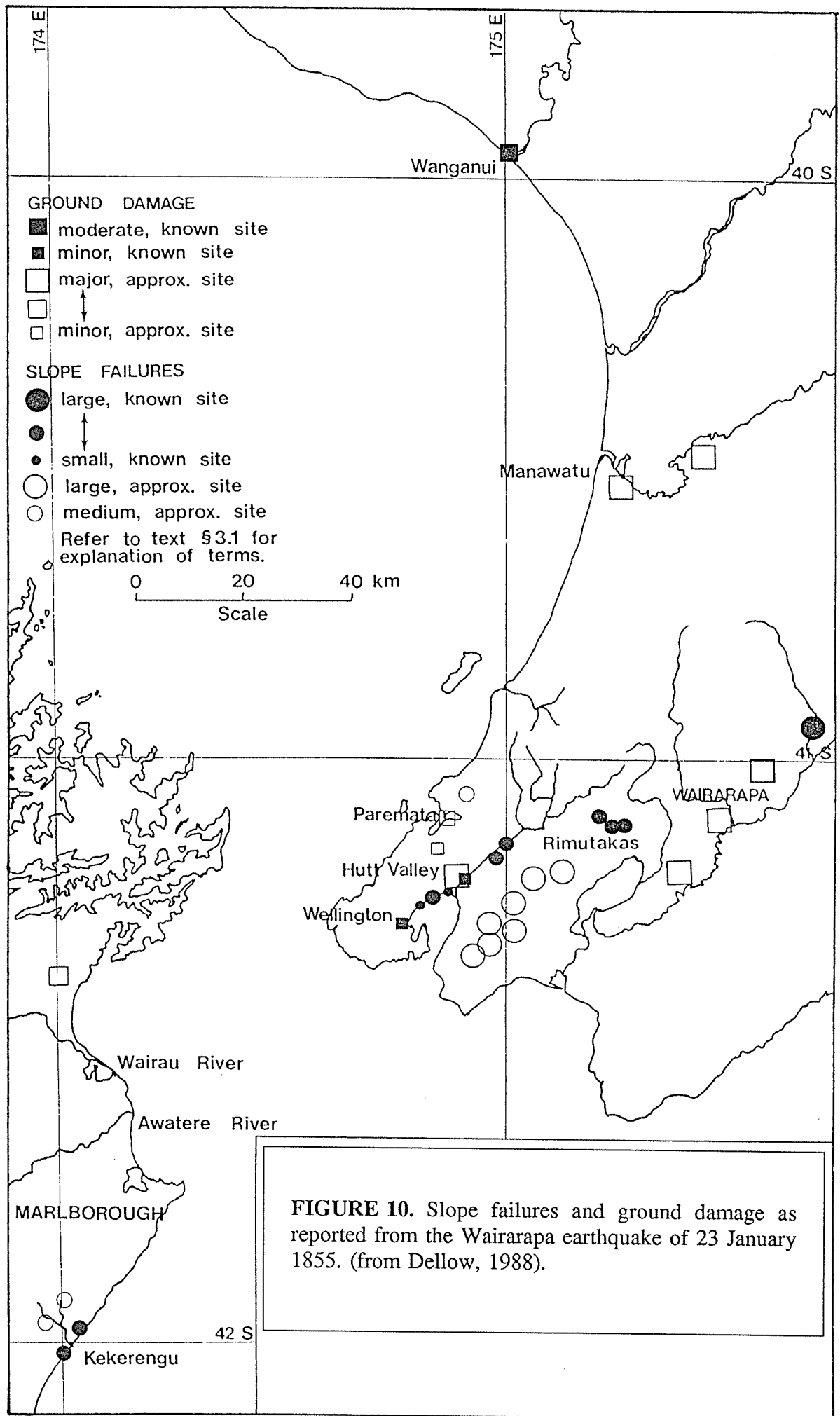


FIGURE 10. Slope failures and ground damage as reported from the Wairarapa earthquake of 23 January 1855. (from Dellow, 1988).

APPENDIX 1

**List of DSIR Reports Prepared for the
Wellington Regional Council Seismic Hazard Studies 1989-92**

- 1 **Begg, J.G.; Van Dissen, R.J., 1992:** Geology and Earthquake Ground Shaking Hazard Assessment of the Upper Hutt Basin, New Zealand. DSIR Geology & Geophysics Contract Report 1992/05 (prepared for Wellington Regional Council).
- 2 **Berryman, K.; Fellows, D., 1989:** Fault displacement hazards in the Wellington region. New Zealand Geological Survey Contract Report 89/17 (prepared for Wellington Regional Council).
- 3 **Dellow, G.D.; Read, S.A.L.; Van Dissen, R.J.; Perrin, N.D., 1991:** Geological setting of the Porirua Basin, including distribution of materials and geotechnical properties. DSIR Geology & Geophysics Contract Report 1991/46 (prepared for Wellington Regional Council).
- 4 **Gilmour, A.; Stanton, B., 1990:** Tsunami hazards in the Wellington region. DSIR Division of Water Sciences Contract Report (prepared for Wellington Regional Council).
- 5 **Heron, D.W.; Van Dissen, R.J., 1992:** Geology of the Kapiti Coast (Pukerua Bay to Otaki). DSIR Geology & Geophysics Contract Report 1992/19 (prepared for Wellington Regional Council).
- 6 **Perrin, N.D.; Campbell, H.J., 1992:** Compilation of geological data, Wellington Area. DSIR Geology & Geophysics Contract Report 1992/24 (prepared for Wellington Regional Council).
- 7 **Read, S.A.L.; Begg, J.B.; Van Dissen, R.J.; Perrin, N.D.; Dellow, G.D., 1991:** Geological setting of the Lower Hutt Valley and Wainuiomata, including distribution of materials and geotechnical properties. DSIR Geology & Geophysics Contract Report 1991/45 (prepared for Wellington Regional Council).
- 8 **Smith, E.; Berryman, K., 1990:** Return times of strong shaking in the Wellington region. DSIR Geology & Geophysics Contract Report 1990/14 (prepared for Wellington Regional Council).
- 9 **Sritharan, S.; McVerry, G.H., 1991:** Quantifying microzone effects in the Hutt Valley using strong motion earthquake records. DSIR Physical Sciences Contract Report (prepared for Wellington Regional Council).
- 10 **Sritharan, S.; McVerry, G.H., 1990:** Analysis of strong motion earthquake records for microzoning central Wellington. DSIR Physics and Engineering Laboratory Contract Report (prepared for Wellington Regional Council).
- 11 **Stephenson, W.R.; Barker, P.R., 1991:** Report on cone penetrometer and seismic cone penetrometer probing in Wainuiomata, Eastern Harbour Bays, Stokes Valley, Kura Park (Titahi Bay), and Whitby. DSIR Land Resources Contract Report 91/21 (prepared for Wellington Regional Council).

- 12 **Stephenson, W.R.; Barker, P.R., 1992:** Report on cone penetrometer and seismic cone penetrometer probing in Wellington City, Kapiti Coast, and Upper Hutt Valley. DSIR Land Resources Contract Report 92/14 (prepared for Wellington Regional Council).
- 13 **Stephenson, W.R.; Barker, P.R.; Mew, G., 1990:** Report on resonant alluvium conditions for part of Porirua Basin. DSIR Land and Soil Sciences Contract Report 90/5 (prepared for Wellington Regional Council).
- 14 **Taber, J., 1991:** Frequency dependent amplification of seismic waves at characteristic sites in the Lower Hutt Valley. Institute of Geophysics, Victoria University of Wellington (report prepared for Wellington Regional Council).
- 15 **Taber, J.J.; Smith E.G.C., 1991:** Frequency dependent amplification of seismic waves at characteristic sites in the Porirua Basin. DSIR Geology & Geophysics Contract Report 91/32 (prepared for Wellington Regional Council).
- 16 **Taber, J.J.; Richardson, W., 1992:** Frequency dependent amplification of weak ground motions in Wellington City and the Kapiti Coast. Institute of Geophysics Report, Victoria University of Wellington (prepared for Wellington Regional Council).
- 17 **Van Dissen, R.J.; Berryman, K.R., 1990:** Seismic hazard assessment of the Wellington-Hutt Valley segment of the Wellington Fault. DSIR Geology & Geophysics Contract Report 90/24 (prepared for Wellington Regional Council).
- 18 **Van Dissen, R.J., 1991:** Ground shaking hazard map for Lower Hutt and Porirua areas: A summary report. DSIR Geology & Geophysics Contract Report 1991/42 (prepared for Wellington Regional Council).
- 19 **Van Dissen, R.J., 1992:** Earthquake ground shaking hazard assessment of the Wairarapa, New Zealand. DSIR Geology & Geophysics Contract Report 1992/10 (prepared for Wellington Regional Council).
- 20 **Van Dissen, R.J.; Taber, J.J.; Stephenson, W.R.; Sritharan, S.; Perrin, N.D.; McVerry, G.H.; Campbell, H.J.; and Barker, P.R., 1992:** Earthquake ground shaking hazard assessment for Wellington City and suburbs, New Zealand. DSIR Geology & Geophysics Contract Report 1992/23 (prepared for Wellington Regional Council).
- 21 **Van Dissen, R.J.; Taber, J.J.; Stephenson, W.R.; Heron, D.W.; and Barker, P.R., 1992:** Earthquake ground shaking hazard assessment of the Kapiti Coast, New Zealand. DSIR Geology & Geophysics Contract Report 1992/20 (prepared for Wellington Regional Council).

APPENDIX 2

Modified Mercalli (MM) Scale of Intensity of Earthquake Shaking (NZ version 1965)

MM I	Not felt by humans, except in especially favourable circumstances, but birds and animals may be disturbed. Reported mainly from the upper floors of buildings more than 10 storeys high. Dizziness or nausea may be experienced.	MM VI	Felt by all. People and animals alarmed. Many run outside. Difficulty experienced in walking steadily. Slight damage to Masonry D. Some plaster cracks or falls. Isolated cases of chimney damage.
MM II	Branches of trees, chandeliers, doors, and other suspended systems of long natural period may be seen to move slowly. Water in ponds, lakes, reservoirs, etc., may be set into seiche oscillation.		Windows, glassware, and crockery broken. Objects fall from shelves, and pictures from walls. Heavy furniture moved. Unstable furniture overturned.
MM III	Felt by a few persons at rest indoors, especially by those on upper floors or otherwise favourably placed. The long-period effects listed under MMI may be more noticeable.		Small church and school bells ring. Trees and bushes shake, or are heard to rustle. Loose material may be dislodged from existing slips, talus slopes, or shingle slides.
MM IV	Felt indoors, but not identified as an earthquake by everyone. Vibration may be likened to the passing of light traffic. It may be possible to estimate the duration, but not the direction. Hanging objects may swing slightly. Standing motorcars may rock slightly.	MM VII	General alarm. Difficulty experienced in standing. Noticed by drivers of motorcars. Trees and bushes strongly shaken. Large bells ring. Masonry D cracked and damaged. A few instances of damage to Masonry C.
MM V	Generally noticed indoors, but not outside. Very light sleepers may be awakened. Vibration may be likened to the passing of heavy traffic, or to the jolt of a heavy object falling or striking the building. Walls and frame of buildings are heard to creak. Doors and windows rattle. Glassware and crockery rattles. Liquids in open vessels may be slightly disturbed. Standing motorcars may rock, and the shock can be felt by their occupants.		Loose brickwork and tiles dislodged. Unbraced parapets and architectural ornaments may fall. Stone walls cracked. Weak chimneys broken, usually at the roof-line. Domestic water tanks burst. Concrete irrigation ditches damaged. Waves seen on ponds and lakes. Water made turbid by stirred-up mud. Small slips, and caving-in of sand and gravel banks.
	Generally felt outside, and by almost everyone indoors. Most sleepers awakened. A few people frightened. Direction of motion can be estimated. Small unstable objects are displaced or upset. Some glassware and crockery may be broken. Some windows cracked. A few earthenware toilet fixtures cracked. Hanging pictures move. Doors and shutters may swing. Pendulum clocks stop, start, or change rate.	MM VIII	Alarm may approach panic. Steering of motorcars affected. Masonry C damaged, with partial collapse. Masonry B damaged in some cases. Masonry A undamaged. Chimneys, factory stacks, monuments, towers, and elevated tanks twisted or brought down. Panel walls thrown out of frame structures. Some brick veneers damaged. Decayed wooden piles broken. Frame houses not secured to the foundation may move. Cracks appear on steep slopes and in wet ground. Landslips in roadside cuttings and unsupported excavations. Some tree branches may be broken off. Changes in the flow or temperature of springs and wells may occur. Small earthquake fountains.

APPENDIX 2 (continued)

MM IX	General panic.	<u>Categories of Non-wooden Construction</u>
	Masonry D destroyed. Masonry C heavily damaged, sometimes collapsing completely. Masonry B seriously damaged. Frame structures racked and distorted. Damage to foundations general. Frame houses not secured to the foundations shifted off. Brick veneers fall and expose frames. Cracking of the ground conspicuous. Minor damage to paths and roadways. Sand and mud ejected in alluviated areas, with the formation of earthquake fountains and sand craters. Underground pipes broken. Serious damage to reservoirs.	<u>Masonry A.</u> Structures designed to resist lateral forces of about 0.1 g, such as those satisfying the New Zealand Model Building Bylaws, 1955. Typical buildings of this kind are well reinforced by means of steel or ferro-concrete bands, or are wholly of ferro-concrete construction. All mortar is of good quality and the design and workmanship is good. Few buildings erected prior to 1935 can be regarded as in category A. <u>Masonry B.</u> Reinforced buildings of good workmanship and with sound mortar, but not designed in detail to resist lateral forces. <u>Masonry C.</u> Buildings of ordinary workmanship, with mortar of average quality. No extreme weakness, such as inadequate bonding of the corners, but neither designed nor reinforced to resist lateral forces. <u>Masonry D.</u> Buildings with low standards of workmanship, poor mortar, or constructed of weak materials like mud brick and rammed earth. Weak horizontally.
MM X	Most masonry structures destroyed, together with their foundations. Some well built wooden buildings and bridges seriously damaged. Dams, dykes, and embankments seriously damaged. Railway lines slightly bent. Cement and asphalt roads and pavements badly cracked or thrown into waves. Large landslides on river banks and steep coasts. Sand and mud on beaches and flat land moved horizontally. Large and spectacular sand and mud fountains. Water from rivers, lakes, and canals thrown up on the bank.	<u>Windows</u> Window breakage depends greatly upon the nature of the frame and its orientation with respect to the earthquake source. Windows cracked at MM V are usually either large display windows, or windows tightly fitted to metal frames. <u>Chimneys</u> The "weak chimneys" listed under MM VII are unreinforced domestic chimneys of brick, concrete block, or poured concrete.
MM XI	Wooden frame structures destroyed. Great damage to railway lines. Great damage to underground pipes.	<u>Water tanks</u> The "domestic water tanks" listed under MM VII are of the cylindrical corrugated-iron type common in New Zealand rural areas. If these are only partly full, movement of the water may burst soldered and riveted seams.
MM XII	Damage virtually total. Practically all works of construction destroyed or greatly damaged. Large rock masses displaced. Lines of sight and level distorted. Visible wave-motion of the ground surface reported. Objects thrown upwards into the air.	Hot-water cylinders constrained only by supply and delivery pipes may move sufficiently to break the pipes at about the same intensity.

(from Hatherton 1984)