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LIQUEFACTION CASE HISTORIES FROM THE WEST COAST OF THE SOUTH ISLAND, NEW ZEALAND

by

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This report is identical to a thesis of the same name submitted to the University of Canterbury in partial fulfilment of the requirements for the Master of Engineering degree.

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Abstract

Liquefaction is a phenomenon that results in a loss of strength and stability of a saturated soil mass due to dynamic excitation such as that imposed by an earthquake. The granular nature of New Zealand soils and the location of many of our cities and towns on fluvial foundations are such that the effects of liquefaction can be very important.

Research was undertaken to build on the past work undertaken at the University of Canterbury studying the effects of the 1929 Murchison earthquake, the 1968 Inangahua earthquake and the 1991 Hawks Crag earthquakes on the West Coast. Additional archival information has been gathered from newspapers and reports and from discussions with people who experienced one or all of these large earthquakes that occurred on the West Coast during the 20^{th} Century.

Further, some twenty Cone Penetrometer Tests were carried out, with varying success, in Greymouth and Karamea using the Department of Civil Engineering's Drilling Rig. These, combined with the basic site investigation information, consolidate and add to the liquefaction case history data bank at the University of Canterbury.

Many of the sites have liquefied in some but not all of the three earthquakes and thus provide both upper and lower bounds for the calibration of empirical models. While a lack of knowledge of the 1929 source location reduces the value of information from that event, the data form a useful set of liquefaction case histories and will become more so as further earthquakes occur. A list of critical sites for checking of the future earthquakes is provided and recommendations are made for the installation of downhole arrays of accelerometers and pore water pressure transducers at a number of sites.

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I wish to acknowledge the help and guidance of my supervisor, Dr. J. Berrill, throughout my research. Siale Faitotonu and Nonga, for their practical assistance and patience in the field and without whom I could not have undertaken the field testing.
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But most of all, thanks go to..
My family, for their love, enthusiasm and support in whatever I choose to do. For their company on some field trips and patience with the many drafts of this thesis!
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1 Introduction

1.1 Soil Liquefaction

Liquefaction is a phenomenon that results in a loss of strength and stability of a saturated soil mass due to dynamic loading, such as that imposed by an earthquake. The granular nature of New Zealand soils and the location of many of our cities and towns on such soils mean that the effects of liquefaction should be considered when assessing seismic risk. Understanding the phenomenon of liquefaction and identifying where liquefaction is likely to occur, should be essential knowledge for civil engineers and city planners, as the stability of the supporting ground beneath inhabited areas plays a key role in the survivability of our cities and towns following an earthquake.

In the past, estimation of the liquefaction potential of a site has been largely based on correlating anecdotal evidence from past earthquakes and the geology and seismicity of the regions. Significant progress in liquefaction modelling and prediction has occurred in the last 20 years, as research has been undertaken world wide to study this phenomenon. The correlation between the anecdotal evidence and the data pertaining to soil properties is the basis for the assessment of the vulnerability of sites in New Zealand and world wide to this phenomenon, though more information is required before the effects and occurrence of liquefaction can be completely understood.

1.2 Thesis Objectives

This research seeks to extend previous studies undertaken at the Geomechanics group of the University of Canterbury into liquefaction in the Buller Region. Many West Coast towns are located on geologically young ground. Hokitika, Greymouth, Westport and Karamea are located at river mouths in a mixed fluvial and lagoonal environment while Reefton, Inangahua, and Murchison have been established in river valleys on near level flood plains. These sites are therefore naturally susceptible to liquefaction and in conjunction with the proximity of the many active faults, including the Alpine Fault, the likelihood of liquefaction is high. These localities are of prime interest to researchers studying this phenomenon.

Earlier studies on liquefaction occurrences have focussed on Inangahua and Westport with little in-depth work being done elsewhere. However, liquefaction occurrences had been reported in Karamea and Greymouth as a result of the 1929 Murchison earthquake and the 1968 Inangahua earthquake, two of the three major earthquakes on the West Coast in the 20th Century. It is these shortcomings that have led to this research project. The aim is to gather anecdotal and documented descriptions of historical liquefaction effects, especially for Karamea and Greymouth sites, carry out further field testing and to combine the present and past data into a coherent set.

Archival information has been gathered from newspaper articles, Government reports and private documents as well as discussions with people who experienced one or all of these earthquakes. These included twelve local, regional and national museums, ten libraries as well as other sources such as the regional councils. Thirteen local and national newspapers were also researched for information. These contact sources and their current contact details are given in Appendix E.

From this historical data, the most useful sites for calibrating models were selected and studied more rigorously. Reports of sand boils and geysers, lateral spreading of the soil deposit, subsidence of the ground surface, and the tilting or sinking of man-made structures were accepted as evidence of liquefaction.

The sites were chosen on the following basis:

- evidence of soil liquefaction in past earthquakes- positive identification of such sites through photographs or reports from multiple sources
- distance from the source of the earthquake
- variation – sites which had liquefied in some earthquakes but not others
- suitability of the soil for CPT testing
- suitability of site for CPT Rig access

On this basis, a number of key sites in Greymouth and Karamea were selected and a preliminary study of each of these sites undertaken. This involved talking to long-time local residents, council staff and others with knowledge of the area. Plans of proposed drill sites were examined in order to determine the general nature of the soil and the location of any underground services. Hand augering was then undertaken to identify any gravel layers close to the surface which would cause difficulties for the light-weight drilling rig available. Around twenty preliminary Cone Penetrometer Tests were undertaken with the Department of Civil Engineering's Drilling Rig, with varying degrees of success.

A number of empirical models, based on case history data and soil properties, have been formulated over the past 30 years to estimate the liquefaction potential of a deposit of soil. The following models were considered and are discussed in Chapter 7.

- Tsuchida and Hayashi (1970) (see National Research Council, 1985)
- Zhou (1980)
- Davis and Berrill (1982)
- Liao, Veneziano and Whitman (1988)
- Shibata and Teparaksa (1988)
- Law, Cao and He (1990)
- Suzuki, Tokimatsu, Koyamada, Taya, and Kubota (1995)
- Olsen (1997)
- Robertson and Wride (1998) –as adopted by NCEER working group (2001)
- Juang, Yuan, Lee, Lin, (2003)
- Toprak and Holzer (2003)

Based on the historical data collected and site investigation work done in both Karamea and Greymouth, a number of locations present themselves as particularly susceptible to liquefaction. These sites, therefore, deserve further study in order to increase understanding into the liquefaction phenomena, at both a local and a global level. It is suggested that the installation of downhole instrumentation be considered for some of these sites.

1.3 Previous Work Done

Studies began in 1984, with Fairless detailing known sites of liquefaction throughout New Zealand. As more recent studies have been undertaken the number of known incidences of liquefaction has increased. At the University of Canterbury a number of postgraduate students and a Post-doctoral research fellow have studied the effects of liquefaction at a number of West Coast localities.

Ooi (1987), Adlam (1988) and Bienvenu (1988) all undertook drilling and research in the Buller region. Bienvenu visited Karamea and undertook preliminary field work including hand augering. All three undertook field work at Inangahua on Three Channel Flat and Walkers Flat., while Bienvenu and Ooi carried out similar testing at Westport. This work was continued by

Dou Yiqiang (Dou and Berrill, 1992) who visited the Buller district six days after the 1991 Hawks Crag earthquake and recorded the effects from a geotechnical perspective. Dou also utilised aerial photographs taken following the 1968 Inangahua earthquake to identify sites of liquefaction in the Inangahua, Westport and Reefton areas.

Liquefaction studies in other areas of New Zealand have been pursued by students at the University of Canterbury including Mulqueen (1989), who intended continuing the work started by Ooi, Adlam and Bienvenu, but due to an extremely wet summer instead undertook CPT and SPT work at Kaiapoi, north of Christchurch where liquefaction occurred in the 1901 Cheviot earthquake. Christensen (1995) studied the effects of the 1987 Edgecumbe earthquake in the Bay of Plenty region of the North Island and Vreugdenhil (1995) investigated the accuracy of a number of prediction models and studied the accuracy of the CPT measurements with respect to thin layers of soil. In the early 1990's, Oliver Guilham undertook a major investigation for the Christchurch Lifeline study (Guilham and Berrill, 1993), while in 1999, a French exchange student, Florence Cassassuce (Cassassuce and Berrill, 1999) undertook another liquefaction study for the Christchurch City Council carrying out both CPT and Seismic CPT testing using the University of Canterbury's Drilling Rig. Most recently a study seeking to understand the effects of liquefaction on lifelines has been undertaken by Wick (2000) in the Wairarapa Region of the North Island.

1.4 Thesis Organisation

Chapter 2 sketches the liquefaction mechanism and outlines the geology of the region under consideration while in Chapters 3, 4 and 5 evidence of liquefaction occurring on the West Coast following the 1929 Murchison earthquake, the 1968 Inangahua earthquake and a number of other smaller earthquakes is given. A summary of the known sites of liquefaction resulting from earthquakes centred on the West Coast of the South Island since the 1920s is given in a table in Chapter 6. Details of the testing undertaken and the limitations of the testing methods are also presented in this chapter. Chapter 7 then presents the different models used to predict the liquefaction potential of a site. The analyses from these models, using data from the field testing undertaken, are presented in Chapter 8. A discussion of these results, illustrating the limitations in current knowledge in the prediction of liquefaction occurrences, is given in Chapter 9. The conclusions and questions raised in the research are detailed in Chapter 10. Location of the test sites, data from the CPT tests, bore logs and particle-size distribution curves are given in Appendix A.

2 Basics of Liquefaction

2.1 Liquefaction Mechanism

The basic mechanism of liquefaction is now reasonably well understood. In order for a soil deposit to liquefy it must be a saturated, loose and fine grained material such as sand or coarse silt. Under shaking conditions, such as those developed by earthquake motions, the sand grains try to pack down into a denser configuration. However, due to the soil's saturated state, water must be expelled from the pore space between the particles in order for a denser configuration to be achieved. If the soil mass is gravel sized or a coarse sand, water can flow out freely from the space between the particles allowing a denser packing of the grains. However, if the sand is finer, the pore space will also be finer, and the water will have insufficient time to flow outwards resulting in a build up of pore water pressure. In response, the particles begin to lose contact with each other and if the shaking is strong enough, the effective stress will vanish turning the soil and water combination into a liquid state. This liquefied soil, carrying the weight of the overlying soil, is under considerable pressure. If the pressures reach a critical level, the slurry mix may be vented though cracks in the overlying strata to form sand boils or geysers at the ground surface. The liquefaction phenomenon is generally limited to the top 10 to 15 metres of strata, as beneath this depth pressure cannot be relieved by venting the material to the ground surface due to the weight of overlying soil (National Research Council, 1985).

2.2 Liquefaction Effects

Liquefaction results in a substantial reduction in ground strength and loss of stability and damage to structures may result. Bearing capacity failures, floatation of buried structures, lateral spreading, differential settlement or minor effects such as sand boils have all been attributed to liquefaction following earthquakes. The human impact of these failure effects is largely dependent on the land use. A brief description of some of the different effects is given below.

2.2.1 Sand Boils

Sand boils are an unambiguous indicator that liquefaction has occurred (Beetham and Hancox, 1992). They are usually fairly easy to recognise and as such have been used as the basis for discerning whether or not a site has liquefied. Sand boils are formed during earthquake shaking when the compaction of granular materials creates zones of high slurry pressure, and to relieve the pressure the water is expelled through the cracks in the overlying strata or through vents it forms vertically through the sand layers to reach the ground surface. This water can flow violently and carries suspended sediment with it, which often settles and forms a conical shape around the vent. An example of this is shown below in Figure 2-1.

Figure 2-1. Sand Boils at Three Channel Flat, Inangahua (photo courtesy Dr. A.J. Carr)

The distribution of the sand boils is thought to depend on the soil deposit. If a deep layer liquefies there are likely to be fewer, but larger, sand boils on the ground surface than if a layer close to the ground surface liquefies. General settlement of the ground occurs if the soil overlying the liquefied material is relatively permeable, though the uniformity of the settlement also depends on the homogeneity of the material. If the overlying material is of lower permeability and cohesionless, it may settle non-uniformly as it becomes suspended on the fluid zone. It is also thought that if cohesive material is overlying the liquefied zone, the upper layer may crack due to the shaking. Through these cracks liquid can vent and elongated and irregular sand boils can form.

The sand and water mixture may not vent at the ground surface if a coarser material such as gravel overlies the liquefied layer. Instead, the liquefied material may be trapped in the pore space between the larger particles (National Research Council, 1985).

2.2.2 Lateral Spreading

Lateral spreading generally occurs on shallow slopes (usually between 0.3 and 3 degrees) where a complex combination of inertia forces and momentary loss of strength of a subsurface layer can cause the upper layer to break up into blocks which move down slope or towards a free face, such as towards a river channel. Fissures generally separate the blocks created, and the area affected can extend to up to tens or sometimes hundreds of metres. This is shown in Figure 2-2.

Lateral spreading is one of the most damaging forms of liquefaction induced ground failure, causing damage to foundations of buildings located on or across the failure surface and rupturing pipelines and other facilities (Kramer, 1997). Ishihara (1996) describes the mechanism in detail. Berrill and Yasuda (2002) give examples of typical damage.

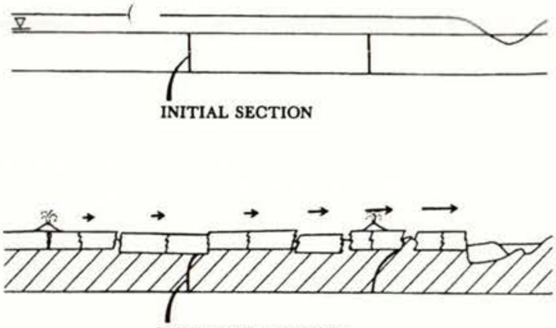


Figure 2-2. Lateral Spreading Mechanism (National Research Council, 1985)

2.2.3 Flow Failure

Flow failures generally involve slopes greater than three degrees and occur in loose saturated sands or silts when liquefaction develops beneath the ground surface. The soil loses strength and flows down a slope. The flows may be composed of blocks of intact material, sliding on a layer of liquefied material, or completely liquefied material, as is shown in Figure 2-3 (National Research Council, 1985).

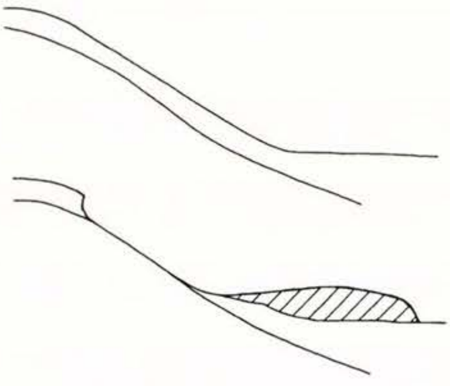


Figure 2-3. Flow Failure Mechanism (National Research Council, 1985)

2.2.4 Ground Settlement

The densification of a soil deposit due to earthquake shaking can result in the subsidence of an area.

2.2.5 Loss of Bearing Capacity

Liquefaction of soil, which results in a substantial reduction of soil strength, can result in large deformation. If liquefied soil is supporting a structure, it can result in a reduction of the foundations support, leaving the structure to both settle and tip, as occurred in Niigata, Japan in

1964 and in Turkey in 1999. This can be a gradual process where the liquefaction slowly propagates up through the overlying soil layers, weakening the soil and allowing the building to slowly settle and/or tip (National Research Council, 1985).

2.2.6 Rise of Buried Structures

When the surrounding soil liquefies, buried structures that are less dense than the surrounding soil, such as pipelines, timber piles and tanks, can rise buoyantly. This generally impacts on lifeline services (National Research Council, 1985).

2.3 ***Assessment of the Liquefaction Potential***

Kramer (1996) notes that the susceptibility of a soil deposit to liquefy is influenced by the following factors:

- density, relative density, and degree of saturation
- particle size distribution
- age of deposit
- depth to the water table
- and the environment in which the deposition process occurs.

The liquefaction resistance increases with age of a deposit. It was written by Youd *et al.* (2001) that, "*sediments deposited within the past few thousand years are generally much more susceptible to liquefaction than older Holocene sediments; Pleistocene sediments are even more resistant, and pre-Pleistocene sediments are generally immune to liquefaction*".

The shaking at a site also needs to be of sufficient severity to induce liquefaction. The following factors all influence the site liquefying in a given earthquake:

- earthquake magnitude
- duration of shaking
- distance from the earthquake source
- other higher order seismological factors such as directivity and site amplification effects.

2.4 ***Geologic Setting***

The West Coast region studied lies to the west of the tectonically active Alpine Fault in the South Island of New Zealand. This fault separates the Indo-Australian and Pacific plates.

As was noted in Chapter 1, many of the towns in North-west Nelson and on the West Coast of the South Island of New Zealand are founded on geologically young ground. Riwaka and Motueka are located on the delta of the Motueka, Riwaka and Moutere Streams, whereas towns such as Reefton, Inangahua and Murchison are all situated on the near level flood plains of the Buller and Inangahua Rivers. Karamea, Westport and Greymouth are towns located at river mouths and in a lagoon environment, and as such are especially susceptible to liquefaction effects.

In the following sections, further details are given regarding the geology of selected towns.

2.4.1 Karamea

The townships of Oparara, Karamea, and Little Wanganui are located on the flood plains of the Oparara, Karamea and Little Wanganui Rivers respectively. Remnants of old sand dunes are visible along the coast back from the present shore for several hundred metres, from Little Wanganui in the south to the Kohaihai River to the north, creating a hummocky landscape except for the estuarine mud flats at the mouth of the Karamea River. Geological maps of the area indicate that the mountains to the east and south of Karamea are composed of granitic zones, siltstones, mudstones, and limestone.

The Karamea River took a different path in 1929 to that taken today, with a now abandoned second channel braiding off to the north and the outlet also to the north and referred to as the Overflow. The map shown in Figure 2-4 was drawn in 1918 and shows that the Karamea Township was situated on what was known as Simpson's Island, which had been formed by the changing course of the braided Karamea River. It is likely to have been built up by the accretion of sand and silt from the river flow, as well as longshore drift along the Karamea Bight. Both the Karamea River and Oparara River mouths have moved considerably over time, as can be seen by comparing the 1918 map with the current map of the area, shown in Figure 2-5. At the river mouth, the river flow rates diminish rapidly as the coast is approached, the rivers become wider and a large amount of sediment is deposited. These deposits are predominantly sand and thus ideally liquefiable materials when saturated. In 1931, the Overflow was blocked off during construction of the flood protection walls. The channel has since filled in, though is visible from above as a small dip in the farm paddocks.

Figure 2-4. Part of a 1918 Map of Karamea (from Archives New Zealand)

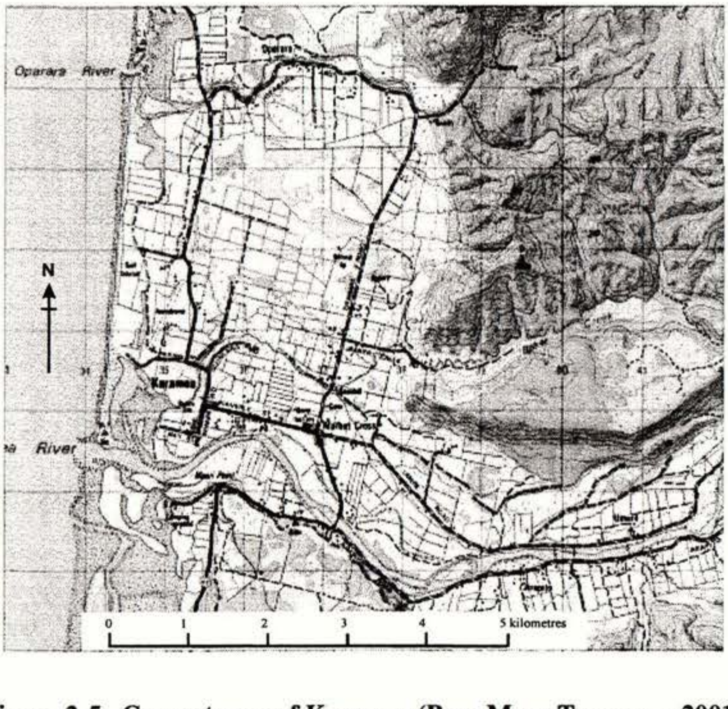


Figure 2-5. Current map of Karamea (Base Map: Topomap, 2001)

Karamea has a history of flooding, as have many West Coast settlements founded in river valleys. Floods occurred in 1893, 1914, 1929 (after the earthquake), 1931, 1963, 1973, 1983 and 1998; each of these events inundated the region with water and left sand and silt deposits.

Frank Wood, in a letter written from Karamea in the days following the 1929 earthquake, wrote:

> *"The flood, (such as it was) came last night, but it was very little more than an ordinary flood, and was mostly caused by ordinary causes- that is, rain on the hills..."*.

These events have contributed to the building up of the coastal plains, through the deposition of young sediments from the mountain catchments and would account for the silt and sand layering which is present over much of the area. Swampy areas of land, such as those between the beach ridge and the mountains at Oparara, have been drained for farming.

2.4.2 Westport

According to Adams and LeFort (1963) "*Westport is situated on one of the Quaternary alluvial plains that have been built up near the coast. Off shore there is a broad continental shelf of low relief, with the 200 metre contour distant 30 to 60 km from the coast*". The following diagram given by Dowrick *et al.* (2003) referred to the work of Suggate and Wood (1979) to illustrate the location of dune sands and alluvial gravel and silt as shown in Figure 2-6.

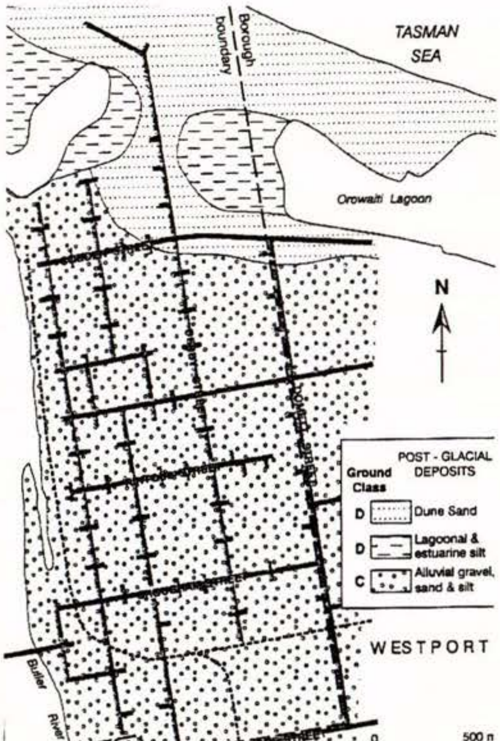


Figure 2-6. Microzoning Map of Westport (Dowrick *et al.*, 2003)

Furkert (1946) gave details on the design of the harbour. He wrote that the harbour acts as a groyne and "*in combination with the material brought down by the river, have built up the seabed for miles to a surprising extent*".

2.4.3 Inangahua

The mountain ranges in the vicinity of Inangahua are formed mainly of gneiss, granite and greywacke. The Grey-Inangahua Depression, filled predominantly with granitic and greywacke-derived sediment carried by the Buller and Inangahua Rivers, extends from Inangahua for approximately 90 kilometres in a south-westerly direction. Near Inangahua the upper layers are described as being tertiary siltstone and sandstone and extending southwards towards Rotokohu are Quaternary gravel deposits (Lensen and Otway, 1971). Sutherland (1969) writes,

> "*Two major rivers flow through the area, the Buller and the Inangahua. From Murchison the former flows through the Upper Buller Gorge and into the Grey-Inangahua Depression where it is now incised approximately 50ft below a broad terrace cut across the sediments. It then flows rapidly to its junction with the Inangahua after which it enters a limestone gorge, passes along the north side of Walkers Flat and enters the Lower Buller Gorge through the Paparoa Range*".

Both Three Channel Flat and Walkers Flat are recent flood plains comprising young, unconsolidated deposits and have large portions of medium to fine grained sands which are angular and composed predominantly of quartz, feldspar, and mica fragments. (Berrill *et al.*, 1987a).

2.4.4 Greymouth

A map of the area drawn in 1879 for the development of the river training wall and wharf indicated a narrow bar on the western side of the Erua Moana Lagoon. Comparing this to Figure 2-7, a map drawn by Dowrick *et al.* (2003) and based on Suggate and Wood (1979), indicates that the coast has been built up considerably over the past hundred years. This is a result of the effects of longshore drift and the sea wall acting as a groyne. Due to the rate of accretion, and the less energetic environment in which the sediment is trapped, the sediments in the Blaketown area are thought to be fairly loose in comparison to sand dunes subject to wave action, and hence more susceptible to liquefaction. It will be seen later that the Blaketown area was indeed one of the worst affected in the 1929 earthquake.

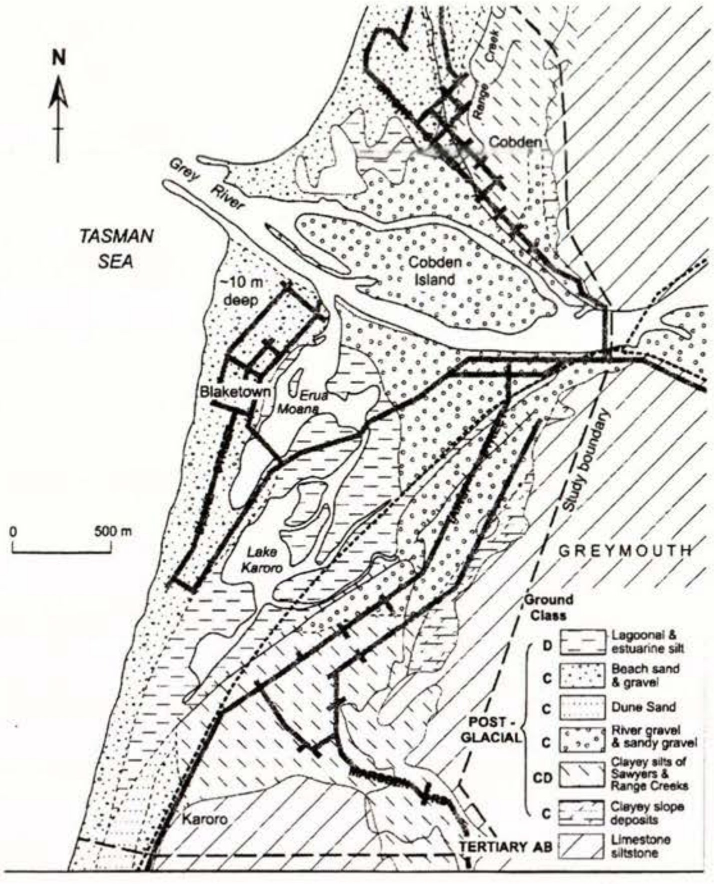


Figure 2-7. Microzoning Map of Greymouth (Dowrick *et al.*, 2003)

Maps, dated 1912, of Greymouth and Karamea are in Appendix D. These indicate the changes in coastline and give an indication of sand deposits.

3 1929 Murchison Earthquake

3.1 Introduction

The 1929 Murchison earthquake, otherwise known as the Buller earthquake, caused severe damage in the Westland and Nelson regions. The earthquake, which has been assigned a magnitude of 7.8 (Dowrick and Smith, 1990) based on intensity reports from 1929, was one of the largest recorded in recent times in New Zealand. Dowrick (1994), wrote that the earthquake occurred in the middle of what was a much wetter June than normal. This statement was based on comparisons made of the June 1929 rainfall levels, with average rainfall levels for four weather stations on the northern West Coast.

The map shown in Figure 3-1 on the following page indicates the location of the epicentre of the earthquake on the White Creek Fault, near Murchison. The areas where liquefaction is thought to have occurred following the earthquake are also indicated on the map with the locations based on documentation cited in the following sections of this chapter. Also drawn on this map is both the known and supposed extent of the rupture surface as given by Dowrick (1994).

3.2 Seismological Parameters

Date: 16 June 1929 (17 June 1929 local time)

Time: 22:47:43 UT (10:17:43 am local time)

Magnitude: 7.8 M_S (Dowrick and Smith, 1990)

Epicentral Intensity: 8 Rossi-Forel Scale (Bastings, 1935)

Epicentre: 41.7° S 171.2° E

Depth: 20 km

The isoseismal map shown in Figure 3-2 illustrates the felt intensities at different locations.

3.3 Location of the Fault Plane

Ferrar and Grange (1929) stated that the release of stress on the White Creek Fault also "*took place simultaneously on or near the Kongahu fault-plane, the trace of which is parallel and close to the West Coast of Nelson Province*". If this were true, it could be considered a double earthquake, as the foci were on two nearly parallel faults or lines of weakness. Bastings (1933) also noted that along the coast near Kongahu Point, a strip of sea- beach 1.5 km long and between 200 and 300 metres wide, was uplifted a height of six to thirty metres and that this might be close to the second epicentre or seat of major disturbance.

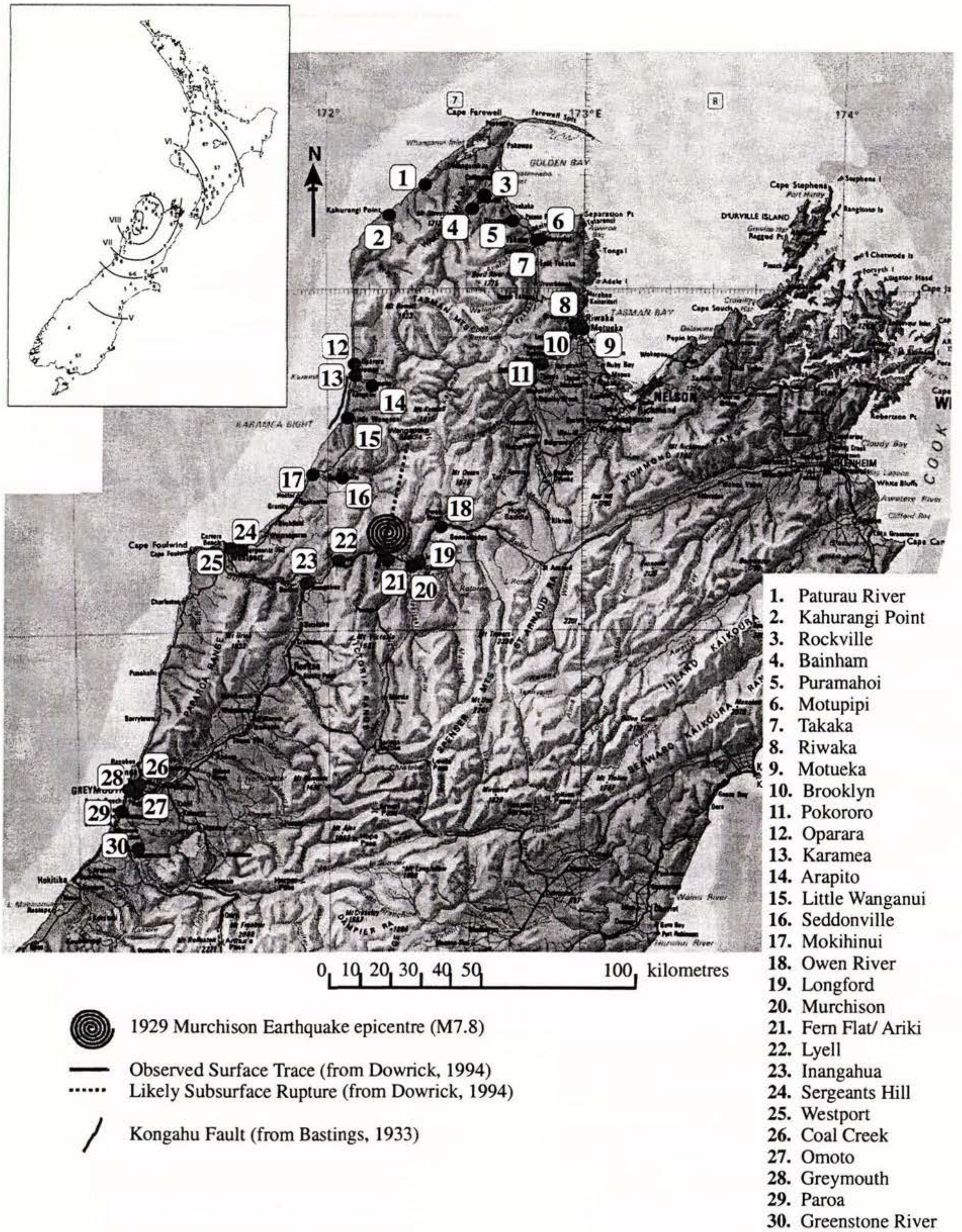


Figure 3-1. Sites of Liquefaction following the 1929 Murchison Earthquake (Base map: Topomap, 2001)

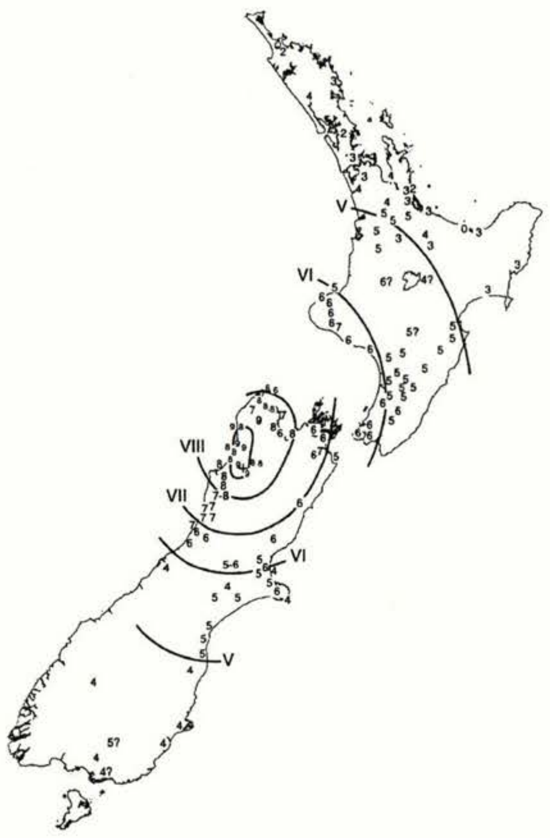


Figure 3-2. Isoseismal Map of the 1929 Murchison Earthquake (from Downes, 1995)

Bastings (1933) wrote that the "*movement may be merely superficial, the result of the slumping of tall cliffs which skirt the beach for some distance in either direction*". The idea of this second seat of disturbance was based on the distribution of aftershocks recorded at the seismological observatory in Wellington and the location of the known Kongahu fault-line. This record, while consistent with noise and surface waves coming from the west, is inconsistent with the dip to the North East of the White Creek fault over the Buller Gorge Road and the Kongahu displacement is far too great to be a result of a direct fault offset. In 1929 the epicentre would not have been well defined and definitely not located better than to within five to 10 km. There is, however, much unknown about the mechanism of the 1929 Murchison Earthquake.

Fyfe (1929) gives a detailed description of the movement on the White Creek Fault. He noted that, "*during the earthquake the country to the east of the fault plane rose relatively to the country to the west, the difference in level of the road-surface on opposite sides of the fracture being 14 ft. 9 in*". This refers to the White Creek Fault and the road through the Buller Gorge. Berryman (1980) stated that there was reverse movement on the White Creek Fault. There was also some sinistral movement, though to a lesser degree. Based on the movement of manmade structures, such as the West Coast Highway, a fence south of the Buller River, a miner's tailrace to the north of the road and the gravel outwash terraces formed by the Buller River, the trace was stated by Henderson (1937) to extend for eight kilometres. Fyfe noted that the fault trace was clearly visible for a distance of about three miles (approx. five km). Dowrick (1994) studied the intensity reports based on damage to houses and seismically induced landslides in the Buller Region to define the extent of fault rupture. He details both the observed surface trace and likely subsurface rupture. The intensity map (Figure 3-2) drawn based on the felt intensities following the earthquake, indicates that the direction of propagation of waves was likely to be in a northwards direction, which correlates well with the extent of damage north of the epicentre and, in particular, to the large amount of liquefaction at Karamea.

Bastings (1933) also noted that between the 16th June and the 30th June 1929, a total of 192 aftershocks for which the S-P interval was measurable and lay between seventeen and twenty-eight seconds, were recorded at stations and listed in the Bulletin of the Dominion Observatory. It was reported in *The Nelson Evening Mail* (Article: Terrific Explosion, Wednesday, June 19, 1929) that for a fortnight before the main earthquake there had been a number of explosions in the mountains behind Murchison. The citizens of the area initially thought that the explosions were a result of someone blasting, however, following the main event they formed the conclusion that the explosions were of subterranean origin.

As many of the reports in this and the next two chapters use the Imperial units of the time, the following conversion table may be useful.

Table 3-1. Conversion of Imperial units to Metric units

Imperial	*Metric*
1 inch	25.4 millimetres
1 foot	0.305 metres
1 yard	0.915 metres
1 chain	20.13 metres
1 mile	1.609 kilometres
1 acre = 10 square chains	0.4 hectares

3.4 ***Observed Effects***

People and the environment were affected by the Murchison earthquake in a multitude of different ways - from fallen chimneys, to landslides threatening and destroying houses, to personal possessions being destroyed and people killed. Many reports detailing the effects of the earthquake describe the state of the houses, fissuring of the roads, slips and other occurrences which proved challenging to individuals and families trying to recover from the earthquake.

Some reports and newspaper articles note the occurrence of sand boils and waterspouts, a strange phenomenon to people, but which were just treated as part of the earthquake and therefore insignificant in comparison to the provision of daily essentials.

As the earthquake occurred 74 years ago, memories are fading and many observations will be lost to history. The following excerpts from newspapers, reports and reminiscences from people who experienced the earthquake and detail the extent of liquefaction that occurred as a result of the earthquake, region by region.

3.4.1 Karamea

In 1929, Karamea was a small town in which the key industries were forestry and farming. The remote location of the settlement meant that access was difficult. There was a road through the hills to Westport, but most provisions came via the harbour wharf, from which logs and other produce were shipped out.

The effects of the earthquake were widespread throughout the Karamea area. This varied from damage to houses, to slips and landslides, fissuring, sand boils, waterspouts and lateral spreading. The destruction of the wharf and loss of the only road into the settlement left it cut off for days following the earthquake. Repairs were slow to come because of the small population, shortage of money and with delays in receiving government and county council support.

The map shown in Figure 3-3 identifies the known sites of liquefaction occurrence in the Karamea region. Most of these sites are located on the river banks or near the main rivers that supply sediment to the region and which have moved considerably over time.

A letter written by Mr. Frank Wood to his mother, details his experience in the earthquake and the days following the event. This letter is housed in the Karamea Museum and has been much referred to in the section regarding the events in Karamea, due to the detail in which Mr. Wood has written.

The people of Karamea have put a considerable effort into recording the events of the Murchison earthquake. In 1989, a '60th Anniversary Reunion' took place and at this event a tape recording was made of individuals' recollections of the damage in the area caused by the earthquake. For the same event, a number of individuals wrote of their experiences. In this section, reference has been made to the taped excerpts and written accounts by the individuals listed in Table 3-2.

Letters and photos stored in the Karamea Museum and Alexander Turnbull Library have also been referred to as well as discussions the author has had with a number of Karamea residents.

Figure 3-3. Sites of Liquefaction in Karamea, 1929 (Base map: Topomap, 2001)

Table 3-2. '60th Anniversary Reunion' Sources

60th Anniversary Tape	*60th Anniversary Written Account*
Mr. Roy Allen	Mrs. Lilywinter
Mr. Jim Richardson	Mr. Cyril Lineham
Mrs. Maisie Richardson	Mr. Stan Lineham
Mr. Jim Hargreaves	
Mrs. Averil Hay	
Mrs. Rose Duncan	

General Karamea Area

A general description of what occurred in Karamea was given in the excerpt printed below. This article, or segments of it, was reproduced in the newspapers listed in Table 3-3 and a book by Rogers (1996). Unfortunately no specific locations are given in this article and the author is unknown.

Table 3-3. Sources giving a general description of the earthquake

Newspaper	*Article Title*	*Publication Date*
The Grey River Argus	Karamea's District Pulverised	21 June 1929
The Greymouth Evening Star	Karamea Devastated	21 June 1929
The Press	Karamea in Ruins	21 June 1929
The Sun	Karamea a heap of ruins	21 June 1929
Danniverke Evening News	Advices from isolated Karamea	21 June 1929

"I don't know where to start to explain the position as regards Karamea. We are isolated, and it looks as though we will be for some time to come. There is practically nothing left whole in Karamea. Every road in the district is closed, and nearly all bridges, including Quinlan's, are down.

The wharf is gone, and the roads are either opened up in all directions or are covered altogether.

There is not a chimney left standing in Karamea, and nearly every tank is gone, and some houses were burnt down. We seem to have been the storm centre. It seems, at first sight, as though we are permanently cut off, as all the hills around us appear to be down, but the bearer of this message, if he gets through successfully, will be able to give you a better idea as to that.

> *There are great openings you could drive a horse and cart into the middle of the road. There are water geysers and boiling sulphur in the middle of the paddock, and, worse than all, Karamea river is blocked- evidently by a big slip in the Gorge.*"
>
> -various newspapers, June 1929

Frank Wood, in the letter referred to earlier, wrote, "*In some places the ground has pushed up into lumps- in others it has opened out to various sizes to one and two feet*", and later "*In many places where the ground has cracked, one side has sunk or lifted. There are many bumps on the road like this up to six inches high and there is one in front of Winstanley House over a foot high...Many people saw geysers spurting into the air, and large mud bubbles similar to those you see in pictures of Rotorua... it was merely due to the opening and closing of the ground, the water being forced up suddenly when then cracks closed. There was quite a flood of water in some places that had been previously dry, while other places which were wet temporarily dried up*".

Berrill *et al.* (1988), reporting the observations of eyewitnesses stated, "*Sand boils were observed at several places around the Karamea township and further up the Karamea River Valley. Also, in numerous places, water and sand was extruded from cracks in the ground, especially in road and railway embankments, indicating liquefaction-driven phenomena*".

Mr. Roy Allen (1989) recalled that there were, "*huge cracks alongside and across the roads, and telegraph poles all and every angle with many lying right down...*". He also noted the waterspouts, which fascinated many people who witnessed them in the earthquake and noted that many left a deposit of sand.

Oparara

Oparara School

Both Rogers (1996) and Brockie (1998), quote Vita Harney, recalling her experience as a schoolteacher at Oparara School at the time of the earthquake. This extract was part of a letter written to 'Andrea' on the 6 July 1929 and is now located in the Alexander Turnbull Library.

> "*I heard a dreadful roar and yelled to the children, 'Come to me'. ... After five minutes I thought we'd better get, so formed threes and marched my company to the road...We happened to be in the only safe piece of road for some distance. Two chains away, on either side, the road was cracking up and heaving like a thing alive. The trees in the*

bush were crashing, mud pools bubbling up in the playground and the school swaying as if crazy." - Vita Harney, 6 July 1929

The letter continues and notes that Mr. Thompson, who owned the property across the road from the Oparara School, was feeding pigs at the time of the earthquake and had to crawl to get to them. He "*had a very trying time getting them to the road for the front paddocks were imitating Rotorua and spouting geysers of water and mud*".

Mr. J. Lowe, a student of Miss Harney in 1929, confirmed this account and located precisely the location of the school in 1929 (pers. comm., January 2003). A plaque installed by the people of the area, shown in Figure 3-4, also indicates where the school was located, immediately south of the present Oparara River bridge on the Karamea-Kohaihai Road.

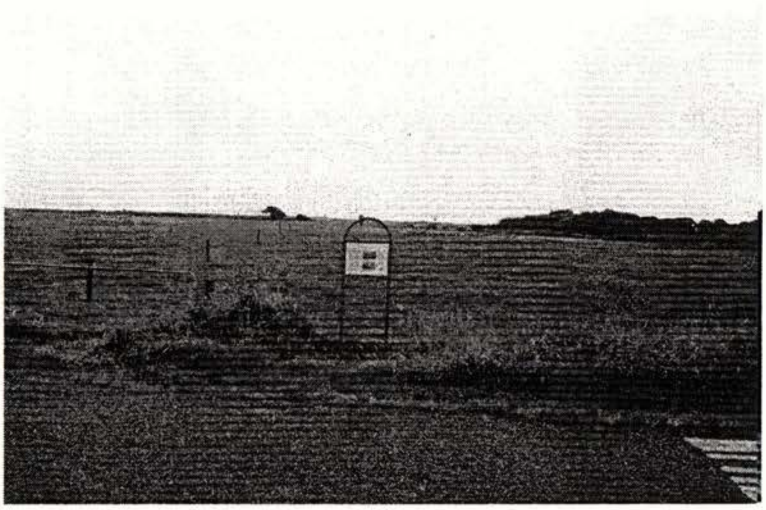


Figure 3-4. Oparara School site, next to the Oparara River (2002)

Along the Oparara River

Mr. Black, M.P., disputed the occurrence of "*quick sands at Karamea Beach*", though noted that there were quicksands in the Oparara River (*The Press*, "Karamea Beach", 5 July 1929).

Mr. Jim Hargreaves (1989) noted waterspouts covering the mudflat north of the Oparara River. He further recalled that "*There was a large house at Oparara that split clean through the middle, right up the passage, leaving a gaping hole 4ft wide and 10ft deep in the ground*". Mr. J. Lowe (pers. comm., January 2003) said that this was old Thompson's house, which is now owned by Mr. Paddy O'Donoghue and is located by number 4 in Figure 3-3. Frank Wood also noted that "*Geo. Thompson's house has broken in halves..*".

Mr. Doug Rhind recalled a relative telling him about sand boils and waterspouts occurring at Fenians property, by the turnoff to Fenians Track, Oparara. This resulted in the settlement of the gully and the present topography of the area. The site is also shown in Figure 3-3.

It was reported by Berrill *et al.* (1988), that "*Mr. Johnson also recalls a line of fence posts toppled just to the west of the pond between the aerodrome and the coast, immediately south of the present golf course*". This occurred in the Aerodrome that lies to the north of the township of Karamea.

Karamea Township

Dowrick (1994) stated that there were two levels of damage in Karamea depending on the location. On firm ground there was less damage to structures compared to those on soft ground or near rivers and streams.

> "*Piles of some buildings tilted towards the waterways, and buildings racked. Some houses, built in stages broke apart at the vertical construction joints... Piles to the wharf tilted badly, probably due to liquefaction.. Road embankments cracked badly and subsided... The three bridges (Karamea, Quinlan's and Oparara) were all usable after the earthquake, despite subsidence of the abutments to Quinlan's bridge..*".
>
> - Dowrick, 1994

In the letter to his mother Frank Wood (1929) described a lot of damage, especially that regarding differential settlements of houses in the Karamea township. He wrote, "*Drummond's new house is sunk in one corner, so has Bill Scarlett's house, near the flagstaff..*", and then, "*The back half of our backyard has dropped three inches*". Frank Wood was living in Market Cross at the time of the earthquake. Further on, it was written "*McLeans had to saw an inch off the bottom of their shop door to get it open, as the floor was on a slope inside, and there is a crack over an inch inside the concrete in front of the shop... I hear Cecil McNabb's house (where old Mr. Gibens lived) is also practically broken in two. The front and the back have sunk, leaving the centre to hold the weight*". He later corrected himself by writing, "*Cecil McNabb's house is not supported in the centre, but sunk in the centre. It is still joined together at the top, but pulled apart nearly a foot at the floor*".

Frank Wood (1929) also commented on the state of the Karamea Hall, which illustrates the effects of lateral spreading, even though the information is conflicting.

> "*The hall has, to outward appearance, shifted an inch inward at both ends. This indicates that the ground has opened out two inches somewhere under the hall. Some of the piles have tilted, some have split, and some have shifted bodily in the ground.*"
>
> -Frank Wood, June 1929

Around the River banks

As indicated in Chapter 2, the Karamea River took a slightly different path in 1929. The Overflow played an important role in the sediment transport process of the river and as is shown in Figure 2-4 meant that the settlement of Karamea was effectively on an island.

Henderson (1937) gave the following description of the area. "*Many fissures opened along the foot of a river terrace of the older alluvium on the north side of the area, and also on the east side along a road on the east bank of one of the channels of the river. The area sunk about 2 ft. 3 in. It is the youngest estuarine flat built by the river and is raised but a few inches above sea-level..*". From the passage the area referred to is likely to have been the area between Quinlans Filling (Point 8 in Figure 3-3) and the rest of the Karamea plain to the north.

Frank Wood (1929) stated that, "*All bridge approaches have dropped from one to three feet... The overflow bridge has both end spans let down about three feet... The big bridge has not shifted a fraction, but you have to climb up two feet on to it at either end*". In another part of his letter he also noted that the cracks in the ground near Quinlans Bridge and the Wharf were wider than in other areas. He wrote, "*In many places where the ground has cracked, one side has sunk or lifted*".

The main bridge across the Karamea River, indicated by number 16 in Figure 3-3, was damaged and was removed following the 1931 flood which inundated Karamea. On removal of the bridge, it was discovered that the piles had been broken off (Dowrick, 1994). This damage to piles, which was likely to have been due to the 1929 earthquake, is indicative of lateral spreading.

Overflow Bridge

A number of residents of Karamea noted the effects the earthquake had on the bridges in the area. The damage was to Quinlans Bridge is illustrated in Figure 3-5. A number of comments were made referring to the bridge over the overflow. No photos were found of this bridge, though it was documented in a letter written by the borough engineer in 1931, when changes were recommended to the overflow channel. The diagram associated with this letter is shown in Figure 3-6.

Mr. Jim Hargreaves (1989) noted, "*The flat land on the river flats sunk about 4 to 5 feet and the Bridges stayed up. The bridge over the overflow by the church dropped at both ends and stayed up in the middle. Hence we drove up hill, across the top and down the other side for years, until the filling was put in and the road straightened*". Frank Wood (1929) noted that the overflow bridge was off its piles and formed an arch shape after both end spans dropped about one metre. Mr. Gordon Duncan (1974), in a letter to the Karamea Museum, also mentioned the overflow bridge and wrote that, "*The road had subsided on each side, the outside spans had sunk at one end and held fast the centre span*".

Figure 3-5. Quinlans Bridge after the earthquake (*Auckland Weekly News*, 17 July 1929)

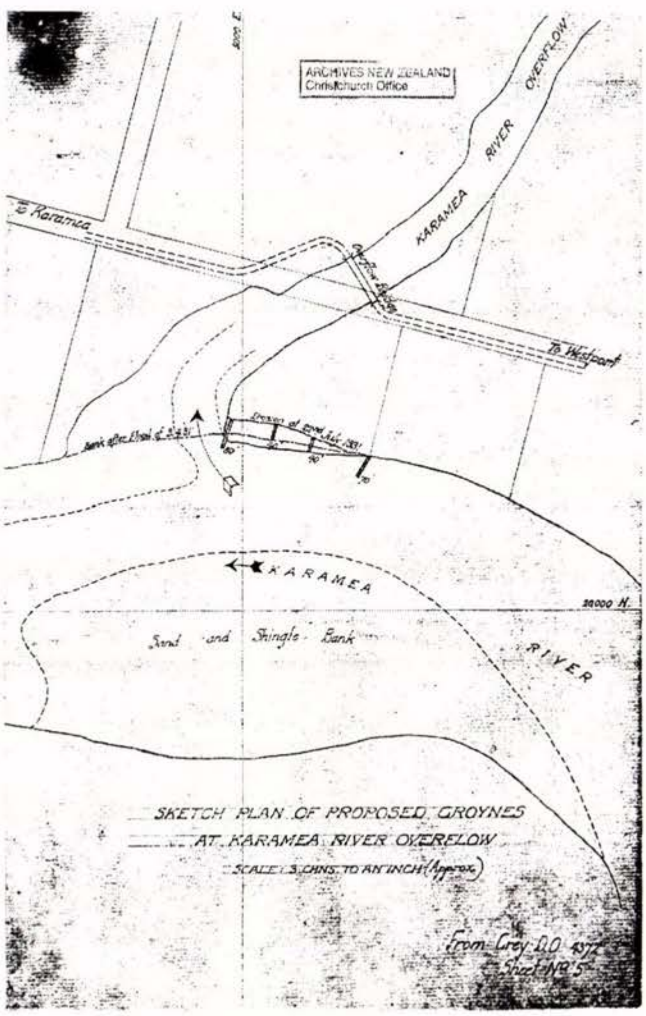


Figure 3-6. Diagram illustrating location of the Overflow Bridge (Archives New Zealand)

Roads around the Karamea area

The damage to the roads also caused disruption to the lives of the people of Karamea. Frank Wood gave the following example in his letter.

> *"The road from here to the bridge used to be perfectly straight, but now it is in and out and up and down all the way. Likewise telegraph poles are leaning this way and that all the way."* -Frank Wood, June 1929

The road at Quinlans Filling is located in Figure 3-3 and the effects illustrated in the photos shown in Figure 3-7and Figure 3-8.

Figure 3-7. Cracks in ground at Quinlans Filling - looking South towards Karamea (photo courtesy of Karamea Museum)

Figure 3-8. Quinlans Filling after the 1929 Earthquake - looking South towards Karamea (photo courtesy Karamea Museum)

Karamea School/ Fensom's Paddock/ Simpson's Paddock

Water spouts and sand boils were widespread in the Karamea school grounds and in the surrounding paddocks. Fensom's paddock is across the road from the school and Simpson's paddock on the Karamea side of the school grounds and domain, as is indicated Figure 3-3. As was shown in Figure 2-4, at the time of the earthquake this area was effectively an island between the river and overflow. The effects of liquefaction were likely to have been enhanced due to the susceptible soils of this particular area and the likelihood that the water table was higher than present because of water on both sides of the island.

Mrs. Lilywinter (1989) stated, "*Cascading up the side of the school was a fountain of water, reaching high into the air*". This was also referred to in Berrill *et al.* (1988) where it was noted that both sand and gravel were ejected immediately in front of the old school building. Berrill (pers comm., July 2003) said that the report was given by Mr. Alex Drummond, who was an 8-year old pupil at the school at the time of the earthquake and whose recollection was very clear.

Mr. Gordon Duncan (1974) stated that all over the recreation ground of the school there were "*Great gaping cracks along the ground and out of them were leaping great streams of water- ten feet, twenty feet, no thirty feet high; dozens of them- just belching out of the earth*". He noted that there were also cracks in the road which they saw as they walked home from school.

Bienvenu (1988) undertook hand augering in front of the school in 1987 and discovered coarse clean sand containing some well-rounded pebbles at a depth of between 2.3 and 2.8 m. She states that the presence of pebbles supported reports that stones were ejected along with sand and water.

Mrs. Maisie Richardson (1989), a pupil at Karamea School at the time of the earthquake, said that there were a lot of spider web sized cracks in the road by the filling that had split right open. She also recalled that there were huge cracks in the paddocks on both sides of Waverly Street that had mud all over them. These cracks extended from the overflow out towards the wharf. Sand boils and waterspouts were also widespread over this area, especially in Simpson's and Fensom's paddocks (Maisie Richardson, pers. comm., March 2003). Mrs. Averil Hay (1989) remembered that there were geysers all over Fensom's paddock, located between the roadway and overflow. This paddock is adjacent to Waverly Street and opposite the school site. Mr. Harry Simpson (pers. comm., January 2003) also recalled that geysers and sand boils were widespread over the school ground and in Simpson's paddocks. He walked past these on his way back to the Karamea Hotel which was owned by his family at the time. These are next to the Karamea Area School and the Domain. (This land is currently owned by Mr. Ross Scarlett, and managed by Mr. Arthur Perkin.)

Karamea Wharf

The effects of the earthquake on the wharf at Karamea caused considerable consternation to the residents of the area since shipping was the chief means of obtaining provisions as well as exporting logs and farm produce. Apart from the road to Westport, which was damaged and remained closed for 15 months (Mr. Roy Allen, 1989), the wharf provided the main access to the outside world.

Henderson (1937) stated that, "*At Karamea a considerable area north of the township moved toward the river, distorting the wharf and training wall and disrupting a tramway*". Further he writes that, "*At Karamea the slumping of the uncompacted alluvium threw the wharf and training wall out of alignment..*". This is also noted in Greyland (1957). Frank Wood (1929) simply stated that, "*..the wharf is done for..*". Later he wrote, "*The* [railway] *line that carries stone for the new Break-wall is twisted all shapes and I think it is sunk altogether at the end*".

Mr. Jim Richardson (1989) wrote the following to illustrate the effects-, "*Phil Ball.. would come down the road, turn right, then back onto the wharf with two locks of his wheels. This morning he missed, cursed himself for being such a fool, pulled ahead again to get straight on to the wharf, looked around and found the wharf was gone about 2ft behind his back wheels. It was in the river!*".

The Nelson Evening Mail ("Postmasters' Reports", Friday June 21, 1929, pg 5) initially gave this information by stating that the wharf was leaning into the river and the training wall at the river mouth was considerably damaged in the earthquake. Subsidence was also responsible for the damage to the wharf sheds. This was also recorded in *The Sun* ("Karamea a heap of Ruins", Friday June 21, 1929, pg 10). This is a clear example of the effects of lateral spreading.

Figure 3-9. Karamea Wharf- circa 1923. (Alexander Turnbull Library)

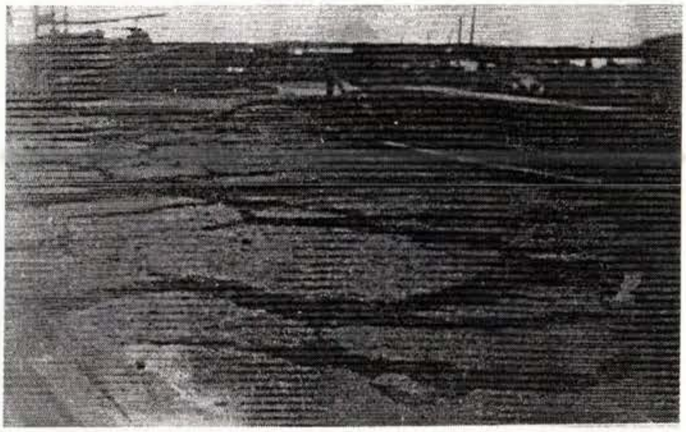


Figure 3-10. Near Karamea wharf after the earthquake (photo courtesy Karamea Museum)

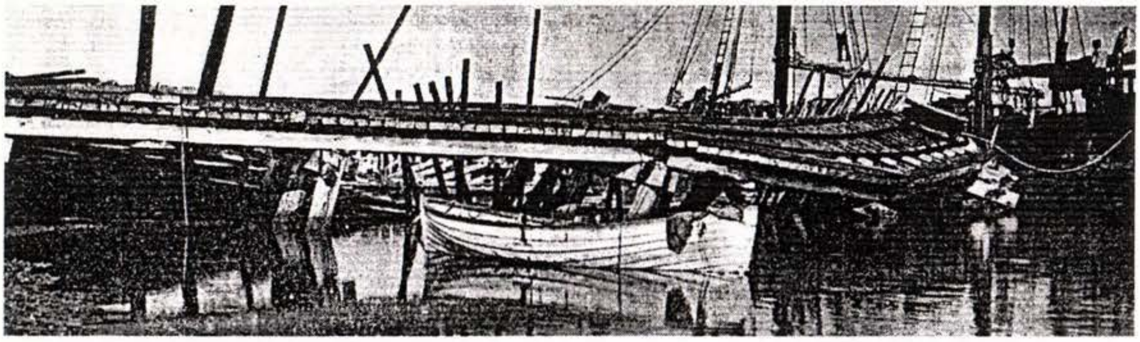


Figure 3-11. Karamea Wharf following the earthquake (*Auckland Weekly News*, 3 July 1929)

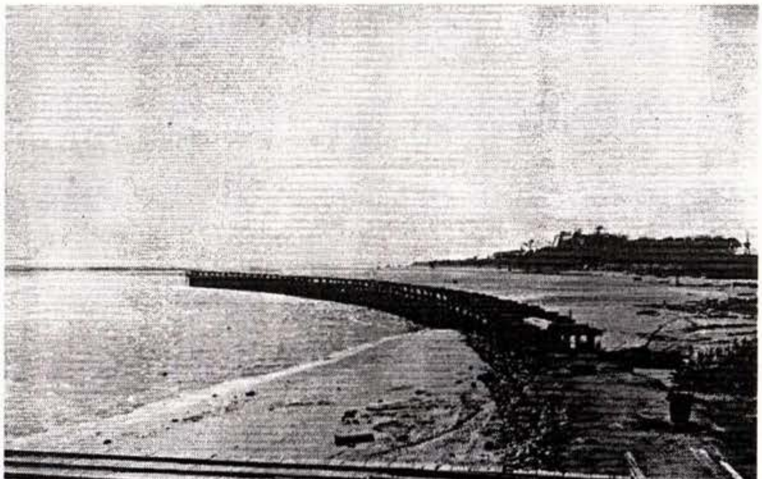


Figure 3-12. Karamea Training Wall seen from the Wharf looking North prior to earthquake (Alexander Turnbull Library)

Figure 3-13. Remains of the Karamea Training Wall, North of the Wharf site, January 2003

Arapito

Mr. Cyril Lineham (1989) who was a school boy at the time of the earthquake, "*remembers seeing sand and water together with some gravel ejected in the land to the south of the present road at Arapito*", at what is now Scarlett's farm. According to Bienvenu (1988), Cyril Lineham remembered, "*seeing during the shaking many stones about 20 or 30 mm in diameter, with sand and water, being ejected from the paddock now owned by Chris Anderson. He said also that this paddock has probably not been ploughed since 1929. Also, his father's well was filled with 'mud', which was probably fine sand*".

Bienvenu also undertook hand augering at this site and it was discovered that very coarse sand containing rounded pebbles was located at depths between 2.0m and 2.5m. It was found from scala penetrometer tests, that in general, there were very soft layers of soil.

The effects of the earthquake on the land surrounding the gravel pit on Sid Simkin's property were detailed in the letter written by Frank Wood. It was reported to Mrs. Dulcie McNabb by Mrs. Maisie Richardson, that Sid Simkin lived at Arapito, past Elfords Creek and over the back on the side of the terrace (pers. comm., June 2003).

> "*The land all round the gravel pit at Sid Simkin's place has shifted away and sunk for a chain or two, and the pit itself has nearly filled up,-just as though someone had taken a pot of porridge and given it a shake to level it up*". -Frank Wood, June 1929

Umere

Mrs. Lilywinter (1989) recalled cracks in the road as she walked to Umere from Karamea School after the earthquake, but did not give any exact locations. Mrs. M. Lowe (pers. comm., January 2003) also recalled fissuring in Umere, but no sand boils.

Kongahu Mudflats - Between Maori Point and Kongahu

The account from *The Grey River Argus* ("Karamea Disasters", 25 June 1929) stated that "*Jones's residence at Kongahu was split into three pieces.. Some hundreds of geysers spouted up from the mudflat*".

Bienvenu (1988) reported Mr. Karl Jones telling her that his grandfather, "*was fishing on the expanse of salt water situated between the road and the sea behind Mr Jones's Farm at Kongahu, some 10 km south of Karamea. He was still on the lake when the earthquake occurred and was very frightened by big geysers of mud around him*". This is near point 17 in Figure 3-3.

Mrs. Dulcie McNabb, recounted Mr. Ken Kees as recalling that the Kongahu Swamp, between the back road and the main road, "*rolled in great waves, with geysers erupting everywhere and leaving behind great areas of sand. Later when farmers were working up the land they came across all these sandy areas*" (pers. comm., September 2003).

3.4.2 Little Wanganui

It was reported by Berrill *et al.* (1988) that both Mr. and Mrs. Duncan remembered seeing sand boils in the locations illustrated by the red areas shown in Figure 3-14. Bienvenu (1988) wrote, that Mrs Duncan remembered seeing geysers of water ejected from the cracks formed by the moving ground. No further details regarding this have been found.

3.4.3 Seddonville

The headmaster of the school at Seddonville, Mr. G.H. Beilby, stated, "*when we emerged from the building we found the playground one quivering mass with geysers of mud and water everywhere, and large cracks opening everywhere. I thought another Rotorua had broken out*" (*The Nelson Evening Mail*, "At Seddonville", Saturday June 29, 1929, pg 8). The Seddonville School was then located at the site of the current Seddonville Camping Ground. This information was confirmed by Mrs. Dulcie McNabb of Karamea, and other local residents.

Sand boils were also reported at Corby Estate, Seddonville, on the banks of the Mokihinui River, between the homestead and the Seddonville Road (Berrill *et al.*, 1988) as shown in Figure 3-15.

3.4.4 Mokihinui

The Press ("Frightful Havoc", 22 June 1929) reported Mr. J. Niven, a member of the Buller County Council in 1929, as stating that the Kynnersley Hotel at the mouth of the Mokihinui River, "*had been lifted up several feet in the middle by a geyser of water*". Sand boils and waterspouts were present all over the beach and the water rose to a height of 10 feet. It was noted that this phenomenon caused the destruction of the football ground at Mokihinui. Mrs. Rose Duncan recalled in the 1989 "60th Anniversary Reunion" Tape, that there was an "*up thrust*" at Mokihinui of about 70 acres out of an area of 100 acres.

3.4.5 Westport

Following the 1929 Murchison Earthquake, there were a number of incidences of liquefaction in the Westport area. As was noted in Chapter 2, Westport is situated on a coastal fan and most of the occurrences of liquefaction were on the banks of the Buller and Orowaiti Rivers. Figure 3-16 details the locations of liquefaction which were located in Westport based on evidence and recollections.

As with Karamea, the wharf at Westport suffered due to the earthquake. According to Henderson (1937) "*Fissures opened parallel with the water front for chains back; the wharf warped slightly; the railway-lines twisted; and the long straight of the break-water mole showed gentle swings from side to side and up and down as the embankment settled unevenly on the uncompacted estuarine deposits*". Following the earthquake these details were reported in *The Nelson Evening Mail* ("Westport Suffered Severely", 18 June 1929). It was written in *The Press* ("Stricken Town - Bad Plight of Westport", Wednesday June 19, 1929), "*Down at the wharves the lines were bent and the wharf itself is knocked out of alignment*".

The Grey River Argus also reported on the effects in Westport and the surrounding areas.

> "*A four-foot fissure is reported to have opened up at Orowaiti, about two miles from town, and another deep fissure is down Romily Street. On the road to the mines large fissures appeared on the road, and one bridge pier has lifted about five feet... Extensive fissures are reported on the road to Cape Foulwind, and one bridge has been completely demolished and others badly damaged.. A fissure is reported to have opened up in one of the back streets in Westport..*"
>
> -*The Grey River Argus*, "Ten Killed near Murchison", June 19, 1929

This excerpt was also reported in *The Nelson Evening Mail* on Tuesday June 18, 1929 in an article entitled "Fissures in Roads".

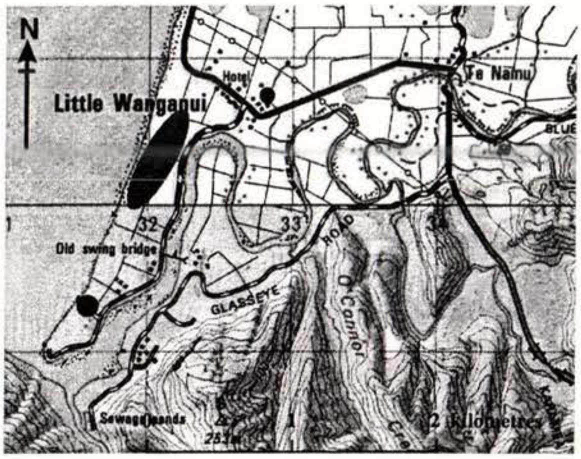
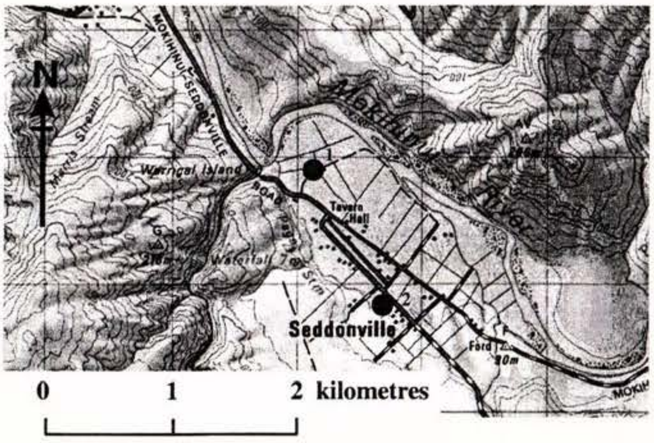
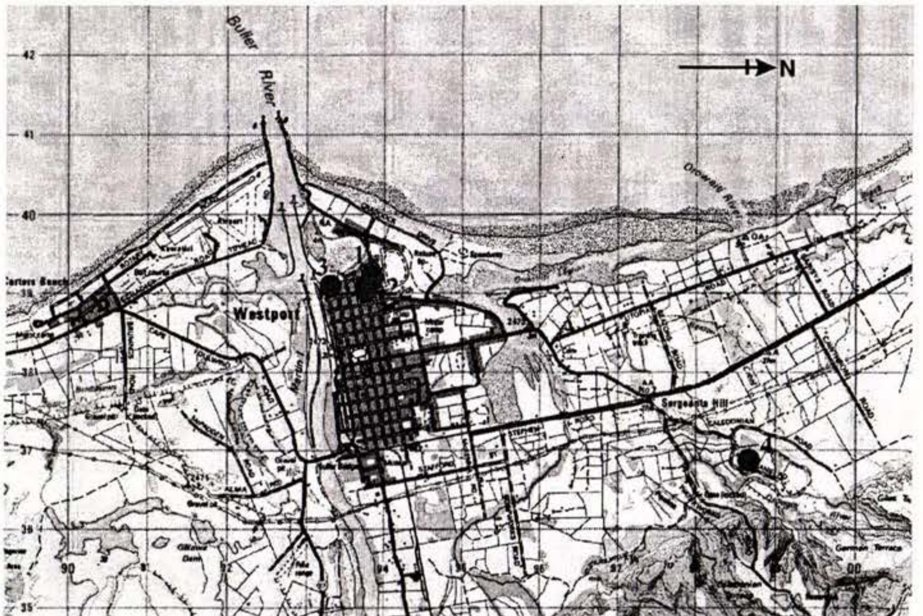


Figure 3-14. Sites of liquefaction at Little Wanganui, 1929 (from Berrill *et al.*, 1988; Base map: Topomap, 2001)



1. Corby Estate
2. Seddonville School

Figure 3-15. Sites of liquefaction at Seddonville, 1929 (after Berrill et al., 1988; Base map: Topomap, 2001)



1. Derby Street
2. Romily Street
3. Wharf
4. Keoghan's Farm

Figure 3-16. Sites of Liquefaction in Westport, 1929 (Base map: Topomap, 2001)

A letter written by Mr. Swinburn, the Borough Engineer, to the Mayor of Westport (dated 15th August 1929) details the damage to the water supply mains, sewers, waterworks headworks and other services. In the letter, currently located at the Christchurch Branch of Archives New Zealand, Mr. Swinburn writes, "*In Derby Street from Cobden to Gladstone Street, a distance of some 15 chains, the 9" main sewer is badly broken and some 30 house connections will have to be renewed.. In Romily Street between Cobden and Bright Streets some 5 chains of 9" main sewers have been broken and about 12 connections*". Numbers 1 and 2 in Figure 3-16 illustrate these locations.

Another occurrence was at Keoghan's Farm, at Sergeant's Hill, north of Westport. Berrill *et al.* (1988) wrote, "*from 1929, when he was a boy of 11, Mr Keoghan remembers geysers of sand and water 5 to 6 feet high, leaving sand cones 2 to 3 feet high*". This occurred in the paddocks behind Mr. Keoghan's house according to Bienvenu (1988). Bienvenu also noted that there seemed to be successive deposits of fine and coarse sand laid down by the Orowaiti River.

Benn (1992) also quotes Palmer (1970), who had a farm at the Orowaiti River near Westport as saying, "*I ran to the house.. and had to jump cracks in the ground which had, in some places dropped a foot or more and water was being forced up through the cracks*".

3.4.6 Murchison

General

Murchison has generally been considered the town closest to the epicentre of the 1929 Murchison earthquake; hence the name was taken from this town to describe the earthquake. Due to the proximity of the town to the fault rupture, town people saw some of the worst effects of the earthquake, including the landslides, which killed a number of people in outlying areas. In comparison to this loss of life, and the general destruction of the township, sand boils and other features caused by liquefaction would probably have been be considered insignificant. At the time of the earthquake, indeed even now, Murchison residents mainly relied on farming, sawmilling and the associated rural service industries for their livelihood.

As was seen earlier, with the occurrences of liquefaction in Karamea, most of the liquefaction in the Murchison region occurred on the flood plains of the Buller River. The sites of liquefaction discussed in the following sections are shown on the map in Figure 3-17.

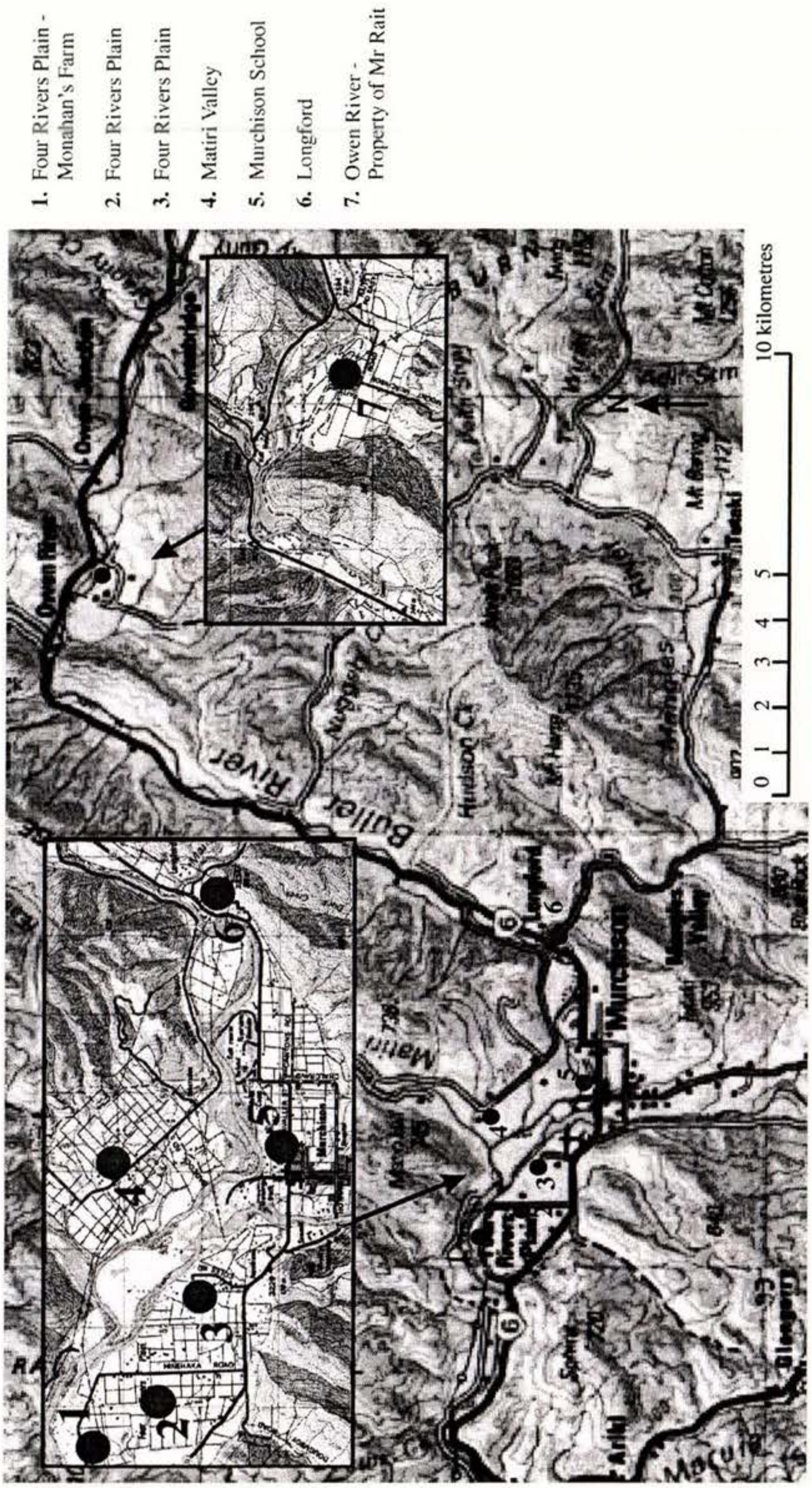


Figure 3-17. Sites of liquefaction in Murchison, 1929 (Base map: Topomap, 2001)

The following reports of liquefaction effects could not be identified from the descriptions given.

Barbara Oxnam noted cracks and waterspouts but did not give further details, whereas Keith Miller noted spouts of water on the flats, possibly at Four Rivers Plain. Mr. Miller attributed the waterspouts to the pressure exerted below the ground (Murchison District Historical and Museum Society Inc., 1999).

The Nelson Evening Mail (date and article title unknown) reported Mrs F. M. Burnett observing water spouts up to five metres high occurring on the river flats, though no exact location is given. This again is possibly describing events at Four Rivers Plain, however, senior citizens of Murchison could not definitely confirm this.

It was also reported in *The Nelson Evening Mail* ("Refugees at Christchurch", Saturday June 29, 1929, pg 7), that the ground opened up at the back of one man's house and sulphur fumes were emitted from the cracks that occurred as a result of the earthquake.

The photo of the camp at Murchison shown in *The Evening Post* (Thursday June 27, 1929), possibly indicates liquefaction with either sand or water in the front of the photo. The camp was located in the school grounds, though following the earthquake there was also a lot of rain. This photo has also been reproduced in the Murchison District Historical and Museum Society Inc. Booklet on the earthquake on page 2.2 and Figure 3-18.

Figure 3-18. School Grounds at Murchison following the earthquake (Murchison District Historical and Museum Society Inc., 1999)

In an article in the *New Zealand Free Lance* ("How Murchison Weathered the Quake", June 25, 1929) it was stated that roadways were split with large fissures, bridges were demolished and isolated sulphur and hot mud-spouts appeared, but once again no specific details were given.

Owen – Glenhope

Morva Borcovsky (Murchison District Historical and Museum Society Inc., 1999) wrote that on the way from the family house on Mr. J. Rait's farm to Mr. Rait's own house following the earthquake there were spurts of water emitted from the cracks in the ground. The number '7' in Figure 3-17 indicates this site. Raits Road leads down to the farm.

Tutaki – Longford

"*Water kept gushing up like geysers everywhere. Geysers in the river were as high as the poplar trees along the opposite bank of the river*", according to Coralie Canton (nee Oxnam). It was also noted that Mr Oxnam, "*concreted the yard round the back of the house as he thought it unsafe because of the many places where marine water and gravel had spouted up from the ground*" (Murchison District Historical and Museum Society Inc., 1999). The site where the Oxnam's house stood in Longford is located on the map shown in Figure 3-17. There are a number of locations that look potentially liquefiable, between Longford and Tutaki, and hence it is hard to ascertain an exact location based on this article.

Matiri Valley

At the Matiri Valley, settlers were reported as observing cracks opening up chains in length and over a foot in width. In *The Nelson Evening Mail* it was reported that, "*patches of ground were pushed up above the surrounding level on what before the big shake was a dead flat. On Mr. A. O'Rourke's farm a lakelet, about four acres in area, surrounded his homestead after the shake, the water being up to about 2 feet in depth and having apparently been squeezed up out of the ground. Mr. O'Rourke states that pieces of blue metal* [possibly gravel] *as big as a man's fist were forced up through the grass in the paddocks. Another settler in this locality Mr. F. L. Edwards reports similar experiences*" ("From the Back Country", Monday June 24, 1929, pg 5).

Four Rivers Plain – Fern Flat

During the earthquake Malcolm Brown (Murchison District Historical and Museum Society Inc., 1999) observed cracks opening and closing in the paddock with water from underground sources spurting out of them. He stated that, "*after a little while of this, the paddock looked as if there had been a flood*". According to Mrs. Bradley (pers.comm., April 2003) of the Murchison Museum, Mr. Brown was living at Four Rivers Plain at the time of the earthquake.

Berrill *et al.* (1988) reported further sites of liquefaction on Four Rivers Plain. "*Mr T. Monahan remembers seeing sand boils distributed over the flat land around his house at Four Rivers plain west of Murchison, and seeing isolated sand boils over most of the Four Rivers Plain area after the 1929 earthquake*". Bienvenu (1988) wrote that the Monahan's farm was situated on an ancient terrace of the Buller River. In 1992, Dou (Dou and Berrill, 1992) undertook further study of this site. Dou spoke to Mr. Hugh Monahan who was living on the farm at the time of the 1929 Murchison earthquake, while his younger brother, Mr. T. Monahan, was away in Christchurch at school. Mr. H. Monahan recalled that the sand boils were located in a paddock to the east of Mr. T. Monahan's house, rather than on the western paddock as previously thought. Dou undertook further site investigation in the paddock to the east of the house, and found that there was very loose medium to coarse sand at a depth of 2m, which was beneath the water table. A scala penetrometer test was conducted next to the hand auger site as the hole collapsed at a depth of 2.7 metres. This test indicated a layer of gravel at a depth of 4.4 metres. In comparison, hand augers undertaken by both Dou and Bienvenu indicated gravel at depths between one and 1.7 metres, and loose sands above this, with no indication of the water table. Dou commented that this indicated that the paddocks to the east of the house were more susceptible to liquefaction than those to the west.

Dou also undertook hand augering in the paddocks to the north of Monahan's farm. There was no evidence of liquefaction at this site following the 1968 Inangahua earthquake and it is not known if the soil at this location liquefied in the 1929 Murchison earthquake either. Dou found that the soil was very loose but that the water table was significantly lower and this may account for the lack of liquefaction.

Alma Peacock, in an article written for the Murchison District Historical Society Booklet, recalled that as her family travelled along the road to Four River Plain, a quarter-mile from Fern Flat school, "*..on the road, full of cracks opening and shutting as the tremors came and went, we saw spurts of water rising up to four feet or a little more as the earth shook, like a lot of little geysers*" (Murchison District Historical and Museum Society Inc., 1999; Badcock, 1979).

Lower Maruia – Glengarry – Ariki

Charles (Chum) James who was at a school in the Maruia Valley noted that on flat ground cracks opened and closed, and from these water spouted out (Murchison District Historical and Museum Society Inc., 1999). The precise location of this is not known.

A handwritten journal by Alice McWha (date unknown), located in the Alexander Turnbull Library, details her experiences in the Lower Murchison area due to the earthquake. She wrote the following, though the exact location has not been ascertained.

> "*.. then at the road, the cracks were across the road, just as tho cut across by a hingne* [??] *knife while just opposite the cream stand, at the gate there was a spring boiling up and bringing up blue clay mixed with blue sand, the same as what is at the bottom of the hill down at the waterfall bridge, over 100 feet below, there were 3 more like this opening and 1 five feet long across the road, the five springs were all within 2 chains, the road had cracked badly below that...*". -Alice McWha, 1929

3.4.7 Lyell

Mrs. K. Lammas noted, "*the river was dry too with only mud on the bottom with little geysers shooting up out of its bed. Even through the hard metalled* [gravelled] *surface of the road these geysers sent up sprouts of sand and mud*" (Murchison District Historical and Museum Society Inc., 1999; Badcock, 1979). In the book *High Noon for Coaches* mention was made of Mr. Dick Powley, a taxi driver, who was driving in the same area. No mention was made of the geysers in this volume.

3.4.8 Inangahua

The settlement of Inangahua also suffered from liquefaction. The township is situated on flood plains near the confluence of both the Buller and Inangahua Rivers. "*At Three Channel Flat, geysers of sand and water were shooting up twelve feet high through cracks in the earth*" (*Inangahua Herald*, "Inangahua District", Wednesday June19, 1929). It was written in *The Sun* ("Coast still Shaking, but no further Damage done", Wednesday June 19, 1929, pg 9), that "*..Paddocks at Three Channel Flat are covered with white sand. At Flaxbush Creek a whole hillside slipped away*" and later in the same article it is noted that, "*The Inangahua Junction Hotel suffered badly. Cracks in the ground fronting the hotel exuded dirty water, while the shock threw men to the ground*".

Benn (1992) cites Berrill *et al.* (1988) for examples of liquefaction. This was based on a similar article in the Wellington *Evening Post* (June 19, 1929) that reported paddocks at Three Channel Flat were covered with white sand.

Mr. James Jenks, a surveyor at Inangahua, stated, "*great chasms opened and yawned at me and into these chasms the river poured, and the river was bone dry. The next instant they were closed up and up shot the water many feet into the air, like geysers*" (*The Nelson Evening Mail*, "Great Chasms Laid Open", Friday June 28, 1929, pg 5).

3.4.9 Greymouth

Some the most remarkable features of the 1929 earthquake noted in Greymouth, occurred in Blaketown. It was written in the *Grey River Argus* ("A Terrific Earthquake from West Coast to Nelson", June 18, 1929) that the damage resulting from the earthquake was worse on the western side of the town, in the vicinity of Blaketown, than other areas. Figure 3-19 shows the extent of the damage in Greymouth, where most occurred in areas of reclaimed land, or where the land has been built up by the combination of longshore deposition and sand trapped by the effective groyne feature, the wharf and breakwater. It is probably worth noting that this material is likely

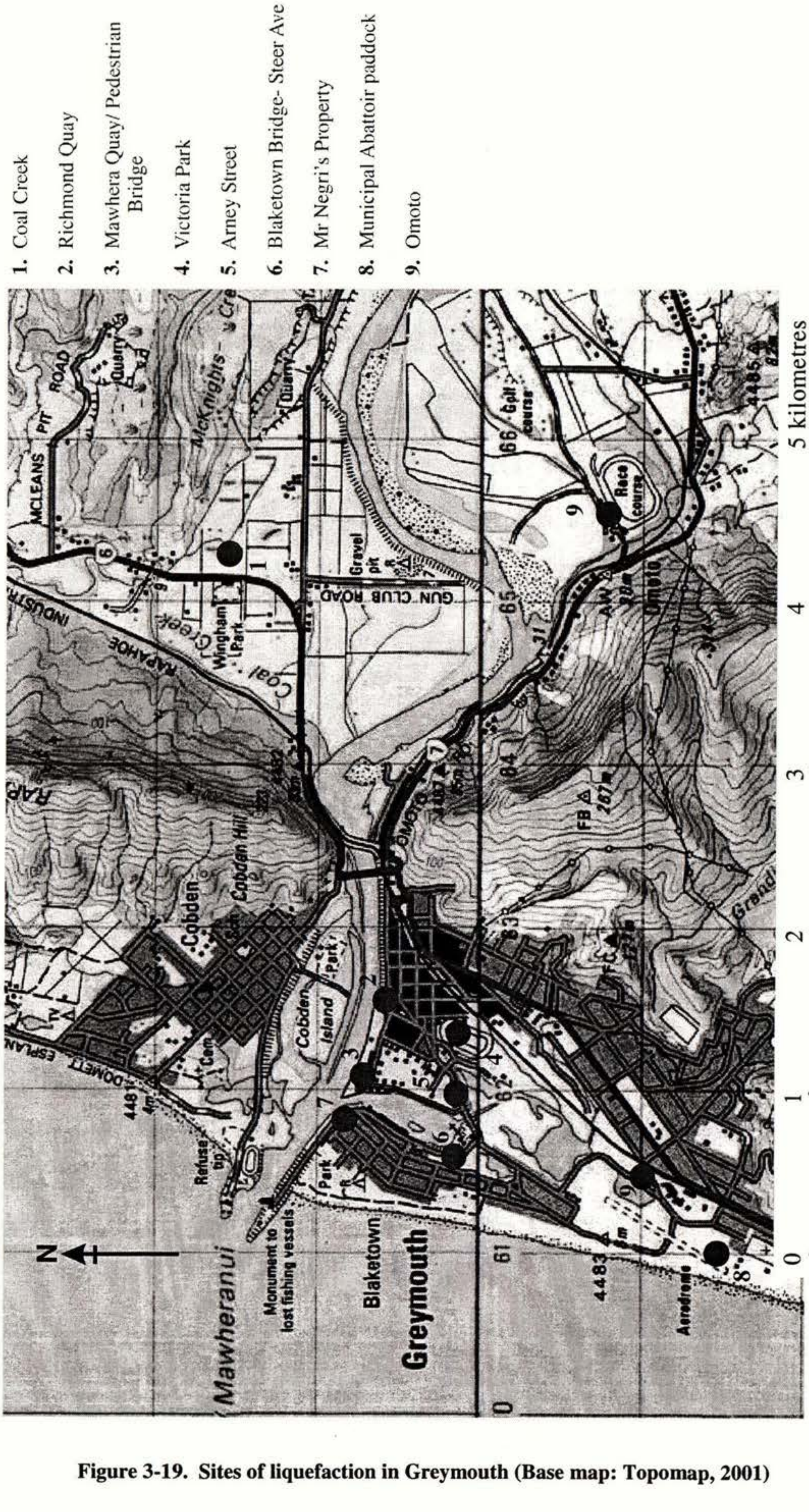


Figure 3-19. Sites of liquefaction in Greymouth (Base map: Topomap, 2001)

to be less compacted than sand dunes built up solely by wave action as the material is trapped in a less energetic environment.

The Nelson Evening Mail ("Greymouth", Thursday June 20, 1929, pg 5) reported that, "*fissures opened in the road at Omoto, near the racecourse*". This is indicated by number 9 in Figure 3-19.

Coal Creek

In an interview with the writer, Mrs. Batey, a resident of Dixon House in Greymouth, recalled her husband telling her about the geysers and sand boils coming out of the paddock on their farm. At the time of the earthquake Mrs. Batey was housebound but clearly remembered that the sand boils and geysers occurred in the paddocks opposite what is now Wingham Park and as shown by number 1 in Figure 3-19. Unfortunately, no further details could be located. The land is currently farmed by the Jones family.

Richmond Quay

Both *The Press* ("Violent Quake- Railways, Houses and Bridges Suffer, Tuesday June 18, 1929, pg 11) and *The Nelson Evening Mail* ("Greymouth", Thursday June 20, 1929, pg 5) had similar excerpts. It was noted in *The Nelson Evening Mail*, "*The footpaths on the seaward side of the town now exhibit fissures in all directions. There are long narrow ones along Richmond Quay, one extending for over 50 yards* [approximately 43 metres]". *The Grey River Argus* gave greater details pertaining to this, writing:

> "*Proceeding thither via Richmond Quay, the first thing to be noted was a series of cracks starting at the Criterion Hotel and extending down nearly to the goods sheds, whilst along the wharf the planks were separated from the earth by the quake. Crossing over the lagoon every footpath was seen to have repeated cracks in the asphalt, showing the shocks were violent near the sea front*".
>
> -*Grey River Argus*, "A Terrific Earthquake from West Coast to Nelson", June18, 1929

On the day of the earthquake *The Greymouth Evening Star* ("Greys Worst 'Quake- Ground Cracks", Monday June 17, 1929, pg 5) reported that narrow cracks occurred in the "*roadway inland from the waterside*" down river from Boundary Street. It was also noted in the same article that the oscillations caused the cracking of the footpath asphalt, near the Grey County Offices.

The water main feeding Blaketown was in the river bank by Richmond Quay and at first it appeared considerably damaged. In the *Grey River Argus* (June 18, 1929, Article: A Terrific Earthquake from West Coast to Nelson) it was noted that, "*There was not a break in the cast-iron pipe, but a joint pulled out in Richmond Quay due to the roadway settling towards the river*". Men working to repair breaks in the water mains on Mawhera Quay were shown in a picture in the *Auckland Weekly News* (July 17th, 1929). Mr. George Webster (pers. comm., April 2003) confirmed this, and stated that the footbridge to Blaketown was at the end of the Quay. The approach embankment to the footbridge appears to have settled from the photo in the *Auckland Weekly News* shown in Figure 3-20.

Figure 3-20. Blaketown Footbridge- note the settlement of the approach embankment (*Auckland Weekly News*, 3 July 1929)

The *Grey River Argus* ("A Terrific Earthquake from West Coast to Nelson", June 18, 1929) also reported the disturbance of the Grey River.

> "*Mr G. Howe of the Harbour staff, observed at the time of the earthquake that the river was affected. Water and debris spouted up for a considerable distance, and later the river fell some inches near the goods shed. The disturbance of the water lasted for some time after the major shake.*" -*The Grey River Argus*, 18 June 1929

Arney Street

In the *Grey River Argus* (July 5, 1929) in an article entitled "Quake Damage" it was written that there were cracks in Arney Street near the bridge.

Some long-term residents of the area also noted that when the Grey River floods and breaches the stop banks this street is one of the first to be flooded.

Victoria Park

The Nelson Evening Mail ("Greymouth", Thursday June 20, 1929, pg 5) also reported fissures near Victoria Park. The location of Victoria Park is shown by number 4 in Figure 3-19.

Preston Road/ Steer Avenue

It was stated in *The Dominion* (24 June 1929) and shown by photographs that there were cracks in the ground in Preston Road. Dowrick (1993) inferred that this was on made ground in a swampy area. The *New Zealand Free Lance* printed the following picture (Figure 3-21) to illustrate the fissures and the creamy silt and salt water that they stated came through these cracks.

Figure 3-21. Cracks in Preston Road at Blaketown (*New Zealand Free Lance*, 26 June 1929)

A number of newspapers wrote similar articles regarding the Steer Avenue area.

> "*..a fissure was caused near the traffic bridge, and it was such that it held up motor cars. People who were in the vicinity state that when the 40 ft crack opened up there, it appeared as though steam escaped. Since the main shock at 10:19 am there has been a series of slight tremors at irregular intervals, but none of them have been sufficient to cause a further exodus into the streets*"
>
> -*The Greymouth Evening Star*, "Greys Worst 'Quake- Ground Cracks", Monday June 17, 1929
>
> "*Every footpath in Blaketown had its asphalt cracked. The main traffic bridge leading to Blaketown was rendered impassable through the road falling at either end, the drop on*

> *the seaward side being about two feet. Three deep fissures, extending each a distance of nearly a hundred feet and widening in places to more than a yard, opened up in the road, and there is a narrow one on the landward side. The bridge itself was evidently pushed westward fully a foot, causing the caps on the piles at that end to split and turn over*"
>
> -*The Nelson Evening Mail*, "Greymouth", Thursday June 20, 1929

A few other details were given in the *Grey River Argus* ("A Terrific Earthquake from West Coast to Nelson", June18, 1929) where it was written:

> "*The bridge lifted on the south side for a few inches at the eastern end, and long narrow fissures were formed in the road leading towards the town. A Power Board high tension pole near the Preston Road bridge was dislocated, and hung at a tipsy angle.*"
>
> -*Grey River Argus*, 18 June 1929

The Greymouth Evening Star ("Greys Worst 'Quake- At Blaketown", Monday June 17, 1929, pg 5) informed readers that the roadway had been, "*built up above the ground level*".

The photo of Steer Avenue, shown in Figure 3-22, was printed in the *Auckland Weekly News* on the 26 June 1929 to illustrate the effects in Blaketown. Mr. George Webster, a taxi driver in Greymouth at the time of the earthquake, recalled this incident (pers. comm., April 2003) and could even identify the owners of the houses in the background.

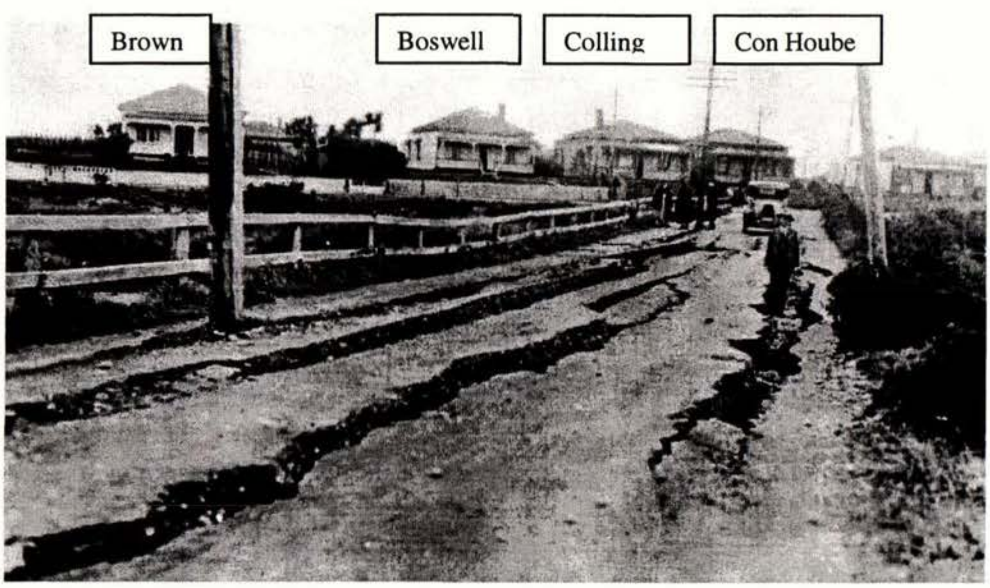


Figure 3-22. Steer Avenue, Blaketown. Photograph looking towards the coast. (Alexander Turnbull Library)

Municipal Abattoir/ Preston Road

Benn (1992) relates part of an article from *The Greymouth Evening Star* (June 18, 1929), "*Between Preston Road and the Municipal Abattoir paddock, scattered amongst the green turf and sand tracks are small conical shaped heaps of blacksand, forced up through vent holes. Beyond the base of each is a layer of straw coloured mud. The sand is of an entirely different texture to that around the lagoon waters. The vent was strong enough to force a heavy corner post entirely out of the ground... around the head of the lagoon near the hospital, sand and mud churned and quicksands formed*". This is indicative of sand being forced up from below the ground surface. More importantly though, for a heavy post to be forced out of the ground, the soil must have liquefied to near the ground surface. A similar passage was written in the *Grey River Argus* ("Ten Killed Near Murchison", 19 June 1929), *The Sun* ("Coast Still Shaking, but no further damage done", Wednesday June 19, 1929, pg 9), and the *Evening Post* (June 19, 1929).

The Greymouth Evening Star ("Lagoon 'Volcanoes'", Tuesday June 18, 1929) wrote, "*Extensive cracks have been formed in the big Abattoir paddock and continue along toward Preston Road, probably in continuity with the cracks at the northern end of Blaketown*". Three horses were also 'bogged' at the head of Preston Road in the quick sand caused by the earthquake, according to the *Grey River Argus* ("A Terrific Earthquake from West Coast to Nelson", June18, 1929).

According to Mr. George Webster (pers. comm., April 2003) you could walk over the ground in front of the hospital when it was dry, however, if it was wet it was just like a quicksand.

Corner of Blake and Collins Streets (Mr. Negri's Property)

What would have been one of the most alarming occurrences for residents of Blaketown occurred at Mr. T. Negri's property. This occurrence was well reported in the daily newspapers.

> "*At one residence in Blaketown belonging to Mr T. Negri, cracks four feet wide opened in the backyard, and spectators were considerably alarmed to see water, mud and sand pouring into the section. The inundation ceased and left a coating almost a foot deep on the back of the yard. A wash-house was also flooded. Several cracked portions of the road and footpaths bear ample evidence of the quake's severity..*"
>
> -*The Greymouth Evening Star*, "Greys Worst 'Quake- At Blaketown", June 17, 1929, pg 5.

This passage was noted by Benn (1992), and was referred to in a number of different articles such as the *Grey River Argus* ("Ten Killed Near Murchison", 19 June 1929).

The *Grey River Argus* ("A Terrific Earthquake from West Coast to Nelson"June18, 1929) gave the following description:

> *"At the property of Mr Anthony Negri the ground subsided and the fissure created was found to have exuded a creamy deposit of silt, which lay over an area of about a tenth of an acre* [i.e. 40 m^2] *several inches deep. Mrs Negri was on the scene at the time, and the fissures surrounded her, so that she did not know which way to turn as the liquid came up and welled around. The subsidence was about ten inches deep. A well, after being blocked up, was nearly filled with water by the quake."*
>
> *-Grey River Argus, 18 June 1929*

The nature of this event was also noted in *The Nelson Evening Mail* ("Greymouth", June 20, 1929, pg 5) where they reported that the deposit was spread, "*over an area of about twenty or thirty square yards*". *The Dominion* (24 June 1929) printed a picture of the cracks in Mr. Negri's back yard at Blaketown. Underneath the picture it was written that the "*water squirted up from these cracks to a height of 6 to 8 feet and covered the yard*". This picture has been reproduced and is shown in Figure 3-23. Mr. George Webster (pers. comm., April 2003) confirmed that Mr. Negri lived on the western corner of Blake and Collins Streets.

An 1879 Plan of the area, drawn up for the design of the wharf and training walls, shows that this property would have been built on reclaimed land, or land which has been built up by longshore drift along the coast.

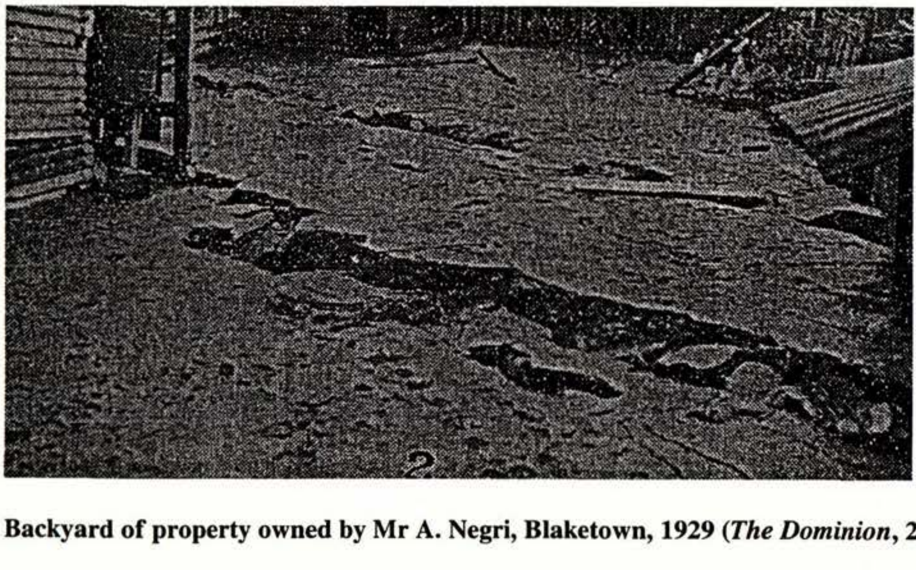


Figure 3-23. Backyard of property owned by Mr A. Negri, Blaketown, 1929 (*The Dominion*, 24 June 1929)

3.4.10 Paroa

A waterspout from a fissure was noted at Paroa, shown in Figure 3-1, in *The Nelson Evening Mail* ("Greymouth", Thursday June 20, 1929, pg 5). Benn (1992) writes using *The Greymouth Evening Star* (18.6.1929) as the source, "*At Paroa, a fountain sprung up in a crack in a residents backyard and was spouting 20 ft (6m)...*". Unfortunately, no further details of this could be found.

3.4.11 Greenstone River

Benn (1992) wrote, "*liquefaction occurred as far south as the Greenstone River at least. It is reported in the Greymouth Evening Star (20.6.1929) that large fissures occurred near the Greenstone Bridge and that: '..curious circular rings in the mud give evidence to some subterranean disturbance'. The same article also mentions that the water in the Greenstone River shot 3 – 4 ft high*". This is one of the most distant records of liquefaction resulting from the Murchison earthquake – about 122km South East of the epicentre (Benn, 1992) and the location is indicated in Figure 3-1.

3.4.12 Motueka- Takaka

It was reported in *The Nelson Evening Mail* ("At Riwaka", Tuesday June 18, 1929, pg 5), that cracks appeared in the ground at Riwaka following the earthquake and from some of these bluish-coloured mud and water was ejected.

Mr. Black, M.P. for Motueka, was reported in *The Nelson Evening Mail* ("Oozing Black Sand", Wednesday June 20, 1929, pg 6), as saying that, "*cold water streams came through the road and paddocks at Motupipi and Riwaka. Water was oozing out of the fissures made on the roads. Dirty black sand, smelling of sulphur, was also exuded. Mr Black had several snapshots in his possession showing the streams*". This was also reported by *The Sun* ("Sulphurous, Black Water oozed out during 'Quake", Thursday, June 20, 1929, pg 11) and *The Press* ("Roads and Bridges", Friday, June 21, 1929, pg 15). These areas are both on river plains, Motupipi near a small estuary. Unfortunately, no exact location can be found at this point and details may now have been lost since the earthquake.

An early observation regarding site effects was noted in *The Nelson Evening Mail* ("Inspection in another area", Saturday June 22, 1929, pg 7). In this article it was stated that, "*although the shock was severe in every district.. it appeared evident that soil conditions modified or intensified the earth movements. The deep, spongy alluvial soil of Brooklyn probably accounted* *for practically all chimneys being shattered in the houses, while in Dovedale valley on the more compact drift soil of clay and gravel the damage was apparently less serious*". It was also noted that in areas where deep alluvial soils occurred such as in Riwaka, Motueka, Puramahoi, Takaka, Brooklyn and parts of Rockville and Bainham, "*seepages were common, mud and water being forced though surface cracks*". A number of residents, 6 or 8 years old at the time of the earthquake and now residing at Abbeyfield House in Motueka (pers. comm., April 2003) disputed this and said that while there were cracks in the ground, they did not recall any mud oozing out of them at Motueka or Brooklyn. However, Miss Lorna Langford (pers. comm., April 2003), recalled mention of mud coming out of the cracks in the ground at Bainham during the earthquake, but unfortunately could give no exact details.

It was reported in *The Nelson Evening Mail* that sand was exuded in a spurting manner like a geyser from one fissure of the many fissures in the streets of Takaka ("Sand Spurts Up", Friday June 28, 1929, pg 5). The only evidence of both cracks and waterspouts in the town of Takaka following the 1929 earthquake came from Mrs. Betty Peart who currently lives in Hokitika (pers comm. April 2003). At the time of the 1929 Murchison earthquake, the family lived about half way down Motupipi Street, as shown in Figure 3-24. Mrs. Peart recalled how her mother thought the water mains had broken, though at the time, there were no mains supplies in Takaka. There was also mud coming out of the ground in the paddocks next to their neighbour's house and later on this ground subsided. Mrs. Peart said that the ground also subsided where Motupipi and Commercial Streets met. This is approximately 110 km North West of the epicentre of the 1929 Murchison earthquake.

3.4.13 Pearce Valley - Pokororo

The Nelson Evening Mail ("Ground Cracked", Saturday June 22, 1929, pg 8) reported that at Pokororo the ground had cracked in multiple directions and Mr. Haycock's raspberry garden, "*had been subjected to a strange disturbance that had left small heaps of bluish grey sand lying about. We picked up some of the sand and found it most astoundingly cold: it also had a sticky feeling as of putty or soft clay*". This site is located by number 11 in Figure 3-1. No further reference to this could be found.

3.4.14 Paturau River

It was reported in *The Nelson Evening Mail* ("Another Experience", Monday June 24, 1929, pg 5), that at Anatoki, near the Paturau River, the "*the earth rose up in bubbles and the river turned muddy almost immediately*".

The Paturau River is a meandering river, indicated by number 1 in Figure 3-1, though the sandy ground nearby contained a lot of stones of boulder size. This can be seen in the photos taken of the area and shown below. Again, no specific location could be found for the waterspouts.

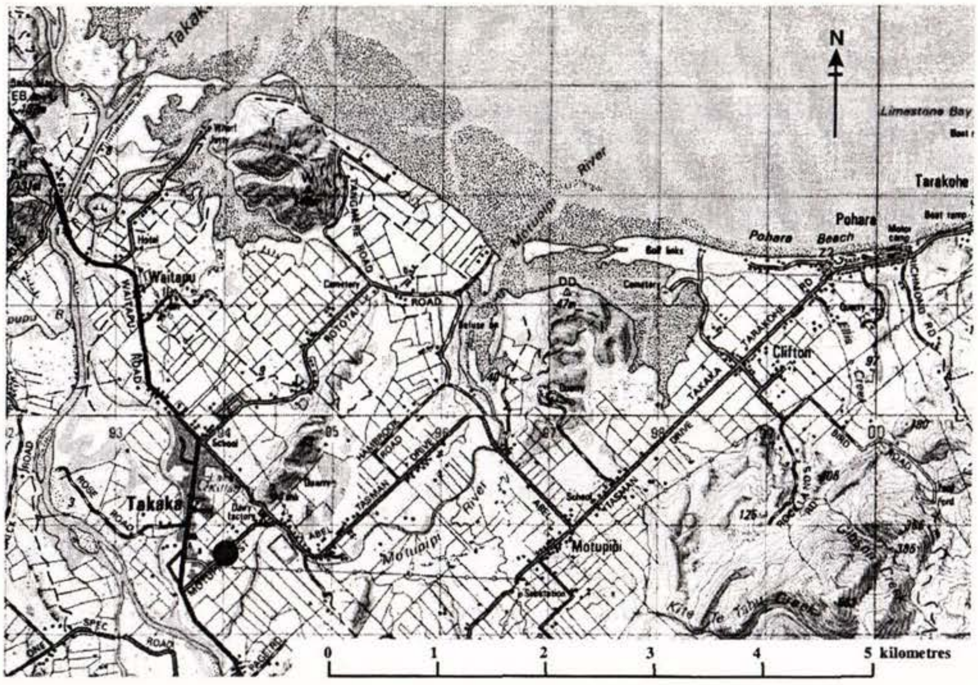


Figure 3-24. Approximate site of liquefaction at Takaka, 1929 (Base map: Topomap, 2001)

Figure 3-25. Near the mouth of the Paturau River (2003)

3.4.15 Kahurangi Point

The Nelson Evening Mail ("At Kahurangi", Saturday June 22, 1929, pg 8) refers to a report of waterspouts by Mr. Page, the Lighthouse keeper at Kahurangi Point. They write, "*Mr Page described one fissure that had opened up near the landing stage at Kahurangi Point. The opening was about four feet wide at the top, and he was unable to reach the bottom with a stick about four feet long. In numbers of the cracks water gushed forth, in some cases several feet* *into the air, and at this particular crack Mr Page felt the water and found it warmer than that in a creek running alongside*". This site is shown in Figure 3-1.

3.5 Summary

Liquefaction was widespread following the 1929 Murchison earthquake in the loose fluvial and dune sands on the West Coast of the South Island. Reports extended from the Paturau River, 120 km north of the epicentre, to the Greenstone River, 122 km south west of the epicentre. These distances are smaller than those indicated by Kuribayashi and Tatsuoka (1975), of distances to the furthest point of liquefaction of between 193 km and 255 km, for a Magnitude 7.8 earthquake.

4 1968 Inangahua Earthquake

4.1 Introduction

The effects of this earthquake were greatly varied. The shallow nature of the earthquake meant that the greatest intensities were felt in the Inangahua district, where few buildings remained undamaged following the earthquake. Photographs taken following the earthquake indicated that landslides and other forms of damage were widespread in the surrounding area, though the effects from liquefaction, such as the formation of sand boils, seemed to be confined to a smaller area. The epicentre was approximately 15 km north of the township and 25 km west of the location of the 1929 Murchison earthquake (Adams *et al.*, 1969).

4.2 Seismological Parameters

Date:	23 May 1968
Time:	17:24:15.6 UT (5:24 am local time)
Magnitude:	7.0-7.1 M_L (Adams *et al.*, 1969)
	7.4 M_S (Dowrick and Smith, 1990
	7.23 M_W (Zhou *et al.*, 1997)
Epicentre:	41.76° S 171.96° E (Anderson *et al.*, 1994)
Depth:	15 km

4.3 Location of the Fault Plane

Dowrick and Sritharan (1993) wrote that the source was a reverse fault rupture, between 15 and 30 km in length. It is noted that the rupture surface dipped between 45 and 50° in a north-westerly direction and that it may have been curved along its strike. In a report written by DTEC Consulting, for the West Coast Regional Council, it was noted that the surface rupture was reported on both the Rotokohu and Glasgow-Inangahua Fault, and was a high angle reverse fault, similar to the fault rupture process that occurred in the 1929 Murchison earthquake. The maximum ground displacement reported occurred near the railway lines at Inangahua on the Glasgow-Inangahua Fault. Adams *et al.* (1968, 1969) wrote "*The epicentre of the Inangahua earthquake lies near the southern limit of the main seismic region of New Zealand, which covers the northern part of the South Island and most of the North Island. In the northern part of the South Island, this fault marks a boundary between two dominant structural trends. To the* *southeast, the major structures share the northeasterly strike of the fault, but to the northwest the dominant strike is more nearly north and south. There is no significant difference in seismicity on the two sides of the fault... Minor seismicity also appears unaffected by the change in geological structure*". Isoseismal maps drawn by Downes (1995) and based on the felt intensity reports are shown in Figure 4-1 and Figure 4-2.

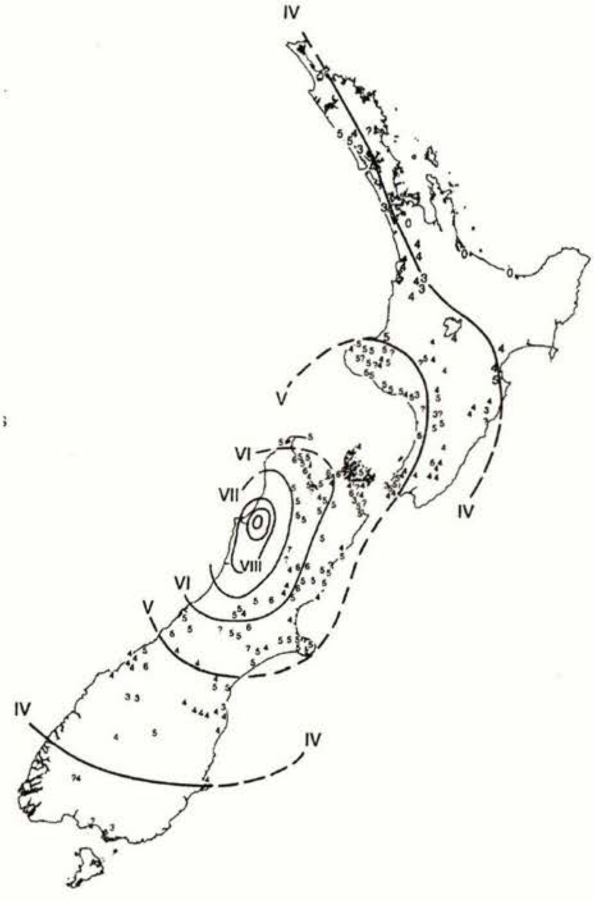


Figure 4-1. Isoseismal Map of the 1968 Inangahua Earthquake (from Downes, 1995)

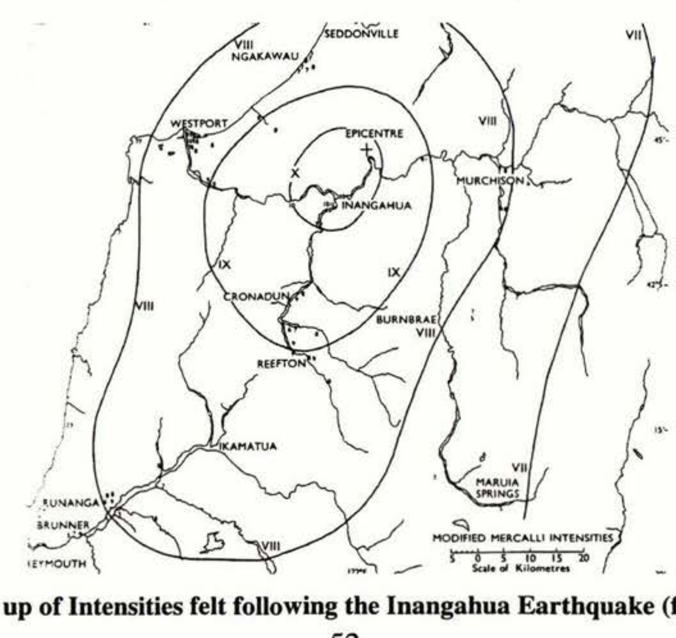


Figure 4-2. Close up of Intensities felt following the Inangahua Earthquake (from Downes, 1995)

4.4 ***Observed Effects***

The Inangahua earthquake seemed to affect the area in a different way to the 1929 Murchison Earthquake. This may be due to the shallow nature of the earthquake, the smaller magnitude, or the different directivity effects. The 1929 isoseismals indicated that the rupture propagated to the North, whereas the 1968 isoseismals were elongated to the South, indicating a rupture propagation focussing energy towards the South. As noted earlier, much of the damage due to slips, liquefaction and general shaking was localised and occurred in the Inangahua – Buller Gorge area and out to the West coast. Some structures were affected as far south as Greymouth but not further north in Nelson. No surface breakage, such as that described as a result of the Murchison earthquake, has been observed for the Inangahua earthquake.

The map in Figure 4-3 indicates the locations of the known sites of liquefaction occurring following the 1968 Inangahua earthquake.

Most of the sites, which liquefied as a result of the 1968 Inangahua earthquake, have been reasonably well studied. Ooi (1987) carried out extensive studies in Westport and Bienvenu (1988), Ooi, Adlam (1988) and Dou and Berrill (1992) all studied sites at Inangahua and Murchison.

4.4.1 Karamea

Eyewitnesses reported that there was no significant damage in the Karamea area; items just moved back and forwards on shelves, according to a number of residents. There was some damage to crockery, glassware and other household items, but overall, financial loss was reported to be small (Myra Perry, 1994).

Mr Peter Sampson, who was living in at Kongahu at the time of the earthquake, did not recall any sand boils, cracks in the ground indicating lateral spreading or other evidence of liquefaction in the Karamea area (pers. comm., May 2002, April 2003). This statement was also supported by Mrs Maisie Richardson, a resident of Karamea at the time of the earthquake (pers. comm., March 2003) who did not recall any cracks forming in the ground or waterspouts. As noted in Chapter 3, Mrs Richardson recalled the liquefaction occurring in the area from the Karamea School out to the wharf in 1929, but could not recall similar effect occurring in 1968.

4.4.2 Seddonville

Berrill *et al.* (1988), noted that sand boils occurred in the backyard of Mr Durkin's house on Excelsior Road. This was thought to have occurred in 1968 and the site is located in Figure 4-3.

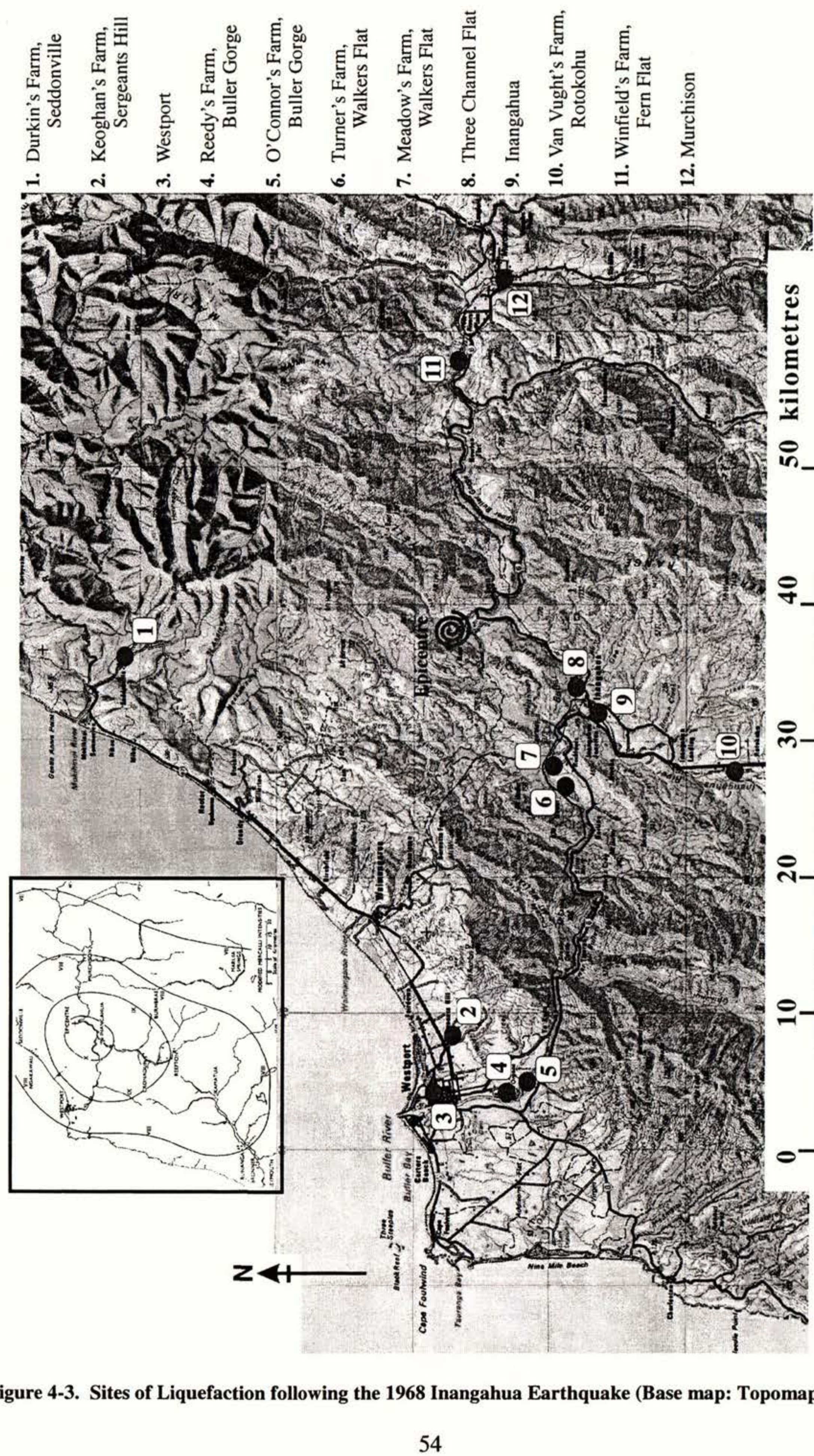
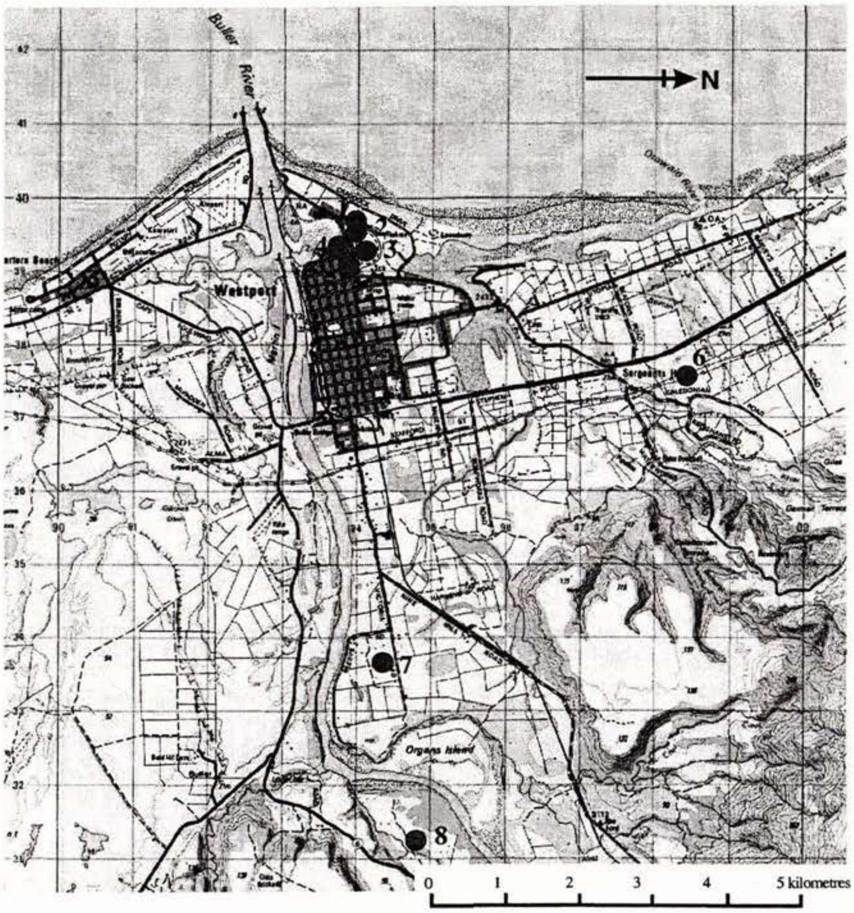


Figure 4-3. Sites of Liquefaction following the 1968 Inangahua Earthquake (Base map: Topomap, 2001)

4.4.3 Westport

The areas of the town built on the old sand dunes and near the Orowaiti Lagoon seemed to be affected considerably more by the earthquake than other areas, effects similar to those observed following the 1929 Murchison earthquake. The known sites of liquefaction in the Westport area are detailed on the map in Figure 4-4 below.



1. Shelswell Street
2. Kilkenny Park
3. Patterson Park
4. Salisbury Street
5. Derby Street
6. Keoghan's Farm, Sergeants Hill
7. Reedy's Farm, Victoria Road
8. O'Connors Farm, Buller Gorge

Figure 4-4. Sites of Liquefaction in Westport following the 1968 Inangahua earthquake (Base Map from Topomap, 2001)

It was stated in *The Press* ("Buildings in Westport Damaged, Roads Cut", Saturday May 25,1968, pg 1) that, "*Three houses in the North Beach area were disjointed and the occupants were surprised to see sea-water bubbling up on lawns a mile from the waterfront, leaving thick layers of sand...... Approaches to the Orowaiti Bridge, the main outlet to the north, and the Waimangaroa overhead bridge were lowered 6in to a foot..*".

It was also written in the same article that, "*At least one residence in the badly-hit North Beach area is beyond repair and sand and seawater continue to seep through some gardens and lawns*" but no specific locations are given.

Following the 1968 Inangahua earthquake the underground services, including both the drainage and water supply, were affected in the coastal areas. This is similar to the damage that occurred following the 1929 Murchison earthquake as noted in Chapter 3. Fairless (1984) cites Andrews (1969) who also noted that buildings were distorted and pavements were also broken in this area. Andrews (1969) also wrote that none of the buildings were damaged to a point of collapse.

Berrill *et al.* (1988), reported in order to reconnect the broken water main, which was located approximately north-south along Derby Street, near Kilkenny Park, 1.2 m of pipe had to be added. Mr Gaynor, who was employed by the Westport Borough Council at the time, supplied the information. Berrill *et al.* (1988) surmised that "*together with the cracking, the broken water main suggests that a large mass of surficial soil had moved towards the coast on a layer of liquefied sand*".

Shelswell Street

Cracks opened in the ground on both the south side of Shelswell Street and at 14 Shelswell Street, where they were found to be passing though the Gaynor's house. Berrill *et al.* (1988) noted that these "*cracks were roughly parallel to Shelswell St, (and to the nearby beach) and were of the order of a few hundred millimetres wide*".

Kilkenny Park

Berrill *et al.* (1988) noted that extensive water and sand ejection occurred in the area surrounding both Kilkenny and Patterson Parks. It was written that, "*Mr M. Dyer remembers seeing numerous sand boils in the paddocks immediately north of Shelswell St and over the land to the west of Patterson Park south of Orowaiti Road... He also remembers boils along the east side of Derby St at Kilkenny Park. Several other eyewitnesses reported sand and water boiling from the ground in Kilkenny Park; these include Mrs MacDonald of Morgan's Lane, and Mr and Mrs Bill Gaynor and Mrs Naylor of Shelswell Street, whose properties back onto Kilkenny Park...*".

Ooi (1987), Bienvenu (1988) and Dou and Berrill (1992) all undertook testing at Kilkenny Park. As part of her research Bienvenu also undertook a number of SPT tests in the park. Ooi undertook a number of cone penetrometer tests and bore holes. From these, it was found that beneath the silty topsoil there were four to five metres of medium dense sand containing lenses of silty sand and dense sand. This was underlain by a one metre thick silty sand layer which in turn was underlain by gravely sand. Ooi found that each of the different methods used to predict the liquefaction potential suggested different layers would liquefy. All the liquefaction potential prediction methods used by Ooi suggested that the top silty sand layer had the greatest potential to liquefy. Dou used this site to check the reliability of the Scala Penetrometer tests he undertook, as well carrying out some CPTU testing.

Patterson Park Racecourse

In *The Nelson Evening Mail* ('Quake damage at Patterson Park ', May 1968) it was written that the ground fissured on the inside training track, and at the entrance to the main straight (which covers an area of approximately 18ft x 24 ft) there was sand and water extruded from the ground. There were also numerous cracks and subsidence of the ground.

Berrill *et al.* (1988) recorded another article from *The Nelson Evening Mail* (31 May 1968) where the occurrence of "*several fissures on the race track and a large 6m by 8m 'boil up' of sand and water at the entrance to the front straight of the inside training track*" were reported. The newspaper also presented photographs of both the sand boils and fissures.

Salisbury Street

Nearby, at 2 Salisbury Street, Mr James Fischer saw a sand boil erupt in the grass verge in front of the house (Berrill *et al.*, 1988).

Derby Street

Berrill *et al.* (1988) gives one of the most extraordinary reports stating "*To the west of Patterson Park at 10a Derby St, sand erupted under a house with sufficient force to lift it, rotate it and leave it skewed on its foundations...*". Ooi (1987) wrote that this occurrence was confirmed by Mr. Wally Forsyth, and the house was subsequently abandoned and demolished.

Berrill *et al.* (1988) also wrote that a, "*less definite report also had water coming out of the ground somewhere near 23 Romily Street, in the same general area*".

Keoghan Farm, Sergeants Hill

Following both the 1929 Murchison and 1968 Inangahua earthquakes, Mr Keoghan noticed sand boils in the paddocks of his farm which is situated on the banks of the Orowaiti River, south east of Sergeants Hill. According to *Berrill et al.* (1988), "*From 1929, when he was a boy of 11, Mr Keoghan remembers geysers of sand and water 5 to 6 feet high, leaving sand cones 2 to 3 feet high. In 1968, smaller cones were formed, about 6 inches high, in the same general area. Small sand boils were also observed by Mr Keoghan in the bed of the Orowaiti River immediately to the south of his house.*"

It should also be noted that this area is also known locally as Giles Creek. It is not known if this occurred again in 1991 following the Hawks Crag earthquakes.

Reedy Farm, Victoria Road, Westport

In a low-lying paddock adjacent to Victoria Road, Mr Reedy observed two large sand boils and a number of smaller ones in 1968. On average, this site floods about once a year; the low lying paddocks being on a flood plain. Berrill *et al.* (1988) undertook hand augering to a depth of 4.5 metres at this site and discovered uniform fine sand underneath 1.6 m of clayey silt. The water table was at a depth of approximately 1.4m. Berrill *et al.* (1987b) noted that clean sand was apparent at a depth of 3m and the silty sand below this layer and the fine sand above had thin interbedded layers of silt.

Berrill *et al.* (1987a) undertook CPT, SPT and piezocone testing at Reedy's farm in 1987. It was shown that the stratigraphy was variable, with clayey silt as a top layer, then fine to medium sands; below that was a sandy silt and coarse gravel below five metres. The liquefaction analysis undertaken by Ooi (1987) indicated that all layers would liquefy under a magnitude 7.1 earthquake, and the loose to medium silty sand layer lies within the most liquefiable soil boundary determined by Tsuchida and Hayashi (National Research Council, 1985) and plotted on a particle size distribution curve.

O'Connor Farm, Buller Gorge Road

Berrill *et al.* (1988) recorded sand boils on the farm of Mr O'Connor as a result of the Inangahua earthquake. These were discovered "*on the Buller River side of the farm road immediately beyond where it crosses Omanu Stream* [and] *on the flat paddock near the mouth of the Buller Gorge, beyond the cowshed*".

Dou and Berrill (1992) undertook both a hand auger and a scala penetrometer test at this site and noted that there was a 0.6 metre layer of silt underlain by a layer of very loose fine to medium sand. The water table was at a depth of 1.9 metres at the time of the test and gravel was encountered at a depth of 4.2 metres.

Adams *et al.* (1968), Lensen and Suggate (1968) and Greyland (1978) wrote that there were earthquake fountains and mud craters at Te Kuha near the western end of the Buller Gorge which is close to Mr O'Connor's farm.

4.4.4 Inangahua

Inangahua is situated above the 1968 rupture surface and seemed to bear the brunt of the earthquake in terms of the liquefaction. Falconer (1968, 1969) described the effects of the earthquakes. He wrote the following excerpt detailing the effects in Inangahua:

> "*Within that area, secondary effects of* [the] *earthquake, visible from the air, included:- conspicuous cracks in the ground, small craters from sand ejected to the surface of pasture on low lying river terraces, a variety of landslips, collapses of domestic chimneys, partial collapses of tile roofs of dwellings, and timber frame dwellings moved on or off their foundations*" - Falconer, 1969.

Both sand boils and waterspouts were prevalent on both Three Channel Flat, to the east of the settlement, and Walkers Flat to the west. A number of reports detailed the effects at these sites which are shown in Figure 4-5.

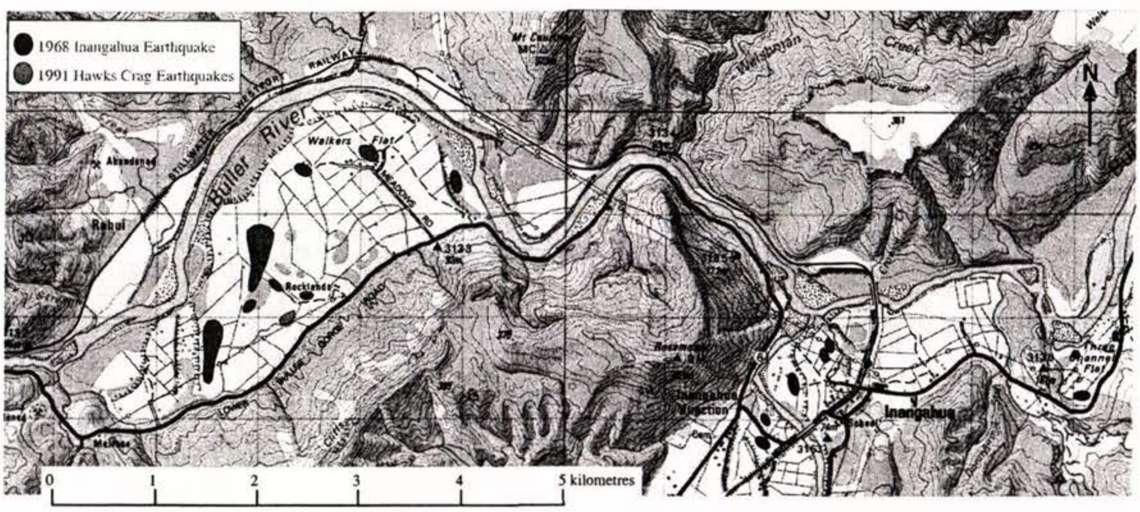


Figure 4-5. Sites of liquefaction following the 1968 and 1991 earthquakes in the Inangahua area (Base map: Topomap, 2001)

Sand boils were also reported by Dou and Berrill (1992) to have occurred in Inangahua on properties belonging to Mr. Lee and Mr. Inwood. This was based on aerial photographs taken following the earthquake. Dou gave the location of Mr. Inwood's farm as, "*on the alluvial terraces bounded by the Inangahua and Buller Rivers and the railway line at Inangahua*". Scala penetrometer undertaken by Dou in a field on the low terrace of Inwood's farm indicated that the soil was very loose and there was gravel near the surface. At Lee's farm, located on a high terrace of the Inangahua River, hand augering was carried out. These suggested that there was very loose silty sand between 1.55m and 3.1 m which was underlain by very dense coarse sand with some gravel to a depth of 4.3m. Beneath this, gravel was found. This was similar to another bore log undertaken nearby, and hence no further study was undertaken at this site.

Lensen and Suggate (1968) and Eiby (1980) gave the following co-ordinates and epicentral distances for occurrences of sand and water ejection.

Table 4-1. Sites of sand and water ejection (Lensen and Suggate, 1968)

Co-ordinates	*Epicentral Distance (km)*
41.86° S 171.96° E	10.5
41.97° S 171.89° E	23.9
41.77° S 172.03° E	0.3
41.86° S 171.86° E	13.4
41.83° S 171.65° E	29.4

Dodd in Shepherd *et al.* (1970a) gave a detailed account of the sand boils. This was also noted in Shepherd *et al.* (1970b).

> *"One of the more unusual effects of the earthquake was the appearance of sand boils on areas of alluvial river-flat. These were seen in two places- about a quarter of a mile from Inangahua Junction towards Murchison, and on Three Channel Flat at about 1 ¾ miles from Inangahua Camp towards Murchison.*
>
> *Fine silty sand with some mica had spread over the surface of the ground in irregular patches up to six feet across and one and two inches thick... In every case, when the sand was removed, a fissure was found in the ground surface beneath. These fissures were roughly parallel to one another and to the foot of the nearby Dee Hill face. This orientation of the fissures could possibly have been due to the fall of spoil from the face of Dee Hill, which caused a 'mud wave' in the alluvial valley floor deposits, parallel to the face of the hill...*
>
> *Samples of the sand layers at 5'-6" and 10'-0" depth were put in plastic bags for laboratory tests. A third sample was taken from sand on the surface...The particle size distribution curves for the three sand samples are alike, and all samples can be classified as fine silty sand. All three sands had the same specific gravity of 2.72...The three sands examined have very similar properties, so it is not possible (nor is it vital) to identify the erupted sand with any of the sub-surface sand strata".*

It is written in Sutherland (1969), and also in Shepherd *et al.* (1970a) that most of the sand flows occurred on the low lying alluvial terraces.

> "*Witnesses reported the presence of a large 'gusher' in the fields immediately north of Inangahua with water being thrown to a height of at least ten feet... it was active for at least an hour and a half after the earthquake... Two distinct area arrangements were apparent. One in which the sand patches defined a straight line over distances in excess of one hundred yards and one in which there were no discernable geometric arrangement. In the latter case, on Three Channel Flat and at Walkers Flat, there was no trace of any fissure beneath the flows...Presumably the sand erupted through isolated holes which have since filled with the surrounding material. A definite fissure could be found beneath the collinear flows and in places the fissure could be traced from one flow to the next. The fissures were apparently a direct consequence of the earthquake and were not caused solely by the extrusion process. By means of an auger one fissure was traced down to the water table at a depth of seven feet. This sand thus came from a greater depth.*
>
> *At both Walkers and Three Channel Flat the extruded sand was of a bluish grey colour having a mean grain size of approximately 0.1 mm. Sand layers having the same colour and grain size were observed between depths of eight and eleven feet in the sides of a drainage channel cut across Three Channel Flat. It is probable that the material was extruded from this depth.*"

Parts of this passage were also noted in Fairless (1984).

Fairless (1984), cites the following excerpt from Lensen and Suggate, in Adams *et al.* (1968):

> "*In water-saturated gravel or interbedded gravel and sand, settling and compaction resulting from the passage of the earthquake waves creates sub-surface water pressure. Where this is sufficient, water is ejected at the surface, commonly carrying sand. In some places the ejected sand had rheotropic properties, being readily made fluid by vibration. Patches of sand were prominent on lower post-Otira Glacial terraces at Inangahua, south to Rotokohu, north-east to New Creek and west over Walkers Flat as far as 6 km west of Inangahua; an isolated report comes from 25km west of Inangahua*".

Three Channel Flat

Three Channel Flat, located about 10 km southwest of the epicentre and very close to the centre of the aftershock pattern, showed the most intensive sand ejection in comparison to other sites according to Berrill (1987, 1988). A photo indicating the extent of the sand boils on this flat can be seen in Figure 4-6.

Figure 4-6. Sand boils on Three Channel Flat following the 1968 Inangahua earthquake (photo courtesy Dr. A.J. Carr)

Following the studies made by Adlam (1988) and Ooi (1987), Berrill *et al.* (1988) wrote the following regarding the fissures on Three Channel Flat.

> "*Presumably the formation of the linear fissure with sand ejection indicated some lateral mass movement, while the unconnected random pattern of boils... mark areas of complete liquefaction on virtually level ground*". - Berrill *et al.* (1988)

As noted in Chapter 2, this area is made up of a series of low river terraces of recent alluvium. The photographs taken following the earthquake indicate the scale of the liquefaction occurring at the site. Both Adlam and Ooi also undertook auger borings to obtain visual inspection of the subsoil, electronic cone penetrometer tests (CPT), hollow stemmed augering and standard penetration testing (SPT) at Three Channel Flat. Hand auger boring undertaken by Adlam and Ooi at the south end of the flat indicated a three to four metre thick layer of silt is underlain by fine sand of variable thickness which is then underlain by gravel. The sand layer increased in thickness at the edges of the abandoned south river channel, and there was a discrete absence of sand boils in the centre of the channel where the sand layer was less thick. Berrill *et al.* (1988) wrote, "*It appears that where there is sand, it was ejected in 1968*".

Adlam (1988) wrote regarding the 1968 earthquake, "*Ooi (1987) in his study stated that liquefaction would have occurred at Three Channel Flat if the Inangahua earthquake had a magnitude of as low as 3. This value appears to be anomalous on the basis of historical data, which does not show any instances of liquefaction for earthquakes with a magnitude of less than 5*". Adlam states that liquefaction could have occurred at Three Channel Flat with an earthquake of magnitude 6.2 on the Richter scale. Dou and Berrill (1992) simply noted that the soils at Three Channel Flat were very soft; allowing them to be liquefied at a lower shaking intensity. However, no mention was made of liquefaction at this site following the 1991 Hawks Crag Earthquakes.

Walkers Flat

Fairless (1984) cited Sutherland in Shepherd (1970a) as writing, "*An eyewitness described the Walkers Flat flows as being slow and continuing until approximately 1030 hrs (five hours after the earthquake)... A second eyewitness reported observing sand flows occurring along the edge of the road near Inangahua. The observations were made 'early in the morning'*".

Dou and Berrill (1992) studied aerial photos taken following the earthquake and located a number of sites on the properties of both Mr. Turner and Mr. Meadows on Walkers Flat. These were mapped and the locations shown in Figure 4-5.

Meadows' Farm

Berrill *et al.* (1988) wrote that on one site at Meadows' Farm, on Walkers Flat, liquefaction occurred as a result of both the main shock and an aftershock. Mr Meadows' farmhouse lies at the east end of the flat and in the paddock immediately north of this sand boils were noted.

> "*Mr Meadows' remembers sand and water being ejected from vents in the vicinity of the farmhouse and cowshed during some aftershocks as well as the main shock.*"
>
> - (Berrill *et al.*, 1988)

Using aerial photographs, Dou and Berrill (1992) inferred that lateral spreading occurred on the easternmost side of the Walkers Flat terrace. Hand augering was undertaken, both in the paddocks to the north of the house where sand boils were confirmed to have occurred by Mr.

Meadows and on the central part of the terrace where no liquefaction was noted. The results from the scale penetrometer tests at the non-liquefied site indicated that there was a 1.5 metre layer of silt underlain by a 1.7 metre thick layer of dense sand. Beneath this was a layer of gravel. At another test site, gravel was encountered at 1.7m, again above the water table. In the paddock which was reported to have liquefied, hand augering indicated that there was a 1.7 metre thick layer of silt underlain by a one metre thick layer of fine sand which contained lenses of silt. This was in turn underlain by a one metre thick layer of silty fine sand which overlaid gravel. Dou also noted that the water table was at a depth of 3.2 metres, and that the scala penetrometer tests indicated that the soil was less dense than that in the paddock which did not liquefy.

Turner's Farm

Dou and Berrill (1992) also undertook testing at Turner's Farm on the eastern side of Walkers Flat. The testing included hand augering, scala penetrometer testing, Standard Penetrometer testing (SPT), and Cone Penetrometer testing (CPTU).

Dou initially used aerial photographs to discern the location of sand boils and hence determine both liquefied and non-liquefied sites. Hand augers and scala penetrometer testing was then undertaken to determine the different conditions and to ascertain why some areas liquefied and others did not.

From the hand augers and scala penetrometer tests, Dou discovered that the sites which did not liquefy had either denser soil, or the water table was much lower than at the sites which liquefied. At one of the sites which liquefied the soil profile was as follows: 3.6 metres of silt underlain by one metre of loose fine to medium sand, followed by one metre of medium to coarse sand with fine gravel, below which was gravel. The water table varied from a depth of 2.9 metres to 3.2 metres beneath the ground surface. Hand augers at other locations revealed a similar pattern, though the top layer of silt was between 2 metres and 2.9 metres thick.

SPT tests were also undertaken at a number of the liquefied and non-liquefied sites. The tests undertaken at different depths at the liquefied sites gave N values between 3 and 9, whereas at the non-liquefied sites the N values ranged from 8 to 19. During a number of tests boiling into the hollow stemmed auger occurred rendering the results questionable.

CPTU tests were also undertaken at these sites to determine the cone resistance and rate of pore pressure dissipation. From these, the cone resistance and pore pressures were plotted and the results analysed to determine layers which may have liquefied. At one of the sites, a very low cone resistance (0.8 to 3 MPa) was discovered in the fine sand layer between 3 and 3.8 metres, which is thought to have liquefied.

4.4.5 Murchison

It was noted in Greyland (1978) that Mr Arthur Ross observed jets of water coming up through the ground, some of which were up to two metres high. Mr Ross owned a farm in the Murchison region and was rounding up his cows for milking at the time of the earthquake.

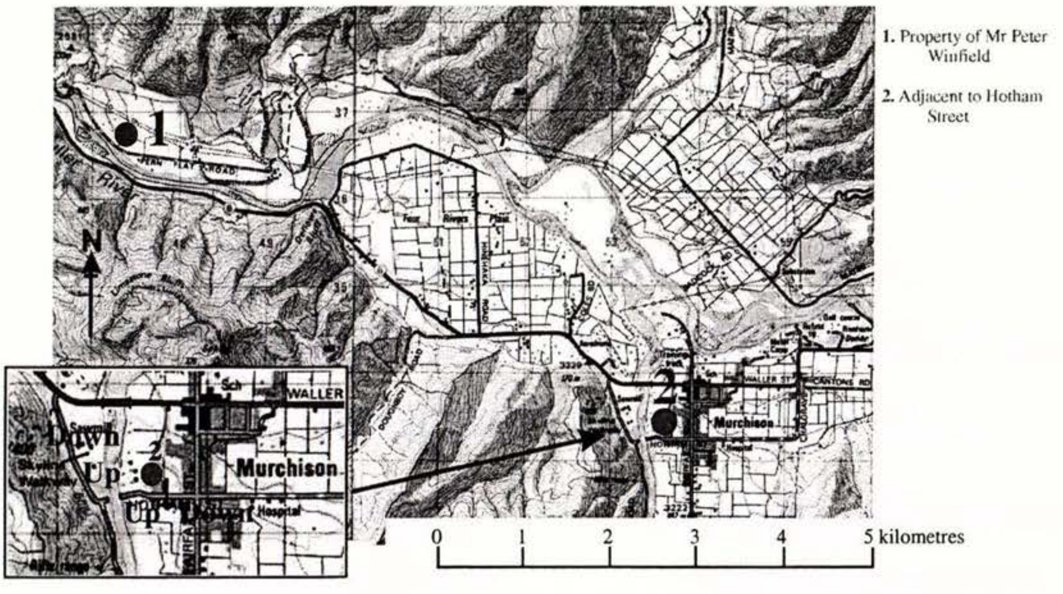


Figure 4-7. Sites of liquefaction in Murchison following the 1968 Inangahua Earthquake (Base map: Topomap, 2001)

Dou and Berrill (1992) also undertook fieldwork on Mr. Peter Winfield's farm on Fern Flat which is located in both Figure 4-3 and Figure 4-7. Aerial photos taken after the earthquake indicated liquefaction at this site, which was confirmed by Mr. Winfield. Precise locations are given for these occurrences in Dou and Berrill. Dou carried out both hand augering and scala penetrometer testing at this site and at one location found a 3.3 metre thick layer composed of silt and fine sand lenses, beneath which was a 0.2 m layer if loose fine sand. Below this layer, 1.5 metres of dense medium sand was followed by a one metre thick layer of coarse sand. Gravel was encountered at a depth of 6 metres and the water table at 4 metres below the ground surface. Nearby, a layer of gravel was encountered at a depth of 3.3 metres. Three SPT tests were also undertaken at this site, though sand heaving into the auger occurred in the third test rendering the result possibly erroneous.

Dr. John Collen, a geologist at Victoria University of Wellington, recalled seeing the sand boils (illustrated in Figure 4-8) in Murchison in the morning following the earthquake and located these on Hotham Street, as shown in Figure 4-7. He remembered seeing the water pulse out of the crack as the crack opened and closed, and continued to do so four hours later, though at this time the flow was smaller and was closer to the centre of the sand boil. Dr. Collen also noted that the fine grained particles flowed out to the edge of the sand boil while the coarser material remained closer to the vent (pers comm., April 2003).

Figure 4-8. Sand Boils adjacent Hotham Street, Murchison (photo courtesy Dr. John Collen)

Dr. Collen wrote, "*The road near the sand boils was fractured with a gap of 10-15cm, upthrown by about the same amount to the west*" (pers. com. May 2003) and this is illustrated by the following photo in Figure 4-9 which was taken nearby.

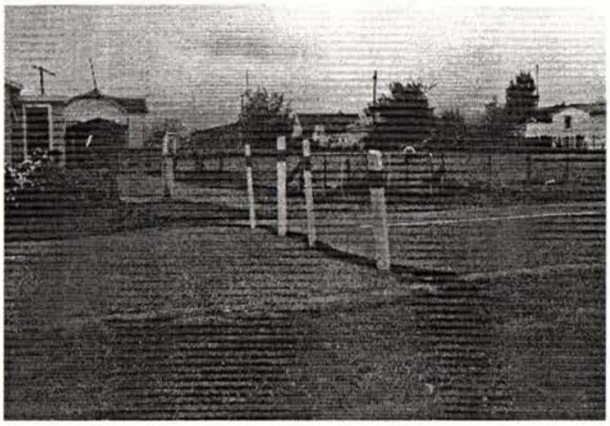


Figure 4-9. Crack across Hotham Street, Murchison (photo courtesy Dr. John Collen)

Dr. Collen also wrote that the road was fractured near the bridge on the west side of the Matakitaki River and was upthrown to the south, as shown in Figure 4-7.

4.4.6 Rotokohu

Dou and Berrill (1992) reported that Mr. Peter Bell, who lived in the area at the time of the Inangahua earthquake, confirmed the occurrence of sand boils that were indicated by aerial photographs on Mr. R. Van Vught's farm. Mr. Bell also recalled sand boils on Mr. Peter Schwass's farm, to the south of Mr. Van Vught's farm. A hand auger investigation and a scala penetrometer test were undertaken at the site. From this it was discovered that there was a 0.7 metre layer of silty sand followed by a 0.4 metre thick layer of very loose fine sand. This was underlain by a medium sand containing gravels. No further testing was undertaken at this site, as a layer of gravel was located at a depth of 1.4m. At the time of testing the water table was at 0.9 metres below the ground surface.

4.4.7 Greymouth

No reported instances of sand boils were located following the earthquake in this region. However, Mr Kevin Boon, Mayor of Greymouth in 2003, recalled that a number of pipes had to be replaced in the town following the earthquake suggesting some lateral spreading. The exact locations were not known, nor the extent of the work. Records have also since been lost (pers. comm., Dec 2002).

4.5 ***Summary***

Based on the known occurrence of liquefaction effects, the occurrences are predominantly in an East-West band across the South Island, mainly following the Buller River. If a comparison is made with the isoseismal map drawn for this earthquake, the effects of liquefaction are confined to the area where the intensities were in the range 9 to 10, with the exception of those that occurred in Westport and Seddonville where the felt intensity was recorded as eight.

5 Other Earthquakes

5.1 ***1913 Westport Earthquake***

Morgan (1913) gives brief details regarding this earthquake. It occurred on the 22 February 1913, and was located, "*few miles westward of Westport*" and at a "*comparatively moderate depth*". It was assigned a Rossi-Forel Magnitude VIII or IX, and Eiby (1968) stated that it was probably greater than local magnitude M_L5.

At Cape Foulwind cracks opened up in the ground, and mud was exuded from one of these (Morgan, 1913). No further details pertaining to this have been found.

5.2 ***1962 Westport Earthquake***

In May 1962, a series of shallow earthquakes were felt in the Westport area. The effects were felt significantly in the immediate area but were also fairly widely felt as can be seen in the following isoseismal map in Figure 5-1.

Date:	10 May 1962 (10 May 1962 Local Time)
Time:	00:27:14.7 UT (12:27:14.7 pm Local Time)
Magnitude:	5.7 M_L
	5.9 M_S (Dowrick and Smith 1990)
Epicentre:	41.67° S 171.44° E
Depth:	12 km

The only known reports of liquefaction have been published in Ooi (1987). Ooi wrote that Mr Keoghan recalled seeing sand boils and geysers of water in the paddocks to the north of his house following the 1962 earthquake. This was on his property, on the banks of the Orowaiti River, near Sergeant's Hill, Westport. Liquefaction was also reported at this site following the 1929 and 1968 earthquakes.

On the O'Connor Farm on the Buller Gorge Road, sand boils occurred at the same site of liquefaction as occurred in the 1968 earthquake. This was described to be, "*on the Buller River side of the farm road immediately beyond the point where it crosses the Omanu Stream*" (Ooi, 1987).

Figure 5-1. Isoseismal Map for the 1962 Westport Earthquake (Downes 1995)

5.3 1991 Hawks Crag Earthquakes

Smith (1992) recorded that two shallow earthquakes occurred within five hours of each other, on the 29 January 1991. Three earthquakes occurred over a period of two days near Hawks Crag in the lower Buller Gorge, approximately 15 kilometres south of Westport. The Richter magnitudes of these were 6.1, 6.2 and 5.9 respectively. Dou (Dou and Berrill, 1992) visited the Buller area six days after the earthquake and confirmed the occurrence of liquefaction at Inangahua and Virgin Flat. The known sites of liquefaction are shown in Figure 5-2.

5.3.1 Observed Effects

Walkers Flat

Dou and Berrill (1992) located sand boils at Wallace Turner's Farm on Walkers Flat following the earthquake. He noted that these sand boils were conical in shape with a maximum height of 0.1 metre and maximum diameter of one metre. The distribution of sand boils was also noted to be random, though in a different location to those that occurred on the flat in 1968, as can be seen in Figure 4-5.

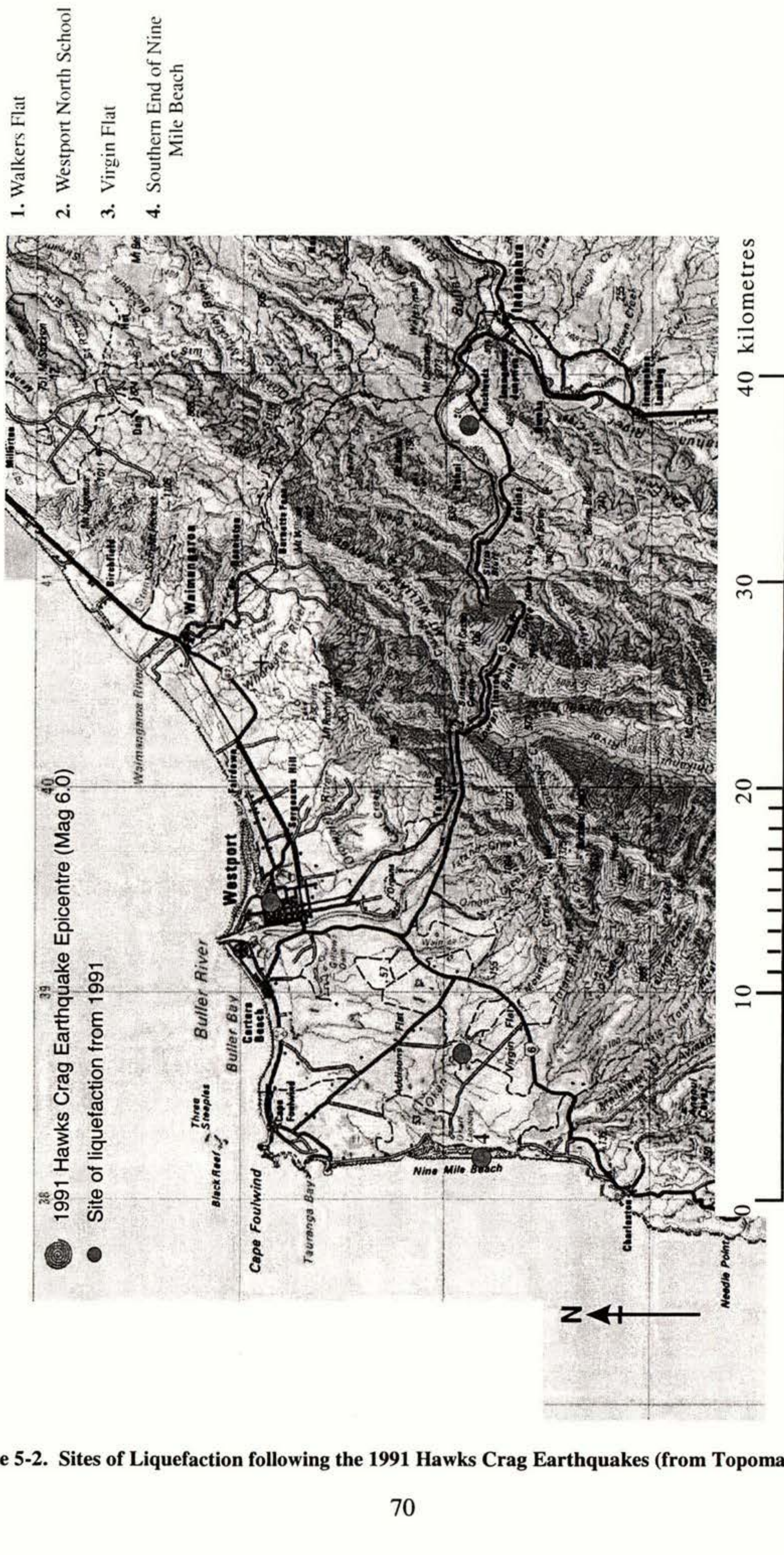


Figure 5-2. Sites of Liquefaction following the 1991 Hawks Crag Earthquakes (from Topomap, 2001)

Westport North School

Sand boils were also formed in the playground at Westport North School during the Hawks Crag earthquake, according to Mr Geoff White, caretaker of the school at the time of the earthquake (pers. comm., Feb 2003). The exact location of these sand boils could not be ascertained and no photographic evidence of this has been located. According to a number of staff at the school, who were living in the area at the time of the 1991 earthquakes, there were no visible effects in the surrounding areas, such as Patterson Park or Kilkenny Park.

The school is situated behind Kilkenny Park, a location that liquefied in the 1968 Inangahua earthquake. It must be noted that the playing fields have also been drained since the earthquake.

Virgin Flat

Dou and Berrill (1992) reported that liquefaction occurred on Virgin Flat, approximately 15 km Southwest of Westport. Cracks in the field indicated the occurrence of lateral spreading, while it was also noted that the settlement of the road had been repaired by the time Dou visited the site approximately a week after the earthquake. The exact site is shown in Figure 5-3.

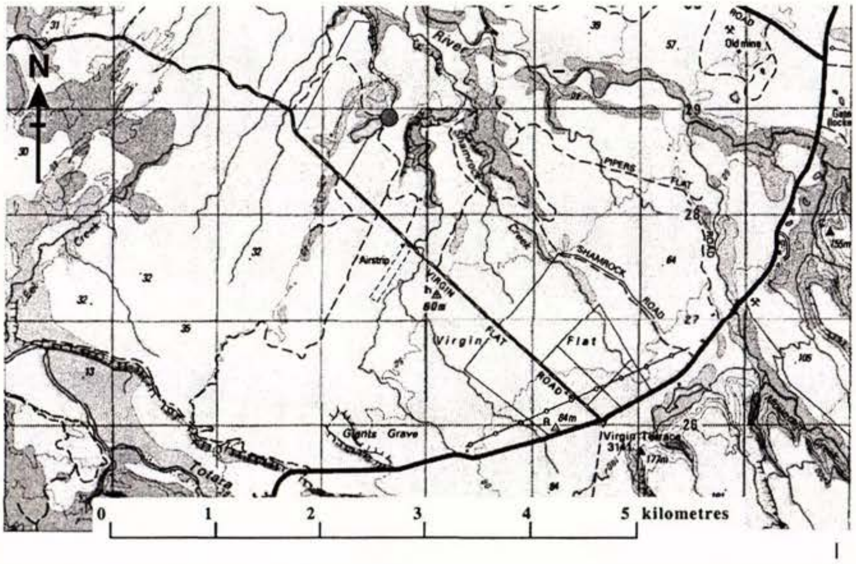


Figure 5-3. Site of Liquefaction on Virgin Flat (from Dou and Berrill, 1992)

It was noted by Benn (1992) that at the southern end of Nine Mile Beach, Charleston, sand boils and cracks were formed. Logs were left "*perched on pedestals of sand*" as subsidence occurred around the large driftwood logs. The photographic evidence shown in Figure 5-4 supports these statements.

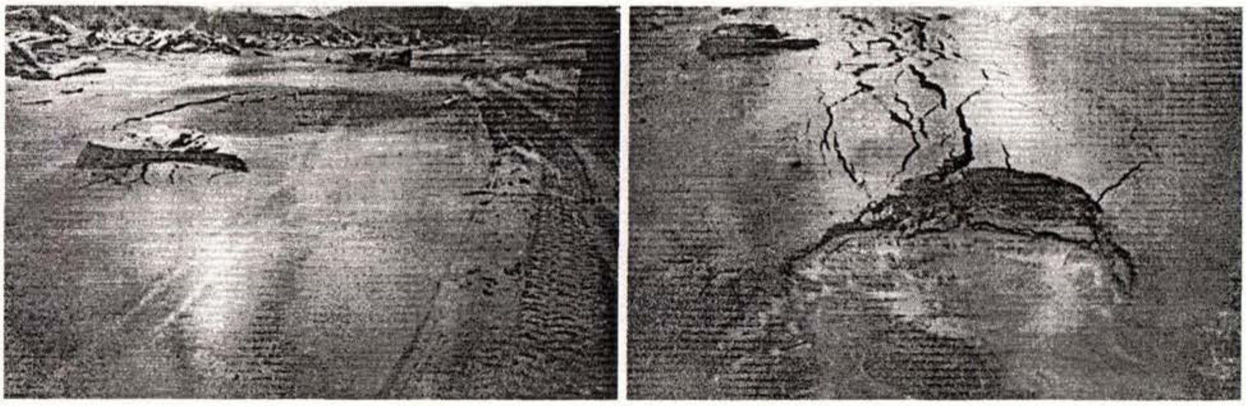


Figure 5-4. Photos of Sand Boils at Nine Mile Beach, Charleston (Benn, 1992)

6 Summary and Sites Tested

6.1 Sites of liquefaction in the Buller District

The known sites of liquefaction based on key indicators such as the occurrence of sand boils are given in Table 6-1 below. The location, date and distance to the epicentre of the earthquake in question, as well a brief description of the evidence obtained is given. Each site, ordered from North to South, is also located on the map shown in Figure 6-1, with the isoseismal maps overlain for both the 1929 Murchison and 1968 Inangahua earthquakes.

Table 6-1. Summary of sites of Liquefaction on the West Coast of the South Island

Site of Liquefaction	*Site No.*	*Date*	*Epicentral Distance (km)*	*Information Source*
Patarau River	1	1929	122	*The Nelson Evening Mail* (1929) - "*earth rose up in bubbles*"
Kahurangi Point	2	1929	105	*The Nelson Evening Mail* (1929) - water spouts
Rockville	3	1929	117	*The Nelson Evening Mail* (1929) - fissuring, mud and water exuded from cracks
Bainham	4	1929	111	*The Nelson Evening Mail* (1929) - fissuring, mud and water exuded from cracks
Puramahoi	5	1929	113	*The Nelson Evening Mail* (1929) - fissuring, mud and water exuded from cracks
Motupipi	6	1929	111	*The Nelson Evening Mail* (1929), *The Sun* (1929) *et al.* - fissuring, mud and water exuded from cracks
Takaka	7	1929	110	*The Nelson Evening Mail* (1929), B. Peart (2003) fissuring, settlement, mud and water exuded from cracks
Riwaka	8	1929	98	*The Nelson Evening Mail* (1929) - fissuring, mud and water exuded from the cracks
Motueka	9	1929	97	*The Nelson Evening Mail* (1929) - fissuring, mud and water exuded from cracks
Brooklyn	10	1929	95	*The Nelson Evening Mail* (1929) - fissuring, mud and water exuded from cracks
Pokororo	11	1929	78	*The Nelson Evening Mail* (1929) - fissuring and sand boils
Oparara-	12			
School		1929	57	F. Wood(1929) , J. Lowe (2003) - mud pools
Mr Thompson's Farm		1929	57	V. Harney (1929) – water/mud geysers

Mr. Rhind's Farm		1929	57	D. Rhind (2003) – sand boils, water spouts, Settlement
Karamea-	13			
Aerodrome		1929	55	Berrill *et al.* (1988) - toppled fence posts
Quinlan's Bridge		1929	53	F. Wood (1929) - bridge approaches dropped
Quinlan's Filling		1929	53	F. Wood (1929) - fissuring
Township		1929	53	F. Wood (1929) - buildings tilted, differential settlement, lateral spreading
Wharf		1929	53	F. Wood (1929), J. Richardson (1989), Newspapers - lateral spreading
Simpson's Paddock		1929	53	M. Richardson (2003), H. Simpson (2003) - sand boils/ water spouts
Fensom's Paddock		1929	53	A. Hay (1989), M. Richardson (2003) - sand boils/ water spouts
School		1929	53	G. Duncan (1974), H. Simpson (2003) - sand boils/ water spouts
Overflow Bridge		1929	53	F. Wood (1929), G. Duncan (1974) - lateral Spreading
Arapito	14	1929	50	Bienvenu (1988) – sand/water geysers
Umere	15	1929	51	Mrs. Lilywinter (1989) – fissuring
Kongahu Mud Flats	16	1929	50	*Grey River Argus* (1929), D. McNabb (2003) – mud geysers, water spouts
Little Wanganui	17	1929	40	Bienvenu (1988), Berrill *et al.* (1988) - sand boils
Seddonville	18			
School		1929	25	*The Nelson Evening Mail* (1929), Berrill *et al.* (1988) - sand boils
Durkin's Farm		1968	25	Ooi (1987) – sand boils
Mokihinui	19	1929	31	*The Press* (1929), R. Duncan (1989) - sand/water geysers
Owen River	20	1929	23	Murchison District and Historical Society (1999) - water spouts
Longford	21	1929	16	Murchison District and Historical Society (1999) - water geysers
Matiri Valley	22	1929	12	*The Nelson Evening Mail* (1929) - fissuring, ground water ejected
Murchison	22	1929	14	*The Nelson Evening Mail* (1929) - sulphur/mud spouts
Hotham Street		1968	30	J. Collen (2003) – sand boils
Four Rivers Plain	22	1929	10	Bienvenu (1988), Dou and Berrill (1992) – sand boils, water spouts

Fern Flat	23	1929	8	Murchison District and Historical Society (1999) - water spouts
Winfield's Farm		1968	24	Dou and Berrill (1992) - sand boils
Lyell	24	1929	15	Murchison District and Historical Society (1999) - sand/water geysers
Inangahua	25			
Three Channel Flat		1929	24	*Inangahua Herald* (1929) – sand/water geysers
		1968	9	Lensen and Suggate (1968), Shepherd *et al.* (1970a, 1970b) - sand boils and waterspouts
Township		1929	26	*The Sun* (1929) – sand/water geysers
Walkers Flat	26			
- Meadows Farm		1968	8	Sutherland (1970), *Berrill et al.* (1988), Dou and Berrill (1992) - water spouts and sand boils
- Turners Farm		1968	11	Sutherland (1970), Berrill *et al.* (1988), Dou and Berrill (1992) - water spouts and sand boils
		1991	21	Dou and Berrill (1992) - sand boils
Sergeants Hill	27	1929	46	Berrill *et al.* (1988) - sand boils
		1962	18	Ooi (1987) - sand boils
		1968	26	Berrill *et al.* (1988) - sand boils
Westport	28			
Shelswell Street		1968	30	Berrill *et al.* (1988) - fissures
Kilkenny Park		1968	30	Ooi (1987), Bienvenu (1988) and others - sand boils and water spouts
Patterson Park		1968	30	Ooi (1987), Bienvenu (1988) and others - sand boils and water spouts
Salisbury Street		1968	30	Berrill *et al.* (1988) - sand boil
Westport North School		1991	16	G. White (2003) - sand boils
Derby Street		1929	49	Archives New Zealand - broken pipes
		1968	30	Ooi (1987), Berrill et al (1988) - sand boil
Romily Street		1929	49	*Grey River Argus* (1929) - fissures in road
Wharf		1929	50	*The Press* (1929) - lateral spreading
Cape Foulwind Road	29	1913	unknown	Morgan (1913) - fissuring
		1929	56	*Grey River Argus* (1929) - fissuring of road
Buller Gorge	30			
O'Connors Farm		1962	20	Ooi (1987) - sand boils
		1968	30	Berrill *et al.* (1988), Dou and Berrill (1992) - sand boils
Reedy's Farm		1968	30	Ooi (1987) - sand boils
Virgin Flat	31	1991	11	Dou and Berrill (1992) - lateral spreading, settlement

Charleston	32			
Nine Mile Beach		1991	15	Benn (1992) - sand boils
Rotokohu	33			
–Van Vught's Farm		1968	21	Dou and Berrill (1992) - sand boils
Coal Creek	34	1929	113	Mrs. Batey (2002, 2003) - sand/water geysers
Omoto	35	1929	115	*The Nelson Evening Mail* (1929) - fissuring in road
Greymouth	36			
Richmond Quay		1929	116	*The Nelson Evening Mail* (1929), *Grey River Argus* (1929) - fissuring
Blaketown Footbridge		1929	116	*Auckland Weekly News* (1929) - settlement of supports
Arney Street		1929	117	*Grey River Argus* (1929) - fissuring
Victoria Park		1929	117	*The Nelson Evening Mail* (1929) - fissuring
Preston Road/Steer Ave		1929	117	*The Grey Star* (1929), *The Dominion* (1929), *The Free Lance* (1929) - fissuring, silt and water exuded through cracks
Municipal Abattoir Paddock		1929	118	*The Grey Star* (1929), *Grey River Argus* (1929) - sand boils, fissuring
Cnr. Collins/Blake Street		1929	116	*The Grey Star* (1929), *Grey River Argus* (1929), *The Nelson Evening Mail* (1929), Benn (1992) - fissuring, sand and water spouts, settlement
Paroa	37	1929	122	*The Nelson Evening Mail* (1929), *The Grey Star* (1929) - fissuring, geyser
Greenstone River	38	1929	122	*The Grey Star* (1929) - fissuring, sand boils and water spouts

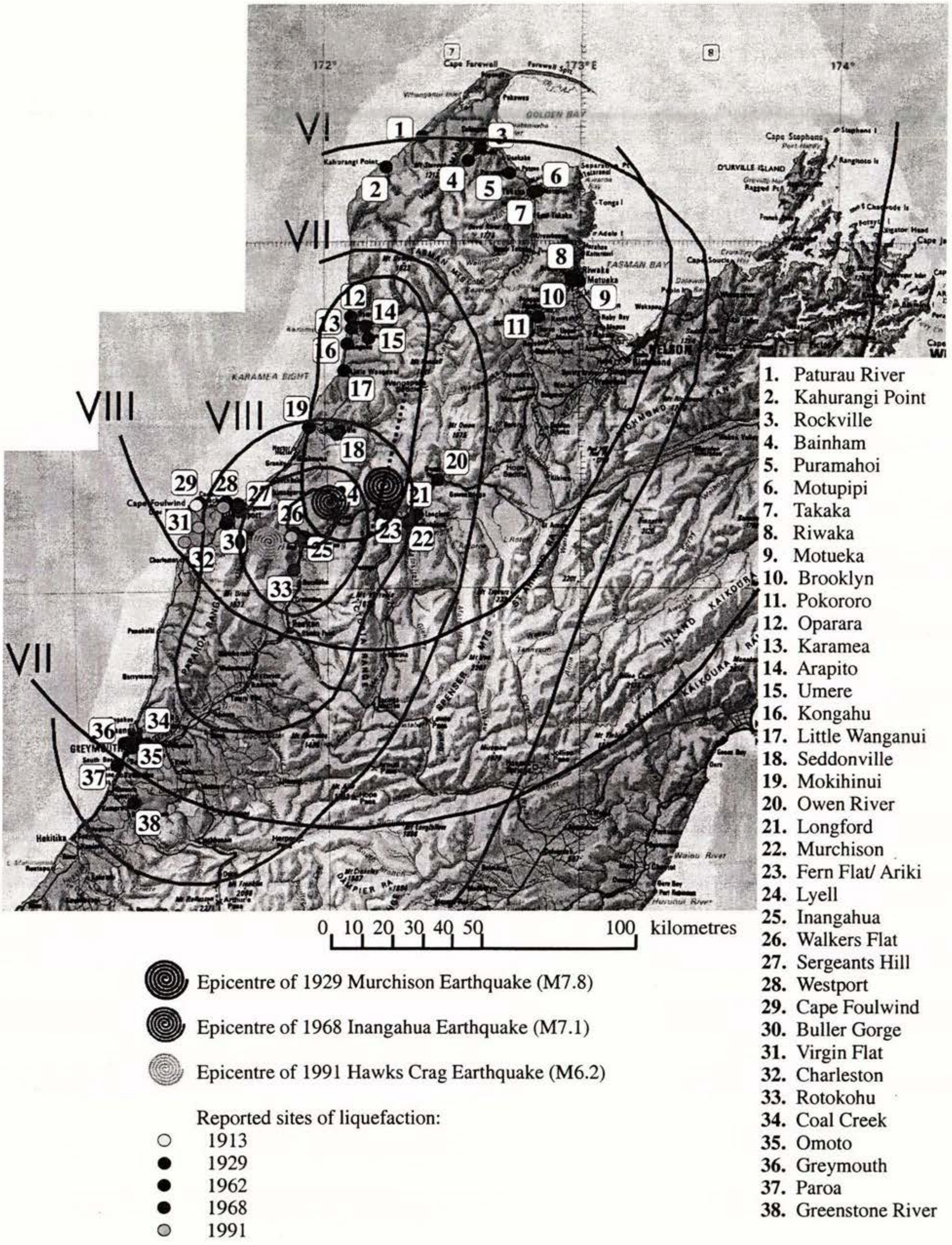


Figure 6-1. Summary map of all known sites of liquefaction (Base map: Topomap, 2001)

6.2 Field Study

Sites for investigative testing were selected based on the following criteria:

- evidence of soil liquefaction in past earthquakes
- positive identification of such sites by photographs or reports from multiple sources
- distance from the epicentre of the earthquake
- variation – sites that had liquefied in some earthquakes but not others
- suitability of the soil for CPT testing
- suitability of site for access by the drilling rig

The phenomenon of liquefaction was not understood at the time of the 1929 Murchison earthquake and neither was it universally recognised in 1968 when the Inangahua earthquake occurred. Due to this, the following surface manifestations were all used to identify sites of liquefaction and base the study upon:

- sand boils and ejection of water and mud from the ground
- lateral spreading
- subsidence of the ground surface
- tilting or sinking of structures.

As noted earlier, a number of liquefaction sites in the epicentral region have been tested by University of Canterbury researchers (Ooi, 1987; Bienvenu, 1988; Dou and Berrill, 1992). The more distant sites at Karamea and Greymouth, which had not been tested, were chosen for this study. These sites are generally the more interesting ones, since because of their distance from the epicentre of the earthquakes, they are closer to the line separating liquefiable from non-liquefiable conditions. Further, since these sites had liquefied in the 1929 Murchison earthquake and not the 1968 Inangahua earthquake, they provide bounds to the dividing condition.

This difference raises questions as to what was different about the rupture mechanisms of the earthquakes, and although the peak ground accelerations can only be assumed for the 1929 earthquake, how this influences the results of the empirical models which have been derived to predict the occurrence of liquefaction. These sites are of key interest as they may be able to help define the limit between the occurrence of liquefaction and no liquefaction. Both towns are also of significant distance from each of the earthquake epicentres as can be seen from Table 6-1.

Earlier studies on the West Coast have focussed on the Buller Gorge areas - towns such as Inangahua, Westport and areas near these. Bienvenu initiated work in Karamea in the 1980's, and this project gave an opportunity to continue the work.

To verify the different sites of liquefaction and to locate them precisely, consultation was held with a number of long-time residents from both Karamea and Greymouth, many of whom remember the water spouts and sand boils seen as children in 1929. A number of these people were living in the same areas during the 1968 earthquake and could confirm that nothing similar happened in that event.

6.2.1 Preliminary Testing

Each potential site was visited to ensure it would be suitable for rig access. The West Coast of the South Island has a reputation for heavy rainfall; this combined with the soft paddocks used for dairying, and the swampy nature of parts of the coast near Karamea could cause difficult access for the drilling rig.

Council staff were consulted, as well as others, with knowledge of the area, to determine the location of underground services. This was vital for Greymouth as testing was planned in a residential area. The Grey District Council staff identified the exact locations of buried services in Greymouth from their GIS database. At each of the sites selected on the basis of rig access, hand auger boring was carried out to enable visual inspection of the subsoil to a limited depth. If gravel was located at a shallow depth, the validity of carrying out CPT testing was questioned as penetration through such layers was likely to be difficult.

6.2.2 Field Testing

Testing in both Karamea and Greymouth included the use of a number of different techniques such as Cone Penetrometer Testing and Standard Penetration Testing.

The Department of Civil Engineering's drilling rig was fitted out to enable the operation of Standard Penetration Testing, Cone Penetrometer Testing (CPT) and as well hollow stemmed auger and flight auger boring. In Greymouth, the hollow stemmed augers were used to drill through gravel layers enabling the continuation of the CPT tests to greater depths. The boring was also used to recover sand specimens from depth.

Standard Penetration Testing (SPT)

The Standard Penetration Test (SPT) has been widely used in the past and can be performed in conjunction with a hollow stemmed auger. However, these tests, while cheap and easy to perform, are very difficult in loose and loose to medium dense sands and corrections for the hole diameter, rod length, sampler type, and energy delivery need to be made in order to standardise the test results. A number of the models predicting whether or not liquefaction is likely to occur at a site are based on SPT data as a large database of case histories is available worldwide.

SPT tests undertaken by Berrill *et al.* (1987a, 1987b) using a hollow stemmed auger and a trip hammer in the Buller region, were found to be unsatisfactory and subsequently rejected due to boiling of sand into the stem of the auger when the plug was removed. SPT tests were attempted at Greymouth in late January 2003, but similar problems were encountered with the boiling of sand in the base of the hollow stem auger used as a casing. Due to the loose nature of the soil in Karamea it was decided not to attempt the SPT following the cone penetrometer testing.

Cone Penetrometer Testing (CPT)

Some twenty Cone Penetrometer Tests were undertaken, with varying success, using the Civil Engineering Department's drill rig in Greymouth and Karamea. CPT results are generally more consistent and repeatable than results from other tests. A detailed interpretation of soil types and soil layers is possible due to the continuous profile obtained.

In the CPT test, a 36mm diameter penetrometer is pushed into the ground at a constant rate of 2 cm/s using a hydraulic ram with extra rods added to the probe as the depth increases. The penetrometer has load cells incorporated into the tip and the sleeve as shown in Figure 6-2 below. These load cells measure the force on the tip and the side friction on the penetrometer. If these continuous force measurements are divided by the projected tip area and the sleeve area the cone resistance, q_c, and sleeve friction, f_s, are obtained respectively (Meigh, 1987). The strength of the soil is indicated by the cone resistance, and when combined with the friction ratio, which is derived by dividing the sleeve friction by the tip resistance, models such as those derived by Robertson and Wride (1998) and Olsen (1997) can be used to indicate probable soil types.

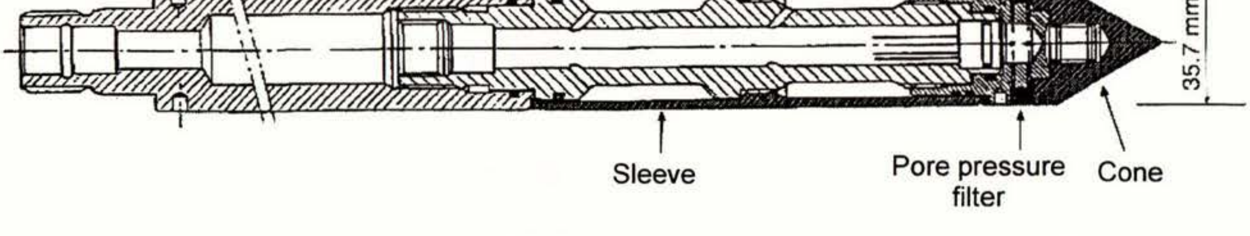


Figure 6-2. Cone penetrometer (from Centre for Advanced Engineering, 1997)

The device type has been standardised by having a standard point diameter and rate of penetration. The only correction is to reduce the point resistance to a standard overburden pressure. The test is reasonably precise; Vreugdenhil (1995) discovered that a layering correction was necessary for only layers thinner than a few hundred millimetres.

Both theoretical (i.e. Vreugdenhil, 1995) and laboratory studies have indicated that layers of stiffer or softer material, above or below the cone tip, influence the cone resistance values. This means that if a thin dense layer is sandwiched between two soft layers of soil, the full penetration resistance will not be measured by the cone. Youd and Idriss (1997) note that, "*the distance to which the cone tip resistance is influenced by an approaching interface increases with stiffness of the stiff layer*". This means that the distance of influence can vary from two to three cone diameters in soft clays or loose sands and up to 20 cone diameters in stiff clays or dense sands. Seed *et al.* (2001) writes that to develop the full tip resistance, the cone must penetrate four to five diameters into a stratum. Youd and Idriss (1997) note that Robertson and Fear (1995), suggest procedures to estimate the layer correction factors and the "*full cone penetration resistance of this stiff layers contained within softer strata*".

Van den Berg (1995) derived a Eulerean finite element model to simulate the effects of the CPT in layered media and found that the response of the soil is different for a loose, liquefiable layer compared to a dense layer, results which are similar to Vreugdenhil and Robertson and Fear. In a loose sand the deformation caused by pushing the cone tip can be observed up to "*a distance of 2 or 3 diameters in front of the cone*", and the "*vortex shaped movement*" deformation pattern is very complicated. In dense sands the deformation pattern is similar to failure planes for bearing capacity theory. It is noted, "*in dense sands a stress build up is created under the cone tip up to a certain maximum, after which suddenly localization into distinct failure planes occurs. At points of localization* [a] *sharp point can be observed in the load-displacement curve, although the material itself is homogeneous*" (Van den Berg, 1995). It is also noted that within the one layer this stress build up and localization can be repeated multiple times. Yu and Mitchell (1998) question the accuracy of the cone factors obtained using finite element modelling, due to the numerical errors associated with the modelling procedure. They state, "*because of the significant errors and numerical difficulties associated with collapse load calculations using displacement finite-element methods, it is unlikely that the incremental finite-element method will provide a completely satisfactory solution for cone penetration in soils*".

Larsson *et al.* (1995) recommend that both classification and interpretation the soil properties of CPT results in very thin layers be done manually as, "*to obtain relevant measured values of cone resistance and sleeve friction, the layer has to have a certain minimum thickness*". The minimum thickness recommended in the paper was between 0.2 and 0.7 metres. It is also noted that thinner seams can be classified by studying the pore pressure profile and relating the variation in the cone resistance to samples taken from the test site.

These all raise questions for the New Zealand environment. As previously noted, most of the areas susceptible to liquefaction on the West Coast of New Zealand are located in estuarine environments or on flood plains where thin seams of material are prevalent.

Other uncertainties in the cone penetrometer testing were raised by Post and Nebbeling (1995). Apart from human error in measurement of depth and the non-vertical inclination of the rods and penetrometer tip, there are uncertainties caused by the heating and cooling of the penetrometer as it is pushed into the ground and due to water pressure on the tip of the cone. They state that the, "*temperature variation in components of the electronics system is a crucial variable that determines the uncertainty in CPT data*" as the zero load output may be shifted by a temperature variation. Temperature control is difficult in the field as the ambient temperature of the soil, the heat generated by the strain gauges in the penetrometer and the heat generated by friction acting on the cone penetrometer, all influence the temperature of the cone penetrometer. The electronics components in the drilling rig could theoretically be kept at a constant temperature. Post and Nebbeling suggested that operators include penetration stops when entering a clay layer following a sand layer to allow the temperature within the penetrometer to stabilise.

The University of Canterbury drilling rig, shown in Figure 6-3, had a total weight of six tonnes and the cone itself belonged to Fugro's sensitive range, with a tip-force limit of 5 tonnes. These two factors limited the depth to which the cone could be driven. For instance, gravel layers between 1.2 and 1.6 metres and again at 5.6 metres limited the use of the CPT in Greymouth as they caused the lifting of the drilling rig. Drilling through these gravel layers, using flight augers, enabled CPT tests to be made to greater depths.

Piezocone tests have been undertaken in the past by the Canterbury University Geomechanics Group. Foray and Berrill (1987) found that even with a very sensitive pore-pressure transducer and data acquisition system due to the high permeability of liquefiable soils, low values of pore pressure were measured in the test areas. The dissipation times were also short. Foray and Berrill concluded that in clean sands at least, pore pressure measurement added little information. In silty sands, where excess pore pressures are higher and more persistent, filter clogging presents a major difficulty. These tests were not undertaken due to these difficulties.

Figure 6-3. University of Canterbury Drilling Rig at Arapito (February 2003)

Seismic Cone Penetrometer Testing

The Seismic Cone Penetrometer Test (SCPT) combines the logging of the CPT test with the seismic downhole testing methods. A seismic cone is pushed to the desired depth in the ground before a seismic pulse is generated at the ground surface by hitting a shear plate on the ground surface with a sledge hammer. When the hammer hits the plate, the shear waves generated are recorded by the accelerometer or triaxial geophones located in the cone. Multiple traces are recorded at each depth before the cone is pushed to the next depth, and from the shear wave velocities the soil profile can be inferred (Robertson *et al.*, 1986).

The method does have disadvantages. The onset of liquefaction is a medium to high strain phenomena whereas the seismic wave measurements are made at very small strains. This means that the thin, low shear wave velocity strata are missed if the interval is too large.

Seismic Cone Penetrometer testing was also attempted, however, excess noise due to the generator used made the data from the SCPT difficult to analyse. This noise problem was not able to be overcome and hence this testing method was abandoned in favour of further simple CPT tests.

6.2.3 Laboratory Testing

Sub-surface samples were obtained for a number of sites to determine particle size distributions, based on sieve and sedimentation testing. The tests were performed to the specification of NZS4402:1986 in the University of Canterbury Geomechanics Laboratory. Both the sieve and sedimentation tests were carried out on oven-dried samples and sodium hexametaphosphate was used as a dispersant in the sedimentation testing. The results are shown in Appendix A.

7 Analysis Procedures

A number of different analysis techniques were used in this study to enable the comparison of the different methods available for predicting the occurrence of liquefaction at a site. A site is considered to have a high probability of liquefying if all methods predict the liquefaction of a particular layer.

7.1 Peak Ground Accelerations and Earthquake Magnitudes

The occurrence of liquefaction depends on both the susceptibility of the soil and the strength of the shaking. Most of the liquefaction prediction models characterise the strength of shaking by the peak ground acceleration. The peak ground acceleration (a_{max} or PGA) is derived, though with much uncertainty, by attenuation relations giving a_{max} as a function of the magnitude of the earthquake and the distance between the site and the source of the rupture. For most New Zealand earthquakes these values must be deduced; no accelerometers existed in the country at the time of the 1929 Murchison Earthquake, and unfortunately the equipment on the West Coast did not function at the time of the 1968 Inangahua earthquake. The PGAs used in the study came from the old DSIR seismoscopes - the scratch plate device. They are quite stiff and thus record PGA directly.

7.1.1 Earthquake Magnitudes

Youd (1997) recommends the use of the moment magnitude for the calculation of the liquefaction resistance of a site as it is calculated from the "*amount of seismic energy released in an earthquake rather than the peak magnitude of an accelerogram trace*" which is used in other methods of determining earthquake magnitudes.

Youd (1997) cited the work of Heaton *et al.* in a 1986 paper, for the relationships between the different magnitude scales. These are shown in the plot in Figure 7-1. From this plot it can be seen that all magnitude scales, except the moment magnitude, saturate at some level. This saturation indicates that the magnitude scales do not distinguish between large and very large earthquakes.

The magnitude of the 1929 Murchison earthquake has been based on felt intensity reports and is given as surface magnitude 7.8 (Downes, 1995). The moment magnitude of the 1968 Inangahua earthquake has been given as 7.23 (Zhou et al., 1997), and the Hawks Crag earthquakes as local magnitudes 6.1 and 6.2 (Zhou et al., 1997).

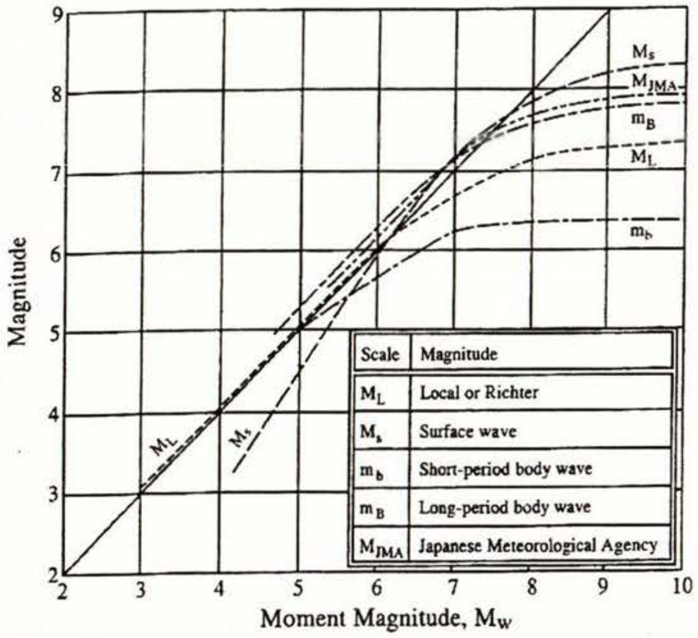


Figure 7-1. Relationship between the different magnitude scales (Youd, 1997)

7.1.2 Magnitude Scaling Factors

The 1996 NCEER and 1998 NCEER/NSF workshops on the Evaluation of Liquefaction and Liquefaction Resistance of Soils recommended an analysis technique to determine the liquefaction potential of a soil based on the SPT, CPT and SCPT tests. The workshop participants also indicated how the procedures should be altered for different magnitude earthquakes.

The 1996 NCEER Workshop participants recommended using magnitude scaling factors to evaluate the likelihood of liquefaction as most prediction methods use a magnitude 7.5 earthquake as the base of their calculations (Youd and Idriss, 1997). A summary paper was written in 2001 outlining the recommendations of the NCEER workshop participants (Youd *et al.*, 2001). In that report it was stated that a consensus was reached and for earthquakes of magnitude less than 7.5, the values given by Idriss should be used as lower bound magnitude scaling factors and those determined by Andrus and Stokoe (1997) used as the upper end of the range. For earthquakes of magnitude greater than 7.5, the Idriss magnitude scaling factors were also recommended as they are more conservative than the original Seed and Idriss (1971) values. The workshop participants felt that, based on the evidence and verification work done so far, these values were conservative enough. As these values are conservative, some areas which were previously classified as non-liquefiable are classified as liquefiable using the recommended factors. The factors recommended by the Youd *et al.* (2001) are given in Table 7-1, and have been used as the scaling factors in this study.

Table 7-1. Magnitude Scaling Factors

M_W	*Idriss*	*Andrus and Stokoe (1997)*
	M < 7.5 Lower Bound; M > 7.5	*M < 7.5 Upper Bound*
5.5	2.20	2.80
6.0	1.76	2.10
6.5	1.44	1.60
7.0	1.19	1.25
7.5	1.00	1.00
8.0	0.84	0.8? (very uncertain value)
8.5	0.72	0.65? (very uncertain value)

7.1.3 Peak Ground Accelerations (PGA)

The peak horizontal ground accelerations, a_{max}, used in most models are those that would occur at the ground surface, at a site, in the absence of liquefaction or pore pressure increases generated by the earthquake (Youd *et al.,* 2001). In order to estimate a_{max}, Youd *et al.* (2001), recommend the use of "*attenuation relationships compatible with soil conditions at a site*", and using relationships that are based on the geometric mean of the peak horizontal accelerations.

The method of Zhou *et al.* (1997) was used in this study to estimate the peak ground acceleration for sites where it was not recorded. This model is predominantly based on New Zealand data but incorporates data from overseas with source distances of less than 10 km to obtain attenuation models giving reasonable predictions of the near source peak ground accelerations. However, in using this data, Zhou *et al.* noted that New Zealand earthquakes may produce different near source PGAs to other parts of the world, though gave no further reference to why this may be. They also note that the New Zealand attenuation appeared to be different to the attenuation models based on European data and those developed in the Western USA while the attenuation was "*similar to that given by models based on Japanese data*" (Zhou *et al.*, 1997).

The New Zealand Loading Standard (NZS 4203: 1992 Loadings Standard) classifies soil into ground classes A, B and C where:

Ground class A = rock or very stiff soil sites with a natural period < 0.25s

Ground class B = intermediate soil sites

Ground class C = flexible or deep soil sites with a natural period > 0.6s.

These classifications have been used by Zhou *et al.* for the New Zealand Recorder sites, though sub classes are used as well to further classify the sites in terms of topographic effects, layer thickness and average shear wave velocity of the soil layers.

In developing the model it was recognised that fault mechanism, tectonic type, ground class, magnitude and source distance all affect the strength of the shaking and as such the model was evaluated based on the following components:

PGA = peak ground acceleration of the stronger of the two horizontal components in units of gravity, *g*

M_W = moment magnitude

r = shortest distance from the fault rupture surface to recording station (km)

d = a constant used to "*restrain the near source PGA prediction*" (Zhou *et al.*)

h_c = the centroid depth of the rupture surface

A residual analysis was carried out in order to form the final model based on different earthquake types and mechanisms, and site conditions as follows:

$$\log_{10}(PGA) = A_1 M_W + A_2 \log_{10}\sqrt{r^2 + d^2} + A_3 h_c + A_4 + A_5\delta_R + A_6\delta_A + A_7\delta_I \quad (1)$$

where

δ_R = 1 for all crustal reverse faults

0 for all other events

δ_A = 1 for all rock sites

0 for all soil sites

δ_I = 1 for all events at the interface between tectonic plates

0 for all other events such as crustal and slab events

The above equation can then be evaluated based on the data available in terms of five different models, for which the parameters A_1 to A_7 are given in Table 7-2, and the models described below:

Model 1: Derived using data from only the 'main seismic region' and solving for all of the parameters.

Model 2: Formulated by only considering soil site data. This model gives similar PGA's as Model 1 for soil sites.

Model 3: This model was developed to calculate the PGA where the site conditions are unknown or not specified and for soil sites this method gives similar results to those of Models 1 and 2 as soil sites dominate the dataset over rock sites.

Model 4: For situations where the focal mechanism and tectonic type are unknown, though the site conditions are known.

Model 5: This is for use when only the distance, depth and magnitude are known; as there are fewer inputs the model has the greatest standard error of all the models.

Table 7-2. Parameters used in attenuation model (Zhou et al, 1997)

	Model 1	*Model 2*	*Model 3*	*Model 4*	*Model 5*
A_1	0.298	0.289	0.297	0.331	0.331
A_2	-1.56	-1.53	-1.58	-1.58	-1.59
A_3	0.00619	0.00611	0.00576	0.00604	0.00566
A_4	-0.365	-0.357	-0.333	-0.509	-0.490
A_5	0.107	0.108	0.101	-	-
A_6	-0.186	-	-	-0.190	-
A_7	-0.124	-0.111	-0.141	-	-
D	19	19	20	19	20

Zhou *et al.* refer to a number of researchers including Dowrick (1991) and state that it has been "*found that earthquakes with predominantly reverse focal mechanisms produce stronger shaking for the same magnitude and distance than do events with predominantly normal or strike-slip mechanisms*".

The authors do not recommend the use of the attenuation models for earthquakes with a moment magnitude greater than 7.4 as this is beyond the range of the data used in developing the equations. For earthquakes with a moment magnitude greater than seven, Zhou *et al.* found that the predicted PGAs are too small for rock sites and too large for the weaker soils. This effect can be seen in the results of the calculations in Table 7-3 below where the calculated values are much greater than those measured for both Westport and Murchison, following the 1968 Inangahua earthquake. The measured values of peak ground acceleration from the 1968 Inangahua earthquake have been used in the analysis where possible. The PGAs for Karamea were evaluated using the attenuation rules derived by Zhou *et al.* (1997) and given in Table 7-3.

For the 1929 Murchison earthquake, peak ground accelerations were inferred from the isoseismal maps and the magnitude of the earthquake. For example, a PGA of 0.4g was used for Greymouth as this was recorded at this site for the smaller Inangahua earthquake and is consistent with the felt intensity reports which were similar for both events.

Table 7-3. Peak ground accelerations used in this study

Location	*1929 Murchison (M 7.8)*		*1968 Inangahua (M_W 7.23)*			*1991 Hawks Crag (M 6.2)*		
	Calculated	Used	Recorded	Calculated	Used	Recorded	Calculated	Used
Oparara	0.228g	0.228g	-	0.135g	0.135g	-	-	-
Karamea	0.250g	0.250g	-	0.148g	0.148g	-	-	-
Arapito	0.269g	0.269g	-	0.150g	0.150g	-	-	-
Murchison	1.134g	1.134g	0.29g, 0.364g	0.425g	0.364g	0.202g, 0.29g	0.070g	0.29g
Inangahua	0.638g	0.638g	-	0.743g	0.6g	0.186g, 0.168g	0.181g	0.186g
Westport	0.279g	0.279g	0.302g, 0.302g	0.338g	0.302g	0.142g, 0.158g	0.246g	0.158g
Greymouth	0.081g	0.4g	0.22g, 0.391g	0.070g	0.391g	0.058g, 0.057g	0.047g	0.058g

7.2 Prediction Methods Used in Analysis

Most analysis techniques to assess the liquefaction potential of a site are based on case history data indicating sites which did and did not liquefy under past earthquake shaking. Empirical relationships are then based on earthquake data and in-situ testing such as cone penetrometer testing to obtain soil resistance measurements. From here there are two schools of thought. The first involves considering the possible dynamic shear stress induced at a site under given earthquake conditions (e.g. Seed and Idriss, 1971; Robertson and Campanella, 1985), and the second, the dissipated seismic energy from the earthquake mechanism (e.g. Davis and Berrill, 1982; Liao *et al.*, 1988).

7.2.1 Stress Approach:

A number of liquefaction potential prediction models are based on the work of Seed and Idriss (1971) and the subsequent modifications made to that work. In this method, curves were determined empirically to predict the no-liquefaction and liquefaction occurrence. Juang *et al.* (2000) note that, "*these empirical curves are essentially performance functions that were established from field observations of soil performance in earthquakes at sites where in situ test data were available*". From this, one can consider the situation a limit state condition where the dividing line between liquefaction and non-liquefaction is where the applied seismic demand is equal to the liquefaction resistance of the soil.

The liquefaction resistance of a soil is usually expressed as the cyclic resistance ratio (CRR) and the applied seismic demand in terms of the cyclic stress ratio (CSR). This means that at the limit state, the shear strength is equal to the shear stress at failure, or CRR is equal to CSR.

All the methods based on this philosophy rely on a degree of engineering judgement to draw the dividing line between liquefaction and no liquefaction, and fix the level of conservatism involved. The methods are also derived for clean sands and based on magnitude 7.5 earthquakes and therefore require other discrete curves to take into account the soil types, fines contents and earthquake magnitudes (Juang *et al.*, 2000).

Law *et al.* (1990) state that, "*The stress method for assessing liquefaction potential is based on the maximum peak horizontal acceleration at the ground surface. This implies that some part of ground motions is selectively accounted for while another part is being ignored*" and hence they feel that there is a deficiency in this method of analysis. The peak accelerations are also directionally dependant; in the stress methods only the peak horizontal acceleration is used, though Law *et al.* cite Atkinson as noting the vertical acceleration may have an influence in the occurrence of liquefaction.

7.2.2 Energy Approach:

An energy approach has been employed by a number of authors as energy is dissipated as the soil deforms under cyclic loading such as that caused by earthquake shaking. The amount of energy dissipated is equal to the area within the hysteretic loop. As noted earlier, the energy dissipated in the soil is related to the volume change due to cyclic loading and functions describing this can be obtained experimentally. Following the hypothesis of Nemat-Nasser and Shokooh (1979), Davis and Berrill (1982) note that the cumulative dissipated energy is related to the excess pore pressure generated in the soil. The relative density, the consolidation pressure and consolidation ratio are also uniquely related to the excess pore pressure.

Law *et al.* in 1990 wrote, "*the total energy travelling through and dissipated in a soil media remains unchanged whether part of the motions is amplified or attenuated*" and due to this "*the complete spectrum of ground motion giving rise to liquefaction failure*" is accounted for in the energy approach.

Bardet *et al.* (1997) wrote that the many energy methods for liquefaction analysis, including those used in this study, need to be validated for large accelerations such as those measured at Port Island, Kobe. They also note that energy based techniques have an advantage over methods based on PGA as they are more representative of the excitations that cause liquefaction as they account for the intensity and duration of the motion. However, Bardet *et al.* feel more research should be done in order to "*characterise earthquake ground motions in terms of energy*".

In this study a number of methods have been employed to predict the occurrence of liquefaction. Table 7-4 indicates the approach taken by the different authors. Each of the methods listed below is based on slightly different requirements and input parameters.

Table 7-4. Methods used in this study

Stress Based Prediction Methods	*Energy Based Prediction Methods*
Shibata and Teparaksa (1988)	Zhou (1980)
Suzuki, Tokimatsu, Koyamada, Taya and Kubota (1995)	Davis and Berrill (1982)
Olsen (1997)	Liao, Veneziano and Whitman (1988)
Robertson and Wride (1998) as NCEER (2001)	Law, Cao and He (1990)
Juang, Yuan, Lee, Lin (2003)	
Toprak and Holzer (2003)	

A number of the prediction methods were derived based on SPT-N values due to the large dataset available to the authors. A number of correlations between the SPT-N values and the CPT-q_c values have been proposed such as that of Meyerhof (1956) where the conversion is given in MPa as follows:

$$q_c = 0.4N \tag{2}$$

Work undertaken by Robertson and Campanella (1985) suggest that the mean grain size D_{50} influences the relationship between the tip resistance, q_c values and N values. Vreugdenhil (1995) compared the N values and q_c values and found that for Loma Prieta data there was a large scatter in the correlation results. Based on the data Vreugdenhil considered reliable data the average q_c/N value was given as 0.4 MPa. The curves derived by both Vreugdenhil and Robertson and Campanella show the same behaviour though the value derived by Vreugdenhil was slightly lower than that calculated by Robertson and Campanella.

Stark and Olson (1995) proposed another correlation between the CPT tip resistance and SPT blow count as shown in Figure 7-2. They note that for given D_{50} values there is a large variation in the ratio of q_c/N_{60}. The conversion from q_c to N values was based on liquefaction potential relationships derived using CPT data and those based on SPT data from field liquefaction performance. Stark and Olson note that while the proposed conversion of CPT to SPT data is more representative than the then existing conversions, it is still more desirable to use site specific conversions.

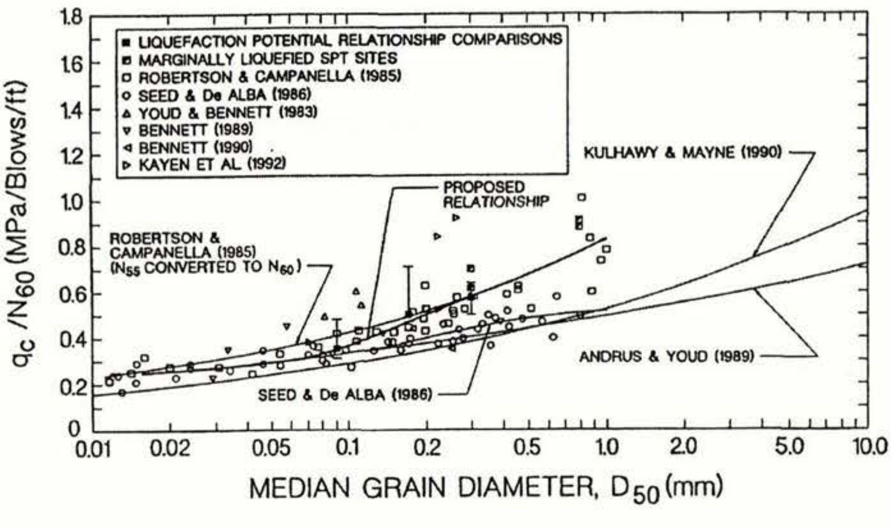


Figure 7-2. Conversion of SPT-*N* values to CPT-q_c values (from Stark and Olson, 1995)

The CPT has the advantage of giving continuous results, with the cone resistance, q_c, giving a measure of the resistance or overall strength of the soil. In cohesionless soils this can be interpreted as a measure of the soil density. Both Olsen (1997) and Robertson and Wride (1998), use the tip resistance and side friction measurements to infer the soil profile. Olsen notes that the, "*CPT sleeve friction resistance is an index of remoulded strength after breakage of the soil structure and after soil has undergone large strain*".

7.2.3 Analysis Models

In the following section, the models used in the analyses will be discussed in more detail.

Tsuchida and Hayashi (1970)

The particle size distributions for the soil samples collected in the field were plotted against curves developed by Tsuchida and Hayashi in 1970 to indicate the potential of certain sized particles to liquefy (National Research Council, 1985). These plots can be found in Appendix A.

Zhou (1980)

Zhou (1980) developed a method to evaluate the liquefaction potential of a site based on static cone penetration tests undertaken in both liquefied and non-liquefied soil in the Tangshan seismic area following the 1976 Tangshan, China earthquake. Recent silt and fine sand of low density make up most of the surface layers in this region.

At any given site the critical cone resistance is expressed as an empirical function based on the critical penetration resistance as a function of the earthquake intensity I and a given density, $q_{co}(I)$ (kgf/cm^2), the depth to the water table H_W (m) and the thickness of overlying strata H_O (m).

$$(q_c)_{critical} = q_{co}(I)(1-0.065[H_W-2])(1-0.05[H_O-2]) \quad (3)$$

To produce a model consistent with the Chinese Building Code, in which the Chinese Intensity scale is used to obtain the seismic loads, Zhou considered q_{co} values based on the actual intensity of the Tangshan earthquake and the average distance to the isoseismals. These were then tabulated; the values of interest are shown below in Table 7-5. Guilhem and Berrill (1993) noted that the Modified Mercalli scale is approximately equivalent to the Chinese Intensity scale.

Table 7-5. Critical Cone Resistance as a function of intensity and distance (from Zhou, 1980)

Intensity	7 (0.1g)	8 (0.2g)	9 (0.4g)	10
Critical Penetration Resistance (kg/cm2)	46.7	116.6	176.9	221.7

The penetration resistance used by Zhou is inferred to be the raw value of q_c. The expression is also given in kgf/cm2 units and conversion to MPa units gives the following where the depth to the water table is given by H_W and the depth of overburden, H_O, both of which are in metre units.

$$(q_c)_{critical} = 0.0981 q_{co}(I)(1-0.065[H_W-2])(1-0.05[H_O-2]) \quad (4)$$

Zhou did note that this method may not be valid for situations where there are there is a light sandy clay foundation with a high clay content as the method had been formulated, based on a uniform medium sand, fine sand and silt foundations.

Davis and Berrill (1982)

The Davis and Berrill (1982) model uses the magnitude and distance to the earthquake source to characterise the seismic energy arriving at a site. This semi-empirical energy based model for estimating the potential of a site to liquefy applies the hypothesis of Nemat-Nasser and Shokooh (1979). Nemat-Nasser and Shokooh proposed that the density of seismic energy dissipated in the soil is related to the increase of pore pressure in the soil.

To obtain the density of seismic energy dissipated in the soil the model combines the Gutenburg and Richter expression for total radiated energy and a geometric spreading relationship. Combining these with laboratory results for the fraction of arriving energy dissipated in the soil and calibrating the resulting expression against case-history data, Davis and Berrill derive an expression for pore pressure increase (Δu) as

$$\Delta u = \frac{450 \cdot 10^{1.5M}}{r^2 \overline{N}^2 \sqrt{\sigma'_o}} \quad (5)$$

where

M = earthquake magnitude

r = epicentral distance (m)

$\overline{N}$ = corrected SPT-N value

σ_o' = initial effective overburden stress

In the original model the relationship for correcting the SPT-N value was one developed by Peck as follows:

$$\overline{N} = 0.77\log_{10}\left(\frac{2000}{\sigma_o'}\right)N \tag{6}$$

At the onset of liquefaction, or the condition of initial liquefaction, the pore pressure increase equals the effective confining stress so that in loose sand the strength becomes small. Using the Meyerhof relationship, a critical $(q_c)_{crit}$ value for the onset of liquefaction can be calculated as follows.

$$(q_c)_{crit} = \frac{0.4}{0.77}\sqrt{\frac{450\cdot 10^{1.5M}}{r^2(\sigma_o')^{\frac{3}{2}}}}\left[\log_{10}\left(\frac{2000}{\sigma_o'}\right)\right]^{-1} \tag{7}$$

The use of the $(1/r^2)$ attenuation rule also means that the method does not work well in near field situations as in this case the rupture cannot be idealized as a point source.

Shibata and Teparaksa (1988)

The model developed by Shibata and Teparaksa is based on the original work by Seed and Idriss (1971) but uses the CPT-q_c values rather than SPT-N values. In using this method, the cyclic stress ratio developed in the field during an earthquake, is calculated from the magnitude of the earthquake, the maximum horizontal ground acceleration at the site of liquefaction (in units of gravity), the effective and total vertical stress and the depth, z, by the following expression:

$$CSR = \left(\frac{\tau}{\sigma_o'}\right)_{ST} = 0.1(M-1)\frac{a_{\max}}{g}\frac{\sigma_o}{\sigma_o'}(1-0.015z) \tag{8}$$

The critical q_c value is based on the cyclic stress ratio and also depends on the median grain size (D_{50}) in mm. This is a limitation to this model as D_{50} must be assumed or determined in the laboratory. For this study, particle size distribution (PSD) curves were drawn from field samples and D_{50} values obtained from these to use in the modelling. The PSD curves can be found in Appendix A.

A normalised critical q_c value ($(q_{c1})crit$) in MPa is calculated from the grain size and the *CSR* as follows:

$$(q_{c1})_{crit} = C_2\left\{5 + 20\left(\frac{CSR - 0.1}{CSR + 0.1}\right)\right\} \tag{9}$$

where

$$C_2 = \frac{D_{50}}{0.25} \leq 1.0 \tag{10}$$

From this the critical cone resistance for liquefaction to occur can be evaluated using the following expression:

$$(q_c)_{crit} = (q_{c1})_{crit}\left[\frac{\sigma'_o + 70}{170}\right] \tag{11}$$

Vreugdenhil (1995), in analysing data obtained from CPT tests undertaken at sites which liquefied in the 1989 Loma Prieta earthquake, found that this model performed well in that it gave a high number of correct predictions of liquefaction at given sites. The disadvantage of the model is that it often assigns a liquefiable status to non-liquefiable materials and seems to be over conservative.

The threshold value of cone resistance, predicted by the Shibata and Teparaksa model, increases with depth. This contradicts the fact that liquefaction is unlikely to occur below a depth of 15 metres.

In the application of both the methods of Davis and Berrill and Shibata and Teparaksa in this study, a model developed by Robertson and Campanella (1985), has been used to identify whether or not soil is liquefiable based on the cone resistance and friction ratio. This, however, is a rough model, and as stated by Robertson, it should only be used as a preliminary indicator as to whether or not liquefaction is likely to occur. This is because a sharp division between a liquefiable and a non-liquefiable soil is unrealistic given the state of knowledge regarding the occurrence, or not, of liquefaction.

Liao, Veneziano and Whitman (1988)

Liao *et al.* (1988), made a rigorous statistical examination of various published models using a catalogue of 283 cases of liquefaction/ non-liquefaction occurrences from 40 earthquakes. They introduced a set of models according to whether they characterised the seismic loading by source parameters or local peak ground accelerations.

A model based on source parameters was used for this study. Liao *et al.* found that while the Davis and Berrill model did not work well for dense sands, nor in the near field of the earthquake source (as the rupture cannot be idealized as a point source) they found that, in general, the loading term characterised the influence of the earthquake well. Liao *et al.* defined the seismic loading term as follows:

$$\Lambda_{EP} = \frac{10^{1.5M}}{(R_{EP})^2(\sigma_v^{'})^{1.5}} \tag{12}$$

This equation uses the earthquake magnitude, M, the distance to the epicentre, R_{EP}, and the effective overburden stress, σ_v'. This can be normalised by using the above equation and the following values: $\sigma_v' = 1.0$ kg/cm2; $R_{EP} = 100$ km and $M = 7.5$ to give Λ_0.

A probability of liquefaction, *PL*, was also defined and is given in the following equation.

$$P_L = \frac{1}{1+\exp(12.922-0.87213\ln(\Lambda_{EP})+0.21056(N_1)_{60})} \tag{13}$$

Liao *et al.* notes that, "*the source models are insensitive to the effects of fines content*" when compared to local models. They write that this may be because the uncertainty in the attenuation relationships used by the source models may obscure the effects of the grain size.

Models are also defined for silty sands and sands based upon the stress approach developed by Seed and Idriss (1971). In these models, the relationship developed by Seed (1985) to calculate the normalised cyclic stress ratio (*CSRN*) is used. This is of the form:

$$CSRN = \left(\frac{\tau}{\sigma_v^{'}}\right)_{M=7.5} = 0.65\left(\frac{a}{g}\right)\left(\frac{\sigma_v}{\sigma_v^{'}}\right)\left(\frac{r_d}{r_M}\right) \tag{14}$$

In equation 14, a/g is the peak surface ground acceleration as a fraction of gravity and σ_v' and σ_v are the effective and total overburden stress respectively. The depth reduction factor is given by r_d and the magnitude normalisation factor to convert the *CSR* to an equivalent M7.5 earthquake is given by r_M. From this regression relationship, models are developed for determining the probability of liquefaction for both clean sand and silty sand according to the case history data set. Youd and Noble (1997) have studied this work and developed further stress approach models. They note that, "*the plasticity of the fines may also have a significant influence on liquefaction resistance that is not accounted for in the present criterion*".

Law, Cao and He (1990)

Law *et al.* (1990) undertook triaxial cell tests and cyclic torsional shear tests to find that under cyclic loading the excess pore pressure for a given sand can be related to the consolidation ratio, consolidation pressure, the relative density and the cumulative dissipated energy as illustrated in the equation below:

$$\frac{\Delta u}{\sigma_h^{'}} = \alpha\left[F_1(K_c)F_2(D_r)\sum\frac{w}{\sigma_h^{'}}\right]^{\beta} \qquad (15)$$

In this equation, $F_1(K_c)$ is the normalising function to account for the consolidation ratio, K_c and $F_2(D_r)$ is the normalising function to account for the relative density, D_r. The cumulated energy per unit volume is given by $\sum w$; Δu is the excess pore pressure and σ_h' is the consolidation pressure in the horizontal direction.

To develop their energy based model, Law *et al.* adopted the widely used relationship by Gutenburg and Richter, published in 1956, to calculate the total energy, in units of Joules, released from an earthquake of magnitude M as follows:

$$E = 10^{4.8+1.5M} \qquad (16)$$

Knowing that only a fraction of the total energy generated at the source will arrive at the site, as the energy will be dissipated due to geometric damping and material attenuation along ray paths, the seismic energy arriving at the site was written in terms of the epicentral distance and the soil state at the site. Law *et al.* give an expression for the, "*attenuation equation for the seismic energy per unit volume arriving at a site (E_1)*", as a function of the hypocentral distance, R.

$$E_1(E,R) = \frac{\theta E}{R^B} \qquad (17)$$

They state that, "*θ is assumed to be a constant and B is a coefficient depending on the properties of the rock though which the seismic waves traverse; its value ranges from 2.5 to 5.0*". Both the stress system and the relative density of the soil affect the stress state. Law *et al.* also found that the energy arriving at different depths was not significantly different as the distance to the epicentre is in reality often tens of kilometres away and the soil layers tens of metres thick.

Using the results from SPT testing, they proposed the following equation where liquefaction will take place if the liquefaction resistance function ($\eta_L(N_1)$) is exceeded by the seismic intensity function ($T(M,R)$), as shown in the following inequality.

$$E_1(E,R) = \frac{\theta E}{R^B} \tag{18}$$

The functions T and η_L were then evaluated based on case history data, using earthquakes which occurred in China, USA, Japan and Chile. Details used included identification of liquefaction occurrence, the earthquake magnitude, epicentre or hypocentre location and the distance to the site in question, the soil profile, depth to the water table, peak ground acceleration, standard penetration resistance and the average induced stress ratio, τ_{av}/σ_v'.

The seismic energy intensity function was then developed from the attenuation characteristics of the seismic energy. For highly fractured rock masses, there is a high energy absorption resulting in high energy attenuation. Law *et al.* cite Hasegawa *et al.* (1981) as noting that for highly fractured rock, such as in western Canada and the Western USA, the coefficient B is 4.3 ± 0.5. This resulted in the following equation where R is in kilometres.

$$T(M,R) = \frac{10^{1.5M}}{R^{4.3}} \tag{19}$$

The corrected SPT resistances were then plotted against the seismic energy intensity functions for both sand and silty sand sites, to determine the boundary between liquefied and non-liquefied sites. Based on this division, the following equations can be written to illustrate the different behaviour. For liquefaction to occur $T(M,R)/\eta_L(N_1)$ must be less than unity, as shown below.

Sand:

$$\frac{T(M,R)}{\eta_L(N_1)} = \frac{10^{1.5M}}{2.28(N_1)^{11.5} \times 10^{-10} \times R^{4.3}} \geq 1.0 \tag{20}$$

Silty Sand:

$$\frac{T(M,R)}{\eta_L(N_1)} = \frac{10^{1.5M}}{1.14(N_1)^{11.5} \times 10^{-9} \times R^{4.3}} \geq 1.0 \tag{21}$$

Suzuki, Tokimatsu, Koyamada, Taya and Kubota (1995)

Suzuki *et al.* (1995) again recommended the equations given by Shibata and Teparaksa (1988), and Tokimatsu and Yoshimi (1983) to calculate the shear stress ratio. Their method differs in that the tip resistance is normalised to σ'_r, an effective overburden pressure of 98 kPa as follows:

$$q_{t_1} = \frac{q_t}{\left(\frac{\sigma'_v}{\sigma'_r}\right)^{0.5}} \tag{22}$$

It is noted that when the modified cone resistance is plotted against the friction ratio for both liquefied sites and non-liquefied sites the boundary separating these states is clearly defined.

Olsen (1997)

Olsen (1997), recommends the following procedure to determine the soil type and liquefaction potential, based on the CPT tip resistance and friction ratio.

1) Calculate the cyclic stress ratio based on the earthquake magnitude and surface ground acceleration

2) Determine the normalised liquefaction cyclic resistance ratio, CRR_1, using the generalised equation given below. Olsen writes, "*this simplified procedure uses a constant stress exponent of 0.7 (for all soil types) to normalise the cone resistance.. and was formulated to produce conservative CRR_1 values for clays and silt mixtures*", in comparison to the more complicated iterative chart method developed by Olsen and Mitchell (1995).

$$CRR_1 = \left(0.00128\frac{q_c}{\left(\sigma'_v\right)^{0.7}}\right) - 0.025 + \left(0.17R_f\right) - \left(0.028R_f^{\ 2}\right) + \left(0.0016R_f^{\ 3}\right) \tag{23}$$

In this equation the following are used: the cone resistance, q_c, has been normalised with respect to $\sigma'^{\,0.7}_v$, the vertical effective stress to give the normalised cone resistance in units of atmosphere, q_{c1}, and the friction ratio, R_f. Olsen writes that "*for vertical effective stresses greater than one atmosphere (atm), an approximating linear stress normalization technique produces results which are increasingly overconservative*". Olsen also notes that equation 23 is very conservative for overconsolidated conditions.

3) The CPT soil classification number (SCN) is then calculated according to:

$$SCN = -3.58 + 2.08\sqrt{(0.76*(1.3 - Log_{10}R_f)^2) + (Log_{10}q_{c1})^2} \tag{24}$$

Equation 24 uses the friction ratio as a percentage and the normalised cone resistance in units of atmosphere as follows:

$$q_{c1} = \frac{q_c}{\sigma_v'^{0.7}} \tag{25}$$

In the expression given in equation 24, *SCN* refers to the soil classification number for a normally consolidated soil and has the following boundaries to enable the soil profile to be inferred.

+ 2	=	boundary between sand and gravel mixture and sands
+ 1	=	boundary between fine sand and silty sand
0	=	represents the behaviour of a pure silt
- 1	=	boundary between clayey silt and a silty clay
- 2	=	boundary between clays and organic peats and unstable soils

These equations are used to give an indication of the soil type and the behaviour such soil represents. This is important in a liquefaction study as the soil type influences the liquefaction resistance of a site and the associated behaviour, and as such is used to improve the accuracy of the results. The soil characterisation chart devised by Olsen and Mitchell (1995) is illustrated in Figure 7-3.

4) Calculate the in situ cyclic resistance ratio (*CRR*) from the normalised CRR (CRR_1) value using the following equation.

$$CRR = CRR_1 * MSF * K_\sigma * K_\alpha \tag{26}$$

In this equation K_σ and K_α represent scaling factors for confining stress and initial shear stress respectively, as defined by Harder and Boulanger (1997). Olsen (1997), notes that both K_σ and K_α factors can be taken as a value of one for non-critical structures and level ground conditions. The cyclic resistance is normalised to a magnitude 7.5 earthquake, using the magnitude scaling factors defined by Idriss and given in Youd and Idriss (1997).

5) Using the above values, Olsen defines the factor of safety against liquefaction as:

$$FS_{liq} = \frac{CRR}{CSR} \tag{27}$$

He continues and states, "*If the FS_{liq} is less than 1.2 then sand boil and slope movement potential should be evaluated.. If the friction ratio is less than 0.8% and FS_{liq} < 1.2 then there is potential* *for slope movement. If the CPT friction ratio is less than 0.5% and FS_{liq} < 1.2 then there is major potential for movement. The potential for sand boil expression is high when the SCN is high, such as SCN > 1...*" (Olsen, 1997).

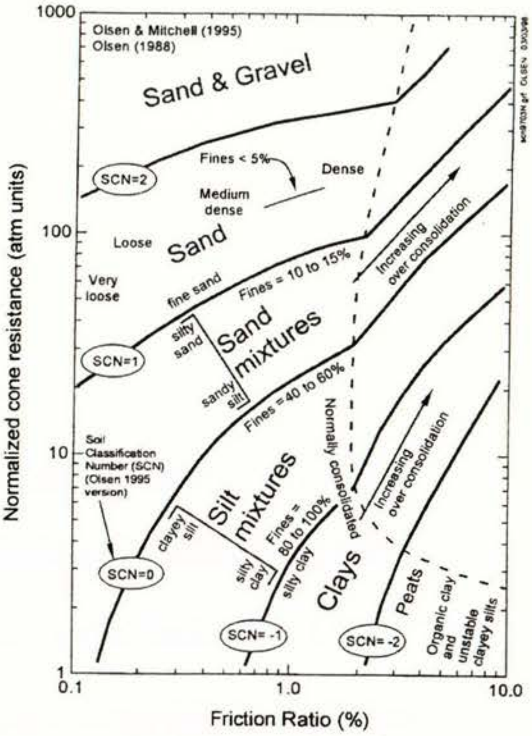


Figure 7-3. Soil Classification Chart (Olsen, 1997)

Robertson and Wride (1998)

For Cone Penetrometer Testing, the method of prediction proposed by Robertson and Wride (1998) has been recommended by the NCEER workshop participants due to its ease of use over other similar methods. It has been built upon the original Seed method and has been developed over the past decade by the authors.

It is noted in Youd and Idriss (1997) that not all members of the 1996 Workshop supported the method. I. M. Idriss felt that there was, "*inadequate development and verification*" of this method and due to this these criteria should not be recommended to the geotechnical profession. Olsen also concluded that, "*the criteria are incorrectly developed and formulated*" and instead recommends the method he has devised. The correlation between the cyclic resistance ratio and cone tip resistance given by Robertson and Wride are, "*empirical and there is some uncertainty over the degree of conservatism in the correlations as a result of the methods used to select representative values of penetration resistance within the layers assumed to have liquefied*" (Youd and Idriss, 1997).

In 1971, Seed and Idriss developed the following equation to evaluate the cyclic stress ratio:

$$CSR = \frac{\tau_{av}}{\sigma'_{vo}} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right)r_d \quad (28)$$

In equation 28, the peak shear stress corresponding to the peak ground acceleration has been scaled by a factor of 0.65 to obtain the average shear stress.

The factor r_d is defined by equation 29, and is stated for use in "*routine practice and noncritical projects*", by Youd *et al.*(2001).

$$\begin{aligned} r_d &= 1.0 - 0.00765\, z && \text{for } z \leq 9.15 \text{ m} \\ r_d &= 1.174 - 0.0267\, z && \text{for } 9.15 < z \leq 23 \text{ m} \end{aligned} \quad (29)$$

Robertson and Wride proposed an equation to obtain, $(q_{c1N})_{cs}$, the equivalent clean sand normalised CPT penetration resistance as follows:

$$(q_{c1N})_{cs} = K_C q_{c1N} \quad (30)$$

In this equation q_{c1N} is the normalised penetration resistance and K_c is a correction factor to take into account the soil grain characteristics. A soil behaviour type chart proposed by Robertson (1990) and shown in Figure 7-4 is used to estimate the grain characteristics.

Robertson and Wride state "*The boundaries between soil behaviour type zones 2 – 7 can be approximated as concentric circles (Jeffries and Davies 1993). The radius of each circle can then be used as a soil behaviour index*" (I_c). This is defined as:

$$I_c = \left[(3.47 - Q)^2 + (\log F + 1.22)^2\right]^{0.5} \quad (31)$$

where

$$Q = \left(\frac{q_c - \sigma_{vo}}{Pa_2}\right)\left(\frac{Pa}{\sigma'_{vo}}\right)^n \quad \text{and} \quad F = \left(\frac{fs}{q_c - \sigma_{vo}}\right) \times 100\% \quad (32)$$

In these expressions, Q is the dimensionless normalized penetration resistance and F is the normalised friction ratio as a percent. Pa and Pa_2 are reference pressures in the same units as σ_{vo} and q_c respectively. The total and effective overburden stresses are given by σ_{vo} and σ_{vo}' respectively.

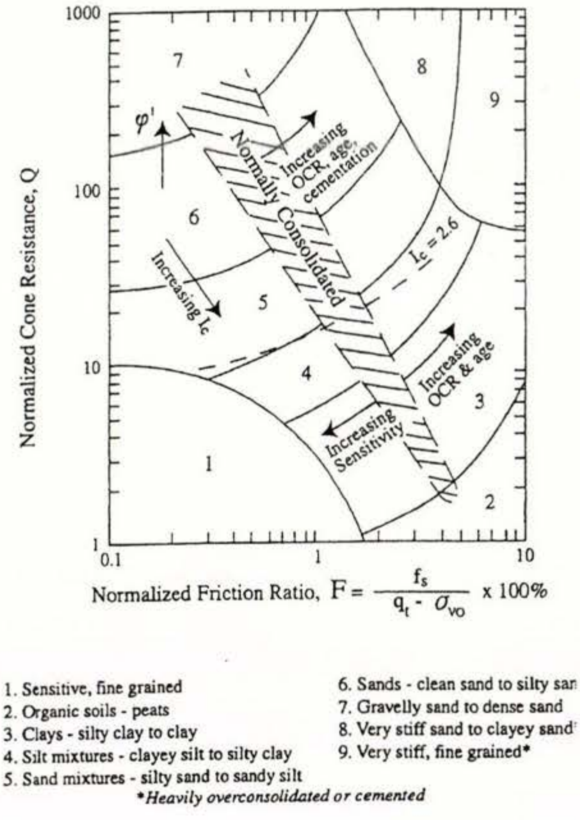


Figure 7-4. Soil behaviour type chart (Robertson and Wride, 1998)

Robertson and Wride also give the soil behaviour type index in terms an I_c value as shown in Table 7-6 below, and drawn as inferred soil profiles, shown in Appendix A and B.

Table 7-6. Soil Behaviour type (Robertson and Wride, 1998)

Soil behaviour type index, Ic	Zone	Soil behaviour type
$I_c < 1.31$	7	Gravelly sand to dense sand
$1.31 < I_c < 2.05$	6	Sands: clean sand to silty sand
$2.05 < I_c < 2.60$	5	Sand mixtures: silty sand to sandy silt
$2.60 < I_c < 2.95$	4	Silt mixtures: clayey silt to silty clay
$2.95 < I_c < 3.60$	3	Clays: silty clay to clay
$I_c > 3.60$	2	Organic soils: peats

Robertson and Wride noted that the soil behaviour type index increases with increasing apparent fines content and soil plasticity along the normally consolidated region. Using this, a simplified relationship was suggested by Robertson and Wride as follows:

if $I_c < 1.26$ apparent fines content FC(%) = 0

if $1.26 \leq I_c \leq 3.5$ apparent fines content $FC(\%) = 1.75\, I_c^{3.25} - 3.7$ (33)

if $I_c > 3.5$ apparent fines content FC(%) = 100

This relationship is approximate as the stress history, mineralogy, soil plasticity and sensitivity are all factors affecting the soil behaviour and hence the response of the CPT.

The correction factor, K_c, can be calculated using the following relationships:

$$\begin{aligned} &if\ Ic \leq 1.64\ \ K_c = 1.0 \\ &if\ Ic > 1.64\ \ K_c = -0.403 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2 + 33.75 I_c - 17.88 \end{aligned} \tag{34}$$

However, if $Ic \geq 2.6$, other criteria must also be used to evaluate K_c. The soil is likely to be non-liquefiable if *F>1%*. Robertson and Write state, "*Caution must be taken in applying the relationship to sands that plot in the region defined by 1.64< Ic<2.36 and F≤0.5%*". This is because sands containing fines and very loose clean sands can be confused. Robertson and Wride suggest that K_c is set to a value of 1.0 in the region to assume that the sand is a clean sand.

A thin layer correction for q_c values is also suggested through the use of a conservative correction factor, developed by Robertson and Fear (1995) for the cone resistance. Robertson and Wride (1998) write "*a slightly improved classification can be achieved if the cone resistance is first corrected for layer thickness before applying the classification charts*".

Using the above equations to determine the equivalent clean sand normalised penetration resistance, the cyclic resistance ratio (CRR) for a magnitude 7.5 earthquake can be calculated using the following equations.

$$\begin{aligned} &if\ \left(q_{c1N}\right)_{cs} < 50 \qquad CRR_{7.5} = 0.833\left[\frac{\left(q_{c1N}\right)_{cs}}{1000}\right] + 0.05 \\ &if\ 50 \leq \left(q_{c1N}\right)_{cs} < 160 \quad CRR_{7.5} = 93\left[\frac{\left(q_{c1N}\right)_{cs}}{1000}\right]^3 + 0.08 \end{aligned} \tag{35}$$

A factor of safety against liquefaction can then be calculated using the CSR and CRR values as follows:

$$FoS = \frac{CRR_{7.5}}{CSR} \times MSF \tag{36}$$

The magnitude scaling factors recommended for use are those given in Youd *et al.* (2001). In this equation a site is considered liquefiable if the factor of safety is less than or equal to a value of 1.0.

The calculation procedure developed by Robertson and Wride (1998) is laid out in the flow chart in Figure 7-5 on the following page.

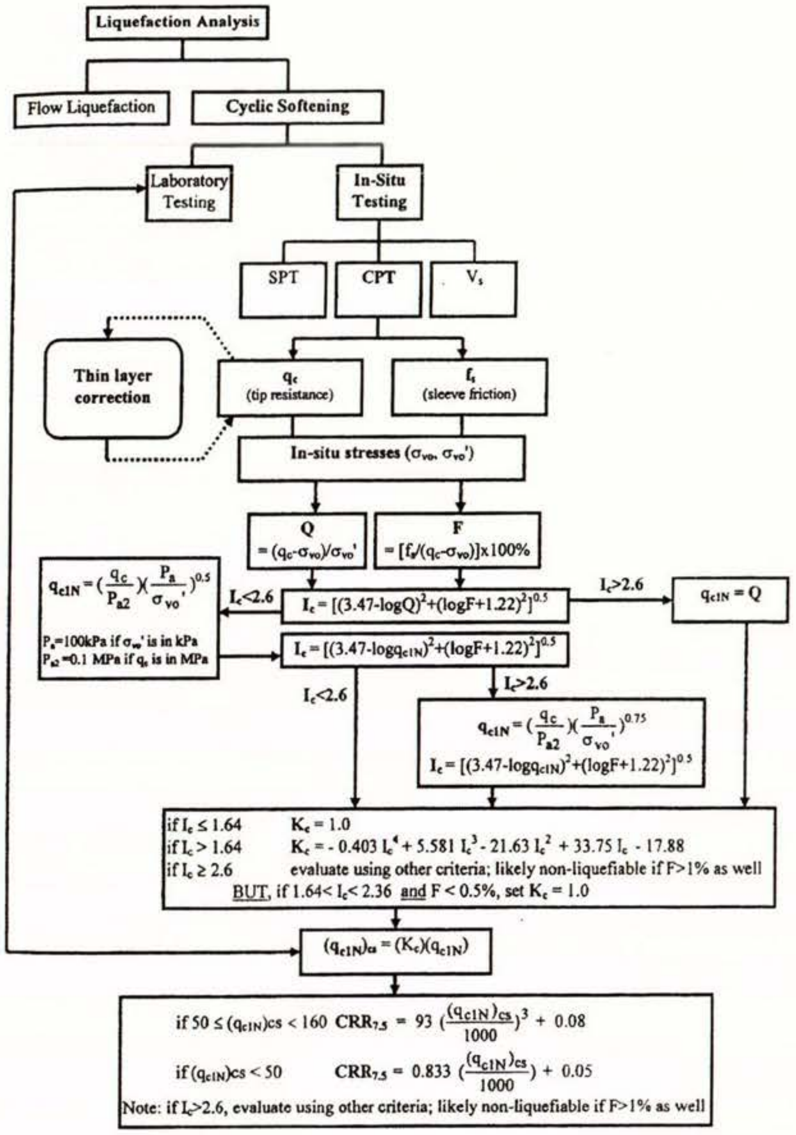


Figure 7-5. Flowchart illustrating the calculation procedure (Robertson and Wride, 1998)

Corrections can also be made for high overburden stresses, static shear stresses and the age of deposit by using K_σ and K_α factors. However, it is noted by Youd *et al.* (2001) that applications of such factors are beyond routine practice and require specialised expertise. Due to this, these corrections have not been made in this study.

Pyke (2003), in a discussion paper regarding the NCEER recommendations, noted that liquefaction resistance described in the paper is only applicable to sites on, "*level to gently sloping terrain, underlain by Holocene alluvial or fluvial sediment at depths less than 15 m*". He also writes that, "*the use of independent curve fits to a number of empirical relationships is questionable*". The simplified procedure is considered to be overly conservative and that it predicts liquefaction where there were no visual evidence of it in terms of sand boils and fissures. He also writes:

> "*.. these standard relationships, which are heavily weighted by data from hydraulic fills and very recent streambed deposits. Additionally, the simplified procedure assumes that soils are horizontally stratified. In deposits such as alluvial fans where soils from successive episodes of deposition tend to be lensed rather than layered, cleaner and looser sands often form a soft inclusion that is surrounded by a stiffer matrix of clayey sands. In such a deposit the strains in the potentially liquefiable material will be controlled by the deformation of the stiffer matrix and the stress developed in the softer material will be much less than those calculated using the simplified procedure.*"
>
> – Pyke (2003)

This is consistent with the results of the present study as can be seen in Chapter 8.

Juang, Yuan, Lee, and Lin (2003)

Juang *et al.* (2003), have developed a method of determining the liquefaction resistance using CPT data, building on from the simplified method originally derived by Seed and Idriss (1971).

They again use the cyclic resistance ratio (CRR), as follows:

$$CSR_{7.5} = 0.65\left(\frac{\sigma_v}{\sigma_v^{'}}\right)\left(\frac{a_{\max}}{g}\right)\left(\frac{r_d}{MSF}\right) \tag{37}$$

In this equation the depth correction factor, r_d, used is that given by Liao *et al.* (1988) and Youd *et al.* (2001):

$$\begin{aligned} r_d &= 1.0 - 0.00765z \quad && for\ \ z \le 9.15\,m \\ r_d &= 1.174 - 0.0267z \quad && for\ \ 9.15m \le z \le 23\,m \end{aligned} \tag{38}$$

The magnitude scaling factors suggested are those derived by Idriss (1998) and recommended by Youd *et al.* (2001), where:

$$MSF = \frac{10^{2.24}}{M_W^{2.56}} = \left(\frac{M_W}{7.5}\right)^{-2.56} \tag{39}$$

These magnitude scaling factors are the lower bound of the range of magnitude scaling factors recommended by the 1996 NCEER and 1998 NCEER/NSF workshop participants.

Juang *et al.* developed a neural network model of the following form where *LI* is the liquefaction index. For liquefaction to occur *LI* = 1, else *LI* = 0 for the non-liquefied case.

$$LI = f_{LI}\left(q_{c1N}, I_c, \sigma_v^{'}, CRR_{7.5}\right) \tag{40}$$

The cone tip resistance, adjusted for the overburden stress, is then calculated based on a formula devised by Lunne *et al.* (1997).

$$q_{c1N} = \frac{q_c / 100}{\left(\sigma_v^{'} / 100\right)^{0.5}} \tag{41}$$

A soil type index, I_c, and normalised friction ratio, F, are both used according to the following equations.

$$I_c = \left[(3.47 - \log_{10} q_{c1N})^2 + (\log_{10} F + 1.22)^2\right]^{0.5}$$
$$F = \frac{f_s}{q_c - \sigma_v} \tag{42}$$

Juang *et al.* note that, "*the soil type index, I_c.. is different from the soil behaviour type index defined by Robertson and Wride (1998). The latter, although more sophisticated, requires multiple steps of calculation, depending on the soil type, whereas the Ic defined..* [above] *involves only one-step calculation*". They also write that the difference between the two definitions is small, between one and seven percent, depending on the I_c value.

An empirical model was established from a series of least square regression analyses, and given below. In these equations, the K_1 parameter used in the model originated from the concept of equivalent clean sand and has no physical meaning according to Juang *et al.*.

$$CRR = C_\sigma \exp\left[-2.957 + 1.264\left(\frac{q_{c1N,cs}}{100}\right)^{1.25}\right]$$

where

$$C_\sigma = -0.016\left(\frac{\sigma_v^{'}}{100}\right)^3 + 0.178\left(\frac{\sigma_v^{'}}{100}\right)^2 - 0.063\left(\frac{\sigma_v^{'}}{100}\right) + 0.903 \tag{43}$$

and

$$q_{c1N,cs} = K_1 * q_{c1N}$$

where

$$K_1 = 2.429(I_c)^4 - 16.943(I_c)^3 + 44.551(I_c)^2 - 51.497(I_c) + 22.802$$

Liquefaction is said to occur if the factor of safety, FS, is less than or equal to a value of 1.0 as follows:

$$FS = \frac{CRR}{CSR} \tag{44}$$

However, Juang *et al.* notes that liquefaction may occur even if $FS > 1$ due to both model and parameter uncertainties. To overcome this, the authors recommend the use of Bayesian theory to calibrate the model.

Juang *et al.* also writes that the methods of Robertson and Wride (1997, 1998) and Olsen (1997), give comparable results in assessing the liquefaction resistance of a site.

Toprak and Holzer (2003)

Toprak and Holzer (2003) build on the Liquefaction Potential Index (LPI) defined by Iwasaki *et al.* in 1978 to predict the liquefaction potential of a site. The method uses CPT data and "*predicts the performance of the whole soil column and the consequences of liquefaction at the ground surface*".

Iwaskai *et al.* assumed that the proximity of the liquefied layer to the ground surface, the thickness of such layers and the amount by which the safety factor was less than 1.0, were proportional to the severity of liquefaction. The computation of the *LPI* was also limited to depths of up to 20 metres, as below such depths surface effects such as sand boils and other permanent deformation are rarely noted (Toprak and Holzer, 2003).

The liquefaction potential index is defined by the following equation:

$$LPI = \int_0^{20m} Fw(z)dz \qquad (45)$$

In this equation, the factor of safety, *FS*, is based on that calculated using Robertson and Wride (1998) and the depth, *z*, is in metres though weighted according to $w(z) = 10 - 0.5\,z$.

$$\begin{aligned} F &= 1 - FS \quad && \text{for } FS \leq 1 \\ F &= 0 && \text{for } FS > 1 \end{aligned} \qquad (46)$$

It is noted that the *LPI* does not clearly discriminate between non-liquefied and liquefied areas, though it can be used, "*as a screening tool to predict liquefaction occurrence, particularly in regional studies involving liquefaction hazard mapping*". Toprak and Holzer (2003), also write that "*an LPI value of 7 or greater generally distinguishes areas where surface manifestations of liquefaction were observed from areas where it was not observed. The minimum observed LPI in the liquefied area is about seven*". They also note that in a 1982 paper Iwasaki proposed that a *LPI* value of less than five should indicate a minor damage, whereas a *LPI* value of greater than 15 would indicate severe liquefaction.

8 General Results

Sites were examined in Karamea and in the Greymouth area in order to test the performance of the prediction models. These sites liquefied in the 1929 Murchison earthquake but not in the 1968 Inangahua earthquake. The information gathered by Dou and Berrill (1992) was also studied with respect to the Murchison, Inangahua and 1991 Hawks Crag earthquakes and results summarised in Table 8-3 to Table 8-7. The plots from the analyses undertaken in 2003 have been included in Appendix A. Appendix B contains the analysis of data that was gathered by Dou in 1992.

The sites were each assigned a three-letter abbreviation as shown in Table 8-1 below. The year in which liquefaction occurred at the site is also noted.

Table 8-1. Abbreviations used to describe sites

	Abbreviation	*Liquefaction occurred in:*		
		1929	*1968*	*1991*
Greymouth				
Coal Creek	COA	Y	N	N
Collins Street	COL	Y	N	N
Steer Ave	STA	Y	N	N
Karamea				
Arapito	ARA	Y	N	N
Fensom's Paddock	FEN	Y	N	N
Simpson' Paddock	SIM	Y	N	N
Paddock near wharf	WHA	Y	N	N
Oparara	OPA	Y	N	N
Inangahua				
Walkers Flat	TUR	?	Y/N	Y/ N
Murchison				
Four Rivers Plain	MON	Y	Y	N
Fern Flat	WIN	?	Y/N	N
Westport				
Kilkenny Park	KIL	?	Y	N

The charts showing the results of the cone penetrometer testing also indicate the liquefiable layers, as defined by the different analysis methods. The format is illustrated by the results from the CPT number 2 at Steer Avenue Greymouth and given in Figure 8-1 below. The first of the three charts indicates the measured tip resistance and friction ratio, as well as critical values for q_c obtained by the methods of Zhou (1980), Davis and Berrill (1982) and Shibata and Teparaksa (1988). The second chart shows either the ratio of critical tip resistance to the measured resistance, or the factor of safety against liquefaction defined as FS=CRR/CSR. Vertical lines have been drawn at both a value of 1.00 and 1.25. The third plot indicates for each of the prediction methods the layers that are deemed potentially liquefiable. In this plot wherever there is a coloured dot the layer has the potential to liquefy according to the particular prediction method. For clarity, numbers corresponding to each prediction method are provided along with colour coding in the key accompanying the results of the analysis presented in Figure 8-1.

The peak ground acceleration values used in the computation of these curves are given in Table 7-3 and epicentral distances are given in Table 6-1.

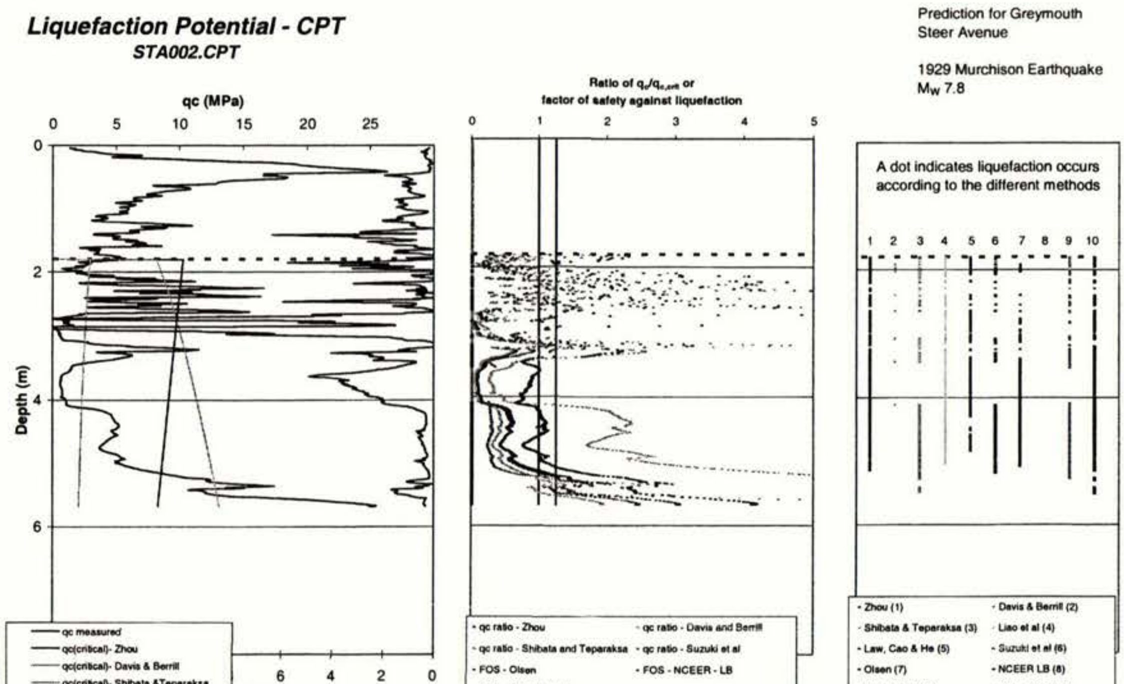


Figure 8-1. CPT test results and analysis for Steer Ave, Greymouth.

A high friction ratio value and a low tip resistance value is indicative of a clay layer and hence not liquefiable.

8.1 Greymouth

Figure 8-2 indicates the three sites where testing was undertaken in January 2003.

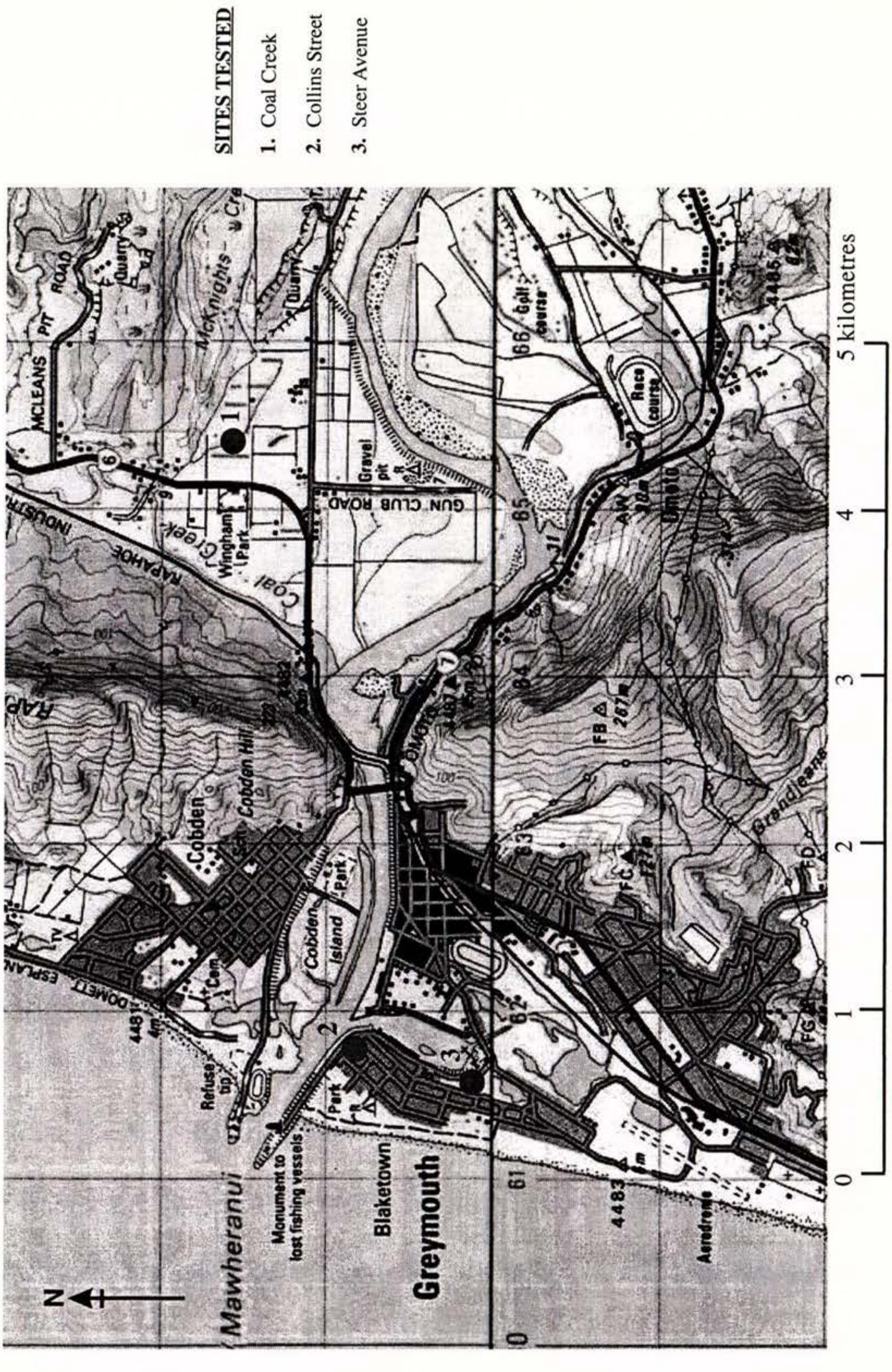


Figure 8-2. Field work sites in the Greymouth area (Base Map: Topomap, 2001)

8.1.1 Coal Creek

This site was identified by Mrs Batey, a resident of Dixon House in Greymouth, and is indicated by number '1' in Figure 8-2. The testing was restricted to one part of the field, due to owners concerns, which limited the area that could be tested. Hand auger testing indicated gravely sand at 2.25 metres and the cone penetrometer was refused at 2.4 metres. All models indicated the site was potentially liquefiable in both the 1929 Murchison and the 1968 Inangahua earthquakes. However, no liquefaction was reported following the 1968 earthquake at this site.

If a peak ground acceleration of 0.07g is used, as was calculated using the relationship of Zhou *et al.* (1997) for the site under the conditions of the Inangahua earthquake; Zhou (1980), Davis and Berrill (1982), Liao *et al.* (1988), and Law *et al.* (1990), all indicate that liquefaction is likely to occur from the depth of the water table, to a depth of 2.4 metres. This peak ground acceleration is much lower than the 0.391g recorded in Greymouth during the 1968 earthquake.

8.1.2 Collins Street

The site selected was a vacant lot shown in Figure 8-3, across the road from where Mr and Mrs Negri lived at the time of the 1929 Murchison earthquake.

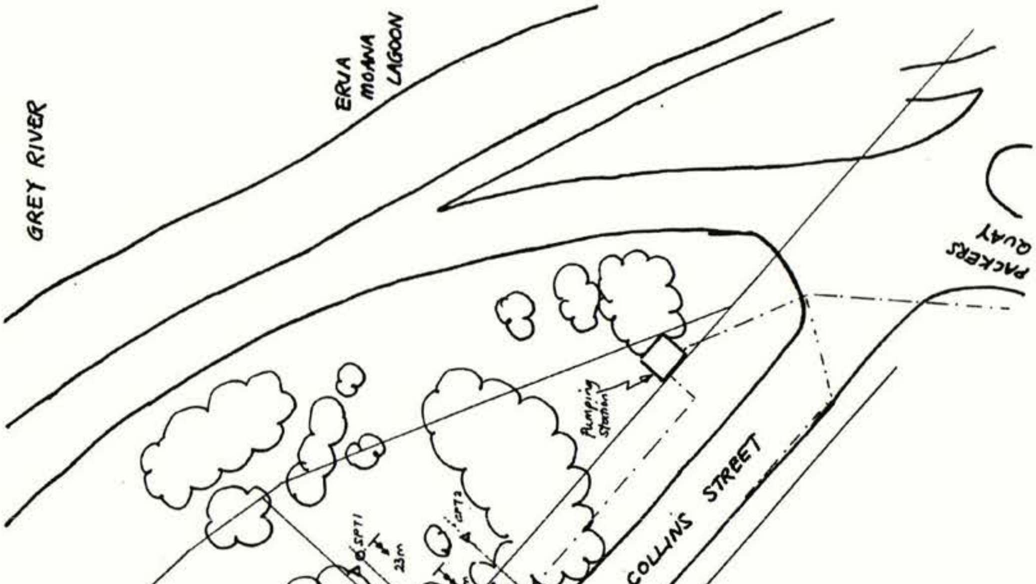


Figure 8-3. Site diagram indicating location of field tests at Collins Street, Greymouth

The strata below the site were predominantly of medium to coarse sands, gravel and fill material as can be seen from the bore log given in Appendix A.

At the site of the CPT test COL001, gravely sand was located at a depth of 1.2 metres. The drilling rig was moved, and two further CPT tests were undertaken. This time, the gravely layers were penetrated, though cone penetrometer refusal occurred at 1.7 metres. A hollow stemmed auger was used to drill to a depth of 2.1 metres and the CPT test continued as COL003.CPT to a depth of 6.15 metres. Drilling was again attempted at this depth, though due to the free running nature of the sand below the water table, the hole collapsed and no further testing attempted. The hollow stem augering allowed a detailed inspection of the soil layers as described in the bore log in Appendix A.

Two SPT tests were attempted at depths of 1.8 and 2.4 metres adjacent to the site of the first CPT test. The results were discounted as in the first test a small stone prevented sand entering the split spoon, and prior to starting the second test there was boiling of sand into the stem of the auger when the plug was removed.

Unfortunately, the first CPT test did not go below the water table. It was discovered during testing that the water table varied from 1.4 metres to 2.4 metres, due to tidal influences.

The CPT tests, COL002 and COL003, indicated the potential of some layers to liquefy. Under the conditions of the 1929 Murchison earthquake only the methods of Zhou, Shibata and Teparaksa (1988), Robertson and Wride (1998) as NCEER (2001), and Juang *et al.* (2003), indicated that the layers between 2.7 and 3.6 metres were potentially liquefiable. The models of Shibata and Teparaksa, NCEER and Juang *et al.* also indicated that lenses of sand between 4 and 5.8 metres are liquefiable. Under the conditions of the 1968 Inangahua earthquake, the thin layers of the soil strata may liquefy according to the prediction methods of Shibata and Teparaksa, NCEER and Juang *et al.*. The plots showing this are located in Appendix A.

8.1.3 Steer Avenue

The third site in Greymouth at which testing was undertaken, was at Steer Avenue, a location where photographs taken following the 1929 Murchison earthquake indicated lateral spreading of the road embankment into the lagoon on either side. The map in Figure 8-2 shows that on the south side of Steer Avenue there is a playing field and a small lakelet. This lakelet has since been reclaimed. The photographs shown in Figure 8-4 and Figure 8-5 were taken on Steer Avenue looking towards the coast.

The old road is shown down the centre of Figure 8-5, with the new road to the left where the lagoon once was. Three CPT tests were undertaken, two to the right of the old road and one to the left as can be seen in the site diagram in Appendix A. The sites are underlain by sand, gravel and fill material.

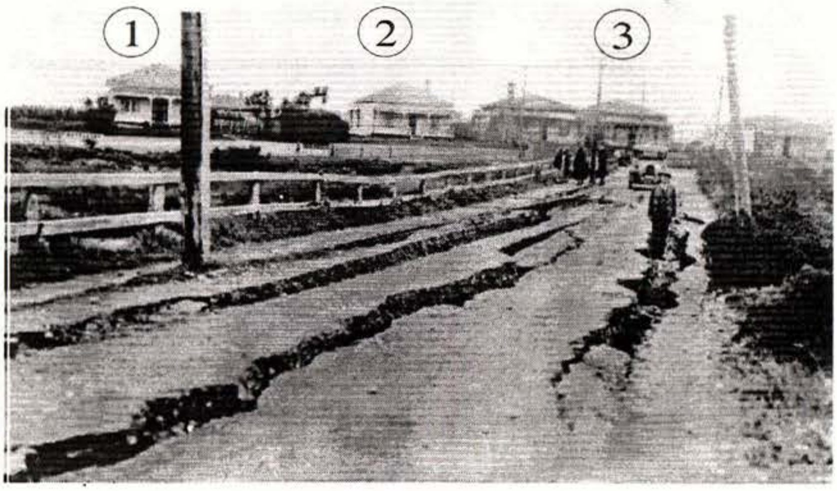


Figure 8-4. Steer Avenue following the 1929 Murchison earthquake (Alexander Turnbull Library)

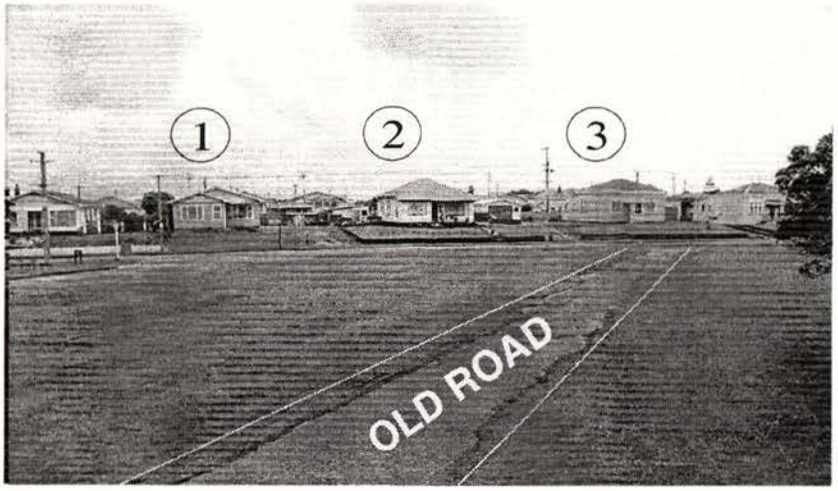


Figure 8-5. Steer Avenue in 2003

The first CPT test was refused above the water table, located at approximately 1.9 metres. This was also observed to change due to tidal influences. Analysing the data from the second CPT test for both the Murchison and Inangahua earthquakes indicated that a number of thin layers were potentially liquefiable under both earthquakes. This can be seen in both Figure 8-6 and Figure 8-7 respectively.

Analyses of data from testing at STA003 indicated few layers had the potential to liquefy. Layers between 1.8 and 2.4 metres were predicted to liquefy for the Murchison earthquake by all the analysis methods, though each method indicated that different sections of the layer would liquefy. Using the earthquake parameters for the Inangahua earthquake, the models indicated that layers between 1.8 and 2.3 metres would have the potential to liquefy.

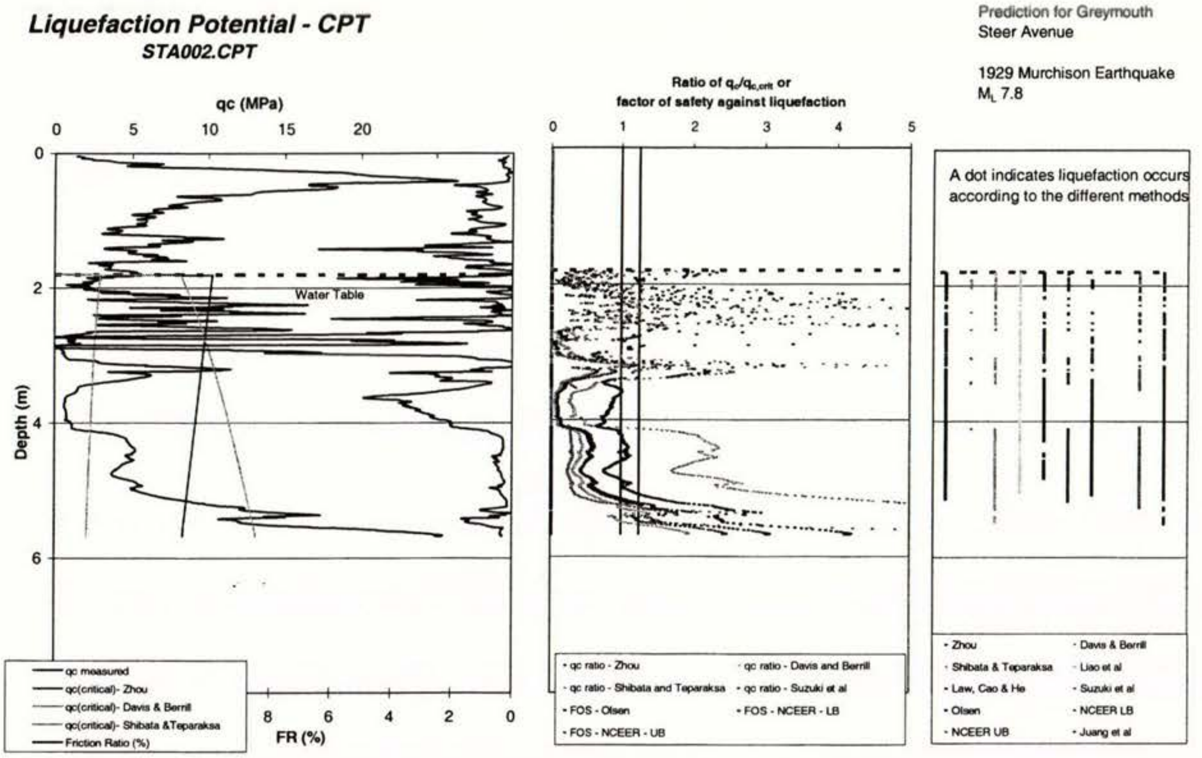


Figure 8-6. Liquefaction analysis plot for Steer Avenue for the 1929 Murchison Earthquake

Figure 8-7. Liquefaction analysis plot for Steer Avenue for the 1968 Inangahua Earthquake

8.2 Karamea

The sites where CPT testing, and borings using a hand auger, were undertaken in February 2003 are shown in Figure 8-8.

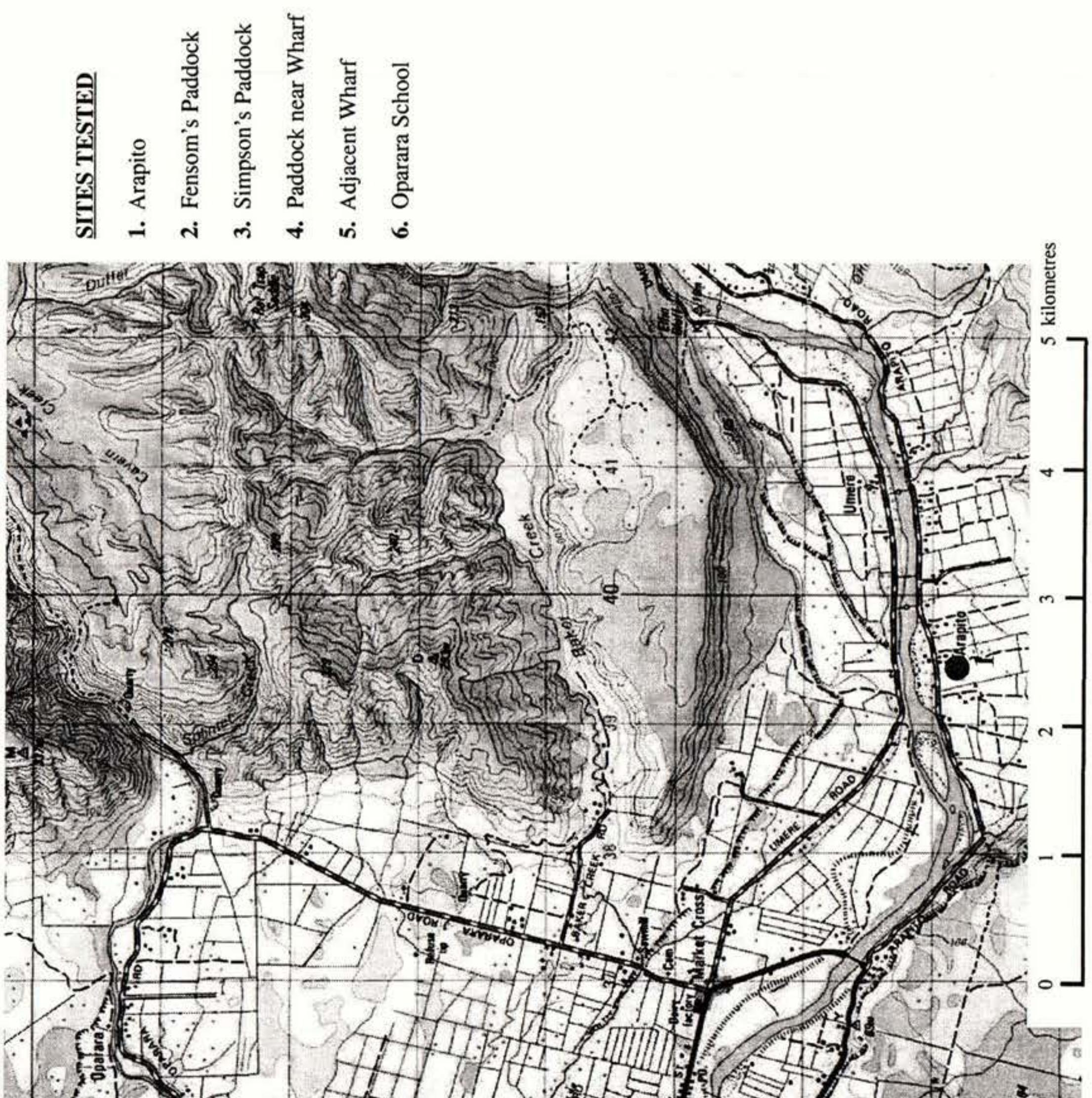


Figure 8-8. Sites in Karamea where field testing was undertaken (Base Map: Topomap, 2001)

8.2.1 Arapito

Testing was undertaken on a property occupied by Mr. Scarlett at Arapito. Bienvenu (1988) referred to Mr Lineham, a previous owner, recalling sand boils and waterspouts occurring in this field as a result of the 1929 Murchison earthquake. No evidence of this occurring in 1968 has been obtained. Three CPT tests were undertaken at the site as well as taking three bore logs using a hand auger.

As can be seen from the bore logs and the particle size distribution curves given in Appendix A, the site is underlain by both silts and sands. Each hand auger was refused at depths between 2.2m and 3.75 m by pebbly or gravely sand. The drilling rig was also limited to depths between 2 and 3.3 metres due to the pebbly nature of the sand at this depth.

The water table was not encountered, though Mr. Scarlett informed us that it was at a depth of approximately 4 metres. If this was the case in 1929, the data obtained cannot be used to predict the liquefaction occurrence.

If the water table was close to the ground surface, analysis of the data indicates that at the front of the paddock (ARA001 and ARA002) all methods predict liquefaction of layers between 1 and 1.5 m and again between 2.4 and 2.8 metres. For the site near the back of the paddock (ARA003) liquefaction is predicted between 2 and 2.2 metres depth.

8.2.2 Fensom's Paddock / Simpson's Paddock:

These sites were chosen due to the numerous reports of surface manifestations of liquefaction such as sand boils, waterspouts and fissuring of the ground. As is indicated by Figure 8-8, these paddocks are across Waverly Street from each other, and adjacent to Karamea School and the Domain. These are sites that were also reported as having liquefied in 1929.

The test sites in Simpson's paddock are underlain by silty sand, sands and silts. At the site of the cone penetrometer test SIM006, a clay layer was present between 3 and 3.8 metres. At each of the test sites, one or two layers of soil were deemed liquefiable according to the prediction methods, as follows.

SIM001: For the 1929 Murchison Earthquake all methods except that of Olsen (1997) predict the potential for liquefaction to occur between 2.5 and 2.9m and also between 3.7 and 4.7 metres. Under the conditions of the 1968 Inangahua Earthquake, the methods, with the exception of Davis and Berrill (1982), and Olsen, predict liquefaction between 2.5 and 2.8 metres and between depths of 3.9 and 4.8 metres. However, no surface effects of liquefaction were reported at this site following the 1968 earthquake.

At the site of the CPT test SIM004, all methods predict liquefaction between 6.2 and 6.5 metres for the conditions imposed by the Murchison earthquake. If the data is analysed for the Inangahua earthquake, liquefaction is predicted between depths of 6.2 and 6.6 metres by all methods except those of Davis and Berrill and Olsen.

The layers between 2.1 and 2.7 metres are deemed liquefiable by all methods in the 1929 Murchison earthquake, whereas all methods predict the occurrence between depths of 2.5 and 2.7 metres in the 1968 Inangahua earthquake at the site of SIM006.

The first two CPT tests undertaken in Fensom's paddock were inconclusive in that they did not get far below the water table. The location of these tests is shown in Figure 8-9. The water table was located at a depth of 2.2 metres. Silty sands and sands were encountered at FEN001; these were very dense just above the water table and prevented further testing at the site. Very dense sand was found below the water table at FEN002, the site indicated by "CPT 2" in the diagram below. The test was stopped at 2.7 metres due to the dense sands. None of the methods predicted liquefaction at this site.

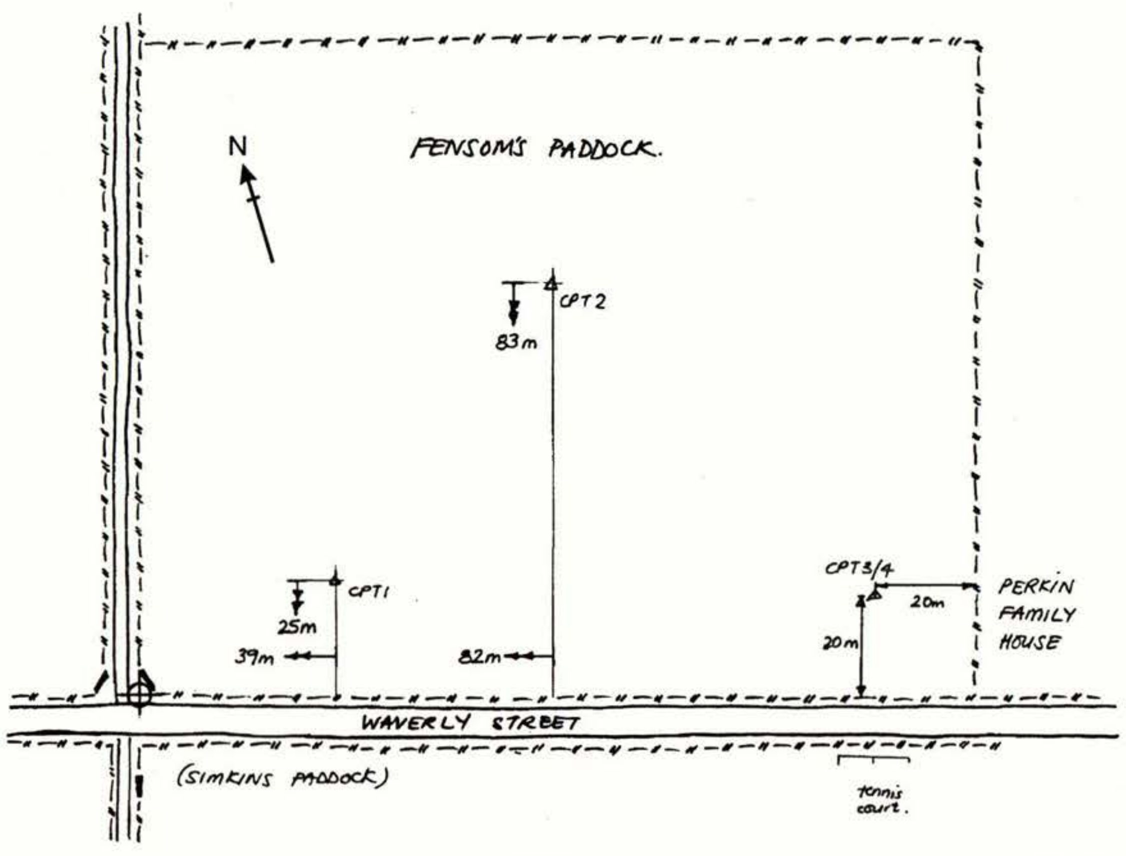


Figure 8-9. Site diagram indicating locations of CPT tests in Fensom's Paddock, Karamea

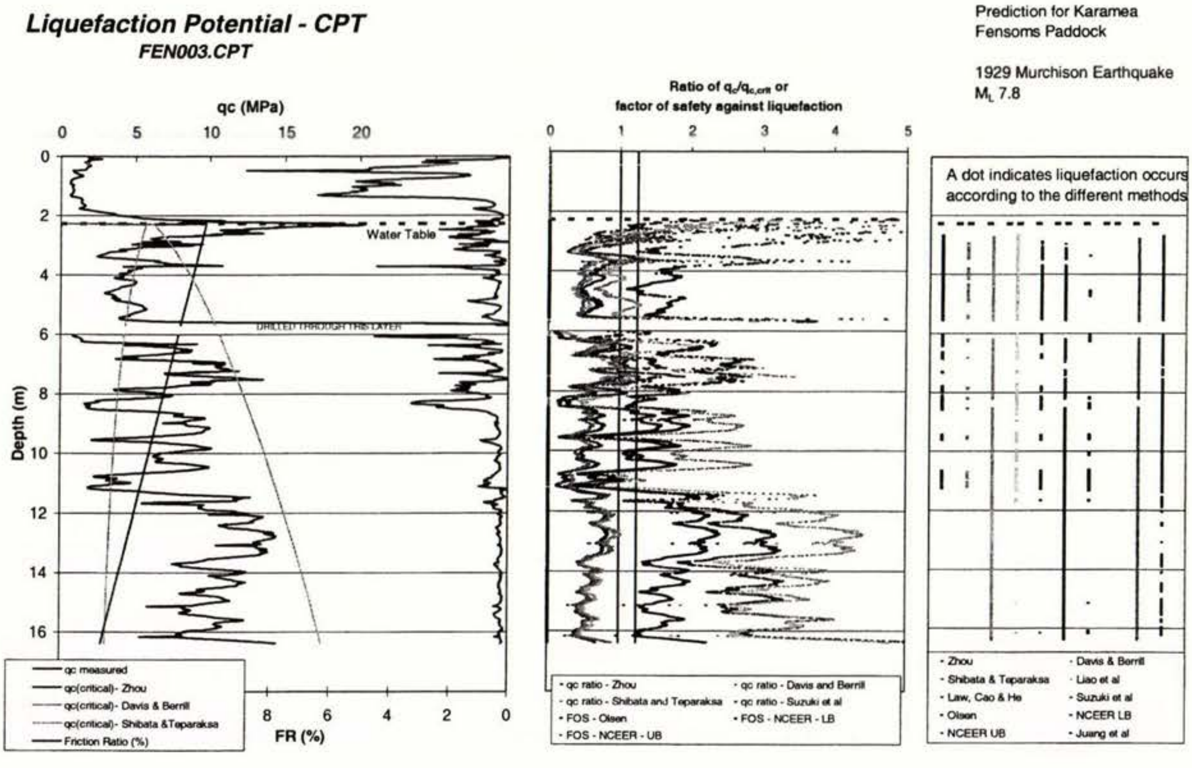


Figure 8-10. Liquefaction analysis plot Fensom's Paddock for the 1929 Murchison Earthquake

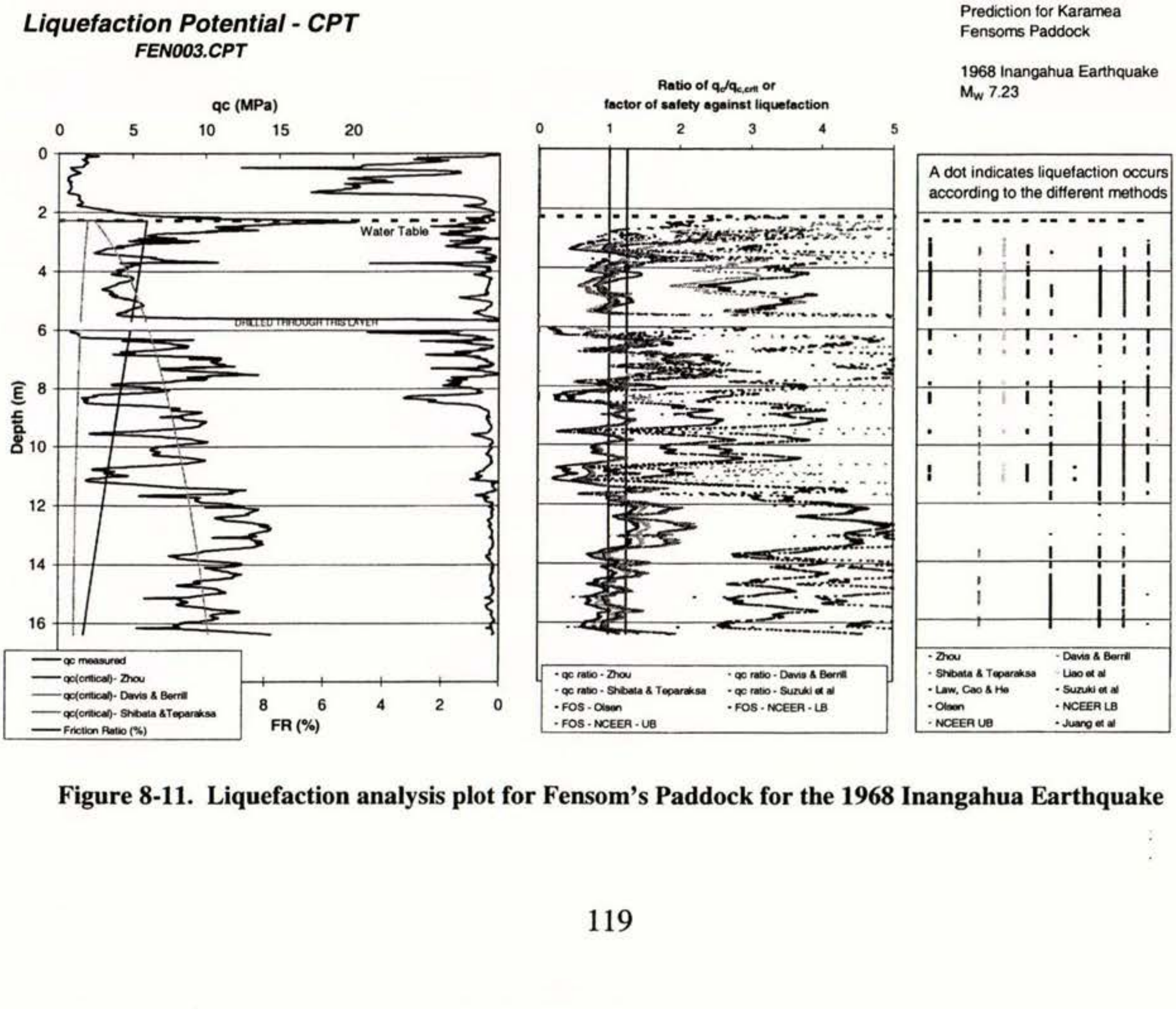


Figure 8-11. Liquefaction analysis plot for Fensom's Paddock for the 1968 Inangahua Earthquake

The third test site was found to be slightly different to the adjacent sites. Soil with a high organic content, silts and clays were found to a depth of 1.5 metres, and below this were layers of sand, silty sands and silts to a depth of 5.7 metres. The cone penetrometer test was stopped at this depth, due to the dense, gravelly nature of the sand. A flight auger was used to drill to a depth of 6 metres, and the CPT test was continued. From a depth of 6 to 16.5 metres, layers of sand, silty sand and sandy silt were present, similar to Simpson's paddock. The water table was located at a depth of 2.2 metres.

All the methods, with the exception of Olsen (1997), predicted liquefaction to occur for the 1929 Murchison earthquake from 3.2 to 5.1 metres, and then for some lenses of sands to depths of 9.8 metres. The method of Olsen predicted liquefaction in multiple thin layers. This can be seen in the analysis chart in Figure 8-10. All methods also predicted liquefaction would occur between depths of 10.9m and 11.4 metres.

The liquefaction induced settlement was calculated using the method of Ishihara and Yoshimine (1992). In this calculation the liquefaction potential was evaluated using the prediction model of Shibata and Teparaksa. The results of this method were used as a greater proportion of the soil column is considered to be potentially liquefiable than for other prediction methods as can be seen in Figure 8-10. It was noted in Chapter 3 that both end spans of the Overflow Bridge dropped approximately 3 feet (0.9m) and the river flats dropped between four and 5 feet (1.2-1.6m). However, the liquefaction induced settlement calculated, using the method of Ishihara and Yoshimine, only indicated a settlement of 0.39m. This implies that either a much greater thickness of soil liquefied or the soil had also consolidated due to shaking. This observation suggests that the settlement calculation has been ascertained on an erroneous assumption, or that the soil column has been altered, either as a result of the earthquake shaking or post earthquake consolidation. The difference between then observed and calculated settlement may be attributed to the simplicity of the settlement and liquefaction models and the possibility that lateral spreading towards the river and overflow caused additional settlement of the river flats. It should be noted that this calculation of the settlement was based on a single cone penetrometer test.

For the case of the Inangahua earthquake the prediction models indicated that liquefaction would occur in thin lenses down the soil column as can be seen in Figure 8-11. However, no liquefaction was reported at this site following the 1968 earthquake.

8.2.3 Wharf

The paddock adjacent to the wharf at the time of the 1929 Murchison earthquake is only accessible at low tide as access is gained by travelling over a sand channel though the lagoon. Due to the difficulties associated with getting the drilling rig out to this paddock, testing was undertaken in the paddock at the end of Ray Street, on the town side of the tidal channel, as indicated by number '4' in Figure 8-8. The particle size distribution curves drawn from the sieve and sedimentation analysis of the soil samples obtained from both sites by hand augering, indicated that testing in the paddock near the wharf is representative of testing in the paddock adjacent to the wharf site due to the similarity of the curves in Figure 8-12 and Figure 8-13. Both the paddock near the wharf and that adjacent to the wharf were underlain by medium to coarse sands as is shown by the bore logs and particle size distribution curves in Appendix A.

The first CPT test was abandoned at a depth of 1.4 metres when the truck started to lift. Using the flight auger it was discovered that there was log at that depth. The truck was moved slightly and two further tests were undertaken, WHA002 and WHA003. Dense sands at a depth of 6.9 metres prevented the continuation of WHA002. Again, a flight auger was used to drill through the dense layer and the test resumed. The sand below the water table, which was found to be at a depth of 2.2m, flowed relatively freely into the hole, though due to the augering was loose enough for the cone to be pushed into the ground. Recording began again at 7.4 metres, the depth at which augering has been stopped, and was again prevented by a dense gravely sand layer at 8.7 metres.

Both the CPT tests WHA004 and WHA005, achieved a depth of 5 metres. The soil profiles at these sites were again sands with silty sand layers, as can be seen in section Appendix A.

For the WHA002/ WHA003 site, all the methods except that of Olsen (1997), predicted the soil to be liquefiable between 1.4 and 4.5 metres depth, under the Murchison earthquake. The method of Olsen, meanwhile, predicted the occurrence between 1.4 and 2 metres and again between 3.7 and 4.4 metres. For the 1968 Inangahua earthquake, Davis and Berrill (1982), indicated that the soil between 1.4 and 2.1 metres was potentially liquefiable. The Olsen model indicated that no liquefaction would occur and the other methods, excluding that of Suzuki *et al.* (1995), predicted liquefaction between depths of 1.4 and 4.6 metres.

For the WHA004 site, most of the methods predicted some liquefaction over the depth. For the Inangahua earthquake, all the methods, except that of Olsen, deemed the soil potentially liquefiable between 3.6 and 4.3 m. The liquefiable layer defined by Olsen began at 3.8 m and went to 4.3 metres. This was similar as that obtained in the Murchison earthquake, where all the methods apart from those of Olsen and Davis and Berrill, indicated that the soil was potentially liquefiable between 3.6 and 4.3 metres.

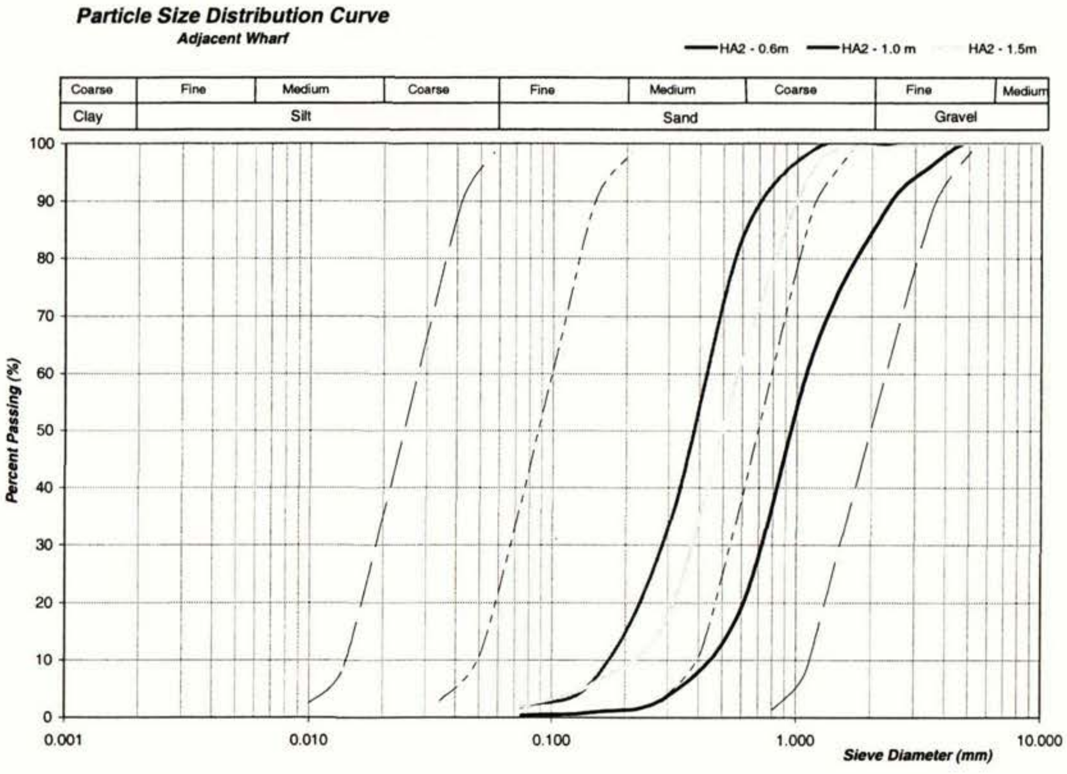


Figure 8-12. Particle size distribution of samples from the paddock adjacent the wharf site

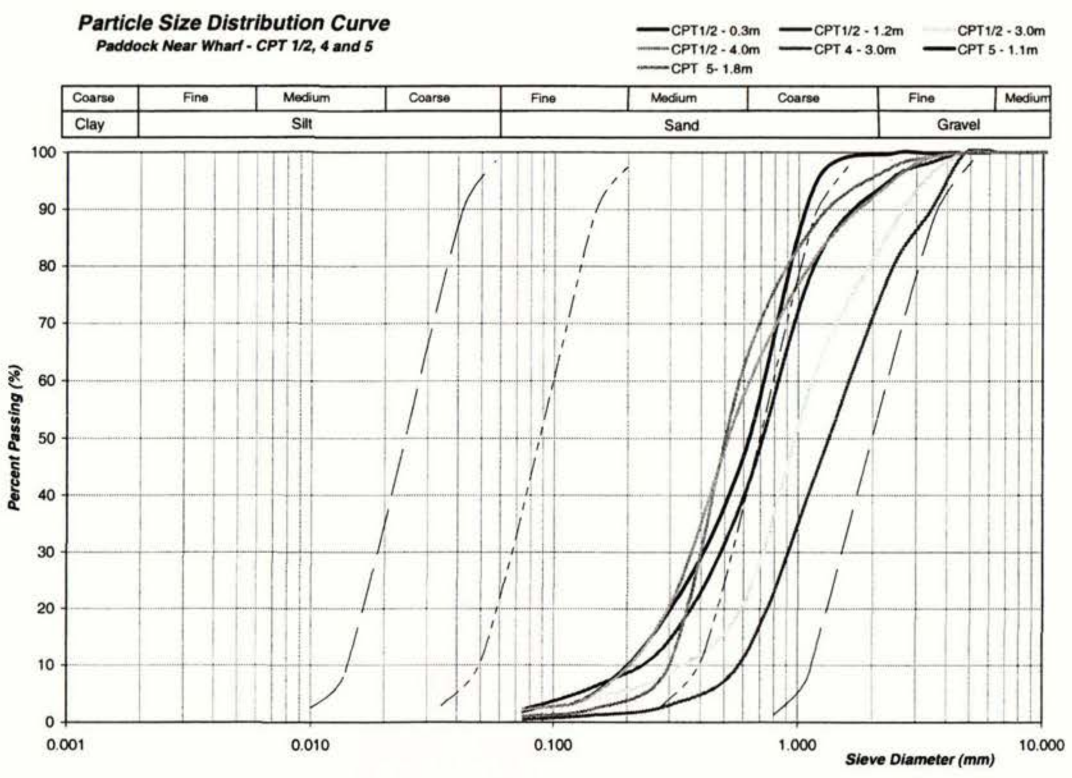


Figure 8-13. Particle size distribution of samples from the paddock near the wharf site

At the WHA005 test site under the conditions of the 1929 Murchison earthquake, all methods, with the exception of Olsen, predict the occurrence of liquefaction from the depth of the water table at 1.2 metres to a depth of 4.1 metres. Olsen predicts that the layers between 3.1 and 4.1 metres have the potential to liquefy. In comparison, the methods of Zhou (1980), Shibata and Teparaksa (1988), Suzuki *et al.*, Robertson and Wride (1998) as NCEER (2001), and Juang *et al.* (2003), all predict liquefaction to occur in the dense sands, to a depth of 4.9 metres.

The methods of Davis and Berrill and Olsen are the only methods that do not predict any significant liquefaction to occur in the 1968 Inangahua earthquake at this site. Suzuki *et al.* predicts the occurrence between 3 and 4 metres and the other methods, except that of Juang *et al.* predict liquefaction from 1.2 to 4 metres depth. The model devised by Juang *et al.* gives a layer from 1.9 to 4 metres as having the potential to liquefy.

8.2.4 Oparara

Following the 1929 Murchison earthquake, Vita Harney wrote a letter describing the waterspouts and mud pools that occurred in the Oparara School grounds due to the earthquake. This letter is currently stored in the Alexander Turnbull Library. Mr. Lowe, an Oparara school pupil at the time of the earthquake, confirmed this report and the school site (pers. comm., January 2003). Three cone penetrometer tests were undertaken at the site.

The particle size distribution plot in Figure 8-14 indicates a very uniform fine to medium sand underlies this site. This can also be seen from the bore log in Appendix A2.

The CPT test OPA001 was stopped by dense sand at 4.6 metres. A flight auger was used to drill to a depth of 5 metres where the test was continued as OPA003. The CPT test OPA004 was also stopped at 5.3 metres due to dense sands.

None of the prediction methods indicated liquefiable layers at either site under the conditions imposed by the 1968 Inangahua earthquake, using the current water table level of 1.95m. Only the method of Zhou predicted a liquefiable layer between 2 and 2.27 metres at OPA001/003 for the 1929 Murchison earthquake. At OPA004, the methods of Zhou, Suzuki *et al.*, Robertson and Wride as NCEER and Juang *et al.* all indicated a liquefiable layer between 4.3 and 4.5 metres.

If the water table were closer to the ground surface, most of the methods predict that the soil would be liquefiable below the water table to a depth of up to 1.6 metres for the Murchison earthquake, and 1.2metres for the Inangahua earthquake. However, the method of Olsen still indicates that no liquefaction would occur at this site.

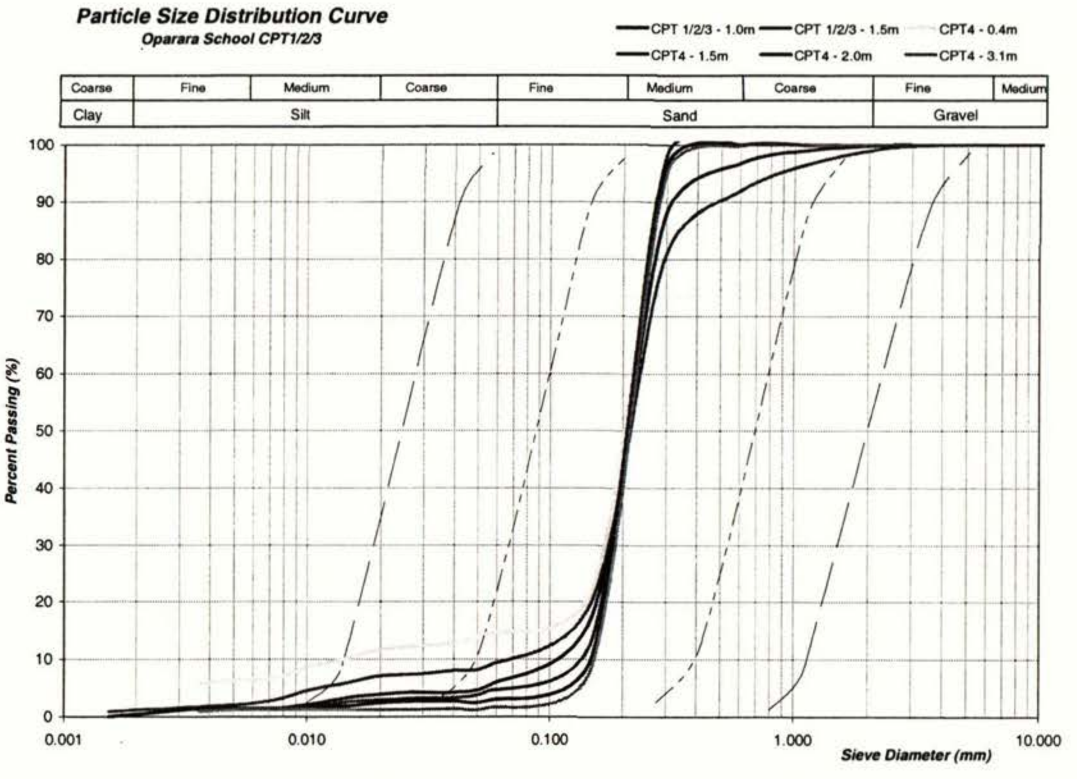


Figure 8-14. Particle size distribution for Oparara School site

At these sites the cone tip resistance value, q_c, increases almost linearly with the depth. This poses the question if the increase in tip resistance is an effect of the confining pressure on a soil of uniform density. If that were true then the following well known relationship should hold.

$$\frac{q_c}{\sqrt{\sigma_v^{'}}} \propto I_D \qquad (47)$$

However, plotting this shows that this supposition is not true as the density index, I_D, is not constant as it shows a generally increasing value with depth.

The fact that none of the methods predict liquefaction at the sites tested also raises questions, as they are known to have liquefied in the 1929 earthquake. It may mean that the shaking in 1929 left the soil in such a dense state that it will now not liquefy unless stronger shaking occurs close to the site. It is known that the school site is located in a dynamic coastal environment, where according to local residents, significant coastal erosion has occurred in the past 10 years. This has been indicated by the Dairy Factory outlet pipe, on the coast near the Karamea golf course, which has been left exposed as can be seen in the photo in Figure 8-15. The implication that sand at this site is of dune origin is strengthened by the uniformity of the grain size seen in Figure 8-14, however, the median grain size is approximately 0.2mm which is similar to other Karamea sites. The sands being more uniformly graded would imply a greater propensity to liquefy. The uniformity of the sand at the site may also explain why the prediction models appear to be more uniform in their outcomes.

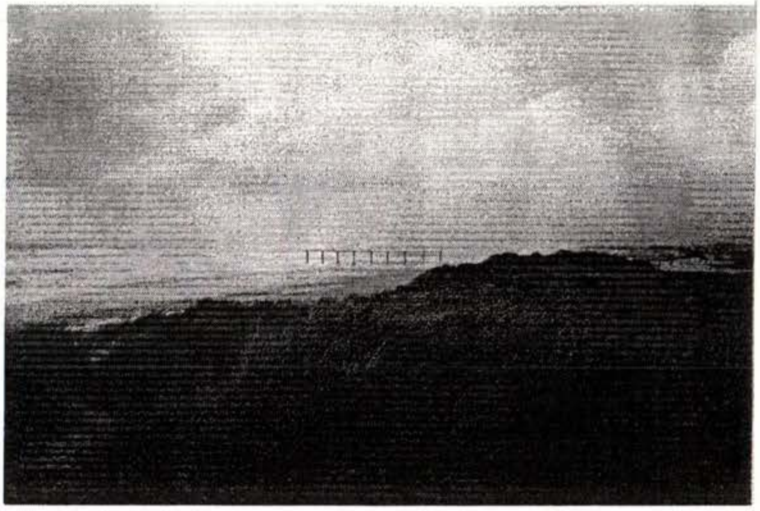


Figure 8-15. Coastal erosion indicated by the Dairy Factory outlet pipe that originally protruded from the sand dunes

It is also not known how many earthquakes have caused liquefaction at this site in the past, since this area was laid down by coastal deposition and material brought down by the Oparara River.

The liquefaction potential index values determined using the method of Toprak and Holzer (2003) are given in the table below. It must be noted that this method sums values over a soil column depth of twenty metres though none of the CPT tests undertaken went as deep as this. This method does not make allowance to depth of probe less than 20 metres as the depth correction factor used in the method is also based on the 20-metre depth. This may lead to erroneous results as any liquefiable layers between the bottom of the probe and a depth of 20 m would increase the LPI.

Table 8-2. Results obtained from the method of Toprak and Holzer (2003)

SITE	*LPI*		*LPI*	
	1929 Murchison Earthquake		1968 Inangahua Earthquake	
Greymouth				
COA 001	7.05	Liquefiable	6.37	Liquefiable
COL001	0.000	Not very liquefiable	0.00	Not very liquefiable **
COL002/3	5.20	Liquefiable	3.65	Not very liquefiable
STA001	0.000	Not very liquefiable	0.00	Not very liquefiable **
STA002	9.72	Liquefiable	8.18	Liquefiable
STA003	2.41	Not very liquefiable	1.70	Not very liquefiable
Karamea				
ARA001	0.00	Not very liquefiable	0.00	Not very liquefiable*,**
ARA002	0.00	Not very liquefiable	0.00	Not very liquefiable*,**
ARA003	0.00	Not very liquefiable	0.00	Not very liquefiable*,**
SIM001/2/3	8.54	Liquefiable	0.00	Not very liquefiable
SIM004	1.632	Not very liquefiable	0.52	Not very liquefiable
SIM006	2.69	Not very liquefiable	0.20	Not very liquefiable
FEN001	0.00	Not very liquefiable	0.00	Not very liquefiable**
FEN002	0.00	Not very liquefiable	0.00	Not very liquefiable**
FEN003	26.21	Highly liquefiable	2.88	Not very liquefiable
WHA001	0.53	Not very liquefiable	0.00	Not very liquefiable**
WHA002/3	14.84	Highly liquefiable	2.91	Not very liquefiable
WHA004	13.45	Liquefiable	1.83	Not very liquefiable
WHA005	15.84	Highly liquefiable	3.26	Not very liquefiable
OPA001/4	0.00	Not very liquefiable	0.00	Not very liquefiable
OPA004	0.21	Not very liquefiable	0.00	Not very liquefiable

* Least well located of the Karamea sites. Place less weight on the results.

** Also shallow CPT tests.

8.3 Analysis Summary

The following tables show for a given site whether or not the prediction models indicate a layer greater than 0.1 metres thick is potentially liquefiable. The asterisk (*) indicates a prediction of liquefaction at a site that is known to have liquefied or a non-prediction at a site that is known not to have liquefied. The annotation (b) indicates that the result is bordering between a yes (Y) and no (N) result.

Table 8-3. Table of results of liquefaction potential prediction models for sites tested in 2003 (Part A)

METHOD	Zhou (1980)		Davis and Berrill (1982)		Shibata and Teparaksa (1988)		Liao *et al.* (1988)		Law *et al.* (1990)		Liquefied?	
SITE	1929	1968	1929	1968	1929	1968	1929	1968	1929	1968	1929	1968
COA 001	Y*	Y	Y*	Y	Y*	Y	Y*	Y	Y*	Y	Y	N
COL001	-	-	-	-	-	-	-	-	-	-	Y	N
COL002/3	Y*	N*	N	N*	Y*	Y	Y*	N*	N	N*	Y	N
STA001	-	-	-	-	-	-	-	-	-	-	Y	N
STA002	Y*	Y	Y*	N*	Y*	Y	Y*	Y	Y*	Y	Y	N
STA003	Y*	Y	N	N*	Y*	Y	Y*	Y	Y*	Y	Y	N
ARA001	-	-	-	-	-	-	-	-	-	-	Y	N
ARA002	-	-	-	-	-	-	-	-	-	-	Y	N
ARA003	-	-	-	-	-	-	-	-	-	-	Y	N
SIM001/2/3	Y*	Y	Y*	N*	Y*	Y	Y*	Y	Y*	Y	Y	N
SIM004	Y*	Y	Y*	N*	Y*	Y	Y*	Y	Y*	Y	Y	N
SIM006	Y*	Y	Y*	N(b)*	Y*	Y	Y*	Y	Y*	Y	Y	N
FEN001	-	-	-	-	-	-	-	-	-	-	Y	N
FEN002	N	N*	N	N*	N	N*	N	N*	N	N*	Y	N
FEN003	Y*	Y	Y*	N*	Y*	Y	Y*	Y	Y*	Y	Y	N
WHA001	-	-	-	-	-	-	-	-	-	-	-	N
WHA002/3	Y*	Y	Y*	Y	Y*	Y	Y*	Y	Y*	Y	Y	N
WHA004	Y*	Y	Y*	N*	Y*	Y	Y*	Y	Y*	Y	Y	N
WHA005	Y*	Y	Y*	N*	Y*	Y	Y*	Y	Y*	Y	Y	N
OPA001/3	Y*	N*	N	N*	N	N*	Y*	N*	N	N*	Y	N
OPA004	Y*	N*	N	N*	N	N*	Y*	N*	N	N*	Y	N

Table 8-4. Table of results of liquefaction potential prediction models for sites tested in 2003 (Part B)

METHOD	Suzuki *et al.* (1995)		Olsen (1997)		NCEER Lower bound (2001)		NCEER Upper bound (2001)		Juang *et al.* (2003)		Liquefied?	
SITE	1929	1968	1929	1968	1929	1968	1929	1968	1929	1968	1929	1968
COA 001	Y*	Y	Y*	Y	-	Y	Y*	Y	Y*	Y	Y	N
COL001	-	-	-	-	-	-	-	-	-	-	Y	N
COL002/3	Y*	Y	N	N*	-	Y	Y*	Y	Y*	Y	Y	N
STA001	-	-	-	-	-	-	-	-	-	-	Y	N
STA002	Y*	Y	Y*	Y	-	Y	Y*	Y	Y*	Y	Y	N
STA003	N(b)	N*	N(b)	N*	-	Y	Y*	Y	Y*	Y	Y	N
ARA001	-	-	-	-	-	-	-	-	-	-	Y	N
ARA002	-	-	-	-	-	-	-	-	-	-	Y	N
ARA003	-	-	-	-	-	-	-	-	-	-	Y	N
SIM001/2/3	Y*	Y	N	N*	-	Y	Y*	Y	Y*	Y	Y	N
SIM004	Y*	Y	Y*	N*	-	Y	Y*	Y	Y*	Y	Y	N
SIM006	Y*	N(b)*	Y*	N*	-	Y	Y*	Y	Y*	Y	Y	N
FEN001	-	-	-	-	-	-	-	-	-	-	Y	N
FEN002	N	N*	N	N*	-	N*	N	N*	N	N*	Y	N
FEN003	Y*	Y	Y*	N	-	Y	Y*	Y	Y*	Y	Y	N
WHA001	-	-	-	-	-	-	-	-	-	-	Y	N
WHA002/3	Y*	Y	Y*	N*	-	Y	Y*	Y	Y*	Y	Y	N
WHA004	Y*	Y	Y*	N*	-	Y	Y*	Y	Y*	Y	Y	N
WHA005	Y*	Y	Y*	N*	-	Y	Y*	Y	Y*	Y	Y	N
OPA001/3	N	N*	N	N*	-	N*	N	N*	N	N*	Y	N
OPA004	N	N*	N	N*	-	N*	Y*	N*	Y*	N*	Y	N

Table 8-5. Table of results of liquefaction potential prediction models for sites tested by Dou, 1992 (Part A)

METHOD	Zhou (1980)			Davis and Berrill (1982)			Shibata and Teparaksa (1988)			Liao *et al.* (1988)			Liquefied?		
SITE	1929	1968	1991	1929	1968	1991	1929	1968	1991	1929	1968	1991	1929	1968	1991
KIL001	Y	Y*	-	Y	Y*	N*	Y	Y*	Y	Y	Y*	Y	?	Y	N
KIL002	Y	Y*	-	Y	Y*	N*	Y	Y*	Y	Y	Y*	Y	?	Y	N
KIL003	Y	Y*	-	Y	Y*	N*	Y	Y*	Y	Y	Y*	Y	?	Y	N
KIL004	Y	Y*	-	Y	Y*	N*	Y	Y*	Y	Y	Y*	Y	?	Y	N
KIL005	Y	Y*	-	Y	Y*	N*	Y	Y*	Y	Y	Y*	Y	?	Y	N
KIL006	Y	Y*	-	Y	Y*	N*	Y	Y*	Y	Y	Y*	Y	?	Y	N
MON001	Y	Y*	-	Y	Y*	N*	Y	Y*	Y	Y	Y*	N*	?	Y	N
TUR001	Y	Y*	-	Y	Y*	Y*	Y	Y*	Y*	Y	Y*	Y*	?	Y	Y
TUR002	Y	Y*	-	Y	Y*	N	Y	Y*	Y*	Y	Y*	Y*	?	Y	Y
TUR003	Y	Y*	-	Y	Y*	N*	Y	Y*	Y*	Y	Y*	Y*	?	Y	Y
TUR004	Y	Y	-	Y	Y	N*	Y	Y	N*	Y	Y	N*	?	N	N
TUR005	Y	Y	-	Y	Y	N*	Y	Y	N*	Y	Y	N*	?	N	N
TUR006	Y	Y	-	Y	Y	N*	Y	Y	Y	Y	Y	N*	?	N	N
TUR007	Y	Y*	-	Y	Y*	N*	Y	Y*	Y	Y	Y*	Y	?	Y	N
TUR008	Y	Y*	-	Y	Y*	N*	Y	Y*	Y	Y	Y*	Y	?	Y	N
TUR009	Y	Y	-	Y	Y*	N*	Y	Y*	Y	Y	Y*	Y	?	Y	N
TUR010	Y	Y	-	Y	Y*	N*	Y	Y*	Y	Y	Y*	Y	?	Y	N
TUR011	Y	Y*	-	Y	Y*	N	Y	Y*	Y*	Y	Y*	Y*	?	Y	Y
TUR012	Y	Y	-	Y	Y*	N*	Y	Y*	Y	Y	Y*	Y	?	Y	N
TUR013	Y	Y	-	Y	Y*	N*	Y	Y*	Y	Y	Y*	Y	?	Y	N
TUR014	Y	Y	-	Y	Y	N*	Y	Y	Y	Y	Y	N*	?	N	N
WIN001	Y	N*	-	Y	N*	N*	Y	Y	Y	Y	Y	N*	?	N	N
WIN002	Y	N*	-	Y	N	N*	Y	Y*	Y	Y	Y*	N*	?	Y	N
WIN003	Y	Y	-	Y	Y	N*	Y	Y	Y	Y	Y	N*	?	N	N
WIN004	N	N*	-	Y	N	N*	Y	Y*	Y	Y	N	N*	?	Y	N
WIN005	Y	N*	-	Y	N	N*	Y	Y*	N*	Y	Y*	N*	?	Y	N
WIN006	Y	N*	-	Y	N	N*	Y	Y*	Y	Y	Y*	N*	?	Y	N
WIN007	Y	N*	-	Y	N	N*	Y	Y*	Y	Y	Y*	N*	?	Y	N
WIN008	-	-	-	-	-	-	-	-	-	-	-	-	?	N	N

Table 8-6. Table of results of liquefaction potential prediction models for sites tested by Dou, 1992 (Part B)

METHOD	Law *et al.* (1990)			Suzuki *et al.* (1995)			Olsen (1997)			Liquefied?		
SITE	1929	1968	1991	1929	1968	1991	1929	1968	1991	1929	1968	1991
KIL001	Y	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
KIL002	Y	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
KIL003	Y	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
KIL004	Y	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
KIL005	Y	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
KIL006	Y	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
MON001	Y	Y*	Y	Y	Y*	Y	Y	Y*	N*	?	Y	N
TUR001	Y	Y*	Y	Y	Y*	Y*	Y	Y*	Y*	?	Y	Y
TUR002	Y	Y*	Y	Y	Y*	Y*	Y	Y*	Y*	?	Y	Y
TUR003	Y	Y*	Y	Y	Y*	Y*	Y	Y*	Y*	?	Y	Y
TUR004	Y	Y	Y	Y	Y	N*	Y	Y	N*	?	N	N
TUR005	Y	Y	Y	Y	Y	N*	Y	Y	N*	?	N	N
TUR006	Y	Y	Y	Y	Y	Y	Y	Y	N*	?	N	N
TUR007	Y	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
TUR008	Y	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
TUR009	Y	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
TUR010	Y	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
TUR011	Y	Y*	Y*	Y	Y*	Y*	Y	Y*	Y*	?	Y	Y
TUR012	Y	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
TUR013	Y	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
TUR014	Y	Y	Y	Y	Y	Y	Y	Y	N*	?	N	N
WIN001	Y	Y	N*	Y	Y	Y	Y	Y	N*	?	N	N
WIN002	Y	Y*	N*	Y	Y*	Y	Y	Y*	N*	?	Y	N
WIN003	Y	Y	Y	Y	Y	Y	Y	Y	N*	?	N	N
WIN004	Y	N	N*	Y	Y*	Y	Y	N	N*	?	Y	N
WIN005	Y	N	N*	Y	Y*	N*	Y	Y*	N*	?	Y	N
WIN006	Y	Y*	N*	Y	Y*	Y	Y	Y*	N*	?	Y	N
WIN007	Y	Y*	N*	Y	Y*	Y	Y	Y*	N*	?	Y	N
WIN008	-	-	-	-	-	-	-	-	-	?	N	N

Table 8-7. Table of results of liquefaction potential prediction models for sites tested by Dou, 1992 (Part C)

METHOD	NCEER – Lower bound (2001)			NCEER – Upper bound (2001)			Juang *et al.* (2003)			Liquefied?		
SITE	1929	1968	1991	1929	1968	1991	1929	1968	1991	1929	1968	1991
KIL001	-	Y*	N*	Y	Y*	N*	Y	Y*	Y	?	Y	N
KIL002	-	Y*	Y	Y	Y*	N*	Y	Y*	Y	?	Y	N
KIL003	-	Y*	Y	Y	Y*	N*	Y	Y*	Y	?	Y	N
KIL004	-	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
KIL005	-	Y*	Y	Y	Y*	N*	Y	Y*	Y	?	Y	N
KIL006	-	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
MON001	-	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
TUR001	-	Y*	Y*	Y	Y*	Y*	Y	Y*	Y*	?	Y	Y
TUR002	-	Y*	Y*	Y	Y*	Y*	Y	Y*	Y*	?	Y	Y
TUR003	-	Y*	Y*	Y	Y*	Y*	Y	Y*	Y*	?	Y	Y
TUR004	-	Y	N*	Y	Y	N*	Y	Y	Y	?	N	N
TUR005	-	Y	N*	Y	Y	N*	Y	Y	N*	?	N	N
TUR006	-	Y	N*	Y	Y	N*	Y	Y	N*	?	N	N
TUR007	-	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
TUR008	-	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
TUR009	-	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
TUR010	-	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
TUR011	-	Y*	Y*	Y	Y*	Y*	Y	Y*	Y*	?	Y	Y
TUR012	-	Y*	Y	Y	Y*	N*	Y	Y*	Y	?	Y	N
TUR013	-	Y*	Y	Y	Y*	N*	Y	Y*	Y	?	Y	N
TUR014	-	Y	Y	Y	Y	Y	Y	Y	N*	?	N	N
WIN001	-	Y	N*	Y	Y	N*	Y	Y	N*	?	N	N
WIN002	-	Y*	N*	Y	Y*	N*	Y	Y*	Y	?	Y	N
WIN003	-	Y	Y	Y	Y	Y	Y	Y	Y	?	N	N
WIN004	-	Y*	N*	Y	Y*	N*	Y	Y	N*	?	Y	N
WIN005	-	Y*	N*	Y	Y*	N*	Y	Y*	N*	?	Y	N
WIN006	-	Y*	Y	Y	Y*	Y	Y	Y*	N*	?	Y	N
WIN007	-	Y*	Y	Y	Y*	Y	Y	Y*	Y	?	Y	N
WIN008	-	-	-	-	-	-	-	-	-	?	N	N

Of the prediction methods studied in this thesis, the method of Davis and Berrill (1982) appears to perform slightly better than the other methods in giving the greatest number of correct predictions of the occurrence or non-occurrence of liquefaction. The Oparara School site with its nearly uniformly graded sand is best fitted by the most of the prediction methods.

9 Discussion

Variations in the estimated peak ground accelerations for the earthquakes reviewed in this study, have introduced a degree of uncertainty to the analysis models used in the prediction of liquefaction potential. This section seeks to outline the effect of this and propose methods that may be applied in the future, to refine and further validate the models presented herein.

In order to improve the understanding of liquefaction phenomena and associated prediction models, a number of key sites in the Grey-Buller District are proposed for permanent instrument installation and further long-term study.

9.1 Modelling

The tables given in Chapter 8 summarising the analysis results illustrate that the performance of the models is not good. A number of the prediction methods indicate liquefaction at sites where no surface evidences of liquefaction were recorded, following the earthquakes experienced on the West Coast, since the late 1920s. This absence of correlation may be attributed to an adopted lower bound approach in the models examined here. For example, Youd *et al.* (2001) writes, when discussing the curve defining the limit between liquefaction and non-liquefaction of sites according to the method of Robertson and Wride (1998), that, "*several studies have confirmed that the CPT criteria .. are generally conservative*". Youd *et al.* (2001) also states that the criteria of Suzuki et al. (1995) are "*slightly more conservative than those of Robertson and Wride (1998)*".

The liquefaction potential models are predominantly based on case history data from the USA, Japan, and China. The lack of knowledge available, regarding liquefaction effects based on New Zealand earthquakes and the applicability of the models to local soils has implications on the application of these models in New Zealand. For example, Zhou *et al.* (1997) noted that the New Zealand acceleration attenuation may be different to that in other countries. Ólafsson (1999) noted that the attenuation rates in Iceland were different to those appropriate for the Western USA or continental Europe, where it is believed that the thickness of the earth's crust is an important factor in the attenuation rule. One of the major factors used in all of the prediction models is the intensity of the shaking at the site. This is usually estimated from the magnitude of the earthquake at the epicentre and rule of the attenuation of the shaking with distance, and expressed as a single term, the peak ground acceleration (a_{max}), or pair of terms, the earthquake magnitude (M) and distance to the epicentre (r). It must be noted that in this study, the accelerations at the epicentre are estimated and with the uncertainties in the attenuation for many of the earthquakes on the West Coast, estimates of the site acceleration are therefore difficult. There are also uncertainties in the epicentral distances.

9.1.1 Uncertainties in Data

Additional inaccuracies in the modelling undertaken for this study may stem from uncertainties in the peak ground acceleration values, the distances from the epicentre of the fault rupture to the site in question, the measured cone tip values or in the models themselves.

The peak ground accelerations used in the modelling process for the 1929 Murchison earthquake and for some sites for the 1968 Inangahua earthquake were simply estimated from qualitative observations rather than discrete measurements. These values were based on the attenuation models of Zhou *et al.* (1997) and the peak ground acceleration values reported by Zhou *et al.* (1997), from scratch plate records.

Martin and Lew *et al.* (1999) wrote, "*the larger magnitude earthquake produces more cycles of strong ground motion than does the smaller magnitude event, even though both may have produced the same peak acceleration*". Liquefaction occurred in Blaketown, Greymouth, in the 1929 Murchison earthquake but not in the 1968 Inangahua event. In the analysis, the same peak ground acceleration was used for the study of the prediction models for both the 1929 and 1968 earthquakes in Blaketown. This was done as the motion was felt strongly in Greymouth in 1929, as indicated by the isoseismal maps. Even when using this estimated value of PGA, not all the sites in the Greymouth area were considered potentially liquefiable, under the 1929 conditions using the various prediction models. This may be a result of the simplicity of the models, densification of the soil due to previous shaking, consolidation over the past 74 years, or some other, as yet, undetermined phenomena occurring to the soil prior to testing in 2003.

Methods such as Davis and Berrill (1982), Berrill and Davis (1985) and Liao *et al.* (1988), estimate the seismic energy arriving at a site from the magnitude of the earthquake and the epicentral distance. These epicentral distances are only known approximately and the rupture at the epicentre may stretch for many kilometres along a fault, making accurate estimates of the distance very difficult. Another factor to consider is that the Davis and Berrill model also assumes a spherical propagation of energy. This assumption, although convenient, neglects directivity effects that can be seen in the isoseismal maps drawn, based on the felt intensities. However, the model could be adjusted for this. For example, for the 1929 Murchison earthquake, a spheroidal propagation of earthquake energy could be used as the isoseismal contours are ellipsoidal in shape. However, this will require further work on the estimation of the appropriate epicentral distances. This modification would introduce another parameter to the model as the energy radiation pattern varies for each earthquake.

Validation of Test Results

As an extension to this study, it would be of benefit to undertake further cone penetrometer tests in the areas studied. Additionally, alternative testing methods such as the seismic cone penetrometer testing, may be used to validate the results. In the cone penetrometer tests, the depths of testing were limited due to the capabilities of the drilling rig used and the nature of the soils in both the Greymouth and Karamea regions. If a test could be made to a greater depth and the seismic cone penetrometer tests undertaken, the soil properties and therefore liquefaction potential of the area could be estimated with a greater level of confidence. Further bore logs would also be valuable in defining the variations in the soil profile across the sites studied.

Greymouth

Undertaking deeper CPT and seismic CPT testing at both Steer Avenue and Collins Street sites in Greymouth would be useful in confirming the results of this study. Other sites of interest would be near the Grey Base Hospital and the adjacent airport runway field. Sand boils were reported in these locales in the 1929 earthquake. While obtaining permission to test may be difficult, these sites will be of great importance following the next large earthquake event, as the hospital and airport would be key lifelines for the West Coast.

Karamea

Specific sites where further testing would be useful are in the paddock adjacent to the old wharf site and near the old Oparara School site. The paddock closest to the wharf site is only accessible via a channel through the estuary at low tide and may require the use of a small track mounted CPT unit to cope with the poor load-carrying surface. Testing at this site could confirm the assumption that the conditions in the paddock near the wharf, where testing was undertaken for this study, are similar to those in the paddock adjacent to the wharf. It would also be interesting to undertake some testing in the paddocks near the Oparara School site, such as those on the other side of the Karamea- Kohaihai Road, which were farmed by Mr. Thompson during the 1929 earthquake. There were reports of waterspouts in these paddocks as well as in the school grounds during this event. The models used in the study do not predict any significant liquefaction at the school site, and it would be valuable to assess the variation in soil profiles in the adjacent area. It is possible that the testing, which was undertaken, was in atypically dense soil conditions.

Further Site Testing

From both a scientific and a civil defence point of view, it would also be prudent to investigate other sites on the West Coast where liquefaction may be expected if earthquakes were to occur in regions other than the Buller Gorge. This could include areas adjacent to the current seismic gap on the Alpine Fault. This might affect areas near Hokitika, Ross and other flood plains along the coast.

Undertaking testing at these locations will greatly expand the current soil profile database and facilitate future refinement of prediction models on examining the occurrence or non-occurrence of liquefaction in these regions in a future earthquake event.

Accelerations

In order to better understand the attenuation of the seismic energy, it is suggested that a number of accelerometers be installed on the West Coast. These will give a measure of the variation of ground acceleration, with distance from the source of the rupture in future earthquake events. There is also the potential to gain useful information from the installation of a series of down-hole arrays and piezometers. These will enable the measurement of the variation in the vertical distribution of ground accelerations and pore-water pressure changes in the various layers of soils that may liquefy in future earthquakes. These aspects will be discussed later in the chapter.

9.1.2 Uncertainties in Modelling

Thin Layer Liquefaction

Many of the prediction models indicated that a large number of relatively thin layers had the potential to liquefy over the depth of the soil columns studied. It was noted by Pyke (1995), that thin liquefiable layers, "*are unlikely to be of practical consequence under level ground, especially when they are well below foundation depth*", whereas they should be considered of importance on sloping ground. The sites considered in this study were all level ground sites. If a thin layer liquefies the increased pore-water pressure is unlikely to be able to build up to the extent required for the surficial effects of liquefaction, such as sand boils, to be seen. This means that liquefaction may have occurred at lower levels, leaving no surface indication of the occurrence. This also means that the model predictions could be correct, but there is no way of knowing for sure. Thin layers of soil may have liquefied in Karamea due to the 1968 Inangahua earthquake, as was indicated by the different prediction models, yet no effects were observed at the ground surface. If the confining layers are sufficiently porous, the increased pore pressure required for liquefaction may have been dissipated into the surrounding soil. It was also noted in Chapter 6 that the tip resistance values measured in the thin layers might be inaccurate due to the deformation caused by the instrument to the soil ahead of the penetrating cone. As a result of this, the ascertained properties of these thin layers may not be known with sufficient accuracy for any modelling based on the data to give accurate predictions.

Soil Profiles

The method of Robertson and Wride (1998), which was adopted by the NCEER Workshop Participants as the recommended method of determining the liquefaction potential of a site, does not appear to represent very well the observed behaviour in the Grey-Buller region. Layers of soil were deemed liquefiable at almost every site under the conditions of either the 1929 Murchison earthquake or the 1968 Inangahua earthquake, even though no surface effects due to liquefaction were reported. It is possible that the normalising process used in this method, provides insufficient acknowledgement of the properties of alluvial soils i.e. layers of gravel and then silts and sand deposited by successive floods.

The discrepancies in the inferred soil profiles in comparison to those obtained by hand auger borings and the particle size distribution plots suggests that soil profile identification through hand augering and drilling is a necessity rather than an option. It is acknowledged that some thin layers of soil may not be identified from hand auger bores or drilling due to the disturbed nature of the soil. However, as noted above, the cone penetrometer probe is likely to disturb the soil for depths of at least 2 to 3 cone diameters ahead of the cone. This is equivalent to a depth of greater than 0.1 metres. This means that the thin layers identified by methods such as Robertson and Wride (1998) and Olsen (1997) are of dubious confidence as the input parameters are suspect.

9.1.3 Sensitivity study

A study was carried out, in order to observe the sensitivity of the various models to changes in the peak ground accelerations and the distance from the site to the epicentre of the earthquake. The results from the following four test sites were used:

- Oparara School Site, Oparara (OPA001);
- Simpson's Paddock, Karamea (SIM001/002/003);
- Fensom's Paddock, Karamea (FEN003); and
- Collins Street, Greymouth (COL002/003).

Peak Ground Acceleration (a_{max}) Modification

The methods of Shibata and Teparaksa (1988), Suzuki *et al.* (1995), Olsen (1997), Robertson and Wride (1998) as NCEER (2001) and Juang *et al.* (2003) all use the peak ground acceleration to characterise the seismic energy arriving at a site and therefore the potential of liquefaction. The peak ground acceleration values were adjusted by ± 10 % of the original estimated value and the changes in the results due to this noted as follows:

At the Collins Street site, the methods of Shibata and Teparaksa, Suzuki *et al.*, and Robertson and Wride, all indicated a small increase in the thickness of the potentially liquefiable layers. The method of Juang *et al.* indicated a greater thickness of the liquefied layer as can be seen by comparing Figure 9-1 and Figure 9-2. With the variation in the peak ground acceleration values, the method of Olsen still indicated that no significant layers would liquefy at this site.

When the peak ground acceleration values were varied for the Oparara School site, there was no change in the thickness of the layers deemed potentially liquefiable under the 1929 Murchison earthquake conditions.

At both the site of SIM001/002/003 and FEN003, four of the five models predicted extensive liquefaction down the soil columns. The methods of Shibata and Teparaksa, Suzuki *et al.*, Robertson and Wride, and Juang *et al.* all indicated slight increases in the thickness of the liquefiable layers, though the small scale of the differences implies that the models are reasonably stable to this sort of change. The prediction method devised by Olsen, indicated that the soil column at SIM001/002/003 would not liquefy and altering the PGA values did not change this result. However, the method did predict a number of layers of varying thickness would liquefy at FEN003. When the a_{max} value was modified for this site, the thickness of the liquefiable layers increased as shown in Figure 9-3 and Figure 9-4. This shows the significance of the tip resistance values on the sensitivity of the results.

These results indicate that the models are not very sensitive to errors in the peak ground acceleration.

Distance R Modifications

Three of the liquefaction potential prediction methods use the magnitude of the earthquake and the distance between the site and the source of rupture to characterise the seismic energy arriving at a site. These models include Davis and Berrill (1982), Liao *et al.* (1988) and Law *et al.* (1990). The changes in the prediction by each of these models are compared when the input parameters are varied.

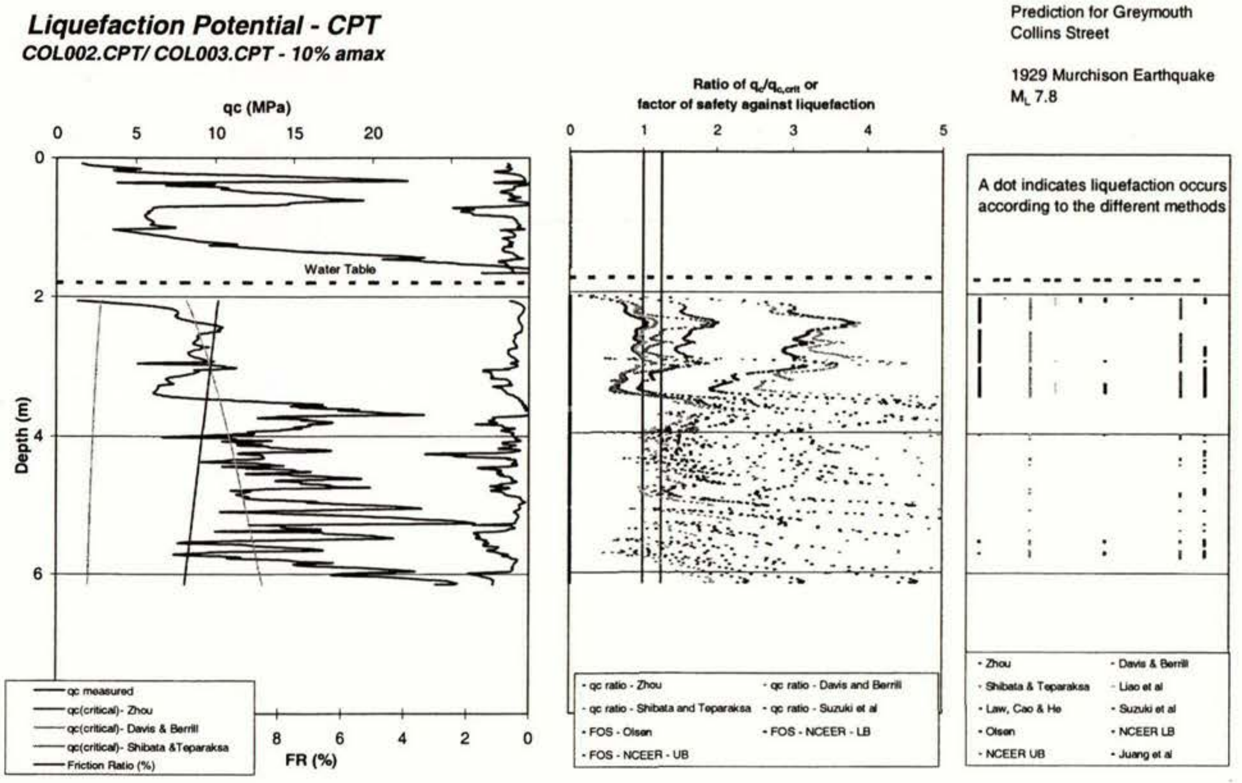


Figure 9-1. Collins Street results for -10% a_{max}

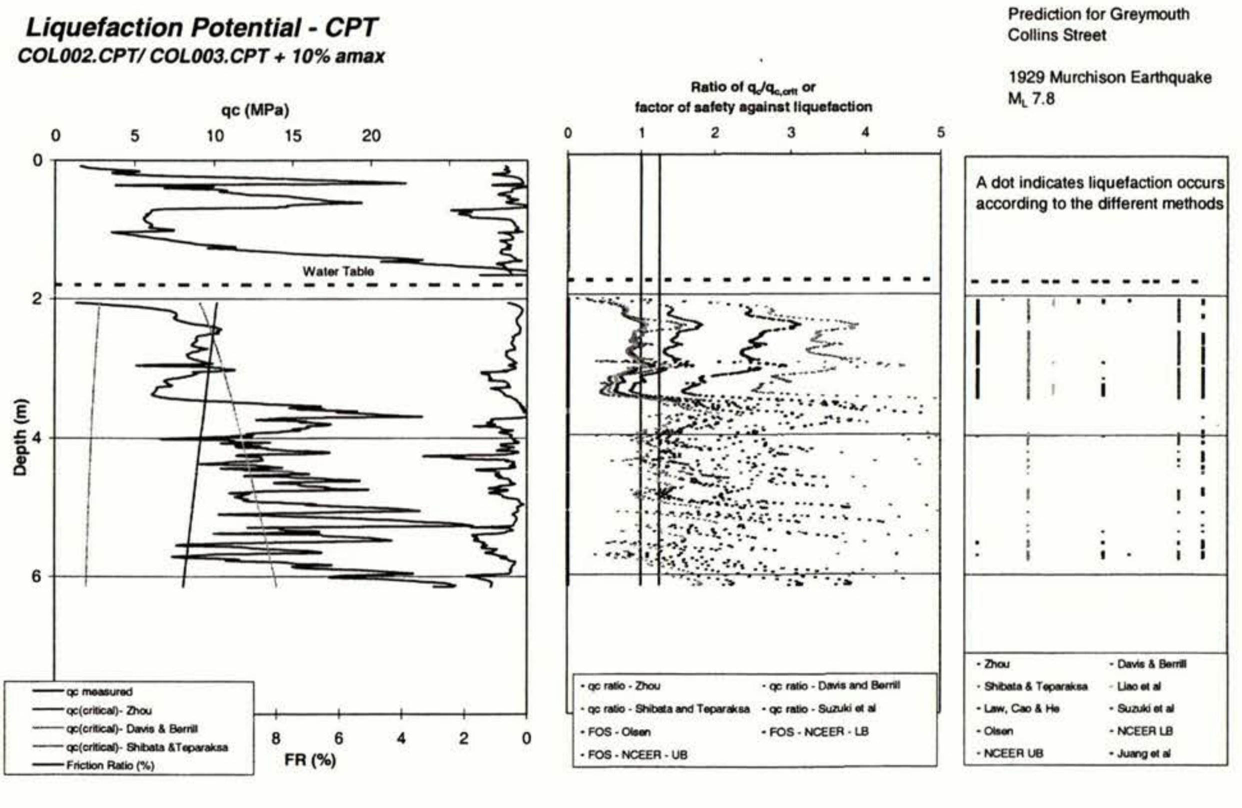


Figure 9-2. Collins Street results for + 10% a_{max}

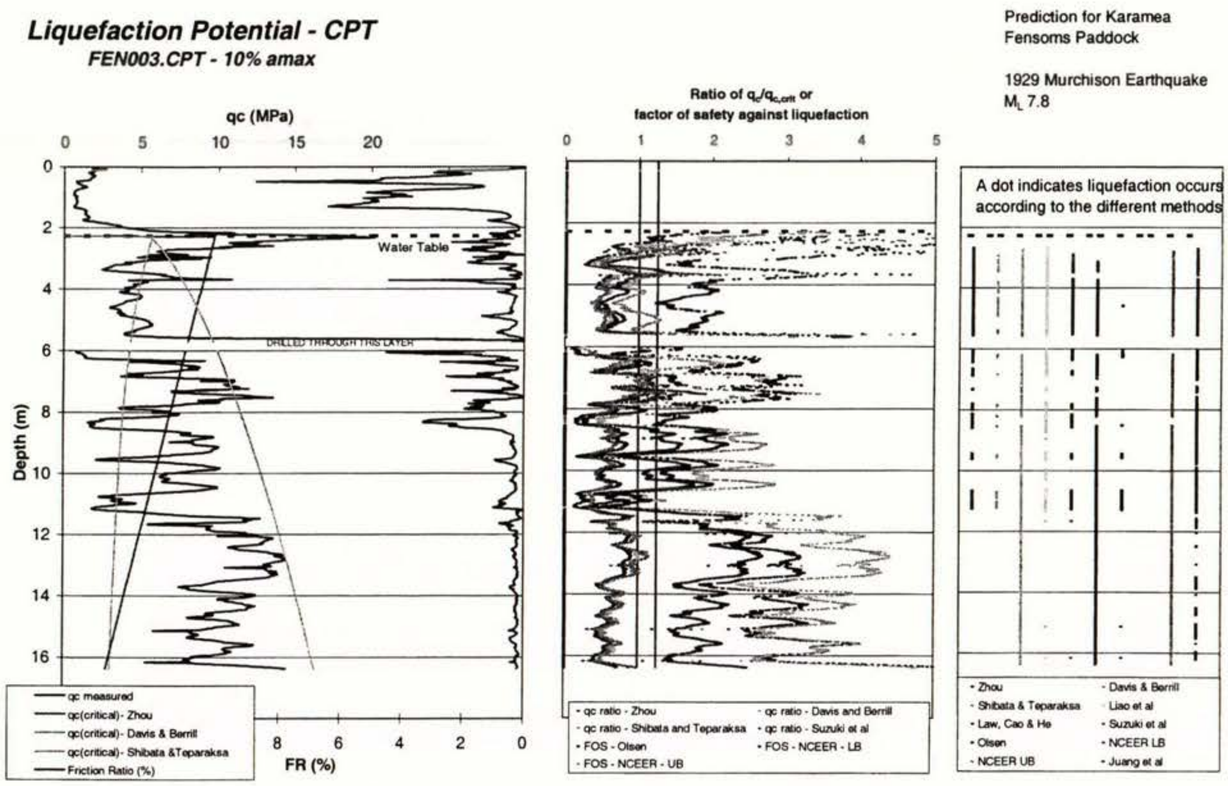


Figure 9-3. Fensom's Paddock results for - 10% a_{max}

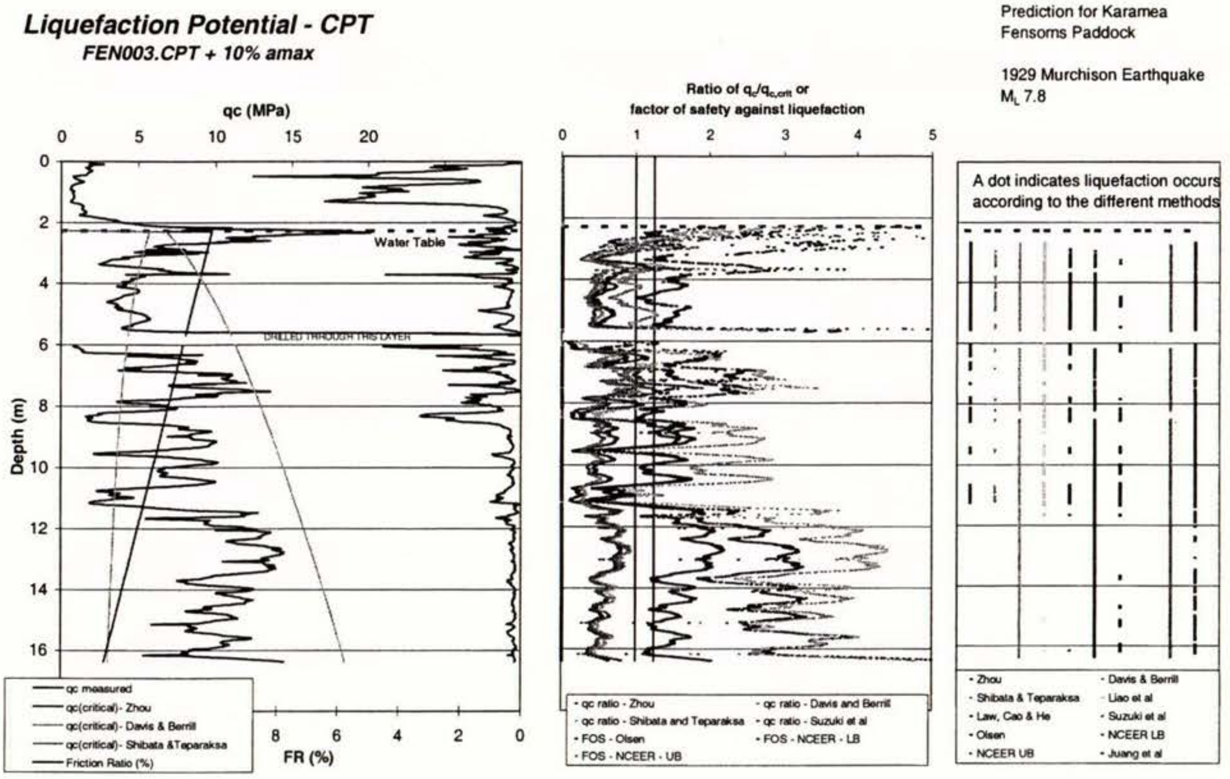


Figure 9-4. Fensom's Paddock results for + 10% a_{max}

For the sites in Karamea, Simpson's Paddock, and Fensom's Paddock, all the methods predicted that some liquefaction would occur. The thickness of the liquefiable layers at Fensom's Paddock varied for each of the three models and when the distance decreased, the thickness of the potentially liquefiable layers increased slightly. This was similar to that found for Simpson's Paddock where when the distance *R* was decreased, the methods of Liao *et al.* and Law *et al.*, predicted slightly varied thicknesses of the potentially liquefiable layers. This shows that the models require reasonably accurate epicentral distance values in order to achieve accurate identification of the liquefaction potential.

Figure 9-5 and Figure 9-6 show a reasonably large variation in the thickness of a liquefiable layer, as predicted by the method of Davis and Berrill. The nature of this layer is reasonably consistent and illustrates the role of tip resistance values in how the models are affected by variations of the parameters such as the distance *R*.

The dense nature of the soil below the water table at both the Oparara School site and at the vacant lot in Collins Street, Greymouth, meant that modification of the values of *R* had a minor effect on the results for only the method of Liao *et al.*. For both sites, the thickness of the liquefiable layer varied slightly according to the Liao *et al.* prediction model. At the Oparara School site, only the method of Zhou (1980), and Liao *et al.* predicted, at the most, a 0.2 metre thick layer to be potentially liquefiable under the excitation of the 1929 Murchison earthquake.

The site liquefied in the 1929 Murchison earthquake, which means the testing could possibly have been undertaken at atypically dense locations, or the water may have been closer to the ground surface at the time of the earthquake. Dowrick (1994) noted that 1929 had a much wetter winter than usual on the West Coast. The shaking caused by the earthquake in 1929 and smaller subsequent events may have left the ground in a more consolidated state than it was prior to the 1929 earthquake, so that it is no longer liquefiable under similar conditions to this earthquake. It must be noted that the testing for this project was undertaken during the summer months.

The values of the factor of safety obtained by Juang *et al.* (2003) also appear to be sensitive to small variations in the data. If the cone tip resistance value is greater than approximately 17 MPa, the safety factor against liquefaction occurring at a site becomes unrealistically large.

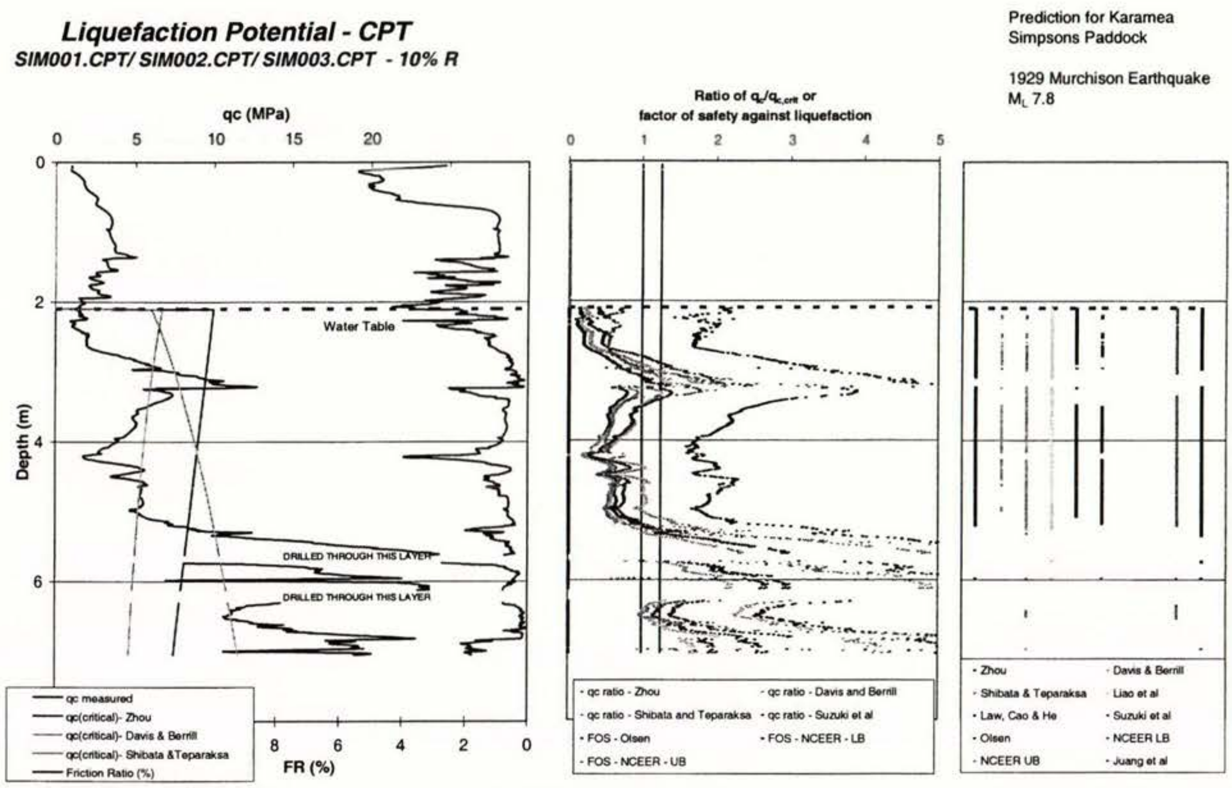


Figure 9-5. Simpson's Paddock site results - 10% R

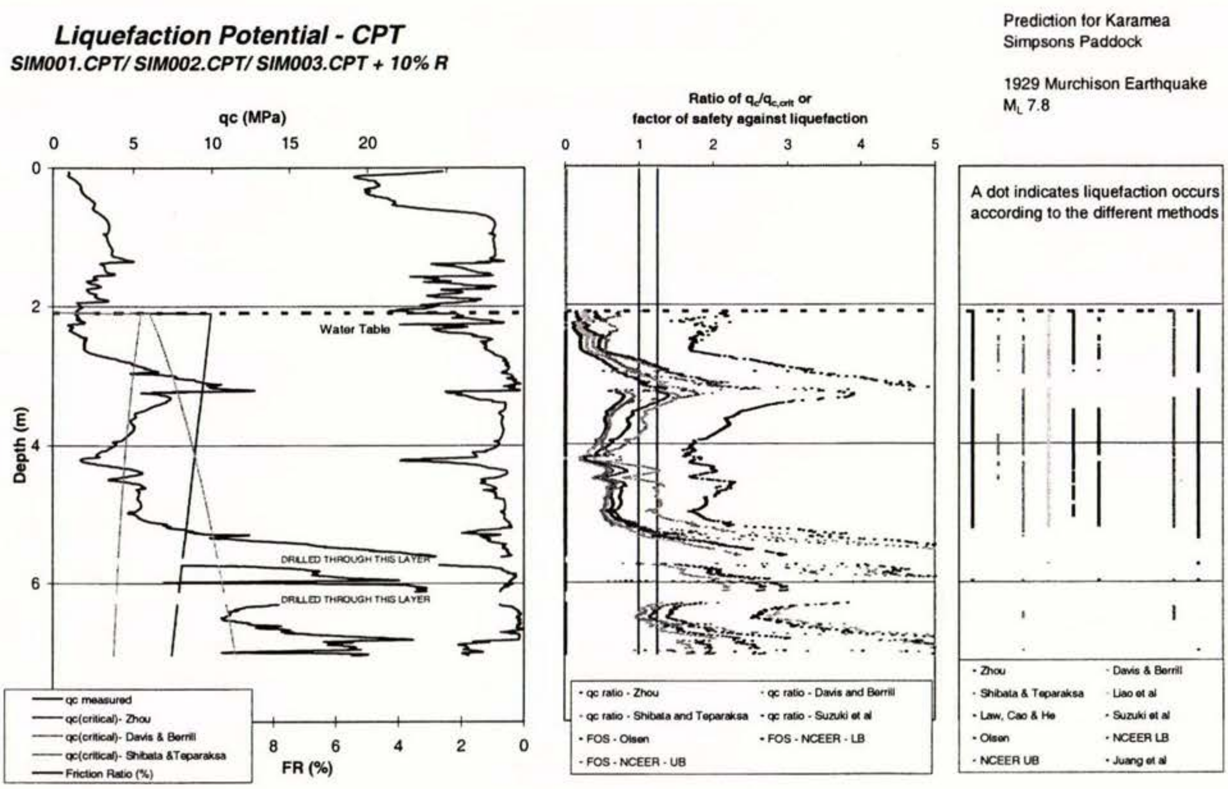


Figure 9-6. Simpson's Paddock site results + 10% R

9.1.4 Improving the Models

In order to improve the accuracy of the prediction models and to increase the understanding of where the phenomenon of liquefaction will occur, details of the sites which may liquefy are needed before an earthquake so that the properties of the sites can be compared before and after the occurrence of liquefaction in the earthquake.

This means that the data obtained in this study can be used as base line for comparison with information obtained at liquefied and non-liquefied sites following the next large earthquake on the West Coast.

An improved model incorporating a greater number of parameters needs to be developed to better represent the complexity of the seismic energy arriving at a site. For example, the thrust events in the Buller region are very different to the strike-slip earthquakes of California, but similar to some events in Japan. As indicated by the results of the analyses, rupture directivity leading to energy focussing is a source characteristic which should also be investigated. The current soil state, in terms of aging effects and particle distribution, also needs consideration as part of the liquefaction model. For example, the simple Oparara site appears to fit the current models better. The improved models should include the associated attenuation rates and duration effects. This needs to replace the current simple models which utilise only the peak ground acceleration value or the magnitude and epicentral distance pair of parameters.

9.2 Instrumentation Arrays

9.2.1 The Need for Instrumentation

There is an immediate need for pre-event instrumentation to be installed at key sites on the West Coast. Any information that can be obtained from such an initiative would be invaluable in terms of validating and further refining the liquefaction potential prediction models.

Accurate descriptions of the peak ground accelerations would be attainable through the installation of arrays of accelerometers at these key sites. These acceleration measurements would enable improvement of the acceleration attenuation modelling, a more detailed description of the variation of seismic energy arriving at a site and as such improve the liquefaction prediction models that rely on this information in order to give accurate predictions.

Most of the liquefaction potential prediction models have been formulated using data from the USA, Japan, and China. This means that the information gathered in the future may be used to validate these models for New Zealand conditions, or alternatively, be used as a basis for New Zealand specific liquefaction prediction models.

9.2.2 Instrumentation Required for the Arrays

In order to obtain the most useful data, the installation of both horizontal and down-hole instrument arrays of accelerometers would be required. The use of these instruments will give more accurate peak ground acceleration values for use in the liquefaction models.

Horizontal arrays of instruments would enable better definition of the seismic energy attenuation relationship as the distance from the epicentre of the fault rupture increases. For instance, the method of Davis and Berrill (1982) assumes a spherical attenuation of seismic energy. When directivity effects and the effects of the altering rate of energy propagation through differing rock and soil outcrops are taken into account, this simplified model may need refinement.

These horizontal arrays would consist of instruments at the ground surface to measure the accelerations in both the North-South and East-West horizontal directions. If enough instruments were available, the measurement of the vertical accelerations could also be recorded.

Installing a number of down-hole arrays would enable the measurement of the vertical variation in horizontal accelerations at a site. This will help identify any possible liquefaction at lower levels that may not result in surface manifestations. Liquefaction at a lower depth may in effect, base isolate the above layers.

Pore pressure transducers should also be incorporated in the down-hole arrays to observe the effects of the earthquake shaking on pore-water pressure. If a layer was to liquefy the pore pressure would increase and there is the likelihood that the horizontal accelerations recorded above that layer will be markedly different from those below the liquefied layer.

9.2.3 Locations for the Instrumentation

Constraints are imposed on the location and type of instruments used in the arrays due to the flood prone nature of many West Coast towns. This means that the instruments need to be able to withstand such events, be accessible and monitored regularly to ensure they are in correct working order. Additionally, they need to be cheap enough so that they can be replaced if needed. The sites selected must be founded on sands and silts as the peak ground accelerations measured at rock sites are very different to those measured on soft soils (Zhou *et al.*, 1997).

The following sites are suggested for locating instruments along with justification for the site selection. Ideally, down-hole arrays would be located at each site, in which case, instruments on the ground surface would suffice for the horizontal array setup.

Karamea

There are two possible sites for instrumentation in Karamea; in the paddock near the location of the old wharf and near the Karamea Area School.

The well documented damage to the 1929 wharf, together with the uniform soils found in the investigation would make this an interesting site for instrumenting. The soils in both the paddocks near and adjacent to the old wharf were reasonably consistent, comprising of medium to coarse sands. In comparison, the ground near the School is underlain by sand and silt layers having been laid down by the river environment. The area of land from the School towards the location of the wharf on the coast towards the west of this site suffered widespread liquefaction in 1929. The current liquefaction potential models predict that the sites studied in 2003 should have liquefied under the conditions of the 1968 Inangahua earthquake, even though no surface effects of liquefaction were visible. In this respect, the land surrounding the school would be a very interesting area to study after another earthquake, especially if the details regarding the strength of the shaking were accurately known. The school ground is also owned by the Ministry of Education and as such may make it easier to set up and maintain an instrument array.

Westport

The studies made by Ooi (1987) and Dou and Berrill (1992) indicated that reasonably consistent sand and gravely sand layers underlie Kilkenny Park. The site liquefied in the 1968 Inangahua earthquake and there were reports of liquefaction in the surrounding streets following the 1929 Murchison earthquake. Local residents could not recall any surface effects of liquefaction, such as waterspouts or sand boils in the park, following the 1991 Hawks Crag earthquakes, though sand boils were located in the nearby Westport North School grounds. As Kilkenny Park has liquefied in the past and the surrounding soils have shown signs of liquefaction, it would be interesting to see how the soil profile may change after further earthquake shaking and to compare the CPT logs with those recorded in the mid 1980s and early 1990s by researchers at the University of Canterbury. Accurate records of the peak ground acceleration would also be very valuable in order to test the current prediction models.

Murchison

A number of areas around the township of Murchison have liquefied in the past, resulting in multiple candidate sites for permanent instrument installation. It is known that parts of Fern Flat and Four Rivers Plain liquefied in both the 1929 Murchison and the 1968 Inangahua earthquakes and photos indicate that areas of the township also liquefied in 1968. Even though no testing has been undertaken in the school grounds in the past, this is likely to be a good location for the instrument arrays. The site is close to the river and hence the soil conditions are likely to be similar to other areas that have liquefied in the past. The site is crown owned land, which would also make instrumenting this area a viable long-term proposition. Testing to determine the 'benchmark' properties of the soil would need to be undertaken prior to installing the instrument arrays.

Inangahua

Under past earthquake events, parts of Walkers Flat have liquefied and some sites re-liquefied in subsequent events. It would be interesting to install an array of instruments in this area to see how the peak ground accelerations vary across the site as well as down a soil column. Other instrumentation, such as piezometers, may also be useful in order to fully characterise the effects at this site and help in the understanding of the liquefaction phenomena. However, this land is currently being used as farmland, which may make installation of equipment difficult at this site.

Greymouth

The proposed location of a down-hole array, would be on the edge of the airport runway, near the hospital. Unfortunately, no soil profiles were drawn from this area for this study. However, unlike the sites alongside Steer Avenue and near Collins Street, the site is unlikely to be underlain by fill material that was dumped during the construction of the breakwater. At the time of the 1929 Murchison earthquake, this field was known as the abattoir paddock. Following the 1929 earthquake, reports were given of waterspouts in this paddock and the soil near the hospital was described as having quicksand like properties when wet. Again, testing to determine benchmark properties must be undertaken prior to the installation of the instrument arrays. Although there has been substantial regrading to form the airport, it is understood that only the in-situ sands were involved. Therefore the installation of an array at this site is recommended, with appropriate drilling and probing to establish the present soil properties and profile.

Hokitika

Instrumenting this area would also be a worthwhile exercise as the next big earthquake may occur on the Alpine Fault and cause problems in Hokitika, another West Coast town built in a flood plain environment. Some preliminary testing would be needed in order to determine a suitable site for setting up the arrays.

By extending the array further south, a greater portion of the West Coast would be instrumented. It is not known where the next earthquake will occur and it would be prudent to extend the net further south.

9.2.4 Monitoring

Monitoring of the site conditions must be undertaken on a regular basis so that the site conditions are reasonably accurately known both before and after a future earthquake event. This includes recording the variation in the depth to the water table and the use of seismic wave velocity measurements to increase understanding of aging effects on soil and hence liquefaction potential.

At nearly all of the sites suggested above, knowledge has already been gathered regarding the soil profiles and results from cone penetrometer tests. It would be prudent to undertake further cone penetrometer tests at the sites where the variation in soil profile is unknown. These results should then be extended using other testing methods such as the seismic cone penetrometer test, or geophysical testing and bore logs, obtained adjacent to the test sites. This data would provide a "datum", and similar testing carried out every five or ten years at each site, could monitor the changes in the soil profiles in terms of effects, such as consolidation and especially water table variations. The water table variations should also be correlated against rainfall records. For instance, Dowrick (1994) reported that the winter of 1929 was an unusually wet winter.

This monitoring may also increase understanding of long-term site effects and indicate why the Oparara site is not considered potentially liquefiable, according to the prediction models, under the conditions of the 1929 Murchison earthquake. As noted earlier, the soil at the site may have become denser under the 1929 earthquake shaking or from consolidation over the past 74 years. The water table may also have lowered over the past 74 years due to the Oparara River cutting down, the alteration of the coast line and drainage of the flat land inland from the coast.

9.3 Sites to Study Following the Next Earthquake

It was discovered during the historical study that a number of sites, which liquefied in either the 1929, 1968 or 1991 earthquakes, have undergone liquefaction more than once.

As a first point of call of sites to investigate for possible liquefaction following the next large earthquake event in the Buller Gorge area of the West Coast of New Zealand, the following sites are suggested:

- Karamea
    - Arapito
    - Paddocks near the school grounds
    - Paddocks near the Wharf site
    - Oparara
- Westport
    - Kilkenny Park
    - Sergeants Hill
    - Westport North School
- Inangahua
    - Three Channel Flat
    - Walkers Flat
- Murchison
    - Four Rivers Plain
    - Fern Flat
- Greymouth
    - Fields at Coal Creek
    - Blaketown – Steer Avenue and Collins Street
    - Airport Runway Field/ Seaward side of the Hospital

The sites listed above are all shown on the diagrams (Figure 9-7 to Figure 9-11) on the following pages. It would also be worthwhile investigating the settlements of Mokihinui and Seddonville at locations where liquefaction was reported, following the 1929 Murchison earthquake. The locations of each of these sites can be seen in Figure 6-1.

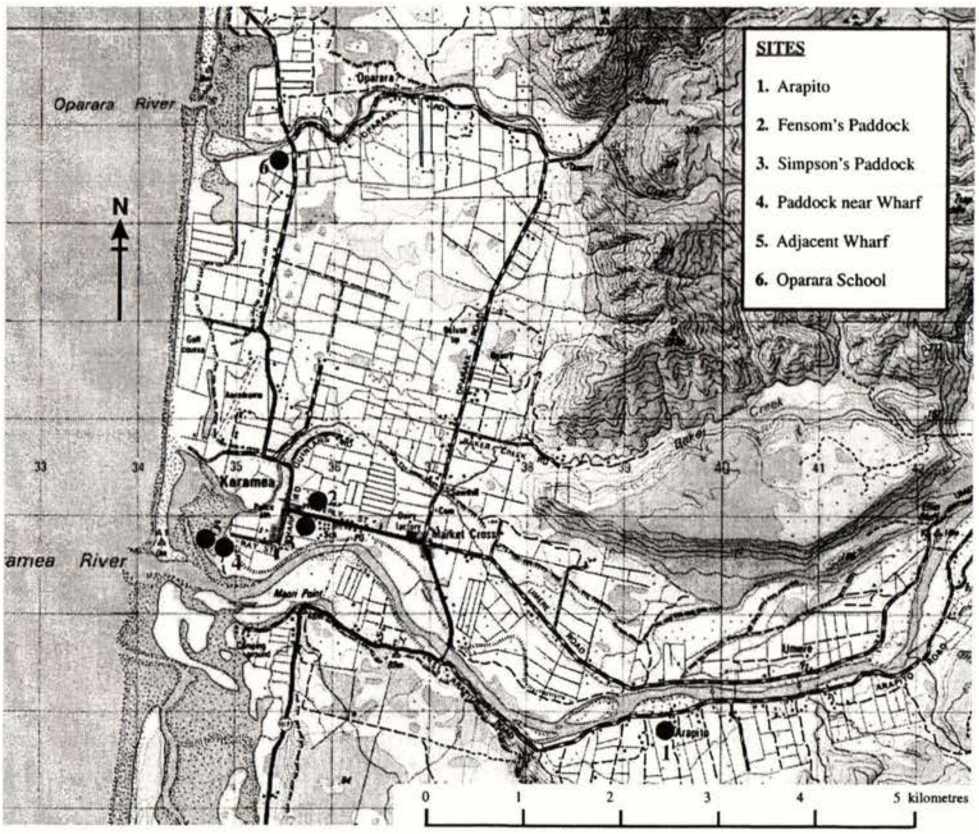


Figure 9-7. Sites to study in Karamea (Base map: Topomap, 2001)

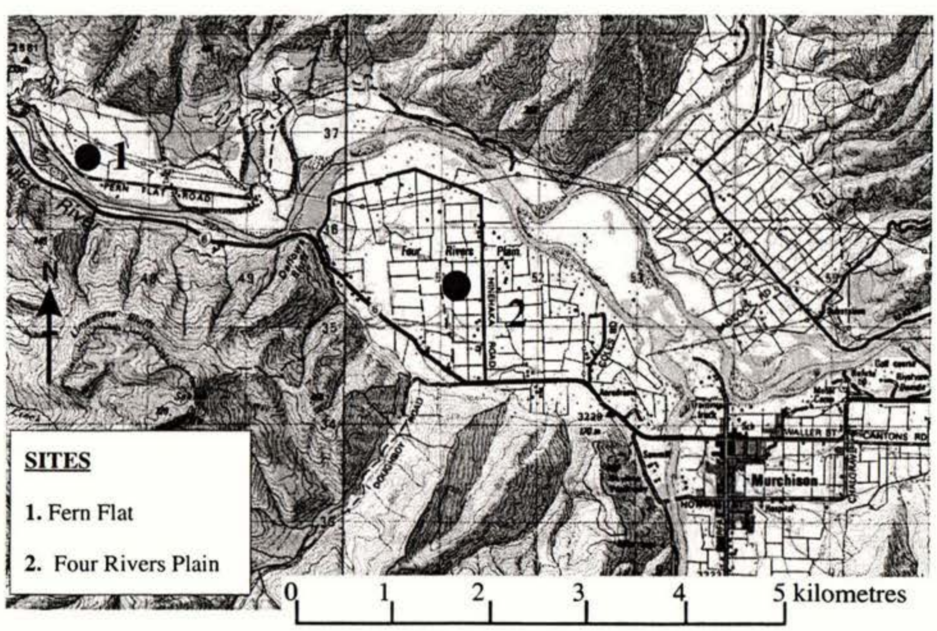


Figure 9-8. Sites to study in Murchison (Base map: Topomap, 2001)

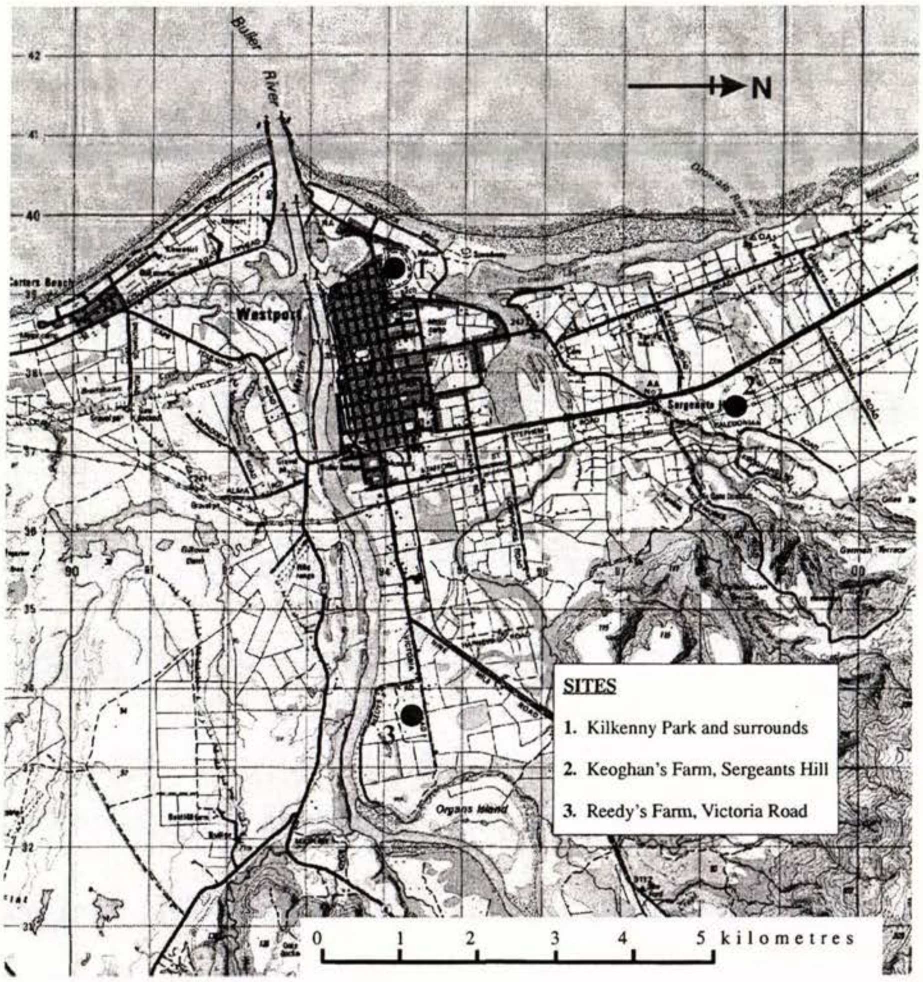


Figure 9-9. Sites to study in Westport (Base map: Topomap, 2001)

Figure 9-10. Sites to study in Inangahua (Base map: Topomap, 2001)

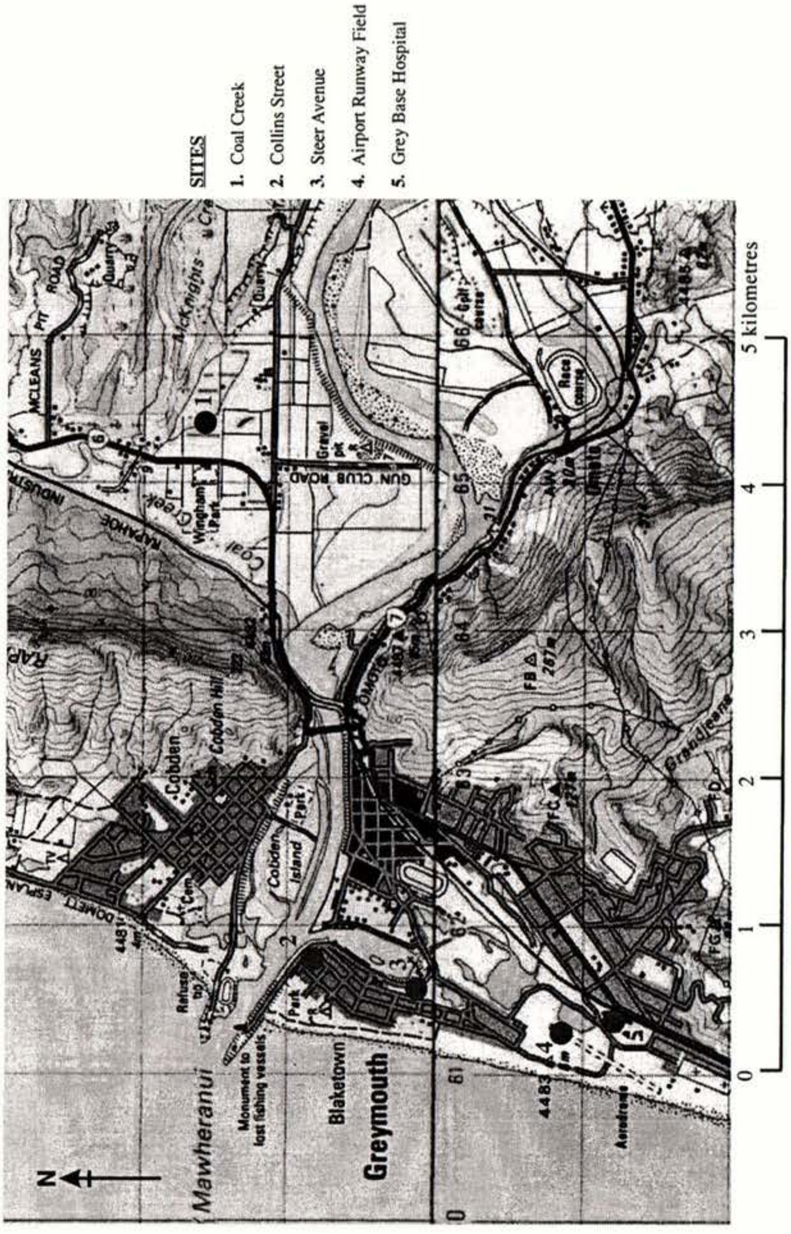


Figure 9-11. Sites to study in Greymouth (Base map: Topomap, 2001)

10 Summary, Conclusions and Suggestions for Future Research

In this chapter the original objectives of the project are compared to the results obtained from the work undertaken and the significance of this study is reviewed. Future work, which should be undertaken to refine this study and further our understanding of the liquefaction phenomena and seismic energy attenuation in New Zealand, is also recommended.

10.1 Summary

The original project objectives were to build upon the current case history database for sites of liquefaction resulting from earthquakes on the West Coast since the 1920s. Site investigation work was performed at sites which liquefied in selected earthquakes and not in others, to ascertain the variation in soil properties and profiles at these sites. Using the information collected, an analysis of the current methods of predicting the liquefaction potential at a site was reviewed. Research into historical records and personal accounts along with the site studies have provided the basis for the suggested sites to be investigated for future instrumentation and further study.

10.1.1 Case History Database

Three major earthquakes on the West Coast of New Zealand since the 1920s- the 1929 Murchison earthquake, the 1968 Inangahua earthquake and the 1992 Hawks Crag events have resulted in damage to infrastructure and loss of life. A number of past University of Canterbury postgraduate students (Ooi, 1987; Adlam, 1988; Bienvenu, 1988) and a Post-Doctoral research fellow Dou (Dou and Berrill, 1992), undertook research into the effects of liquefaction in the Buller Gorge area. These researchers focussed their investigations on areas close to the epicentres of the earthquakes, and obtained case history information for these areas. Bienvenu initiated work in the Karamea area in the late 1980s.

In the course of this study, much additional archival information has been collated from museums, newspapers and from discussions with residents who experienced the earthquakes on the West Coast since the 1920s. This project has greatly extended the University of Canterbury case history database, with sites of liquefaction identified as far north as the Paturau River, and as far south as the Greenstone River on the West Coast as a result of the 1929 Murchison earthquake. These sites are indicated in Figure 6-1.

10.1.2 Fieldwork in Greymouth and Karamea

Soil testing was undertaken at sites in both Greymouth and Karamea. These towns are located a significant distance from the epicentres of the 1929 and 1968 earthquakes, and a number of areas in each town liquefied in the 1929 event but not as a result of shaking from the 1968 event. Due to these differences the results of testing at these localities may be used to help identify the dividing line between potential liquefaction and non-liquefaction of a site.

Preliminary site investigations were carried out at sites that were confirmed to have liquefied as a result of the 1929 Murchison earthquake by a number of residents or by photographic evidence. Hand augering was undertaken to ensure that there were no gravel layers close to the ground surface which may inhibit the cone penetrometer testing and the site assessed for adequate drilling rig access.

Cone penetrometer testing was undertaken in both Greymouth and Karamea. In Greymouth, standard penetration testing was attempted, though the results were discounted due to boiling of sand into the base of the hollow stem auger. SPT testing was not attempted in Karamea due to the very loose nature of the cohesionless soil beneath the water table which was likely to cause similar problems. Hand augering and flight augering were also undertaken in order to define the soil profiles.

10.1.3 Liquefaction Model Study

A number of simple, empirical models have been proposed over the past 30 years to determine the potential of a site to liquefy under earthquake shaking. The results from the CPT testing were analysed according to the following models:

- Zhou (1980)
- Davis and Berrill (1982)
- Shibata and Teparaksa (1988)
- Liao, Veneziano, and Whitman (1988)
- Law, Cao and He (1990)
- Suzuki, Tokimatsu, Koyamada, Taya and Kubota (1995)
- Olsen (1997)
- Robertson and Wride (1998) as adopted by NCEER (2001)

- Juang, Yuan, Lee, Lin (2003)
- Toprak and Holzer (2003)

These models can be categorised into two major groups, those based on an energy approach and those based on a stress approach, both of which are derived semi-empirically from field data.

None of the methods correctly predicted the potential for liquefaction at sites which liquefied in the 1929 earthquake in every instance. The models were also studied in order to ascertain the sensitivity to changes in the input parameters, such as the epicentral distance (the distance between the epicentre and the site in question) and peak ground accelerations at the site. The peak ground accelerations are not known with any degree of certainty for the earthquakes studied. Variations in these parameters were used as they define the basis for the different types of models. The methods of prediction for Davis and Berrill (1982), Berrill and Davis (1985), Liao *et al.* (1988), and Law *et al.* (1990), all use the epicentral distance and the magnitude of the earthquake in question to characterise the seismic energy arriving at a site. In comparison, the methods of Shibata and Teparaksa (1988), Suzuki *et al.* (1995), Olsen (1997), Robertson and Wride (1998), and Juang *et al.* (2003), all use the peak ground acceleration at the site to characterise the number of cycles of shaking. The sensitivity studies indicated that the models devised by Liao *et al.* and Law *et al.* both required fairly accurate epicentral distance estimates. Accurate estimates of the epicentral distances are often difficult as the epicentral distances are only known approximately and the rupture may stretch for many tens of kilometres along the fault.

The peak ground accelerations at the site are often obtained from the magnitude of the earthquake using an empirical attenuation expression. As the magnitude of the 1929 Murchison earthquake is itself an estimate based on felt intensity reports, this adds to the uncertainty and makes any estimate of the peak ground acceleration difficult to apply with any degree of confidence. The uncertainty of relationships between intensity and magnitude, such as the Gutenberg and Richter expression, has become much more apparent with the growing base of data. Thus, the peak ground accelerations at a site were also varied to observe the sensitivity of the models to such changes. It was observed that the models of Shibata and Teparaksa, Suzuki *et al.*, Robertson and Wride and Juang *et al.* are all reasonably stable with respect to small variations in the peak ground acceleration values. When the PGA was altered for the method of Olsen (1997), the change in thickness of the potentially liquefiable layers was strongly affected by the critical cone tip resistance. This was shown by the marked variation in the results, where the results changed only for one of the sites studied and not the others.

The data obtained by Dou and Berrill (1992) was also analysed using the above methods and plots shown in Appendix B.

10.1.4 Sites for Future Study

Sites have been identified where seismic cone penetrometer tests would be helpful to confirm the soil properties obtained from the cone penetrometer testing. In particular, the seismic cone would give another method of evaluating the soil profile and the properties of such soil layers. Further bore logging would also be valuable in ascertaining the variation in profiles, across the sites studied.

Several sites have been proposed for locating future instrumentation arrays, which would enable the recording of horizontal acceleration and pore pressure data and their variations following the future earthquakes on the West Coast. This will provide a better understanding of the attenuation of acceleration on the West Coast and also the vertical distribution of horizontal accelerations (i.e. "site effects") and pore pressure changes and would aid the understanding of the liquefaction phenomena in locally stratified soil deposits, as well as allowing the recent, more analytical, energy approach of Davis and Berrill (1999) to be tested.

Maps have also been drawn up to indicate areas which should be initial points of investigation, to see whether or not there are any signs of liquefaction, for a liquefaction study following the next large earthquake in the Buller Region of the West Coast.

10.1.5 Issues Raised by this Study

The prediction models did not perform as well as expected. This is partly due to the uncertainties in the input data in terms of the peak ground accelerations and epicentral distances, but also because models may be too simple to capture the complex phenomena of liquefaction and the seismological aspects of the earthquake and hence do not incorporate all of the important parameters. Both the attenuation model of Zhou *et al.* (1997), and the peak ground acceleration values obtained from scratch plate records were used to estimate the accelerations for sites where no recordings were available. Zhou *et al.* (1997), also noted that the attenuation of seismic energy in New Zealand may be different from that in other parts in the world. If this is the case, the liquefaction prediction models may need to be adjusted. New Zealand is not on a continental land mass, and as has been found in Iceland, the attenuation rates may be very different to those appropriate for the continental USA or Europe (Ólafsson, 1999).

The distance from the epicentre to the site in question is also approximate as the rupture may extend for many kilometres along a fault. This adds another uncertainty to the data used in the study, though the effects on the West Coast may, in future, be reduced by installing a greater array of accelerometers in the region.

Installing horizontal accelerometers at the ground surface together with down-hole accelerometer arrays, as suggested above, will improve the data recorded. This should help our understanding of the fundamental mechanism and therefore lead to improved modelling of the liquefaction phenomena. A horizontal array of instruments will improve the knowledge of the acceleration attenuation relationships for the West Coast and for New Zealand as a whole, as well as providing the peak ground accelerations in areas where they have been estimated in the past. The down-hole arrays will indicate vertical distributions of the accelerations and as such will indicate the effects of soil liquefaction in lower soil layers and possibly show base isolation-like effects in a soil. Installing piezometers alongside the downhole arrays will illustrate the changes in pore pressure in the soils and hence the onset of liquefaction.

Automatic soil profiling models were examined and it was seen that the models do not always interpret the soil profile correctly. The discrepancies in the inferred soil profiles in comparison to the results of the hand augering illustrate that the automatic models should be used with caution and hand augering or drilling is a necessity at a site rather than an optional extra.

10.2 Conclusions

None of the current prediction models for the boundary between soils likely to liquefy and those not likely to liquefy in an earthquake, appear to perform particularly well in the Grey- Buller Region, with 87 percent correct predictions of liquefaction and 40 percent correct predictions of non-liquefaction. Much of the current data is not very precise as it is based on distant memory and best estimates of parameters such as epicentral distances and peak ground accelerations. This study has indicated the likely reasons for this poor prediction and possible measures to improve both the database and understanding of the liquefaction phenomenon.

10.3 Significance of Research

As a result of this study, the case history database of liquefaction occurrences has been increased. Further sites have been identified as having liquefied in the past and as such are now localities which should be reviewed, following future earthquakes in the Buller Region. Drilling was undertaken at a number of sites in both Greymouth and Karamea and from this base line data and site parameters have been recorded for use in future studies. A number of residents were made aware of the liquefaction phenomena and while the consequences may not be as severe as in highly populated countries such as Japan, risks are still significant.

A review of the current state of prediction modelling was undertaken. This indicated that the models may not be as accurate as would be liked. Small changes in the input data did not have a great effect on the results indicating that the models are not sensitive to variations of the peak ground accelerations within 10%. The study also highlighted the need for soil profile investigations as the models did not correctly interpolate the fluvial nature of the soils in the region. The shortcomings of the cone penetrometer test were also noted. The deformation caused to the soil ahead of the cone tip invalidates the thin layer predictions of soil types and correction for thin layer thickness is required.

Maps indicating locations of sites which should act as a first point of call for a liquefaction study following the next large earthquake in the Buller region of the West Coast have been given in Chapter 9. These are areas which have liquefied in past events and the baseline data has been gathered from site investigation in many of the sites.

10.4 Suggested Future Investigative Work

- Further testing is required to enhance the value of the results obtained by the CPT. These tests were carried out to a limited depth due to the nature of the soil and the capabilities of the drilling rig. The standard penetration tests are not really viable in the areas studied due to the free flowing nature of the sand beneath the water table resulting in boiling problems with the hollow stemmed auger and the SPT test apparatus. It is suggested that seismic cone penetration tests be undertaken in both Greymouth and Karamea to confirm the soil profiles indicated by the CPT. Further bore logging and drilling at the key sites would also be useful in fully characterising the properties of the soils at the sites.
- Further experimental work needs to be undertaken to better understand the liquefaction of thin layers of loose materials overlain by non-liquefiable materials. A number of sites in the study region are stratified deposits where thin layer liquefaction is likely.
- Installation of arrays of accelerometers, both horizontal distributions on the ground surface and down-hole, would be very useful in refining the acceleration attenuation relationships for both the West Coast Region and New Zealand in general. The measurement of pore pressure changes and the accelerations will all improve the future modelling of the liquefaction phenomenon through increased understanding of the effects.
- This study has focussed on the Buller Region. It would be worthwhile for future studies to cover the flood plain regions south of Greymouth, such as the coastal river flats around

Hokitika, Ross, Harihari and Whataroa, and inland basins such as Kokatahi. This region is within what is known as a 'seismic gap' on the Alpine Fault and in close proximity to the fault. Site investigations are needed in this area to clarify the potential for these regions to liquefy. With an Alpine fault event these towns, like Greymouth may have both access problems and loss of vital services.

- Geological and seismological studies are required to better define the source of the 1929 Murchison earthquake. This may be achieved by extending work undertaken by Dowrick (1994) on landslides.

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Personal Communications

Mrs. Batey, January 2003, Greymouth

Dr. J. Berrill, July 2003, University of Canterbury, Christchurch

Mr. K. Boon, December 2002, Greymouth

Mrs. Bradley, April 2003, Murchison

Dr. J. Collen, May 2003, University of Victoria, Wellington

Miss L. Langford, April 2003, Bainham

Mr J. Lowe, January 2003, Oparara

Mrs. M. Lowe, January 2003, Oparara

Mrs. D. McNabb, June 2003, September 2003, Karamea

Mrs. B. Peart, April 2003, Hokitika

Mr. D. Rhind, January 2003, Karamea

Mrs. M. Richardson, March 2003, Karamea

Mr. P. Sampson, May 2002, April 2003, Karamea

Mr. H. Simpson, January 2003, Arapito

Mr. G. Webster, April 2003, Greymouth

Mr. G. White, February 2003, Westport

Appendices

The appendices have been split into a number of sections as follows:

Appendix A contains the data obtained in 2003 from field work undertaken in Greymouth and Karamea. An overall area map is given first, followed by a site map, the hand auger results if applicable, inferred soil profiles and the results of the CPT testing.

The charts showing the results of the cone penetrometer testing also indicate the liquefiable layers, as defined by the different analysis methods. The format is illustrated by the results from the CPT number 2 at Steer Avenue Greymouth and given in Figure A-1 below. The first of the three charts indicates the measured tip resistance and friction ratio, as well as critical values for q_c obtained by the methods of Zhou (1980), Davis and Berrill (1982) and Shibata and Teparaksa (1988). The second chart shows either the ratio of critical tip resistance to the measured resistance, or the factor of safety against liquefaction defined as FS=CRR/CSR. Vertical lines have been drawn at both a value of 1.00 and 1.25. The third plot indicates for each of the prediction methods the layers that are deemed potentially liquefiable. In this plot wherever there is a coloured dot the layer has the potential to liquefy according to the particular prediction method. For clarity, numbers corresponding to each prediction method are provided along with colour coding in the key accompanying the results of the analysis presented in Figure A-1. The peak ground acceleration values used in the computation of these curves are given in Table 7-3 and epicentral distances are given in Table 6-1.

The following is a key for the soil profiles used both in Appendix A and B:

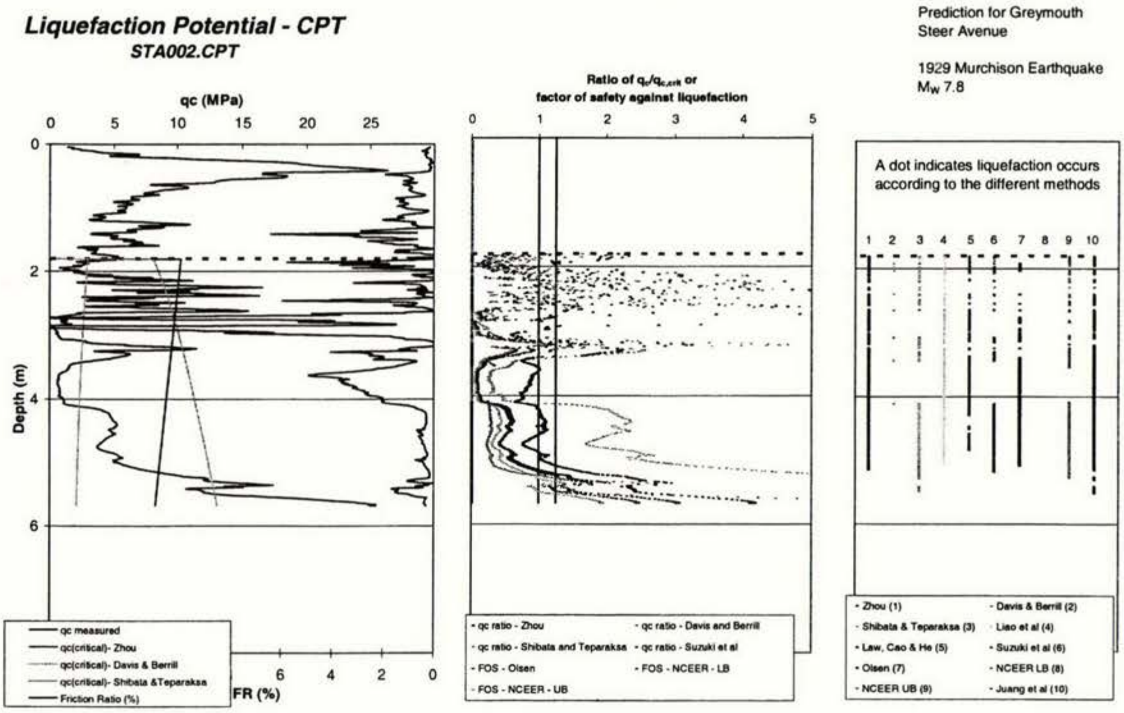


Figure A- 1. CPT test results and analysis for Steer Ave, Greymouth

Appendix A: Results from Greymouth and Karamea.

A1.1 GREYMOUTH

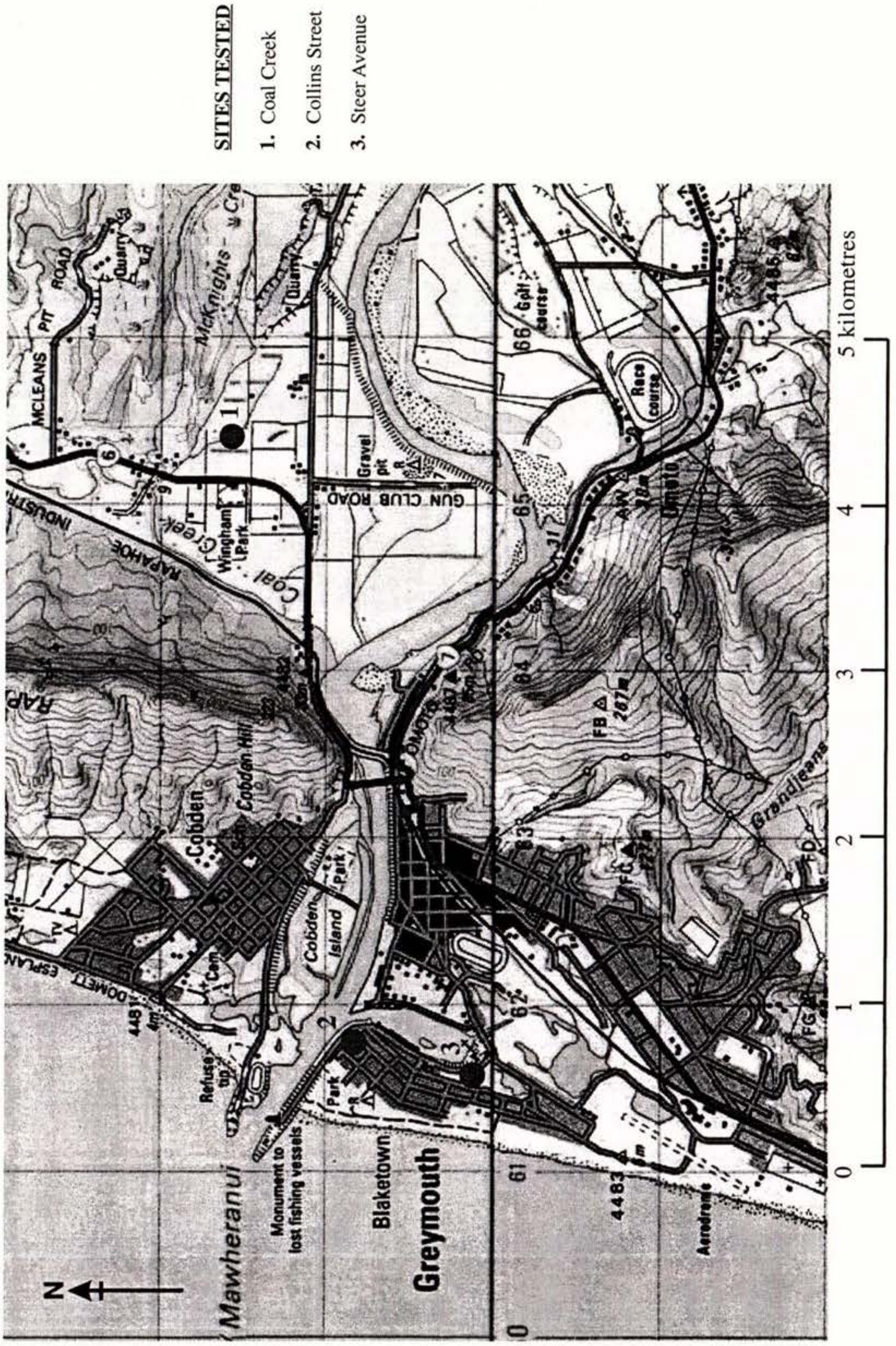


Figure A- 2. Location of field test sites in Greymouth

A1.1.1 Coal Creek

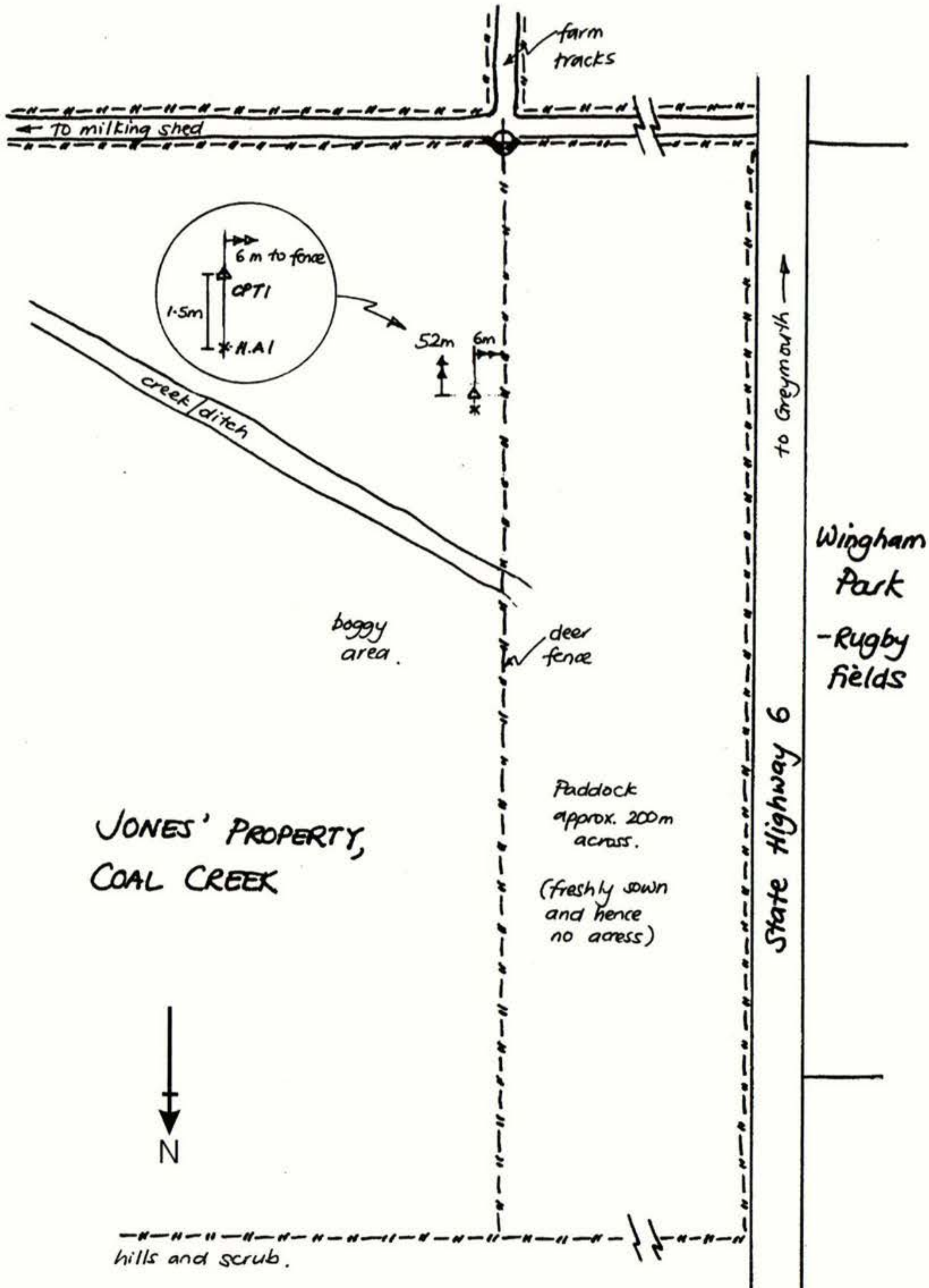


Figure A- 3. Site Diagram for Jones' Property at Coal Creek

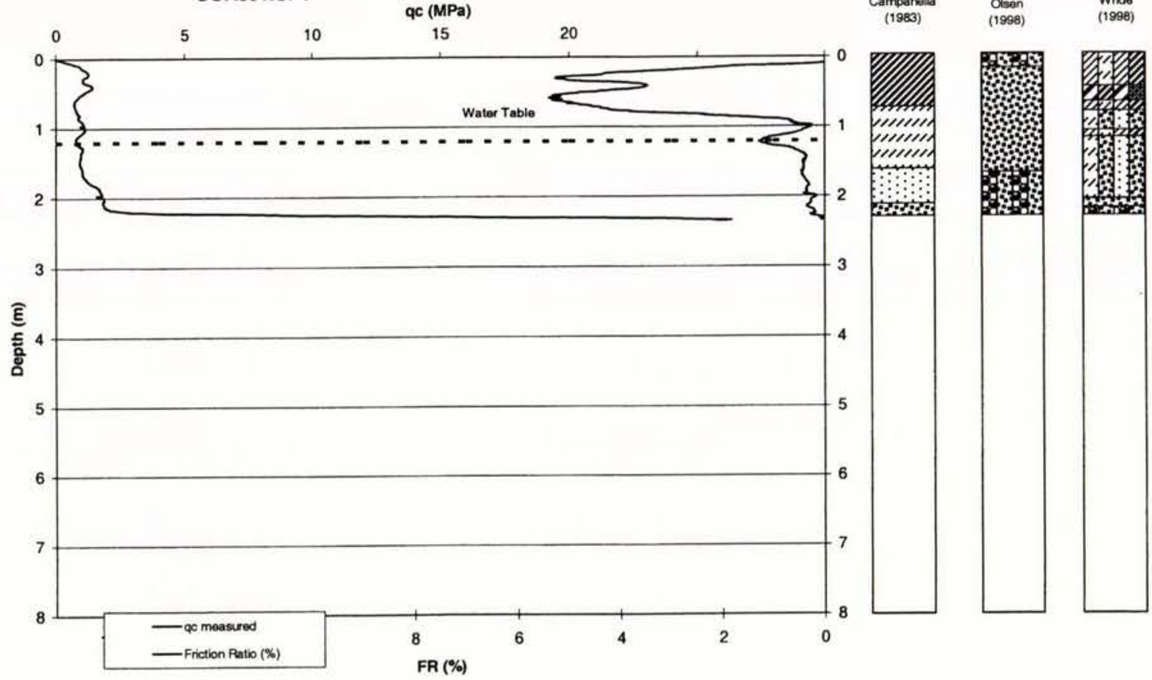
COA001.CPT

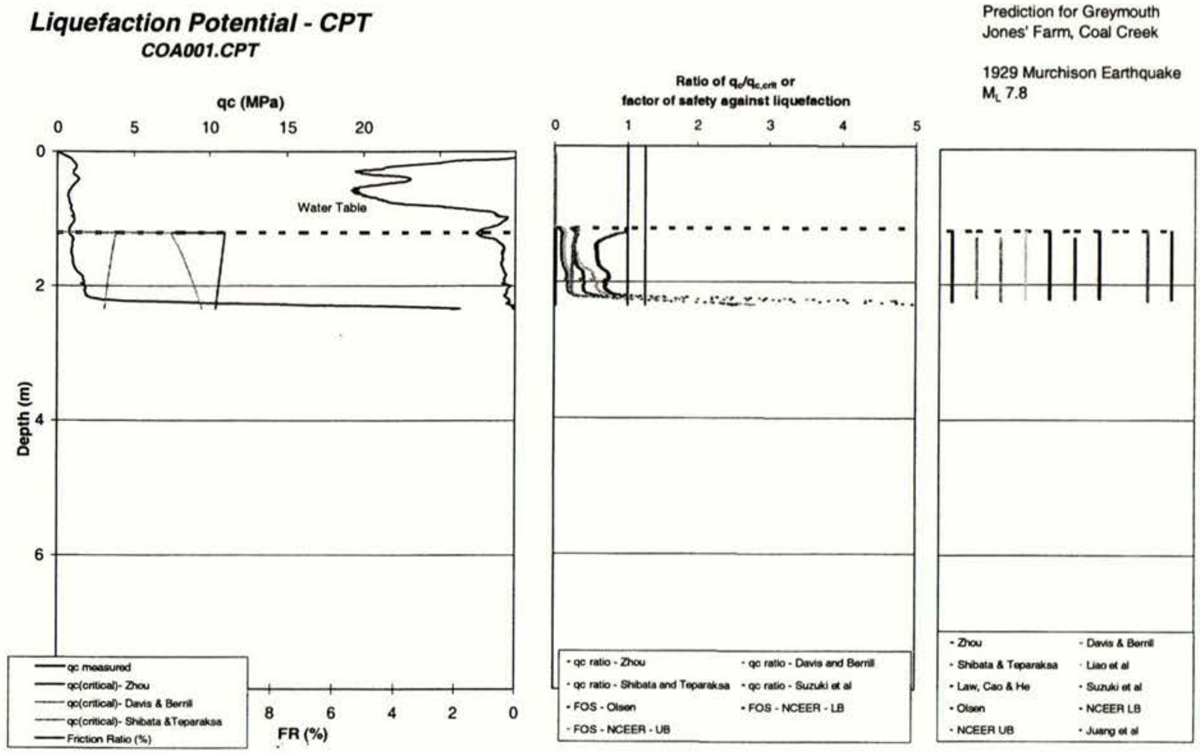
Test Boring Log

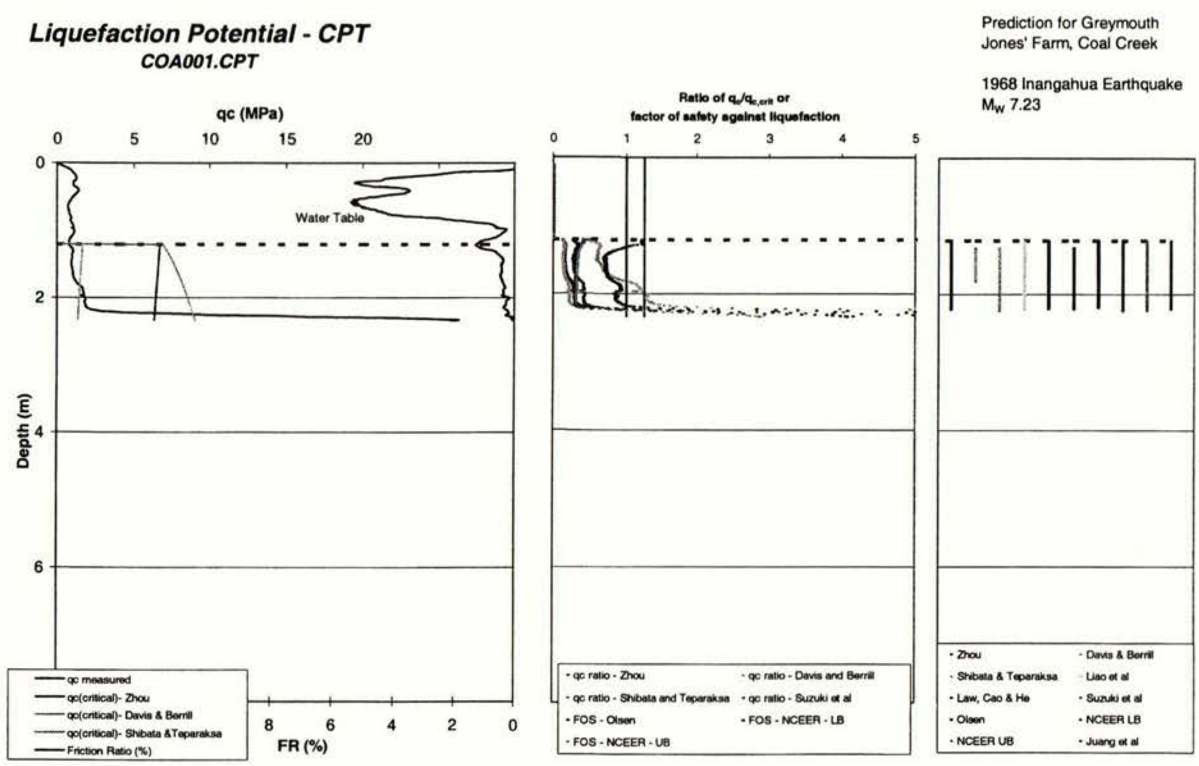
Job	Coal Creek- Jones' Property
By	Kirsti, Siale
Date	30/01/2003
Type	Hand Auger
Boring Number	1

DEPTH (m)				Comments
0.00				Silty TOPSOIL
0.10				
0.20				Soft brown-grey SAND
0.30				
0.40				
0.50				
0.60				
0.70				
0.80				
0.90				
1.00				
1.10				
1.20				
1.30				
1.40				
1.50				
1.60				
1.70				
1.80				
1.90				
2.00				
2.10				
2.20				
2.30				2.25m Encountered Gravel - BORE TERMINATED
2.40				
2.50				
2.60				
2.70				
2.80				
2.90				
3.00				
3.10				
3.20				
3.30				
3.40				
3.50				
3.60				
3.70				
3.80				
3.90				
4.00				

***Inferred Soil Profiles** - based on experimental data*
COA001.CPT




Liquefaction Potential - CPT
COA001.CPT
Prediction for Greymouth
Jones' Farm, Coal Creek
1929 Murchison Earthquake
M_L 7.8
qc (MPa)
0
5
10
15
20
Water Table
Depth (m)
0
2
4
6
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8
6
4
2
0
FR (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0
1
2
3
4
5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al


Liquefaction Potential - CPT
COA001.CPT
Prediction for Greymouth
Jones' Farm, Coal Creek
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0
5
10
15
20
Water Table
Depth (m)
0
2
4
6
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8
6
4
2
0
FR (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0
1
2
3
4
5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

A1.1.2 Collins Street

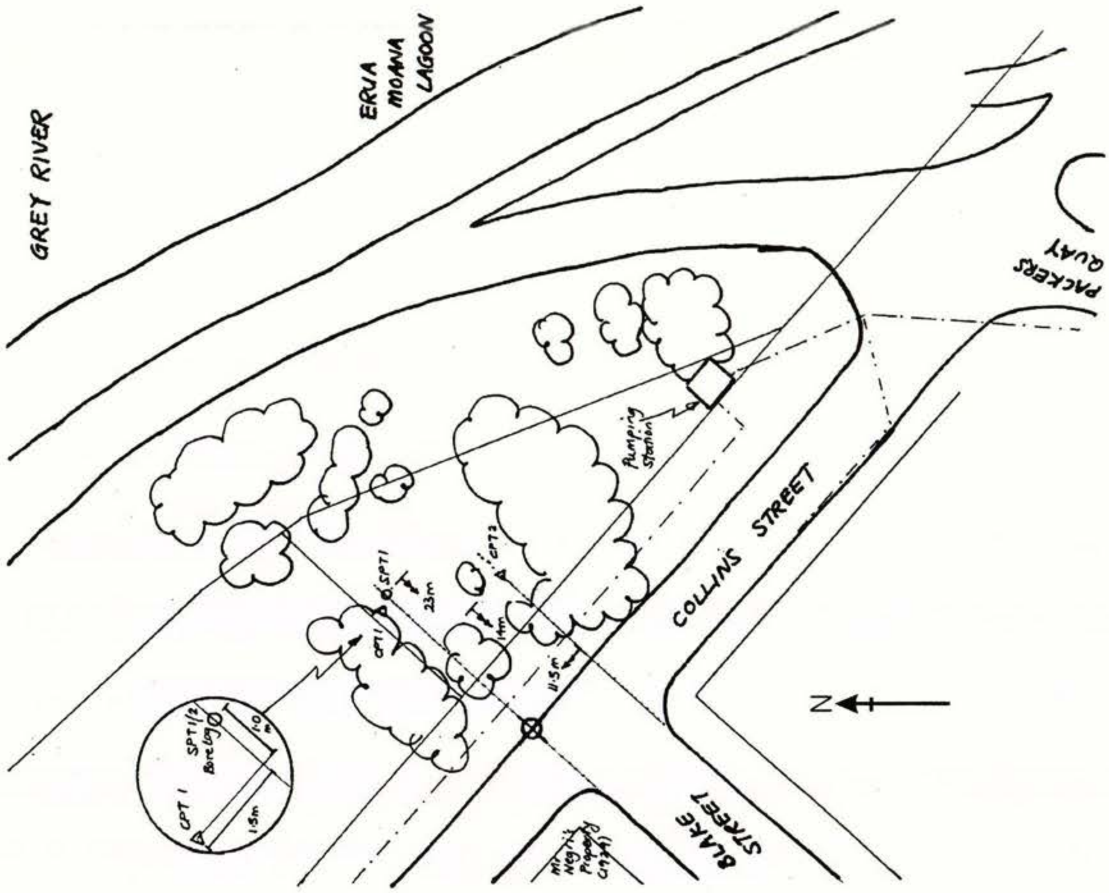
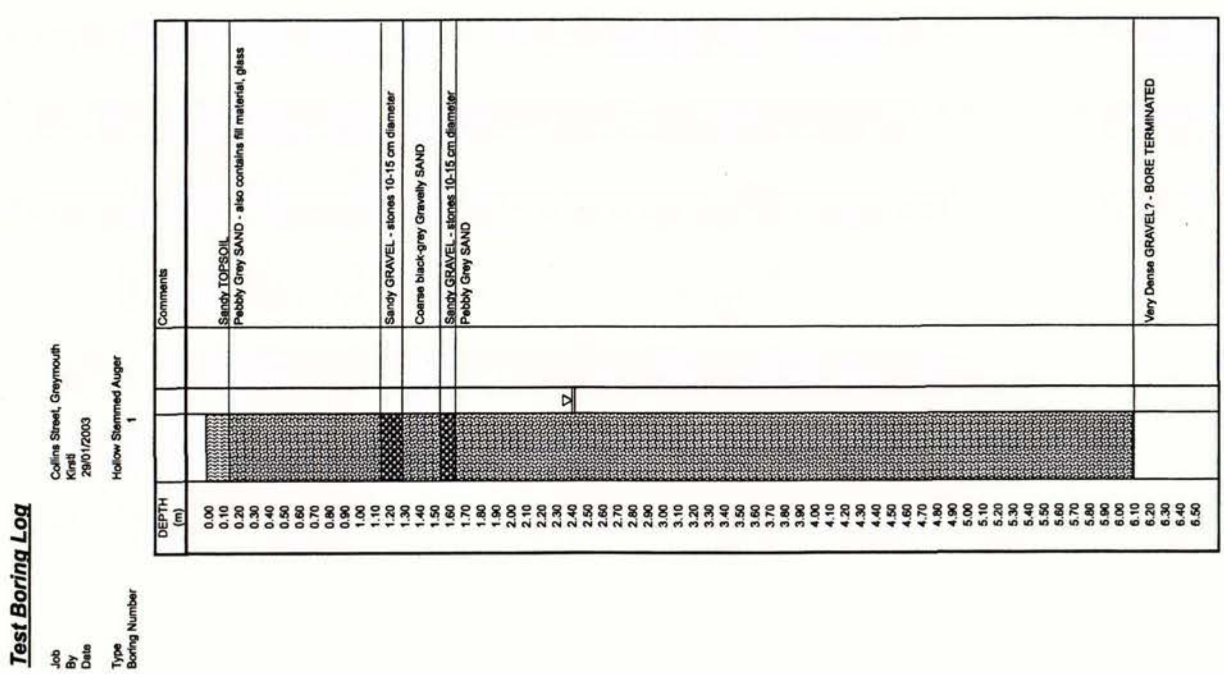
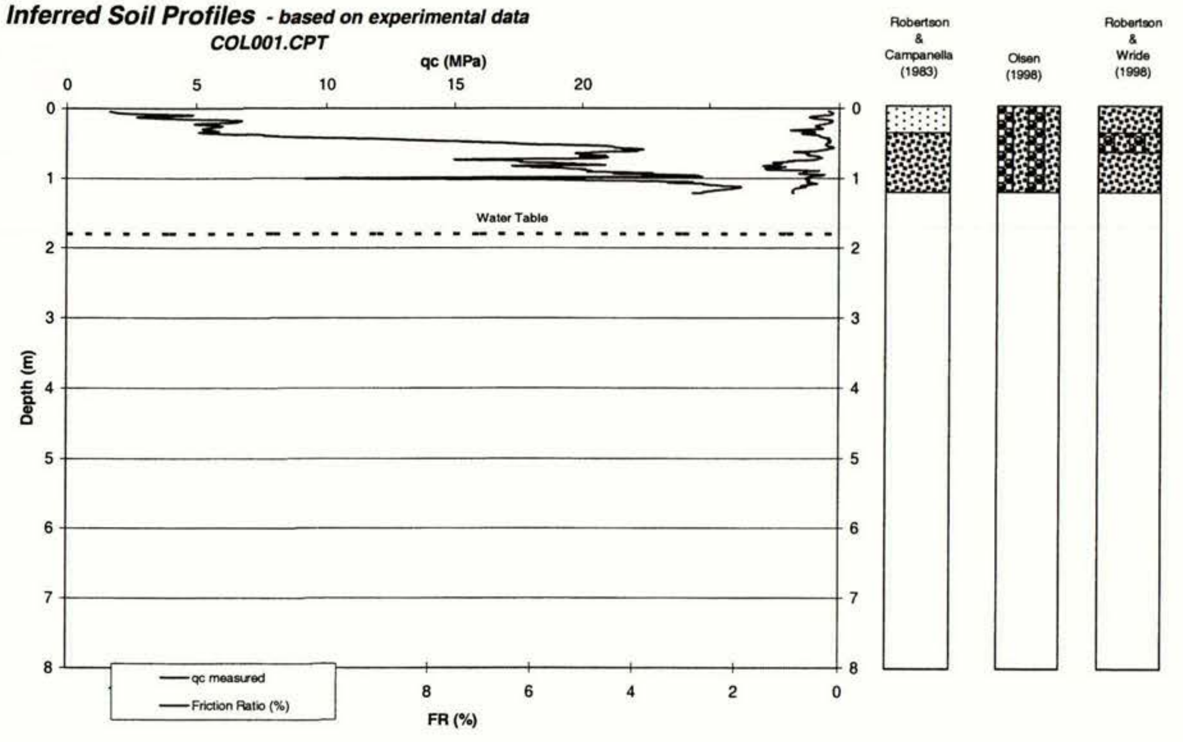
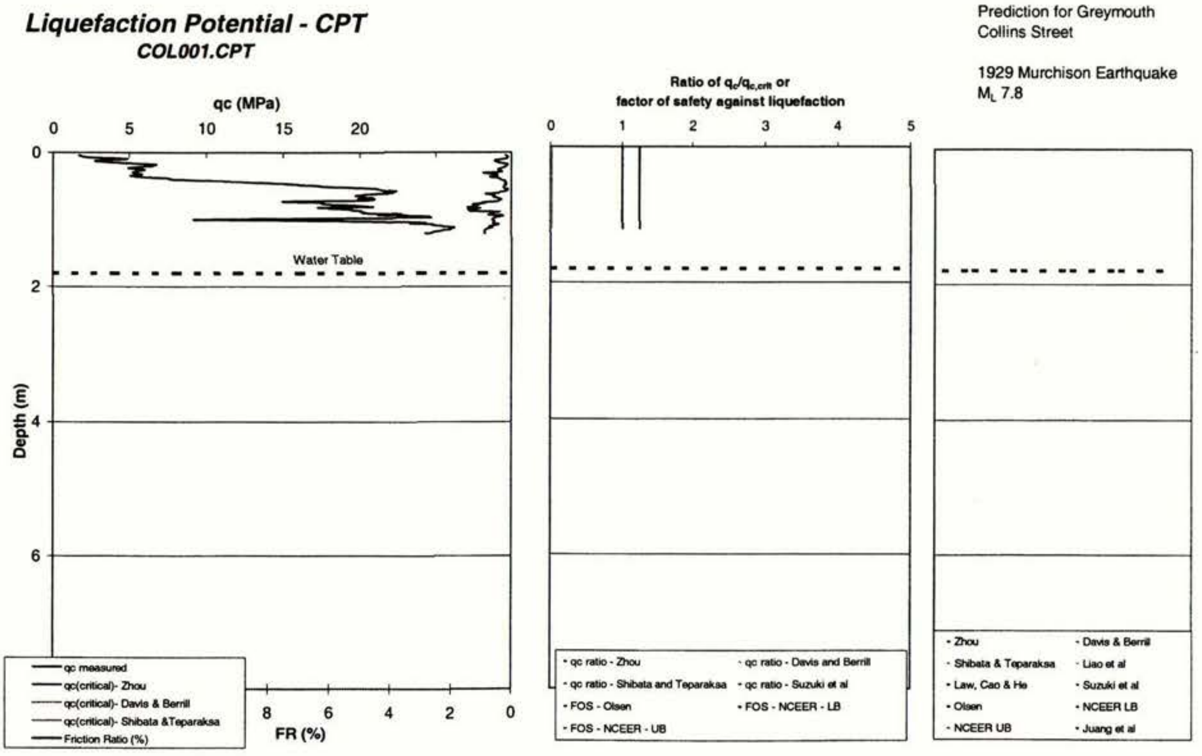


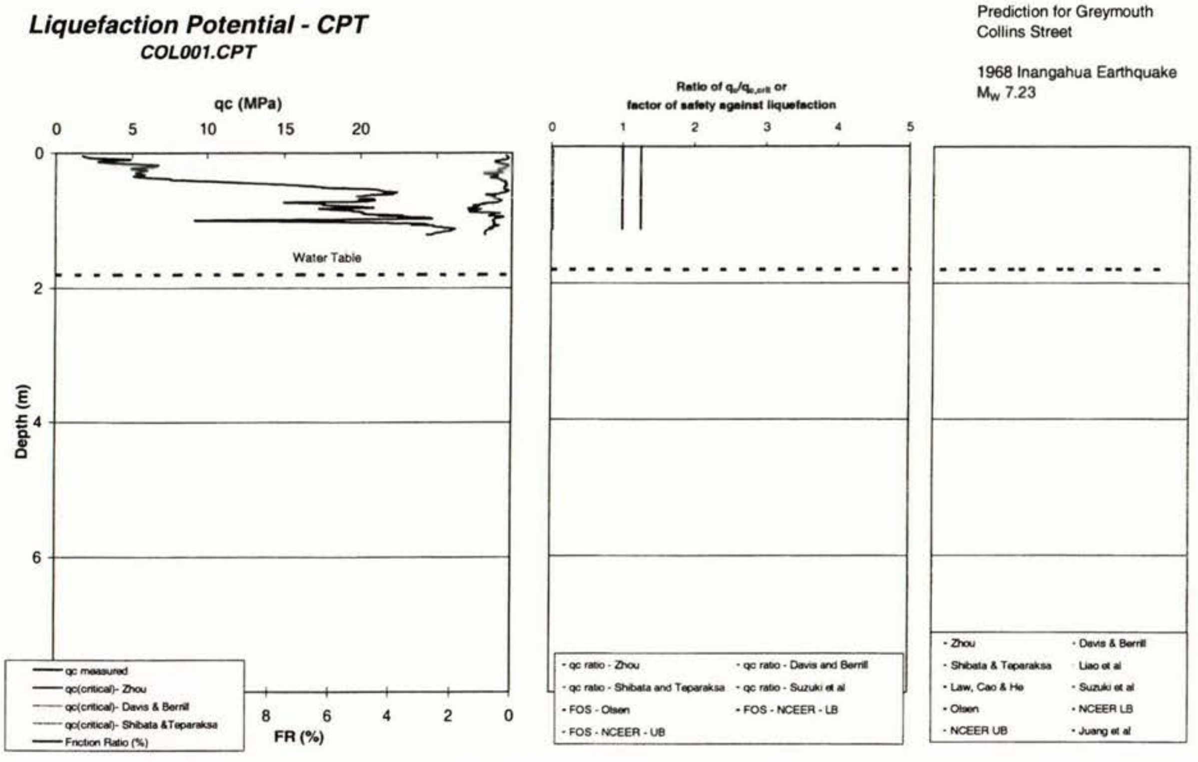
Figure A- 4. Site Diagram for field testing



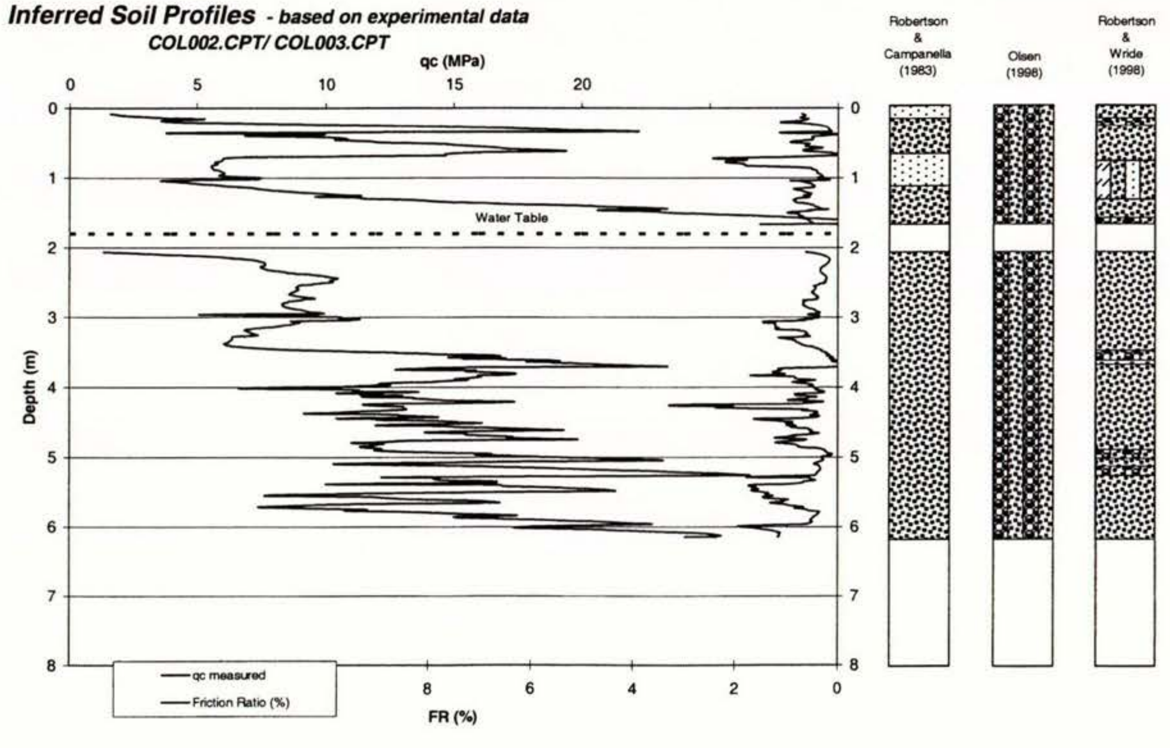
COL001.CPT

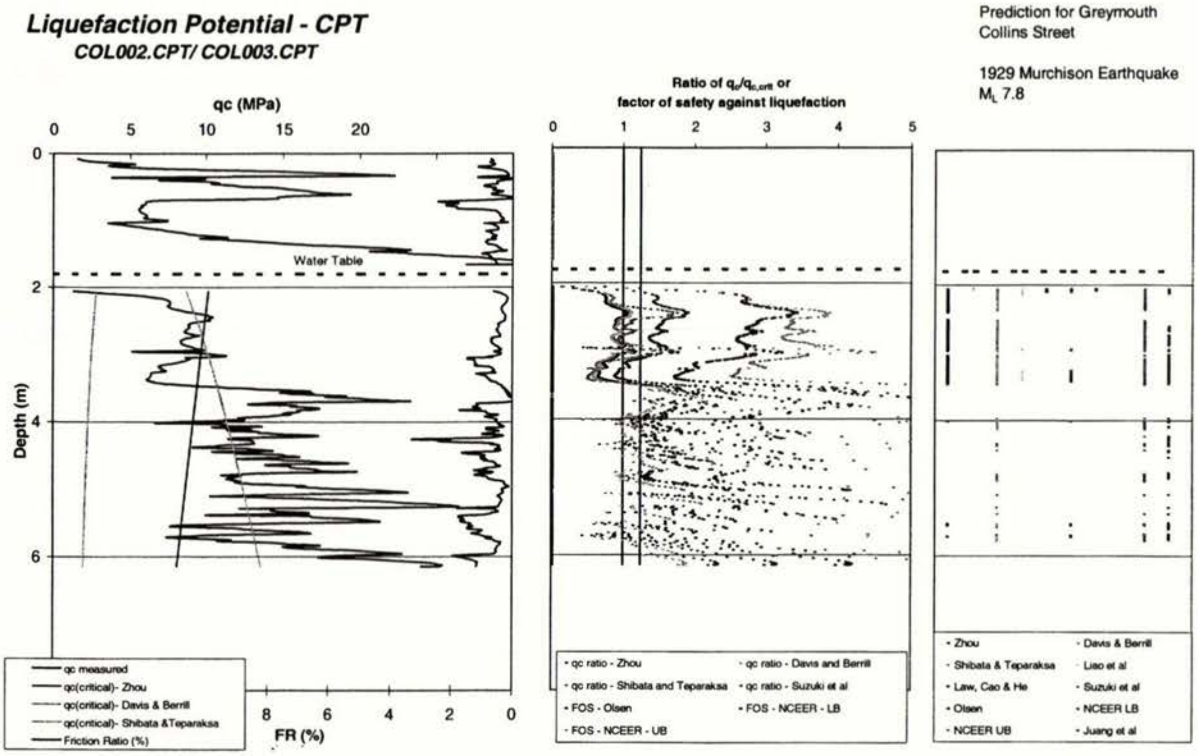

Inferred Soil Profiles - based on experimental data
COL001.CPT
qc (MPa)
0
5
10
15
20
Water Table
Depth (m)
0
1
2
3
4
5
6
7
8
qc measured
Friction Ratio (%)
8
6
4
2
0
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)

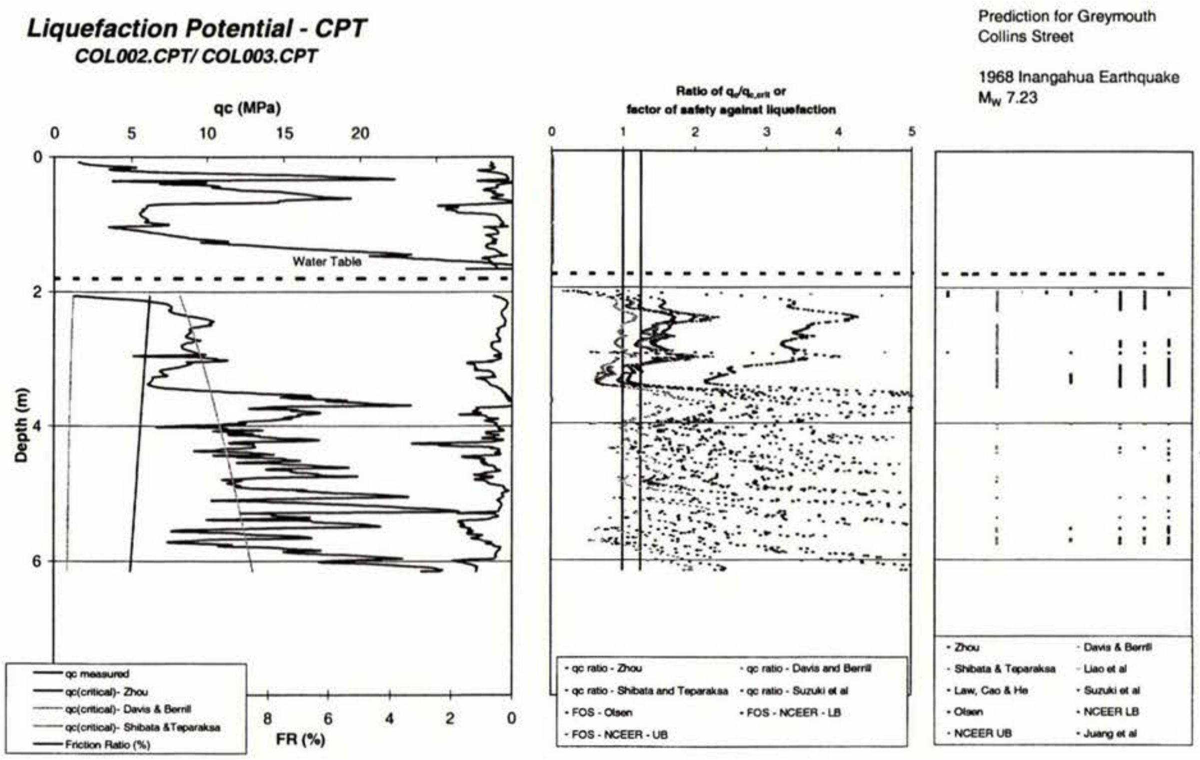

Liquefaction Potential - CPT
COL001.CPT
qc (MPa)
0
5
10
15
20
Water Table
Depth (m)
0
2
4
6
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8
6
4
2
0
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
0
1
2
3
4
5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Prediction for Greymouth
Collins Street
1929 Murchison Earthquake
M_L 7.8
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al



COL002.CPT / COL003.CPT




Liquefaction Potential - CPT
COL002.CPT/ COL003.CPT
Prediction for Greymouth
Collins Street
1929 Murchison Earthquake
M_L 7.8
qc (MPa)
0 5 10 15 20
Depth (m)
Water Table
FR (%)
8 6 4 2 0
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of qc/qc,crit or
factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al


Liquefaction Potential - CPT
COL002.CPT/ COL003.CPT
Prediction for Greymouth
Collins Street
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0 5 10 15 20
Depth (m)
Water Table
FR (%)
8 6 4 2 0
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of qc/qc,crit or
factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

A1.1.3 Steer Avenue

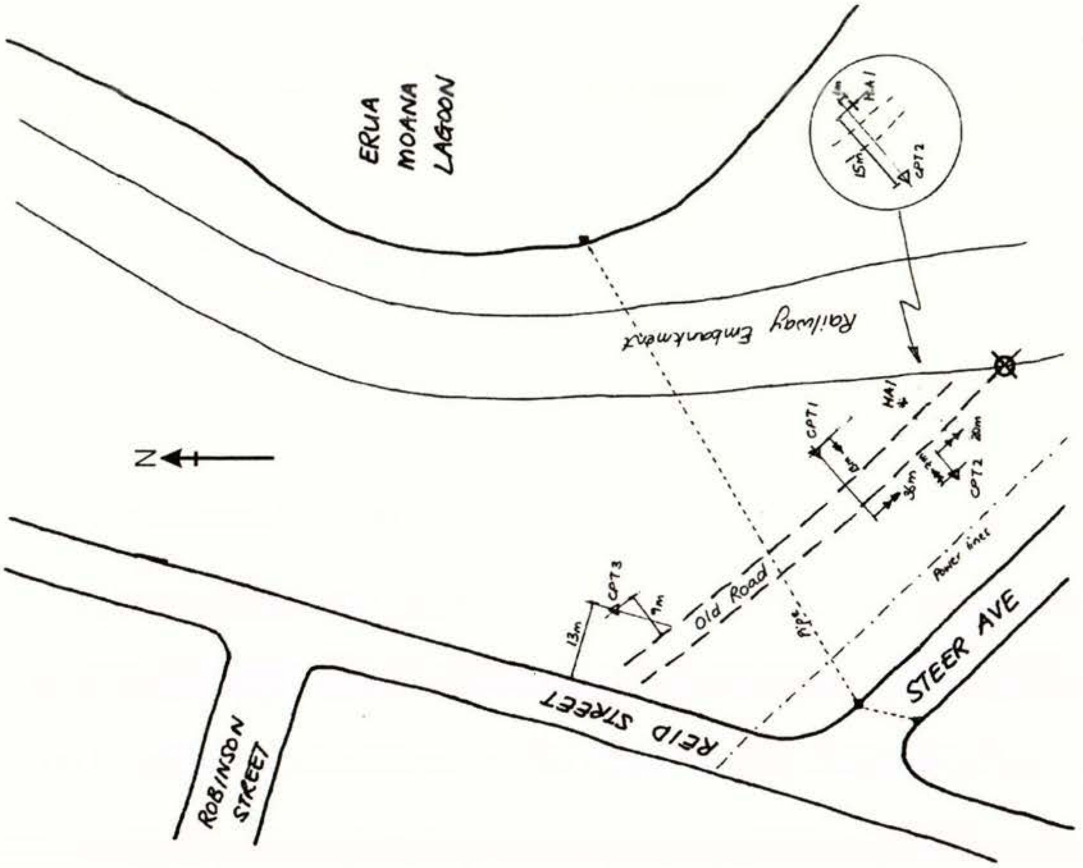
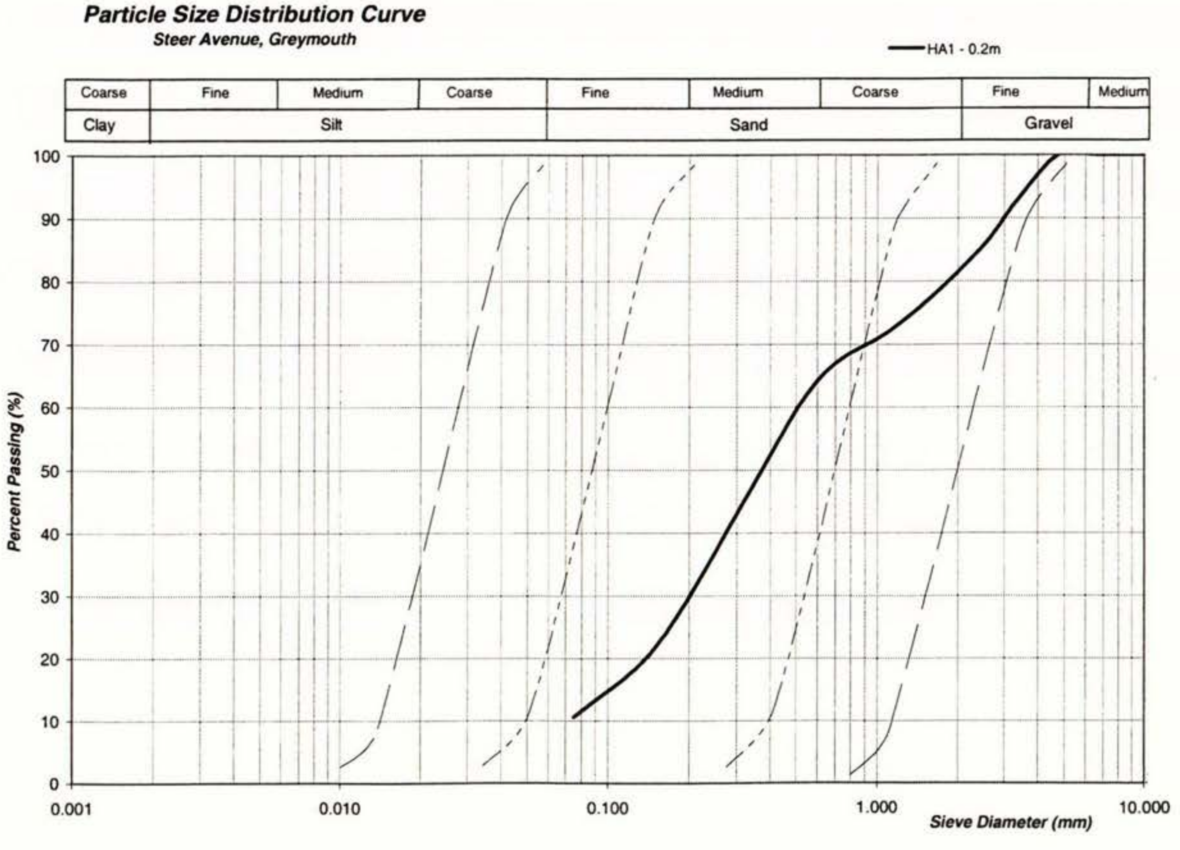


Figure A- 5. Site Diagram for Steer Avenue, Greymouth



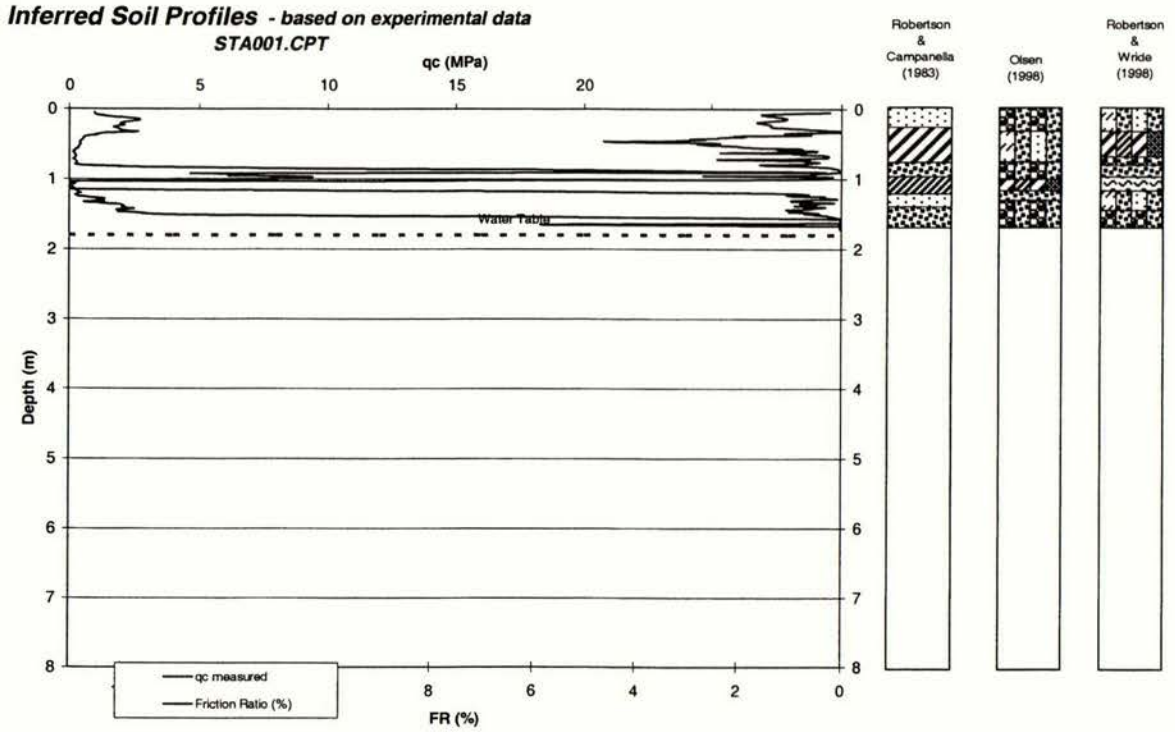
STA001.CPT

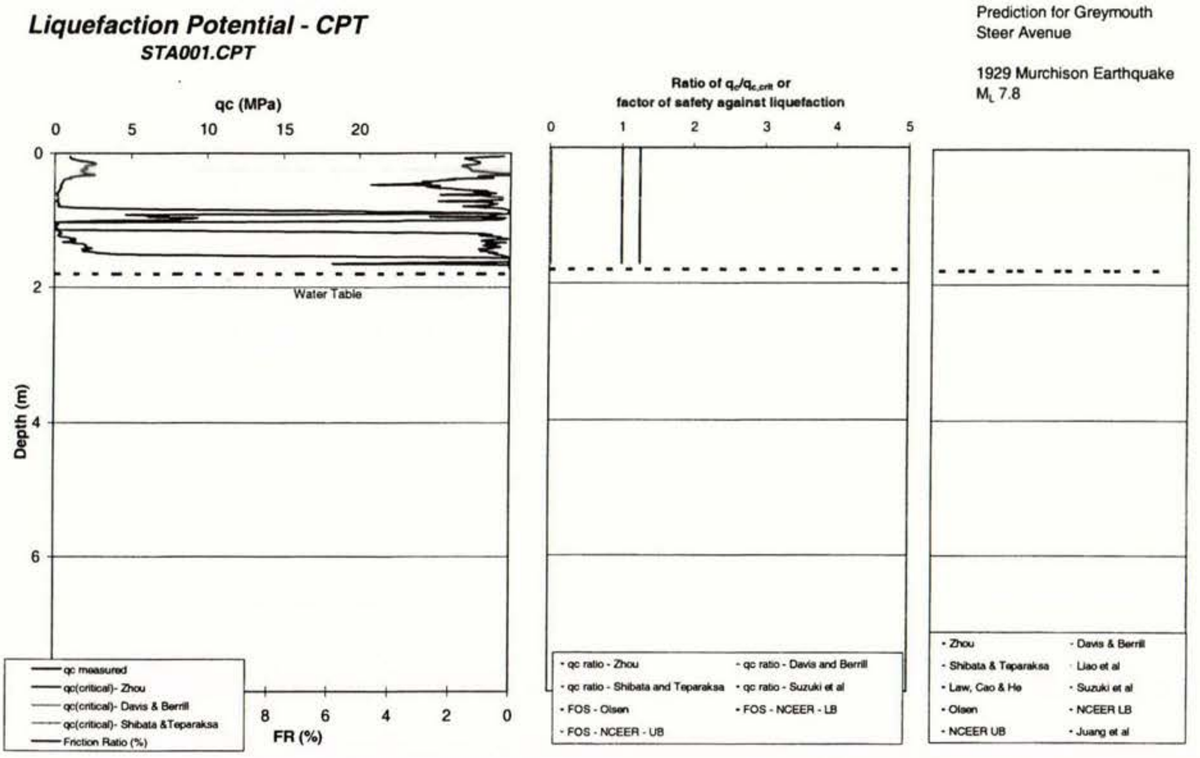
Test Boring Log

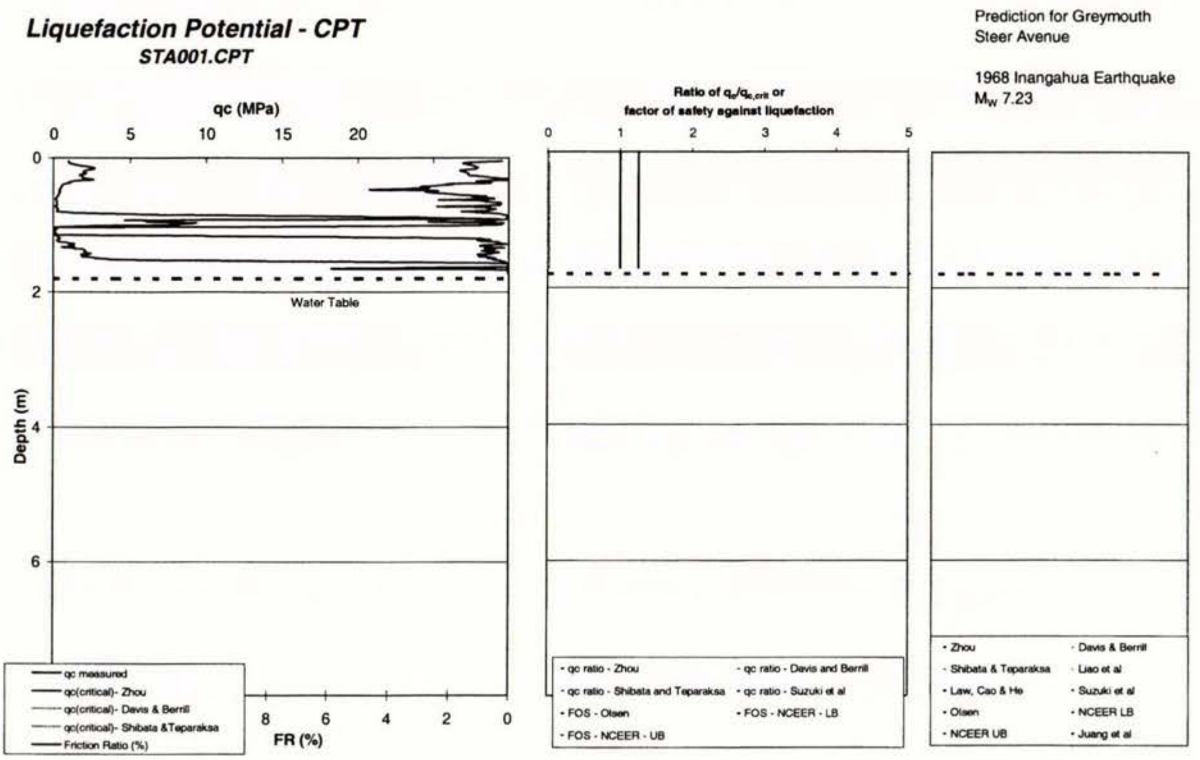
Job: Steer Avenue, Greymouth
By: Kirsti
Date: 8/01/2003

Type: Hand Auger
Boring Number: 1

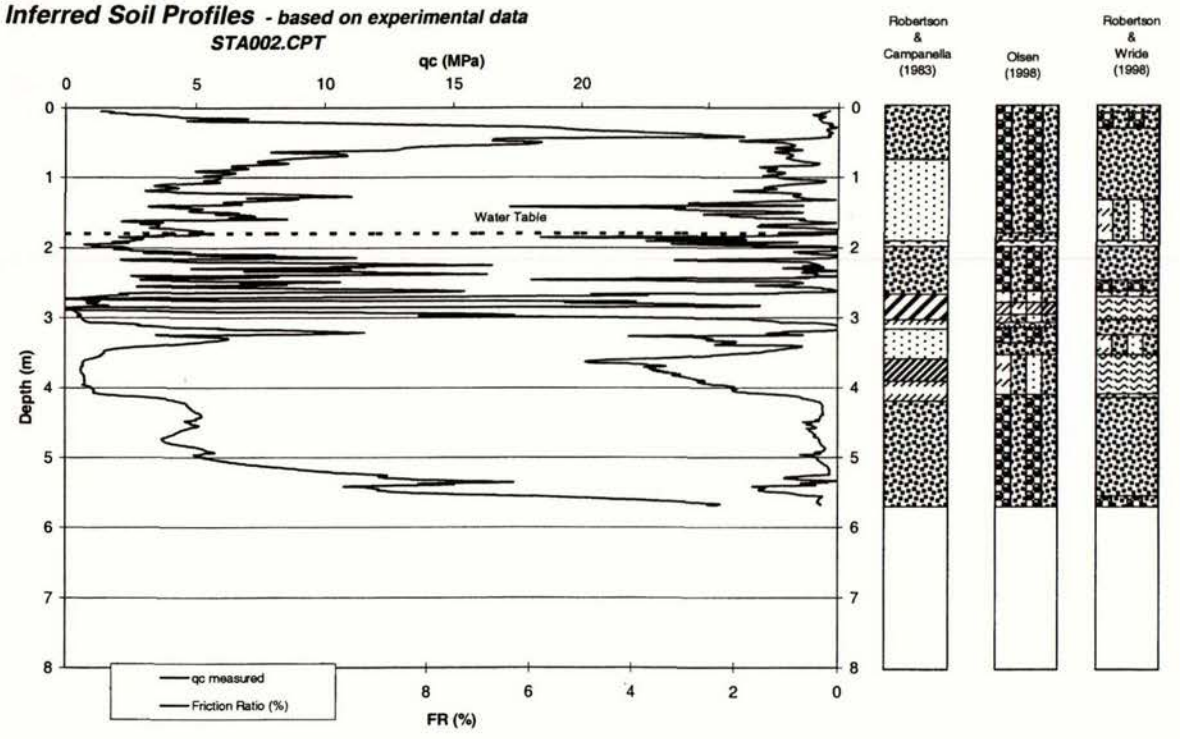
DEPTH (m)	Comments
0.00	
0.10	Sandy TOPSOIL
0.20	Brown SAND containing Coal fragments
0.30	
0.40	Black-brown Pebbly SAND containing coal dust
0.50	Black-Brown Pebbly SAND with iron staining
0.60	
0.70	
0.80	Moist dense SAND and glass fragments - also iron staining
0.90	
1.00	
1.10	Grey SAND -coal rich
1.20	
1.30	Grey Pebbly SAND
1.40	
1.50	
1.60	
1.70	Pebbly Grey SAND and glass fragments
1.80	
1.90	
2.00	2.0m hole caving in- BORE TERMINATED
2.10	
2.20	
2.30	
2.40	
2.50	
2.60	
2.70	
2.80	
2.90	
3.00	
3.10	
3.20	
3.30	
3.40	
3.50	
3.60	
3.70	
3.80	
3.90	
4.00	

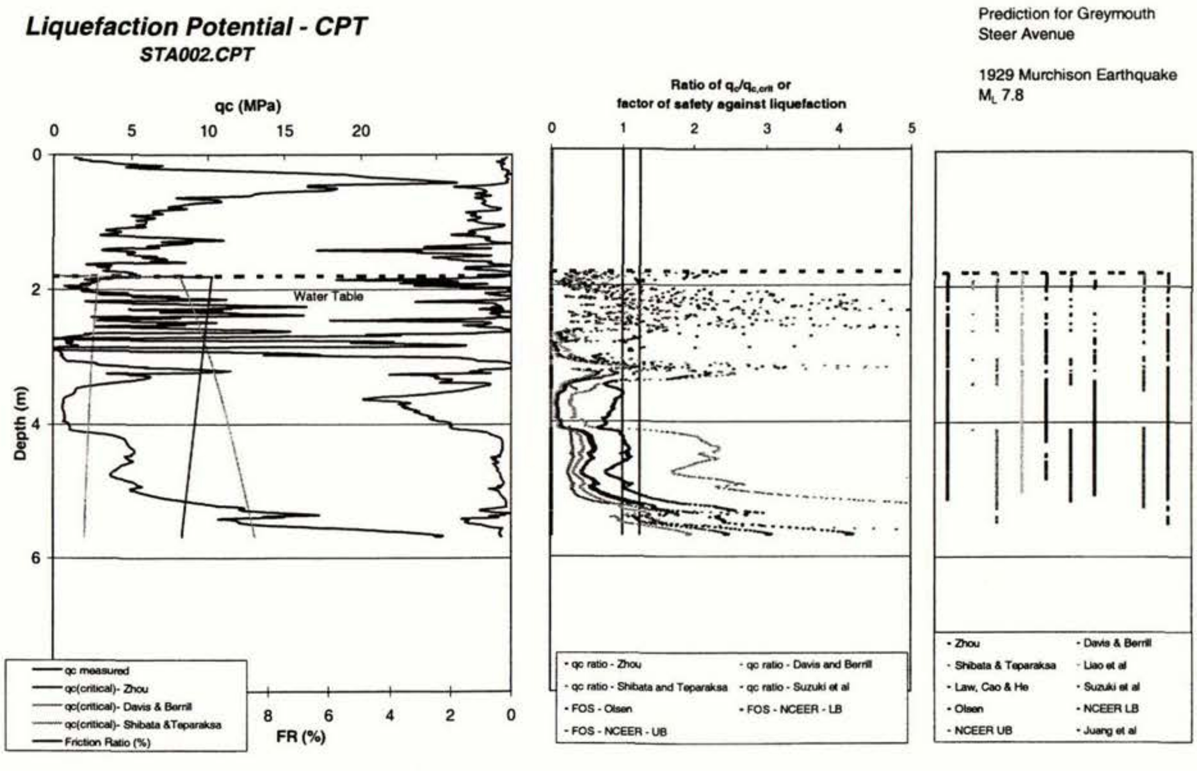



Liquefaction Potential - CPT
STA001.CPT
Prediction for Greymouth
Steer Avenue
1929 Murchison Earthquake
M_L 7.8
qc (MPa)
Ratio of qc/qc,crit or
factor of safety against liquefaction
Water Table
Depth (m)
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al


Liquefaction Potential - CPT
STA001.CPT
Prediction for Greymouth
Steer Avenue
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
Ratio of qc/qc,crit or
factor of safety against liquefaction
Water Table
Depth (m)
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

STA002.CPT

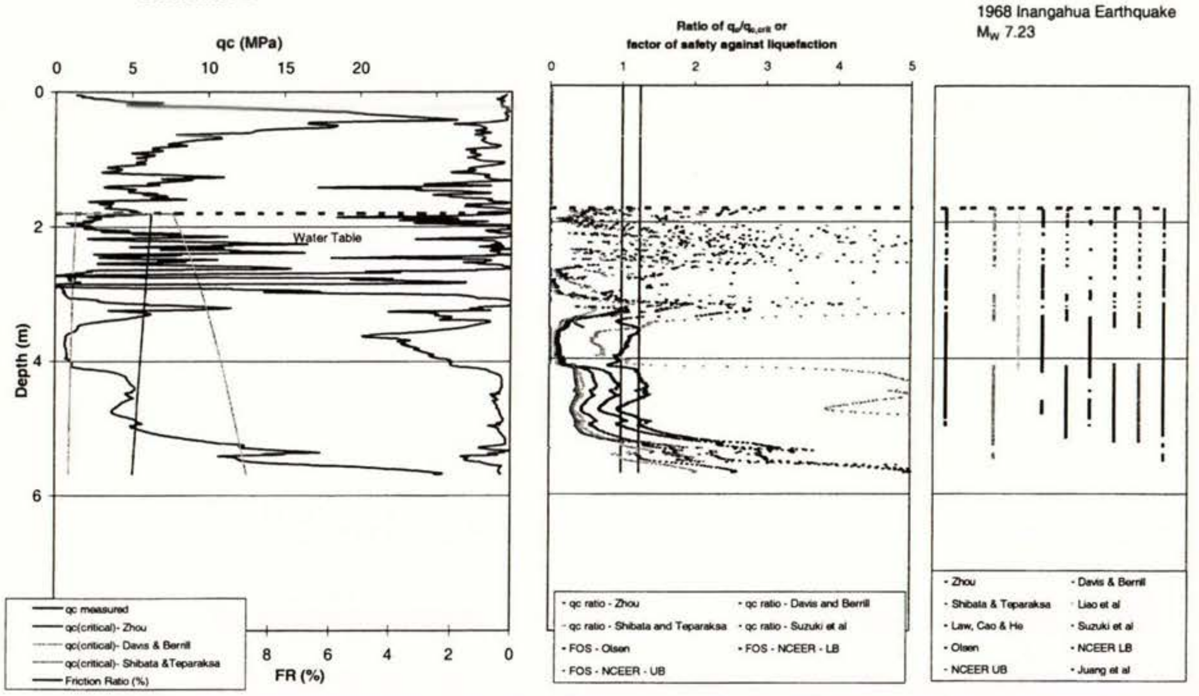

Inferred Soil Profiles - based on experimental data
STA002.CPT
qc (MPa)
0 5 10 15 20
Depth (m)
Water Table
qc measured
Friction Ratio (%)
FR (%)
8 6 4 2 0
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)


Liquefaction Potential - CPT
STA002.CPT
qc (MPa)
0 5 10 15 20
Depth (m)
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
8 6 4 2 0
Ratio of qc/qc,crit or factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Prediction for Greymouth
Steer Avenue
1929 Murchison Earthquake
M_L 7.8
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

Liquefaction Potential - CPT
STA002.CPT

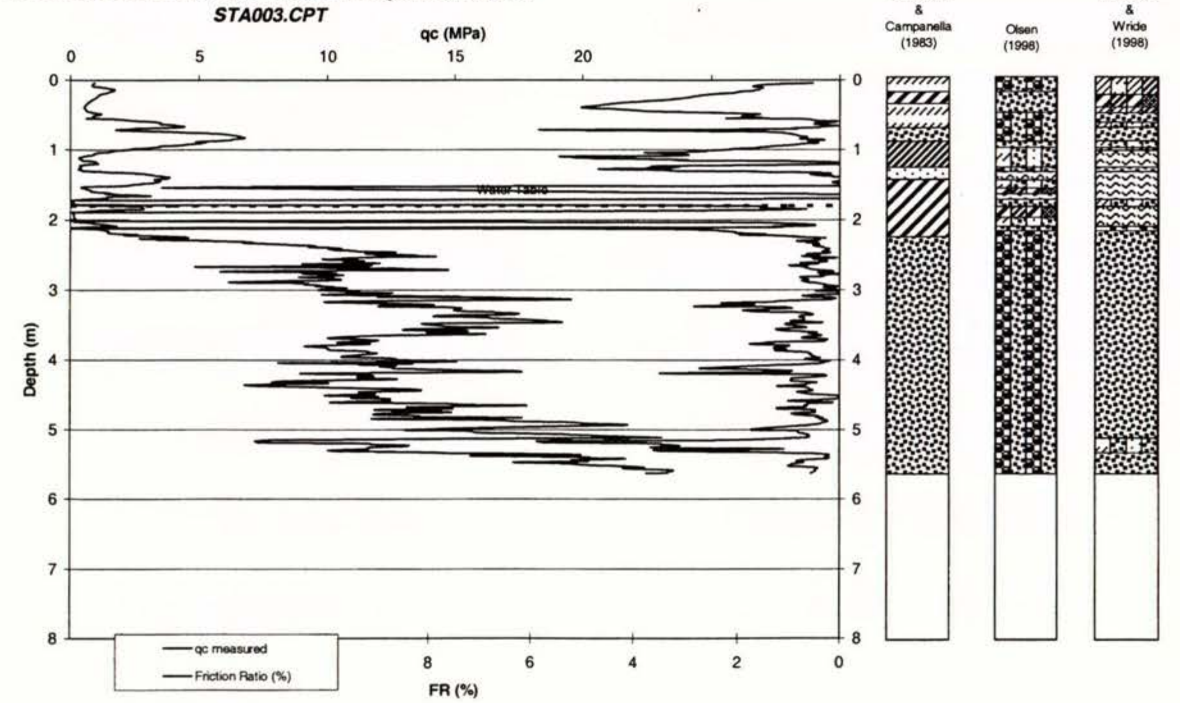
Prediction for Greymouth
Steer Avenue

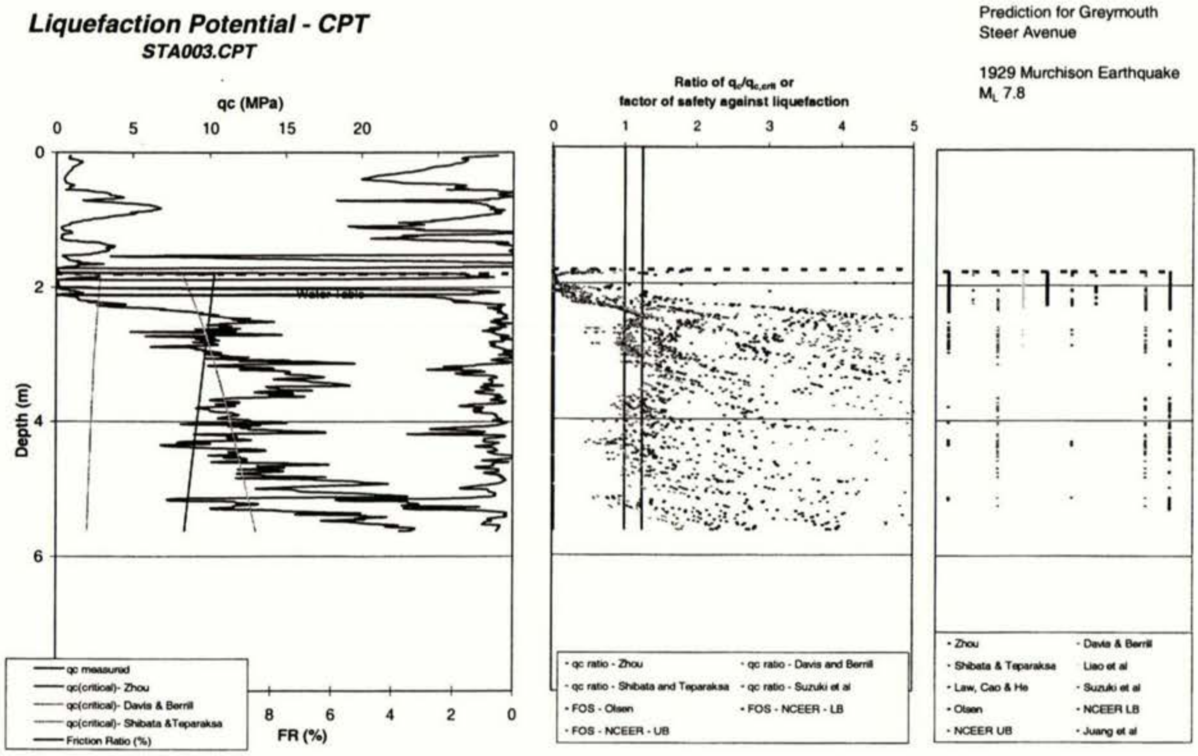
1968 Inangahua Earthquake
M_W 7.23

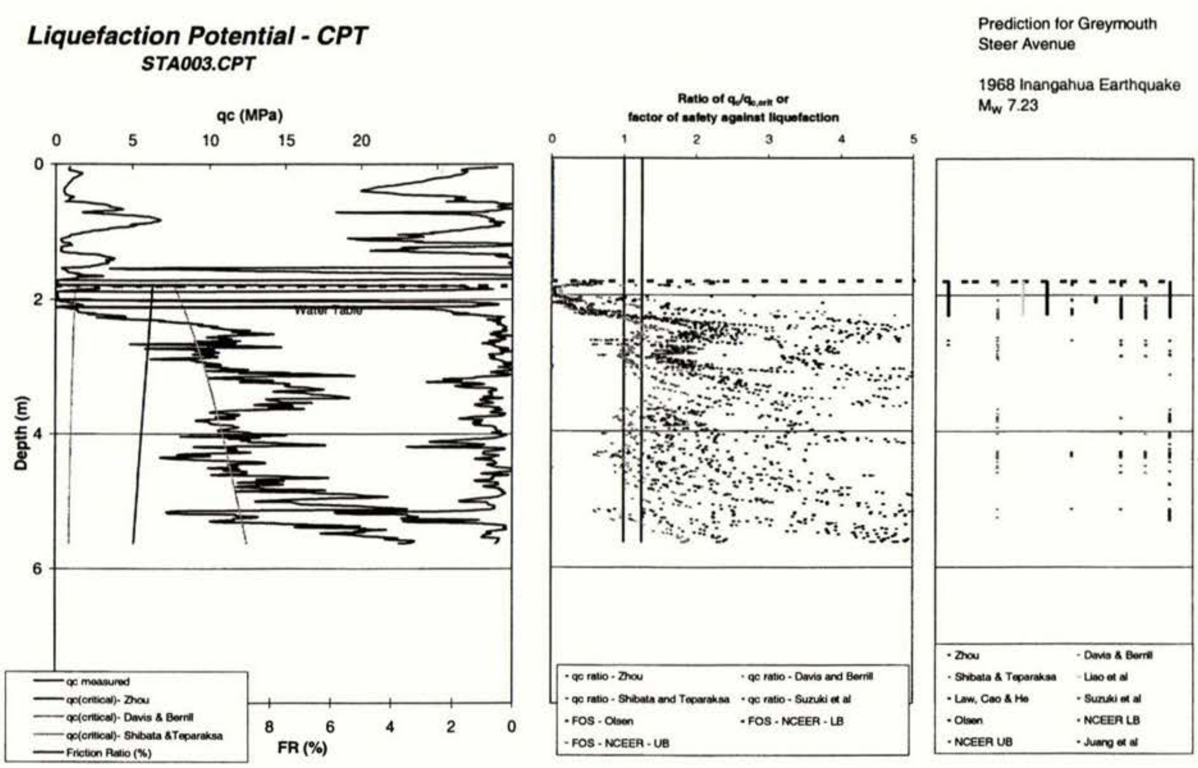


STA003.CPT

Inferred Soil Profiles - *based on experimental data*
STA003.CPT




Liquefaction Potential - CPT
STA003.CPT
qc (MPa)
0 5 10 15 20
Depth (m)
0 2 4 6
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Prediction for Greymouth
Steer Avenue
1929 Murchison Earthquake
M_L 7.8
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al


Liquefaction Potential - CPT
STA003.CPT
qc (MPa)
0 5 10 15 20
Depth (m)
0 2 4 6
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Prediction for Greymouth
Steer Avenue
1968 Inangahua Earthquake
M_W 7.23
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

A1.2 **Westport North School**

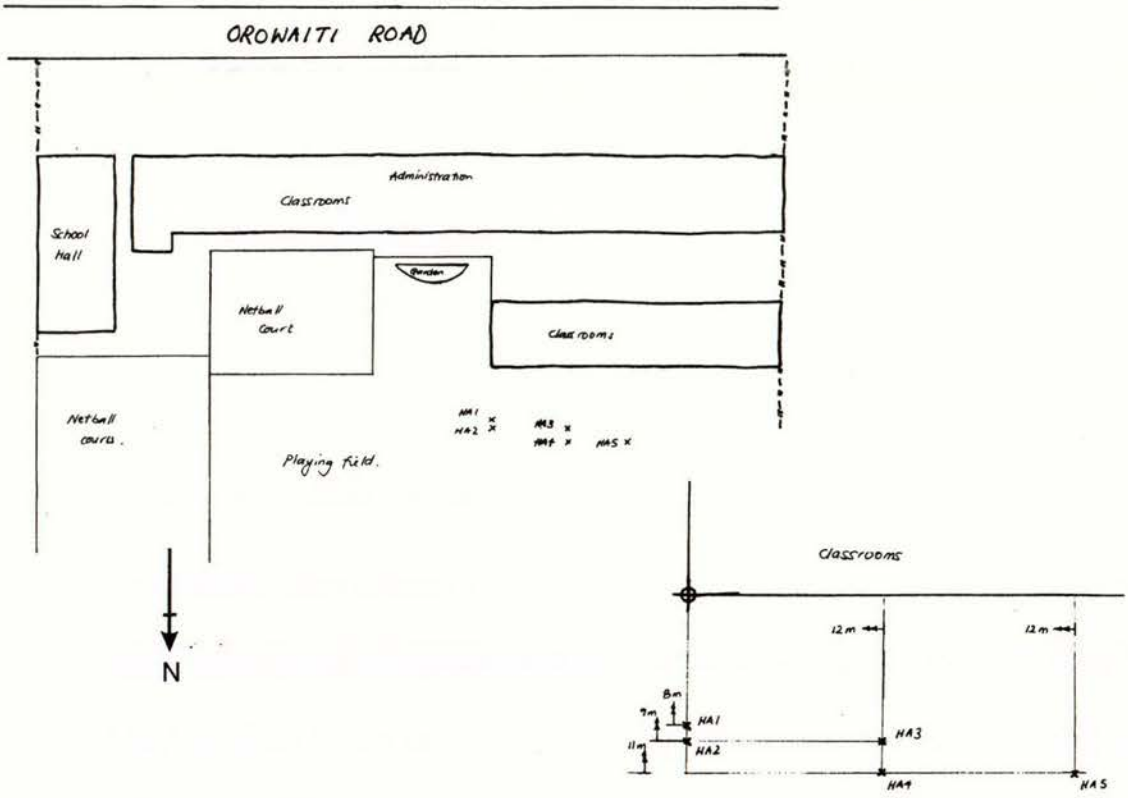
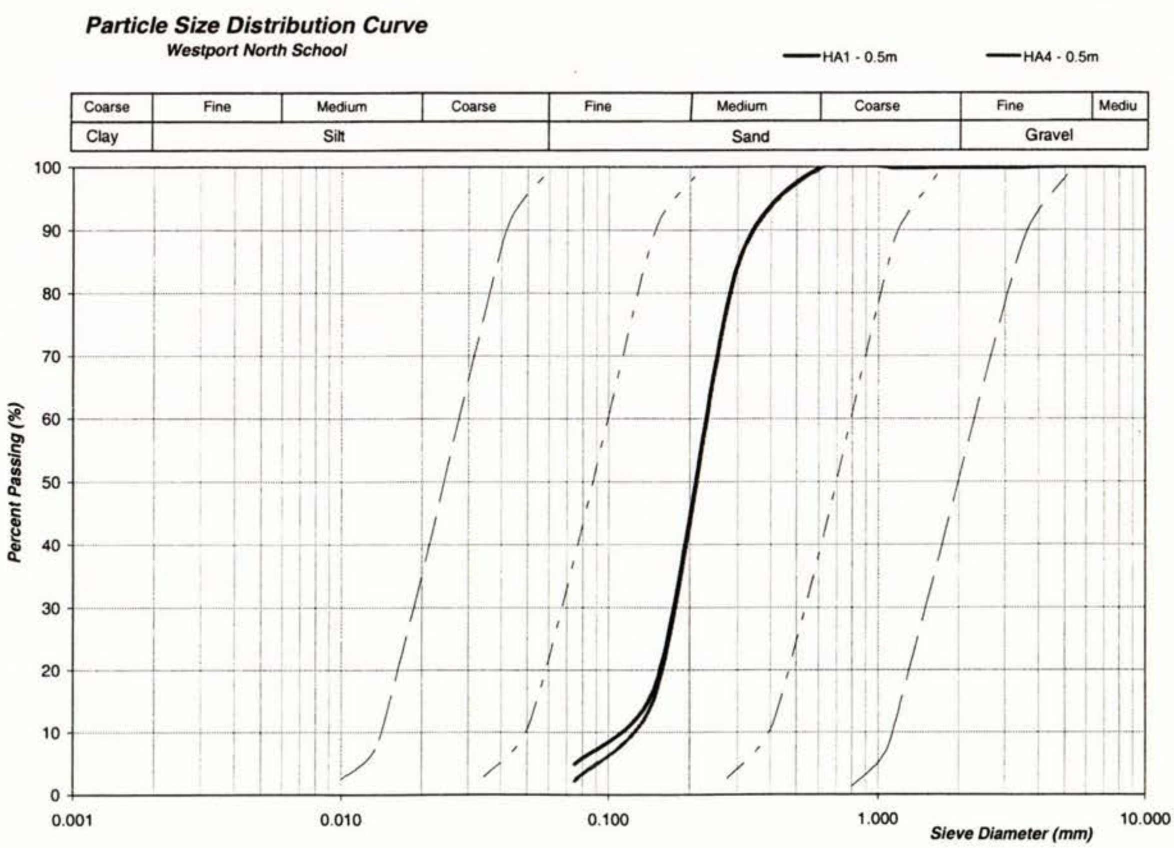


Figure A- 6. Site diagram indicating location of hand auger bores at Westport North School, Westport



Test Boring Log

Job: Westport North School
By: Kirsti
Date: 29/04/2003

Type: Hand Auger
Boring Number: 2

DEPTH (m)	Comments
0.00	
0.10	Brown Sandy TOPSOIL
0.20	
0.30	Medium light brown SAND, little damp
0.40	
0.50	
0.60	Very fine grey brown SAND
0.70	
0.80	0.7m Encountered Gravel - BORE TERMINATED
0.90	
1.00	
1.10	
1.20	
1.30	
1.40	
1.50	
1.60	
1.70	
1.80	
1.90	
2.00	
2.10	
2.20	
2.30	
2.40	
2.50	
2.60	
2.70	
2.80	
2.90	
3.00	
3.10	
3.20	
3.30	
3.40	
3.50	
3.60	
3.70	
3.80	
3.90	
4.00	

Test Boring Log

Job: Westport North School
By: Kirsti
Date: 29/04/2003

Type: Hand Auger
Boring Number: 1

DEPTH (m)	Comments
0.00	
0.10	Dark Brown Sandy TOPSOIL
0.20	
0.30	Medium to coarse grey-brown SAND, contains mica
0.40	
0.50	
0.60	
0.70	0.6m Encountered Gravel - BORE TERMINATED
0.80	
0.90	
1.00	
1.10	
1.20	
1.30	
1.40	
1.50	
1.60	
1.70	
1.80	
1.90	
2.00	
2.10	
2.20	
2.30	
2.40	
2.50	
2.60	
2.70	
2.80	
2.90	
3.00	
3.10	
3.20	
3.30	
3.40	
3.50	
3.60	
3.70	
3.80	
3.90	
4.00	

Test Boring Log

Job Westport North School
By Kirsti
Date 29/04/2003

Type Hand Auger
Boring Number 4

DEPTH (m)				Comments
0.00				
0.10				Fine brown Sandy TOPSOIL
0.20				
0.30				Fine to medium grey- brown SAND
0.40				
0.50				Medium to coarse grey-brown SAND - contains mica
0.60				
0.70				Medium grey-brown Pebbly SAND
0.80				
0.90				0.8m Encountered Gravel - BORE TERMINATED
1.00				
1.10				
1.20				
1.30				
1.40				
1.50				
1.60				
1.70				
1.80				
1.90				
2.00				
2.10				
2.20				
2.30				
2.40				
2.50				
2.60				
2.70				
2.80				
2.90				
3.00				
3.10				
3.20				
3.30				
3.40				
3.50				
3.60				
3.70				
3.80				
3.90				
4.00				

Test Boring Log

Job Westport North School
By Kirsti
Date 29/04/2003

Type Hand Auger
Boring Number 3

DEPTH (m)				Comments
0.00				
0.10				Brown Sandy TOPSOIL
0.20				Fine Pebbly SAND containing coal fragments
0.30				
0.40				
0.50				Medium to coarse grey-brown SAND
0.60				~~hole caving in~~
0.70				0.6m Encountered Gravel - BORE TERMINATED
0.80				
0.90				
1.00				
1.10				
1.20				
1.30				
1.40				
1.50				
1.60				
1.70				
1.80				
1.90				
2.00				
2.10				
2.20				
2.30				
2.40				
2.50				
2.60				
2.70				
2.80				
2.90				
3.00				
3.10				
3.20				
3.30				
3.40				
3.50				
3.60				
3.70				
3.80				
3.90				
4.00				

Test Boring Log

Job	Westport North School
By	Kirsti
Date	29/04/2003
Type	Hand Auger
Boring Number	5

DEPTH (m)				Comments
0.00				
0.10				Fine brown Sandy TOPSOIL
0.20				
0.30				Fine grey SAND
0.40				
0.50				Fine to medium grey-brown SAND
0.60				
0.70				
0.80				
0.90				0.85m Encountered Gravel - BORE TERMINATED
1.00				
1.10				
1.20				
1.30				
1.40				
1.50				
1.60				
1.70				
1.80				
1.90				
2.00				
2.10				
2.20				
2.30				
2.40				
2.50				
2.60				
2.70				
2.80				
2.90				
3.00				
3.10				
3.20				
3.30				
3.40				
3.50				
3.60				
3.70				
3.80				
3.90				
4.00				

A1.3 ***KARAMEA***

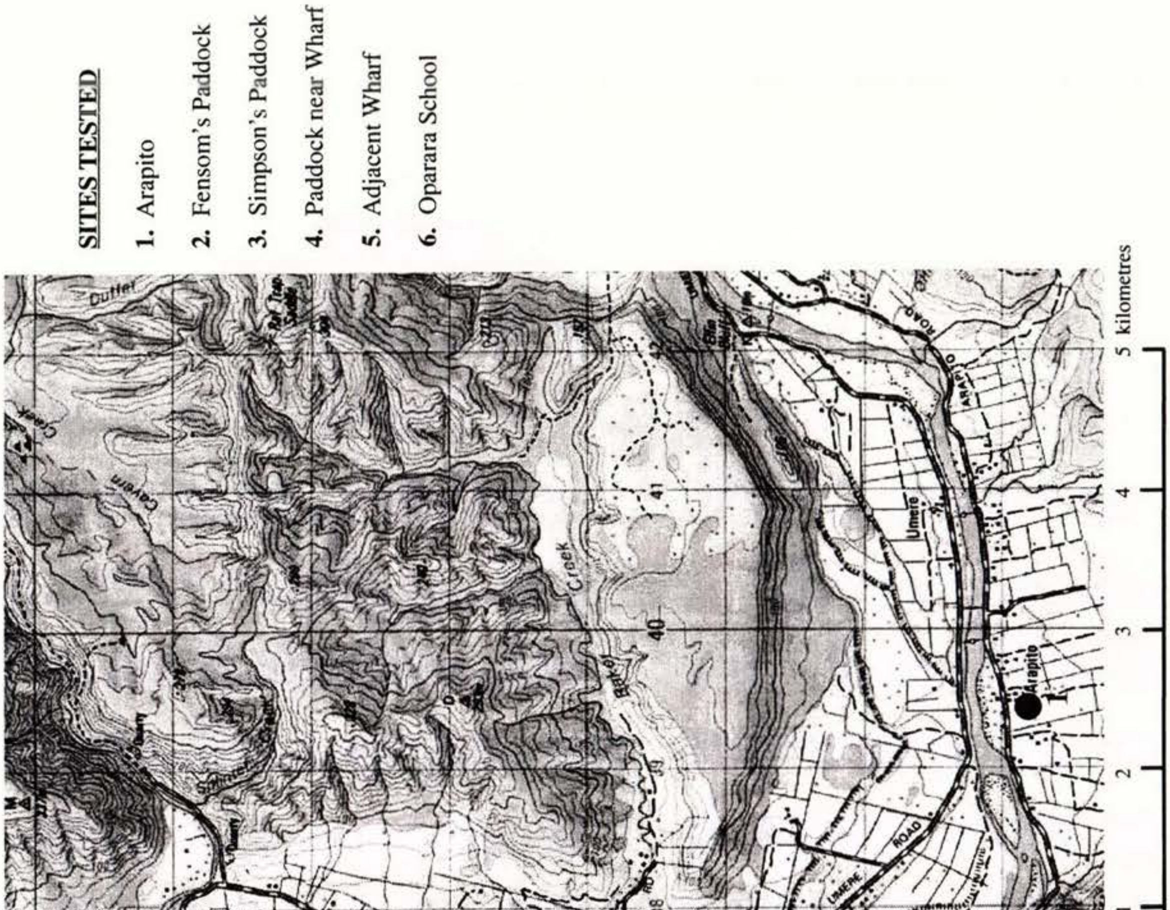


Figure A- 7. Location of field test sites in Karamea

A1.3.1 Arapito

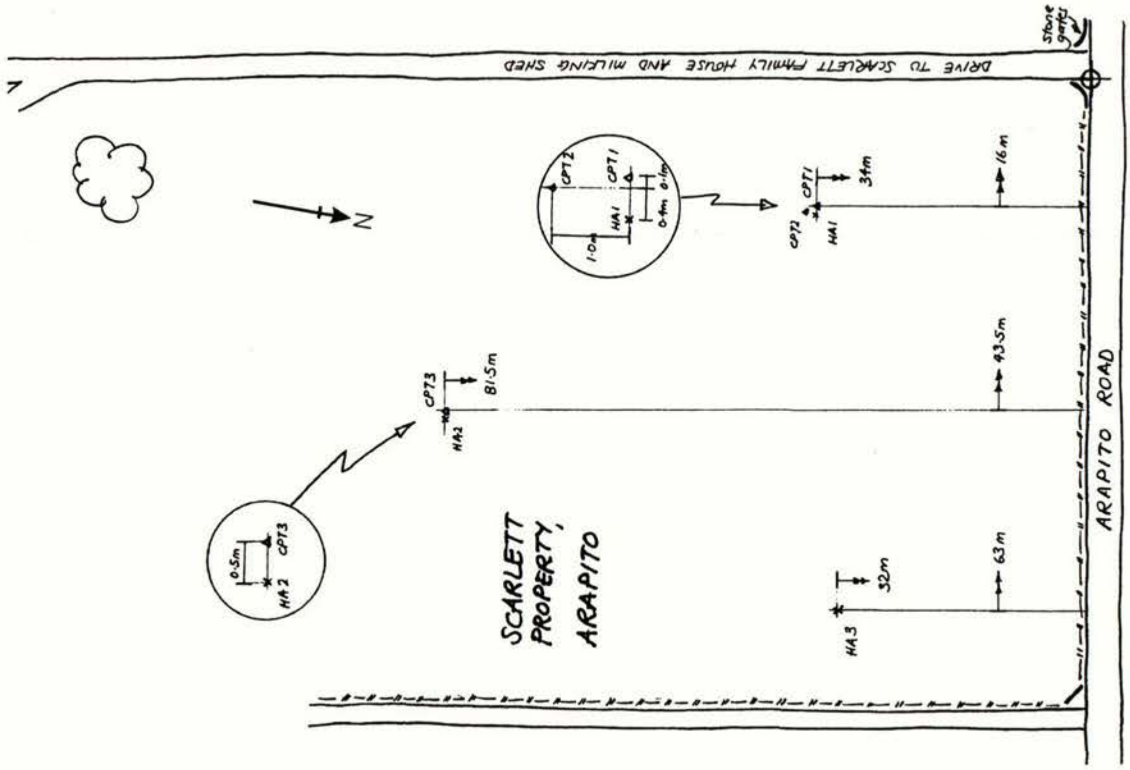
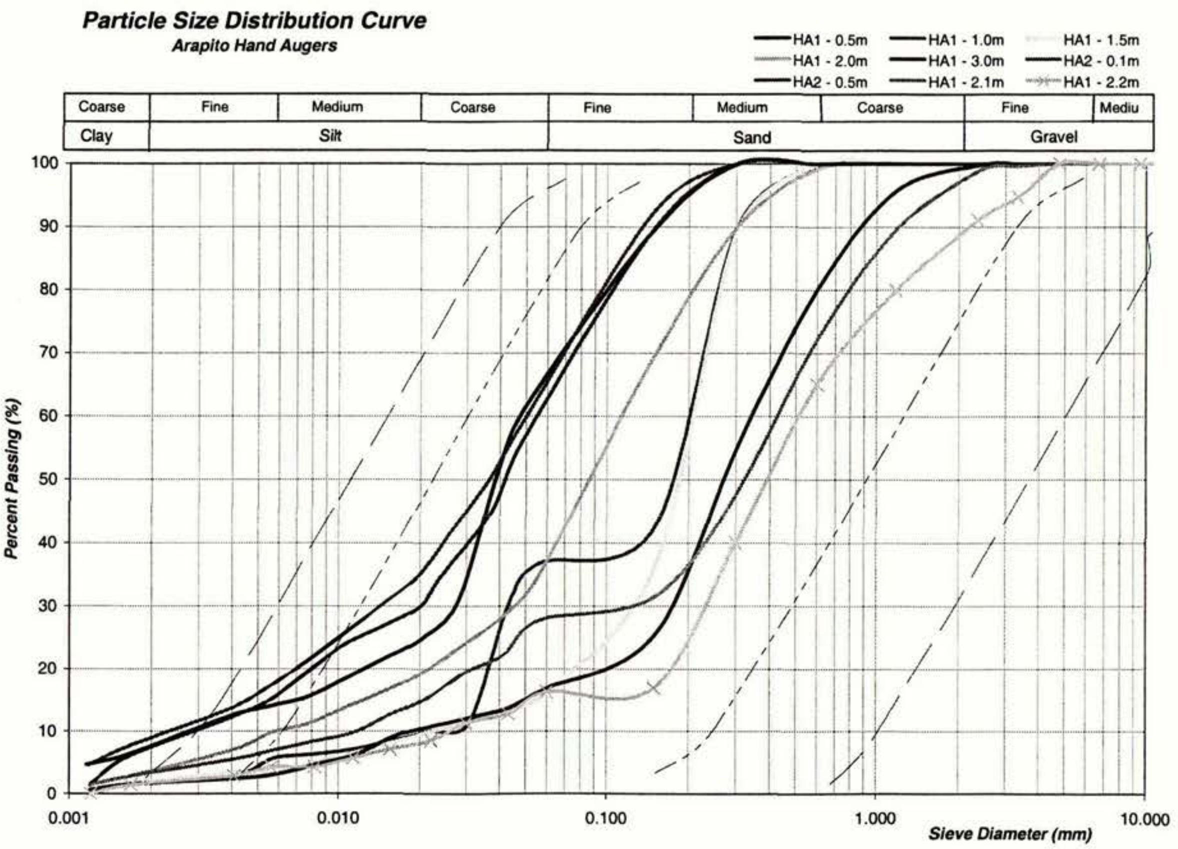


Figure A- 8. Site Diagram for testing undertaken at Arapito



ARA001.CPT

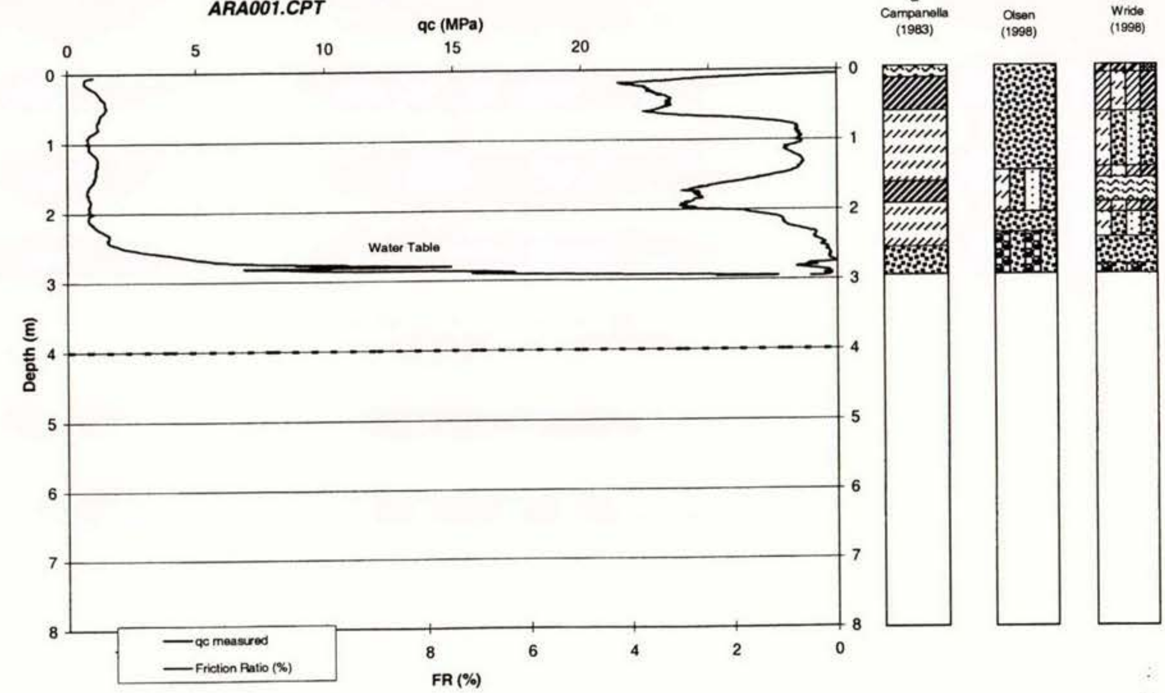
Test Boring Log

Job: Arapito
By: Kirsti, Siale
Date: 21/02/2003

Type: Hand Auger
Boring Number: 1

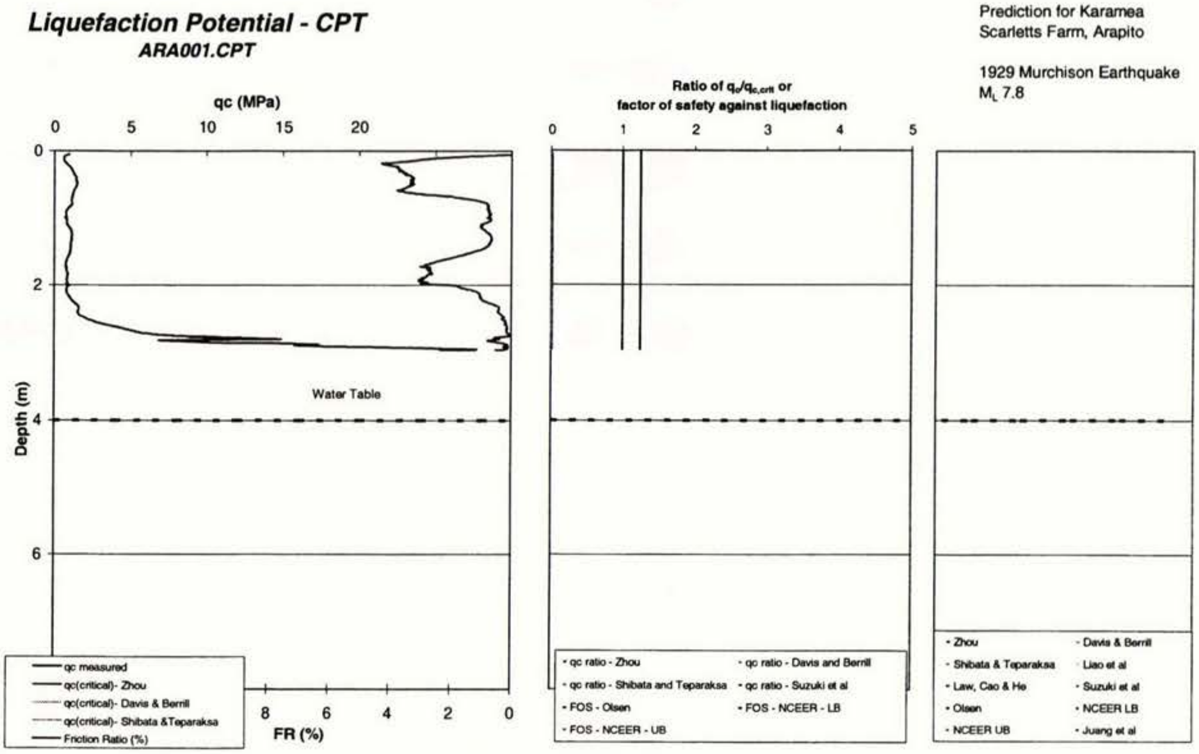
DEPTH (m)	Comments
0.00	
0.10	Silty Topsoil - light brown
0.20	
0.30	
0.40	
0.50	
0.60	
0.70	Sandy SILT light brown in colour
0.80	
0.90	
1.00	
1.10	
1.20	
1.30	
1.40	
1.50	very fine Silty SAND brown in colour very consistent layer
1.60	
1.70	
1.80	
1.90	
2.00	
2.10	
2.20	
2.30	
2.40	
2.50	
2.60	
2.70	cream- white medium SAND very dry and loose, sides of bore collapsing 2.83m Encountered pebbly Sand - BORE REFUSED
2.80	
2.90	
3.00	
3.10	
3.20	
3.30	
3.40	
3.50	
3.60	NB. Did not encounter water table
3.70	
3.80	
3.90	
4.00	

Inferred Soil Profiles - *based on experimental data*
ARA001.CPT



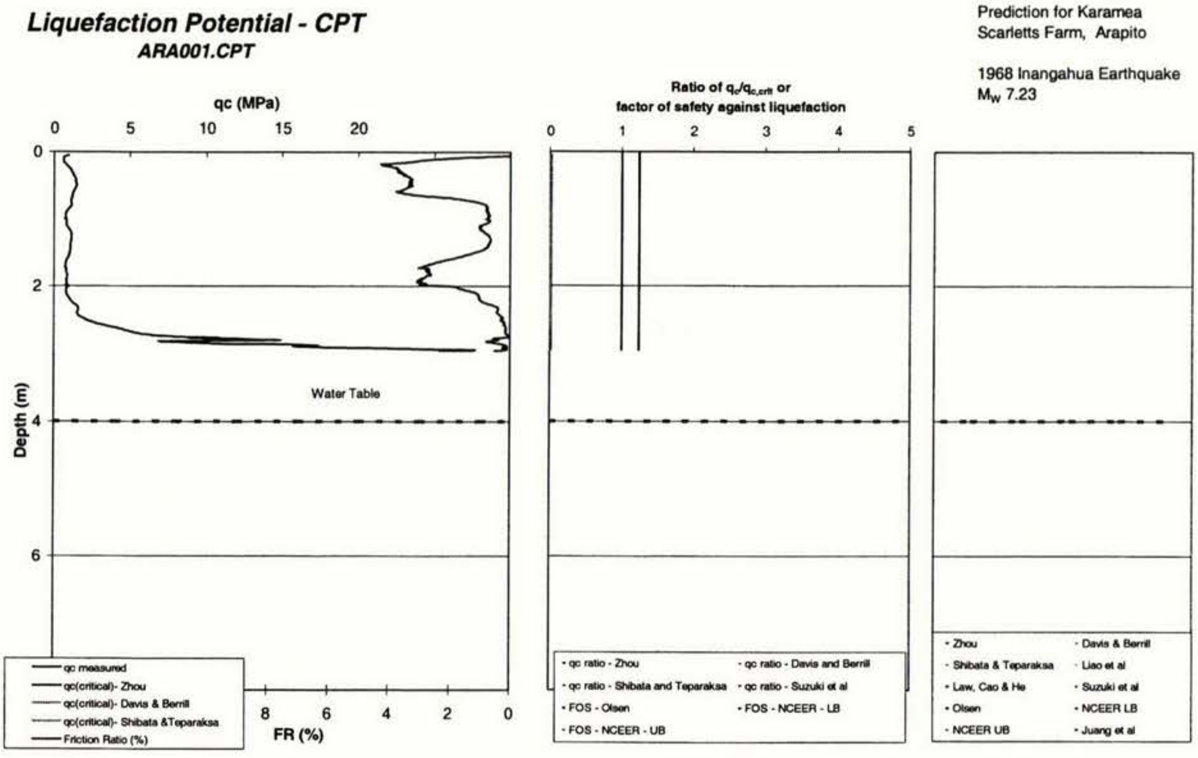
Prediction for Karamea
Scarletts Farm, Arapito

1929 Murchison Earthquake
M_L 7.8

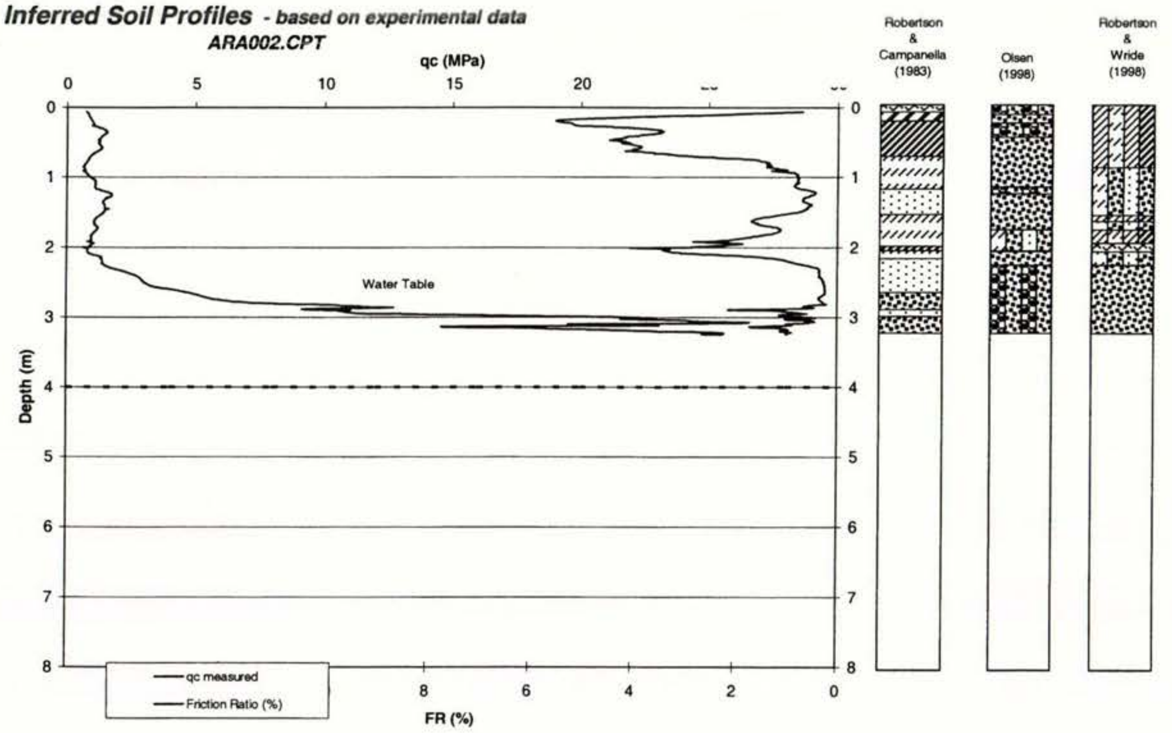


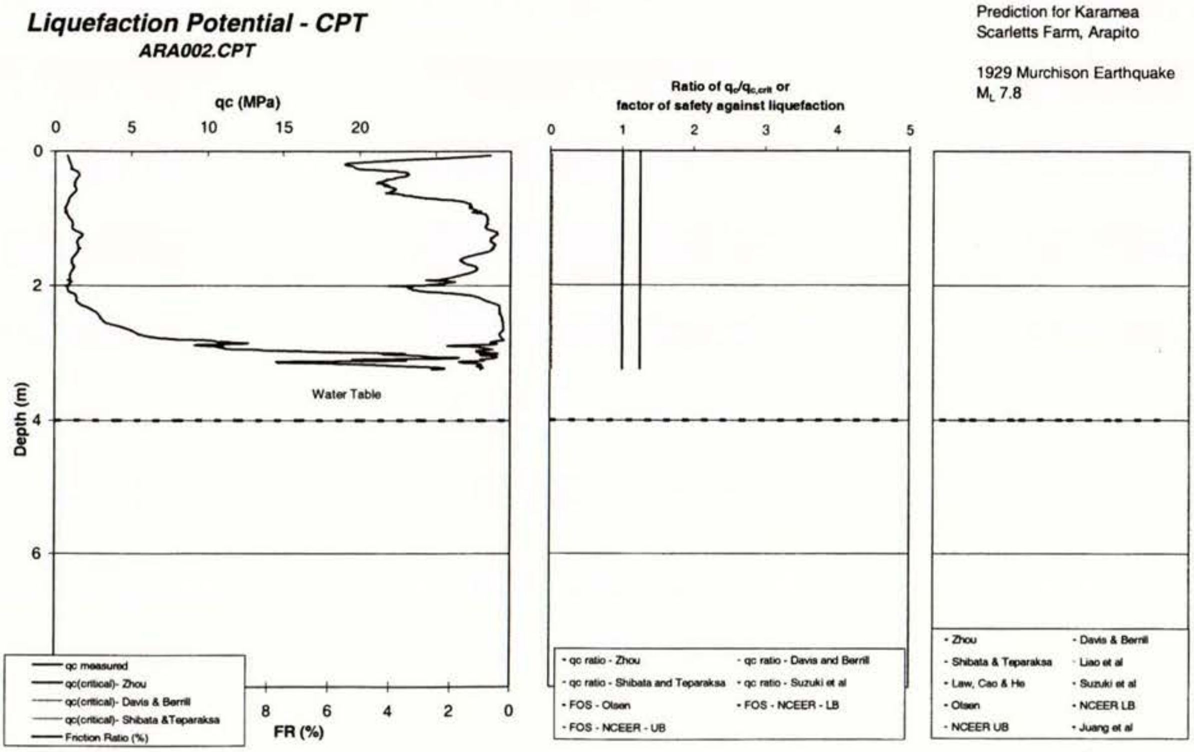
Prediction for Karamea
Scarletts Farm, Arapito

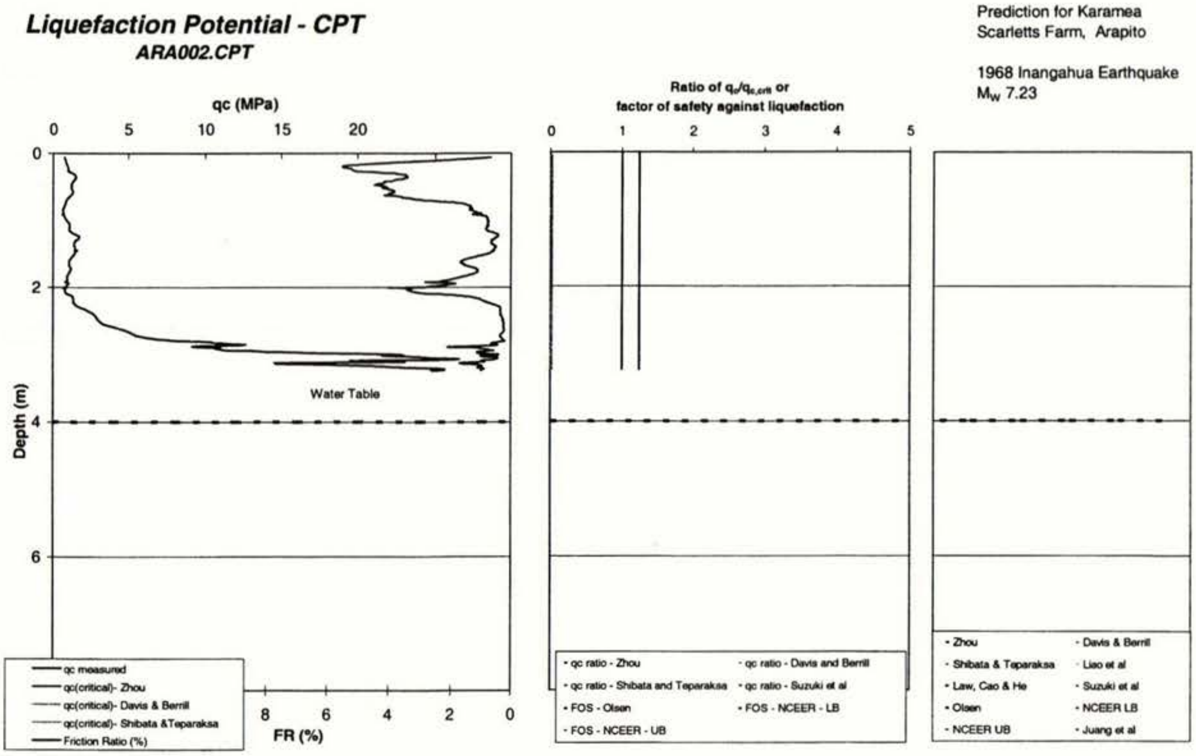
1968 Inangahua Earthquake
M_W 7.23



ARA002.CPT


Inferred Soil Profiles - based on experimental data
ARA002.CPT
qc (MPa)
Depth (m)
Water Table
qc measured
Friction Ratio (%)
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)


Liquefaction Potential - CPT
ARA002.CPT
qc (MPa)
Depth (m)
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al
Prediction for Karamea
Scarletts Farm, Arapito
1929 Murchison Earthquake
M_L 7.8



ARA003.CPT

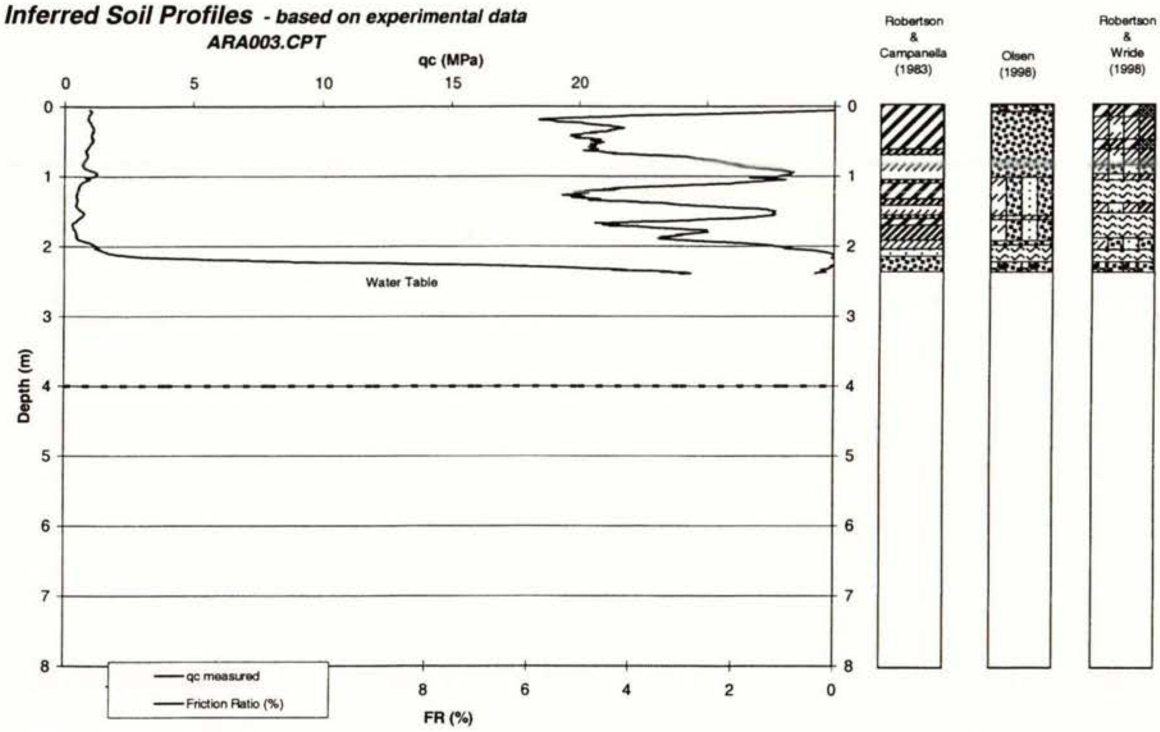
Test Boring Log

Job Arapito
By Kirsti, Siale
Date 21/02/2003

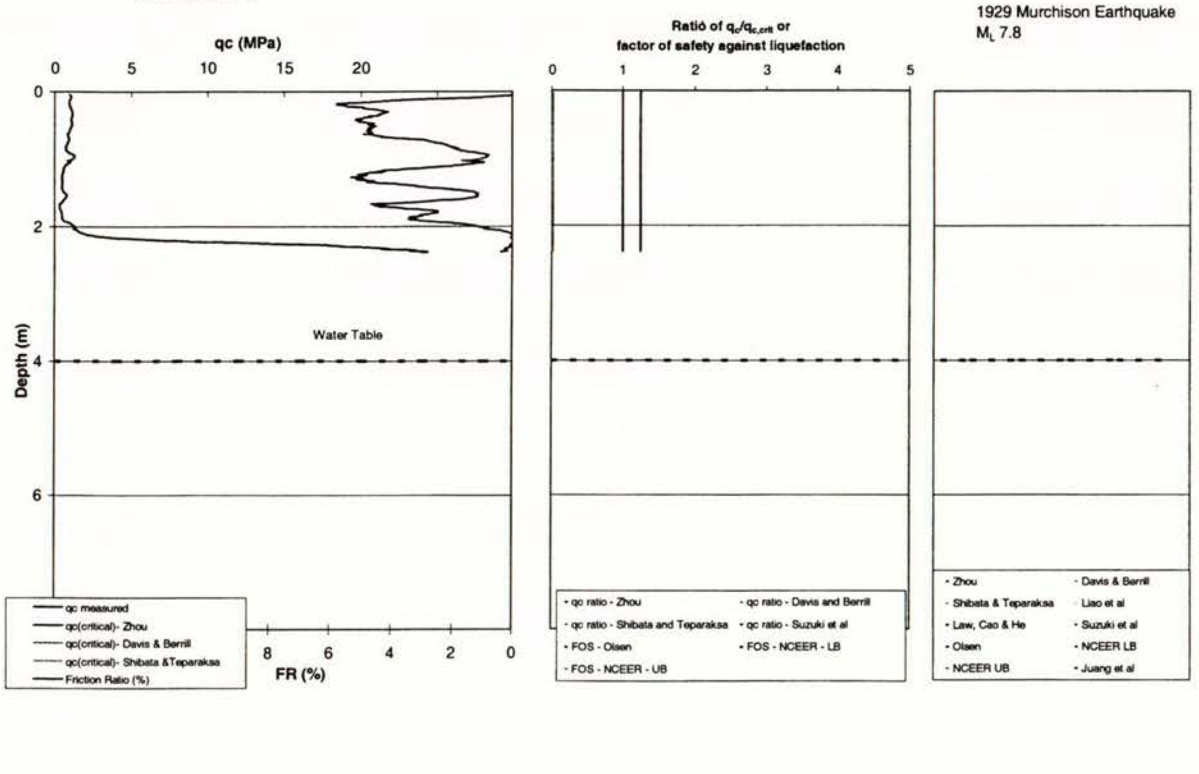
Type Hand Auger
Boring Number 2

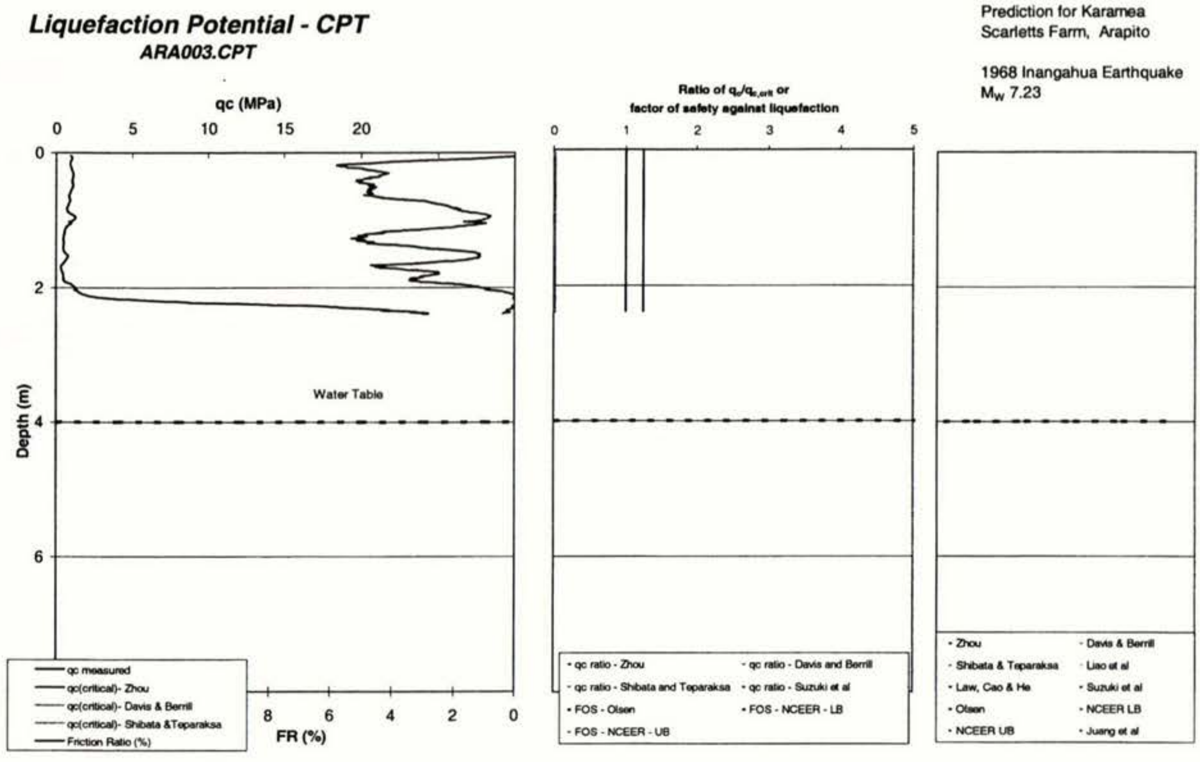
DEPTH (m)	Comments
0.00	
0.10	topsoil
0.20	Sandy SILT light brown in colour
0.30	
0.40	
0.50	fine SILT, some clay present light brown in colour
0.60	
0.70	
0.80	
0.90	
1.00	fine grained silty SAND grey in colour
1.10	
1.20	
1.30	
1.40	
1.50	Sandy SILT - moist and brown in colour
1.60	~~Very soft- can easily push auger though soil~~
1.70	Sandy SILT - brown in colour Much stiffer than before
1.80	
1.90	
2.00	Silty SAND - Whiteish in colour hole collapsing
2.10	
2.20	
2.30	2.20m Pebbles/gravel in base of auger - BORE REFUSED
2.40	
2.50	
2.60	
2.70	
2.80	
2.90	
3.00	NB. Did not encounter water table
3.10	
3.20	
3.30	
3.40	
3.50	
3.60	
3.70	
3.80	
3.90	
4.00	

Inferred Soil Profiles - based on experimental data
ARA003.CPT

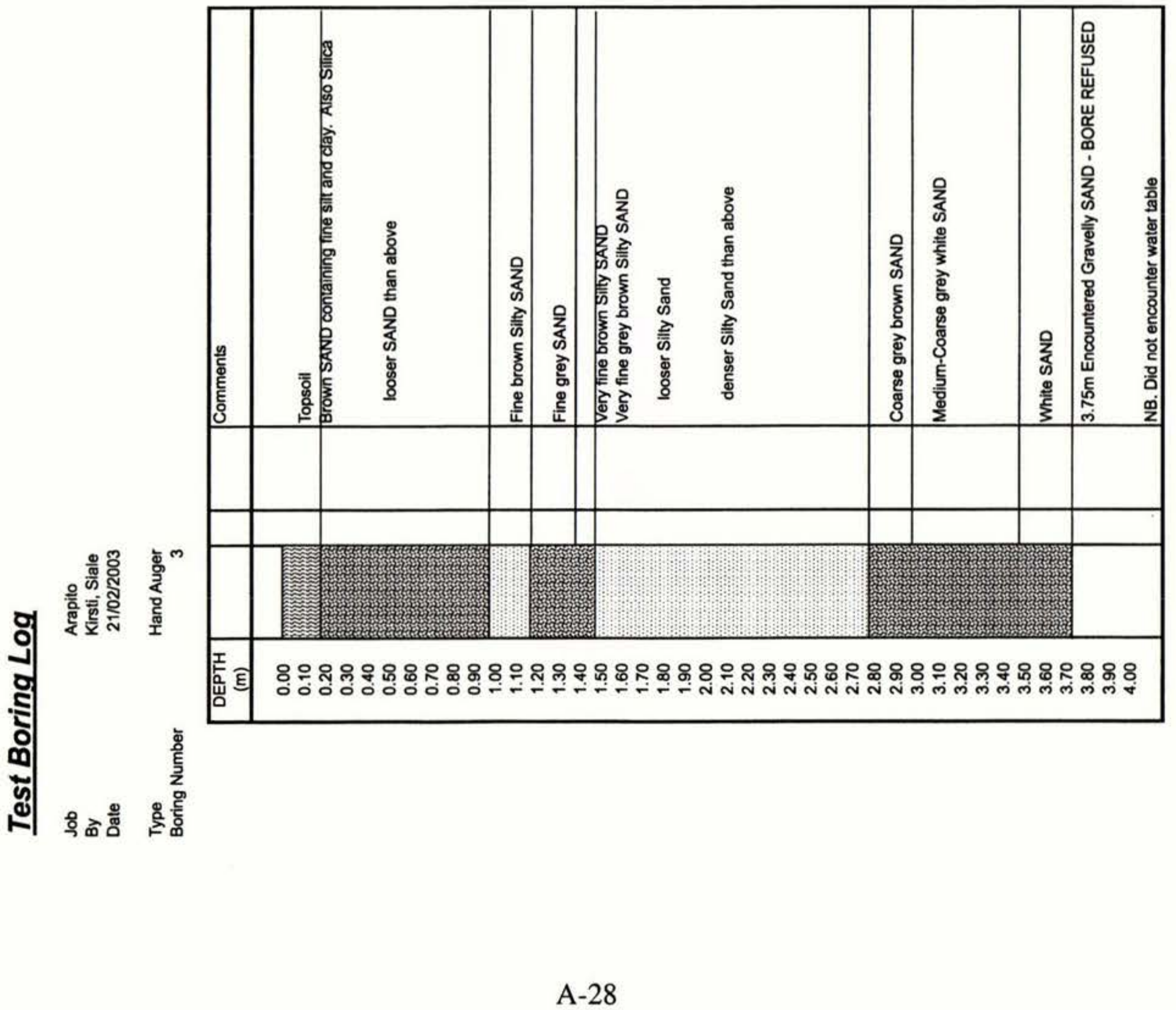


Liquefaction Potential - CPT
ARA003.CPT




Liquefaction Potential - CPT
ARA003.CPT
Prediction for Karamea
Scarletts Farm, Arapito
1968 Inangahua Earthquake
Mw 7.23
qc (MPa)
0 5 10 15 20
Ratio of qc/qc,crit or
factor of safety against liquefaction
0 1 2 3 4 5
Depth (m)
0 2 4 6
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

Hand Auger 3


Test Boring Log
Job Arapito
By Kirsti, Siale
Date 21/02/2003
Type Hand Auger
Boring Number 3
DEPTH (m)
0.00 0.10 0.20 0.30 0.40 0.50 0.60 0.70 0.80 0.90 1.00 1.10 1.20 1.30 1.40 1.50 1.60 1.70 1.80 1.90 2.00 2.10 2.20 2.30 2.40 2.50 2.60 2.70 2.80 2.90 3.00 3.10 3.20 3.30 3.40 3.50 3.60 3.70 3.80 3.90 4.00
Comments
Topsoil
Brown SAND containing fine silt and clay. Also Silica
looser SAND than above
Fine brown Silty SAND
Fine grey SAND
Very fine brown Silty SAND
Very fine grey brown Silty SAND
looser Silty Sand
denser Silty Sand than above
Coarse grey brown SAND
Medium-Coarse grey white SAND
White SAND
3.75m Encountered Gravelly SAND - BORE REFUSED
NB. Did not encounter water table

A1.3.2 Fensom's Paddock

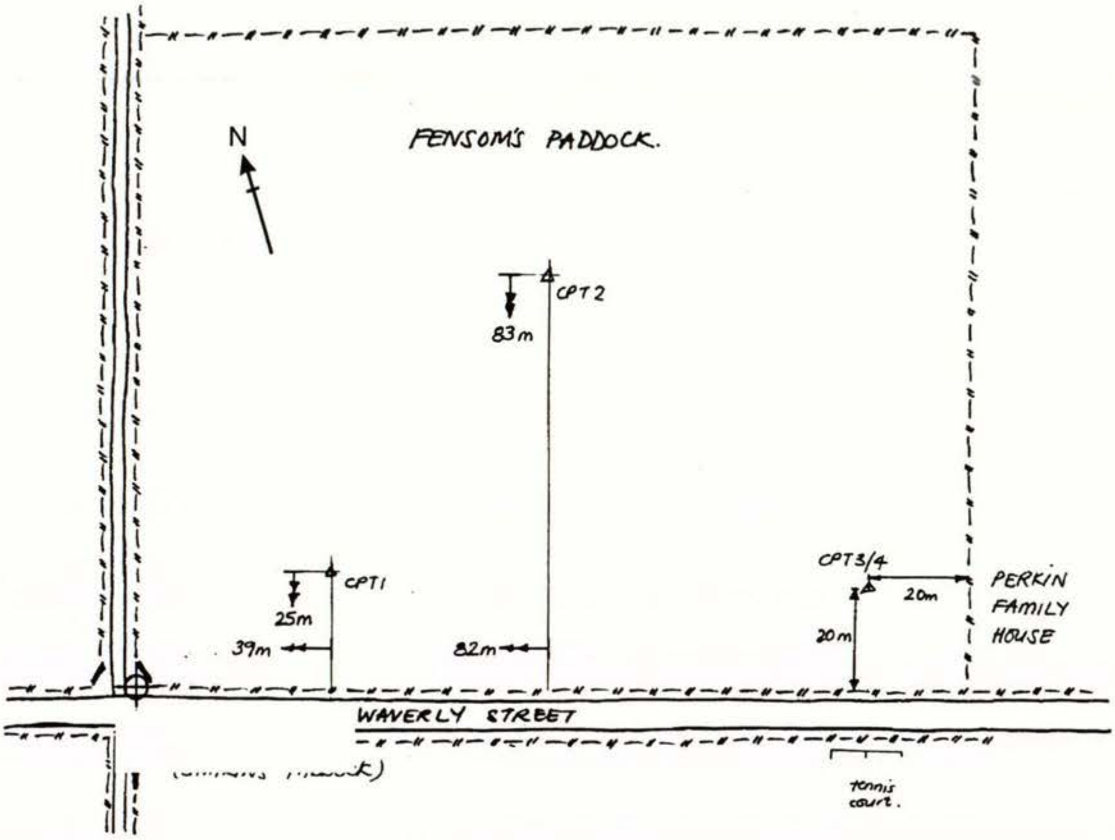
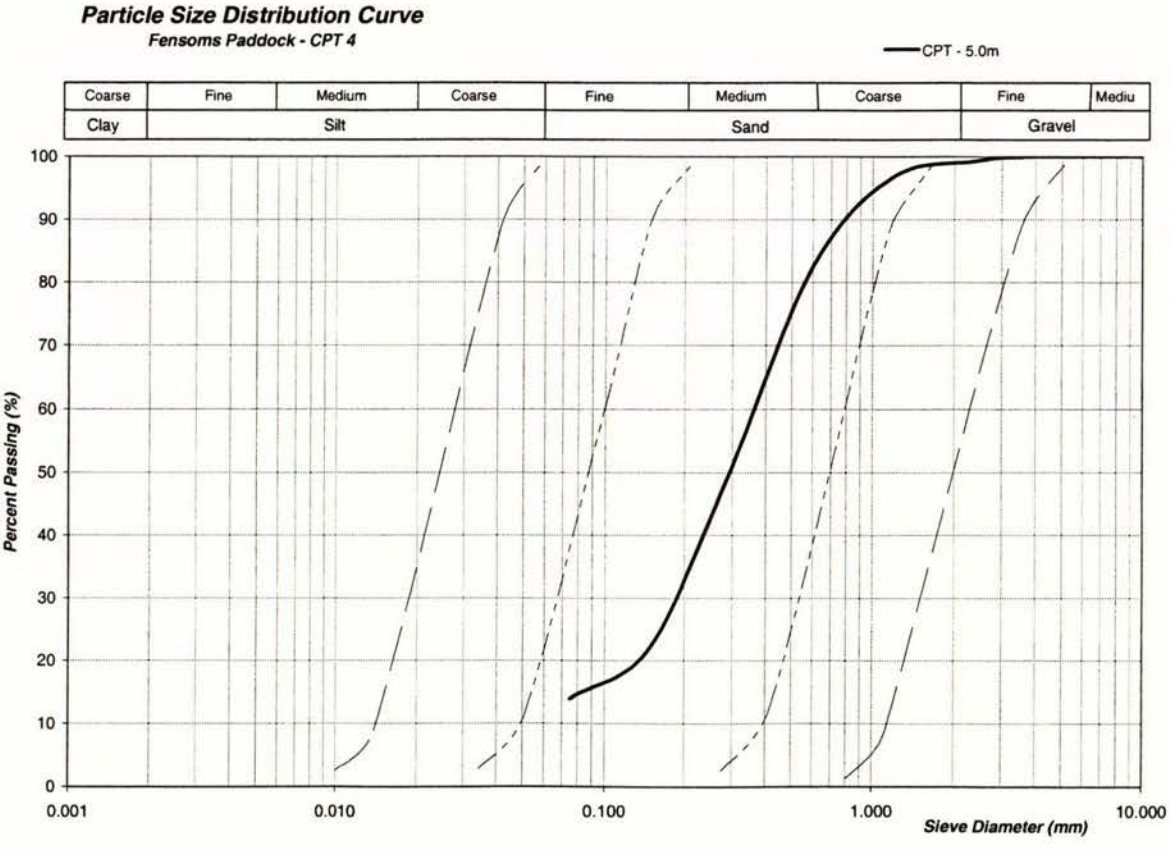
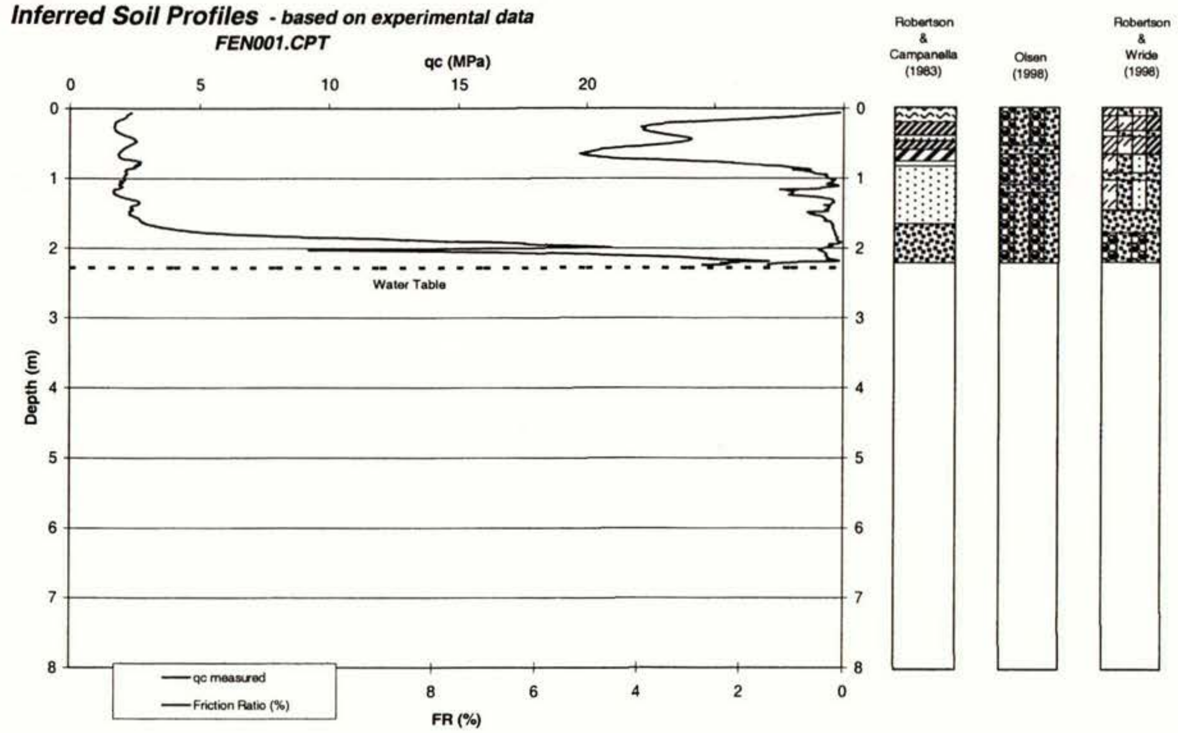
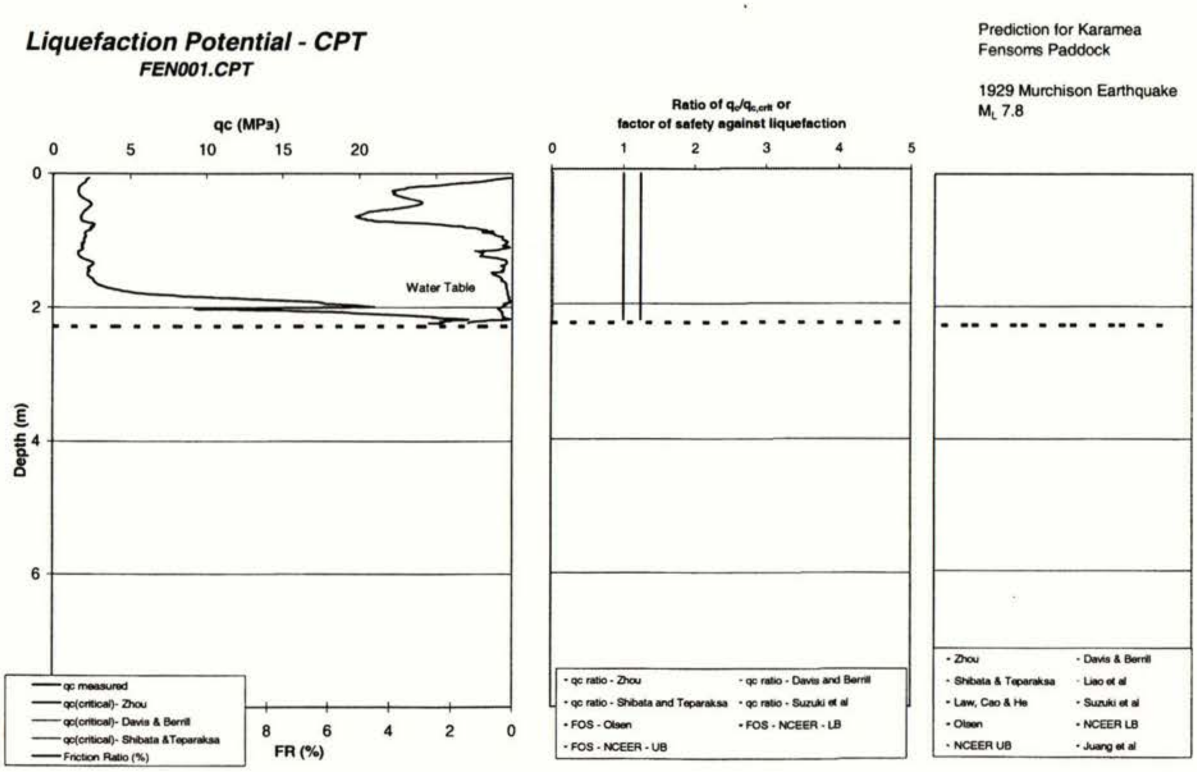


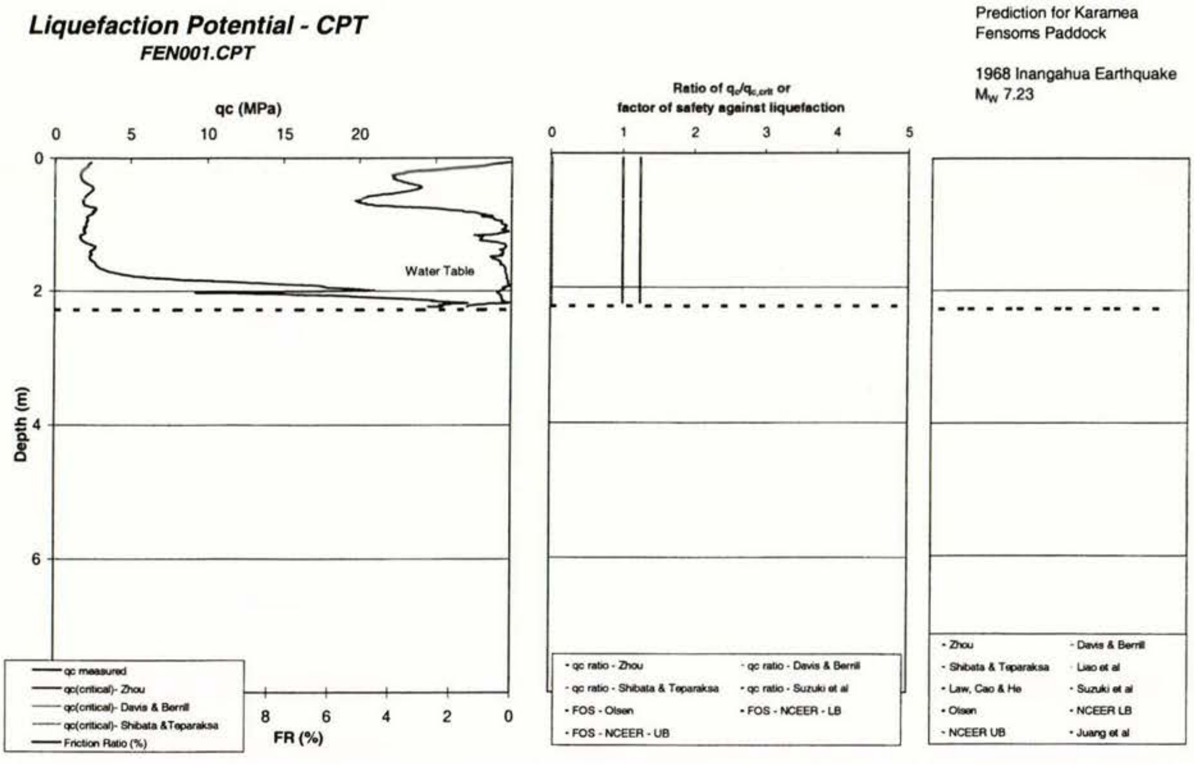
Figure A- 9. Sites Diagram for testing undertaken at Fensom's Paddock, Karamea



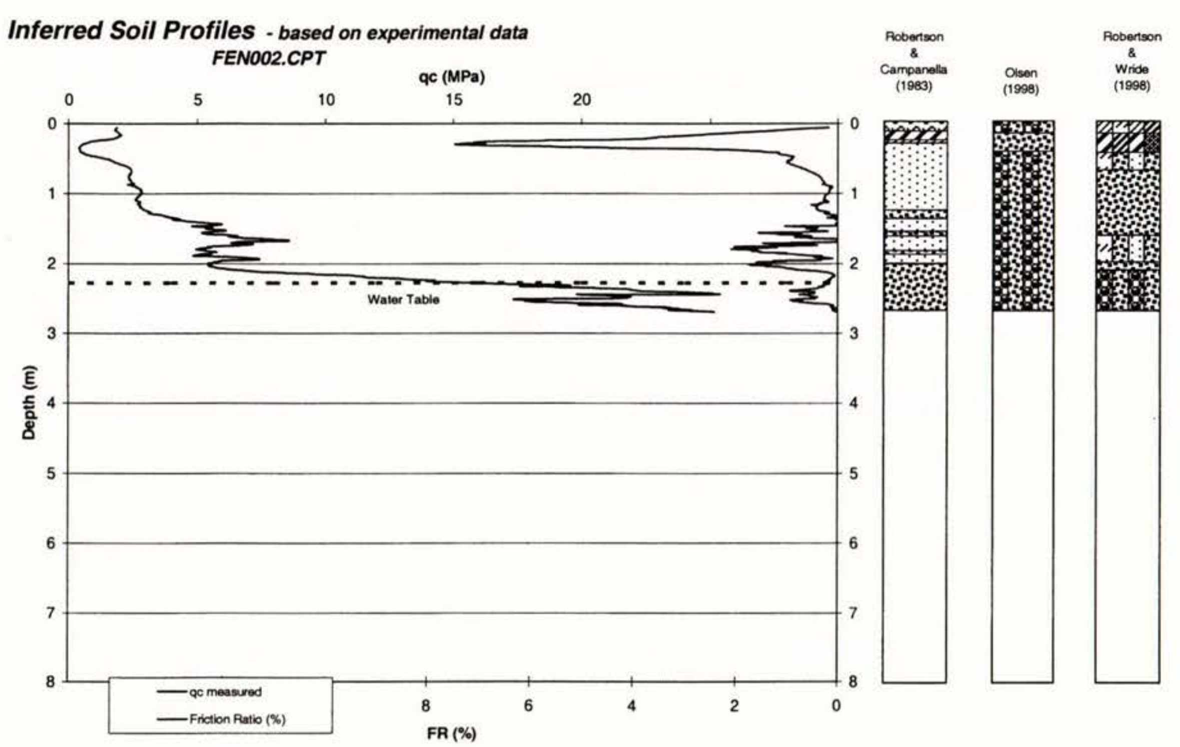
FEN001.CPT

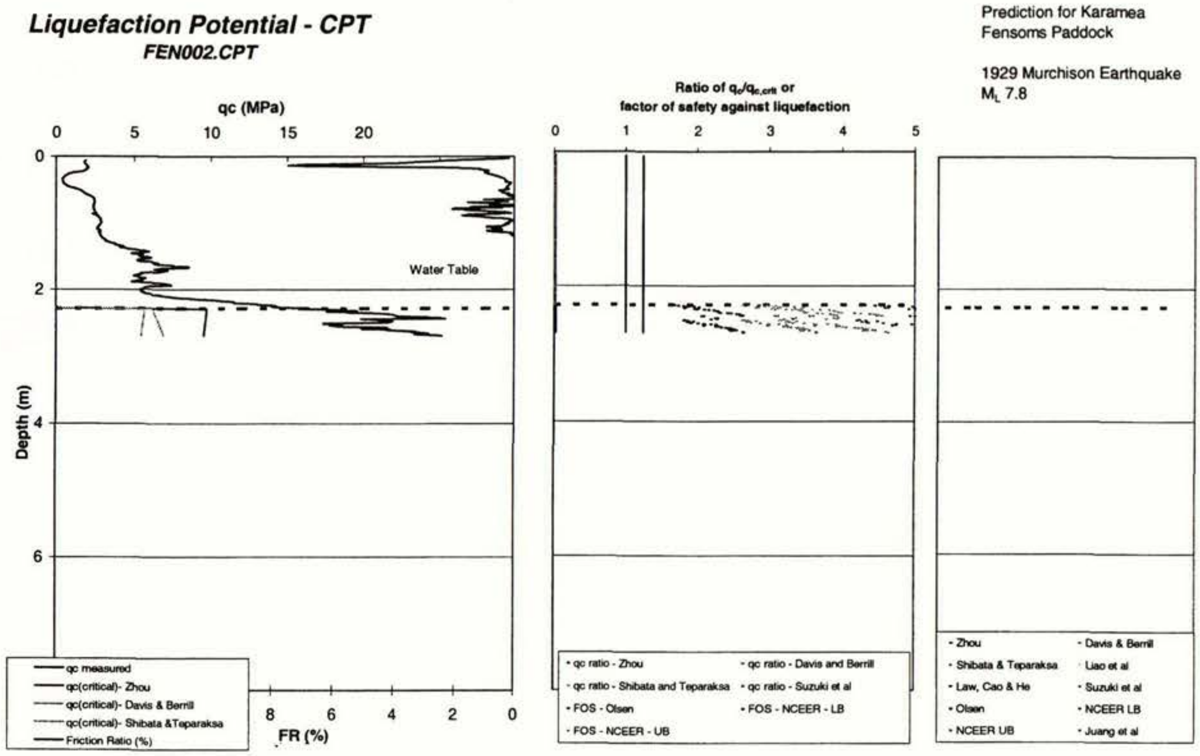

Inferred Soil Profiles - based on experimental data
FEN001.CPT
qc (MPa)
0
5
10
15
20
Depth (m)
Water Table
qc measured
Friction Ratio (%)
FR (%)
8
6
4
2
0
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)

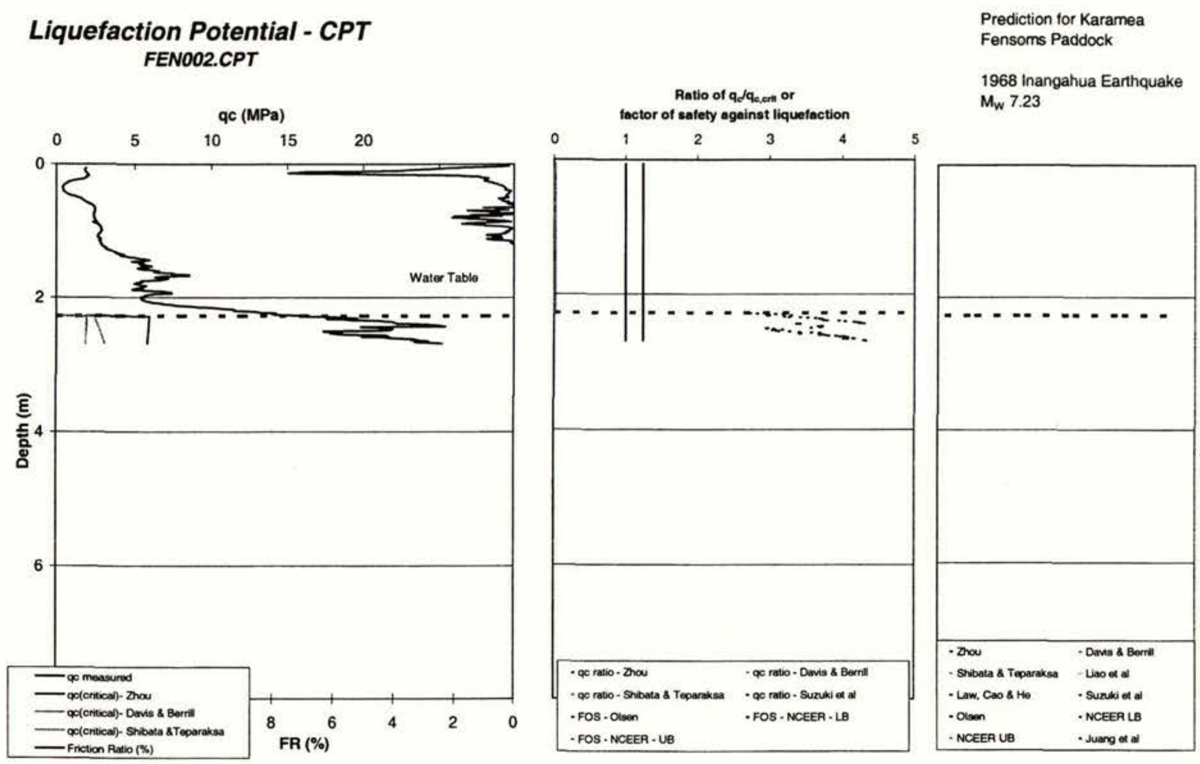

Liquefaction Potential - CPT
FEN001.CPT
qc (MPa)
Depth (m)
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
- qc ratio - Zhou
- qc ratio - Davis and Berrill
- qc ratio - Shibata and Teparaksa
- qc ratio - Suzuki et al
- FOS - Olsen
- FOS - NCEER - LB
- FOS - NCEER - UB
Prediction for Karamea Fensoms Paddock
1929 Murchison Earthquake M_L 7.8
- Zhou
- Davis & Berrill
- Shibata & Teparaksa
- Liao et al
- Law, Cao & He
- Suzuki et al
- Olsen
- NCEER LB
- NCEER UB
- Juang et al


Liquefaction Potential - CPT
FEN001.CPT
Prediction for Karamea
Fensoms Paddock
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0
5
10
15
20
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0
1
2
3
4
5
Water Table
Depth (m)
0
2
4
6
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8
6
4
2
0
FR (%)
qc ratio - Zhou
qc ratio - Davis & Berrill
qc ratio - Shibata & Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

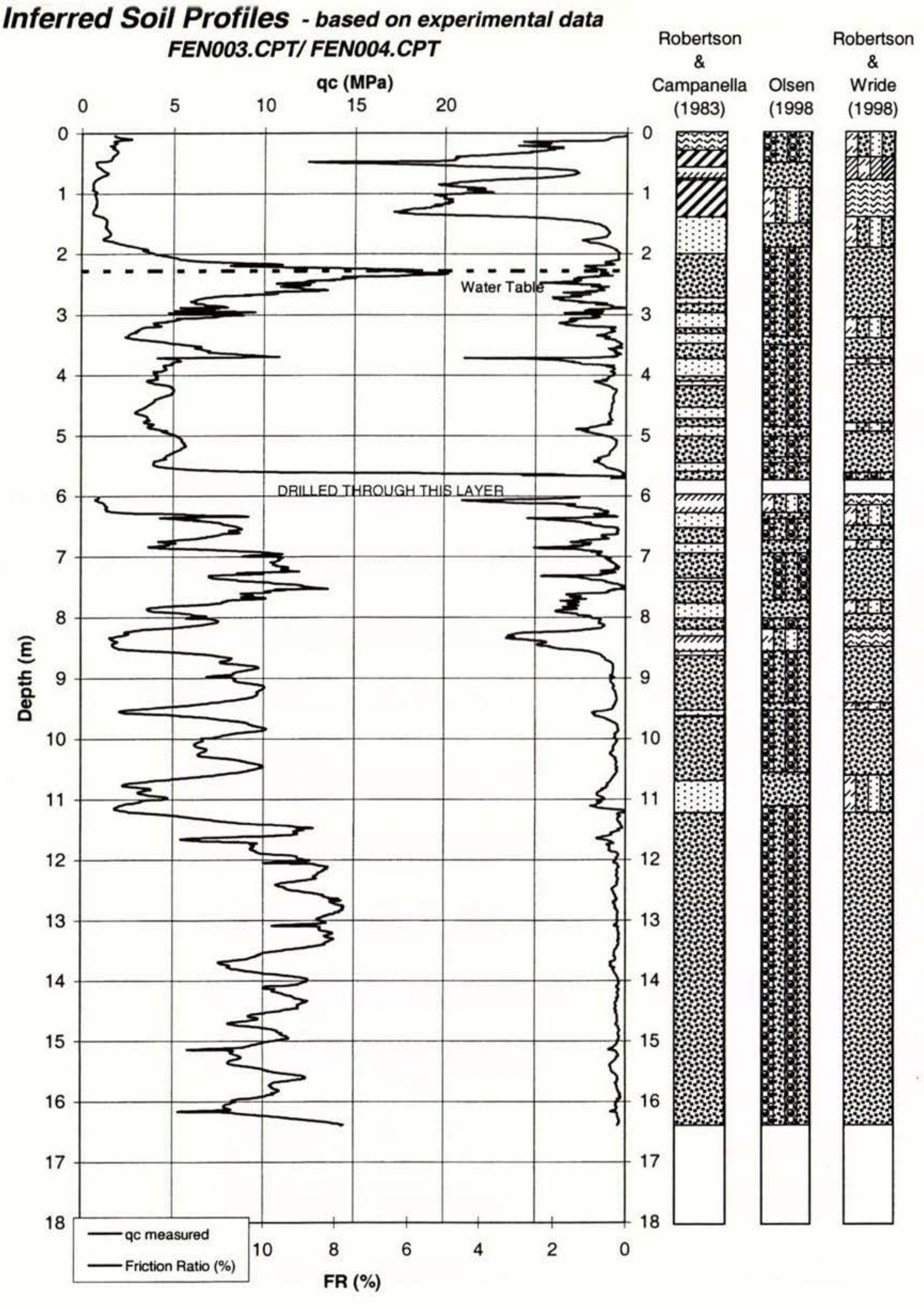
FEN002.CPT


Inferred Soil Profiles - based on experimental data
FEN002.CPT
qc (MPa)
0
5
10
15
20
Robertson
&
Campanella
(1983)
Olsen
(1998)
Robertson
&
Wride
(1998)
Water Table
Depth (m)
0
1
2
3
4
5
6
7
8
qc measured
Friction Ratio (%)
8
6
4
2
0
FR (%)


Liquefaction Potential - CPT
FEN002.CPT
Prediction for Karamea
Fensoms Paddock
1929 Murchison Earthquake
M_L 7.8
qc (MPa)
0 5 10 15 20
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
Water Table
Depth (m)
0 2 4 6
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al


Liquefaction Potential - CPT
FEN002.CPT
Prediction for Karamea
Fensoms Paddock
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0 5 10 15 20
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
Water Table
Depth (m)
0 2 4 6
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
qc ratio - Zhou
qc ratio - Davis & Berrill
qc ratio - Shibata & Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

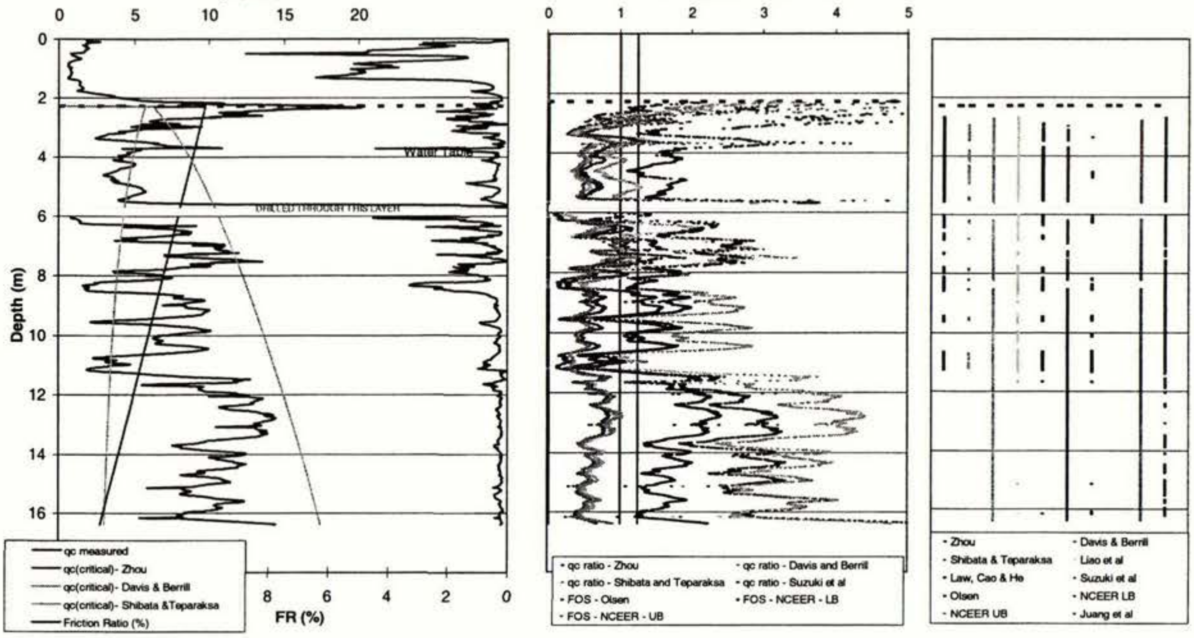
FEN003.CPT


Inferred Soil Profiles - based on experimental data
FEN003.CPT/ FEN004.CPT
qc (MPa)
0
5
10
15
20
Depth (m)
Water Table
DRILLED THROUGH THIS LAYER
qc measured
Friction Ratio (%)
10
8
6
4
2
0
FR (%)
Robertson & Campanella (1983)
Olsen (1998
Robertson & Wride (1998)

Liquefaction Potential - CPT
FEN003.CPT

Prediction for Karamea
Fensoms Paddock

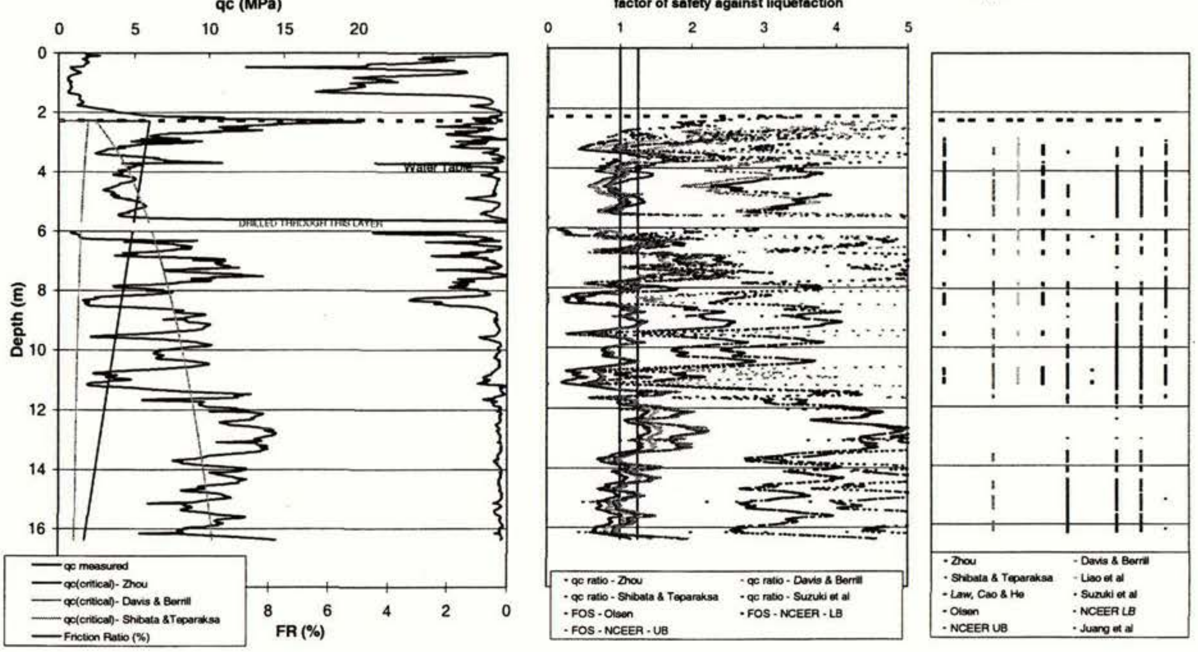
1929 Murchison Earthquake
M_L 7.8



Liquefaction Potential - CPT
FEN003.CPT

Prediction for Karamea
Fensoms Paddock

1968 Inangahua Earthquake
M_W 7.23



A1.3.3 Simpson's Paddock

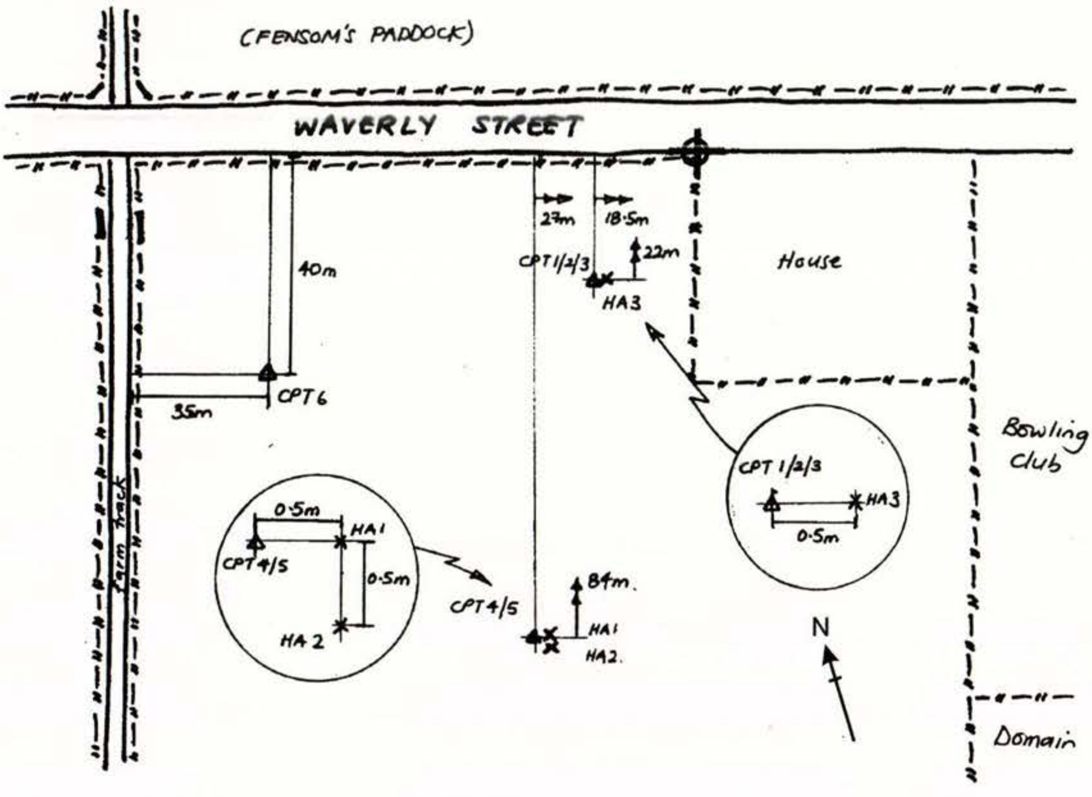
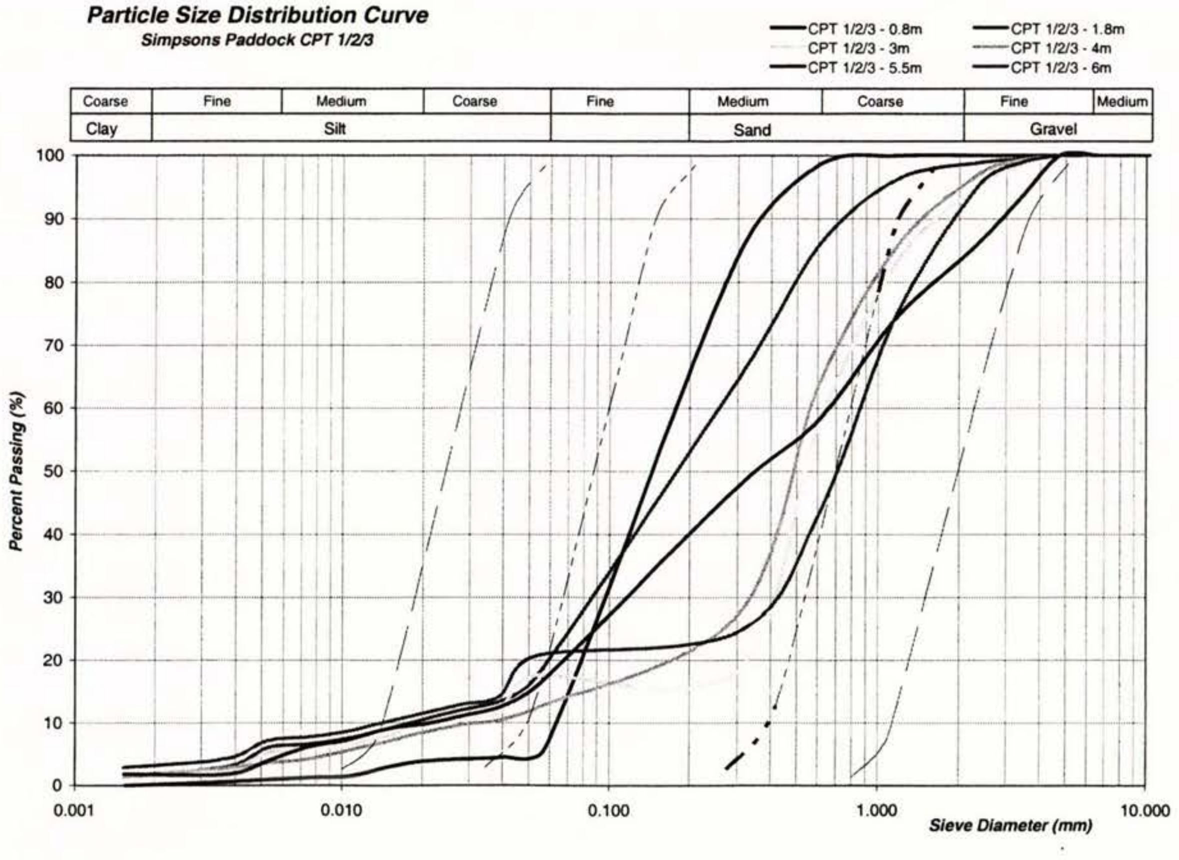


Figure A- 10. Site Diagram for Simpson's Paddock Field Testing



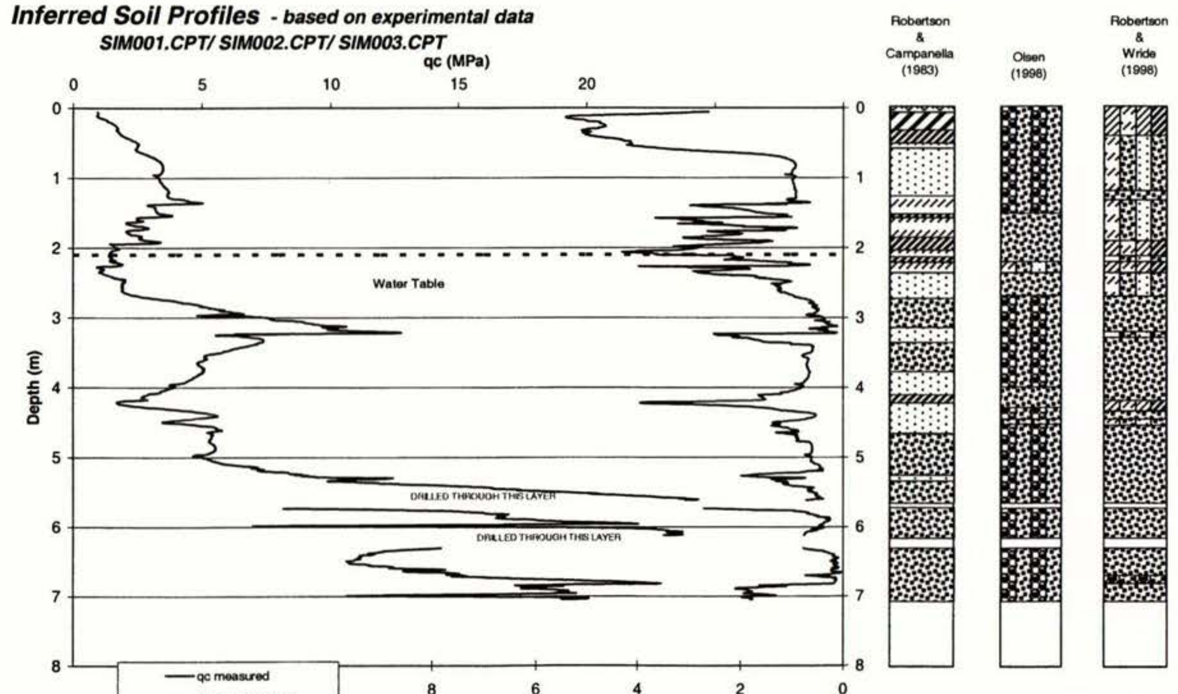
SIM001.CPT / SIM002.CPT / SIM003.CPT

Test Boring Log

Job Simpsons Paddock
By Kirsti
Date 22/02/2003

Type Hand Auger
Boring Number 3

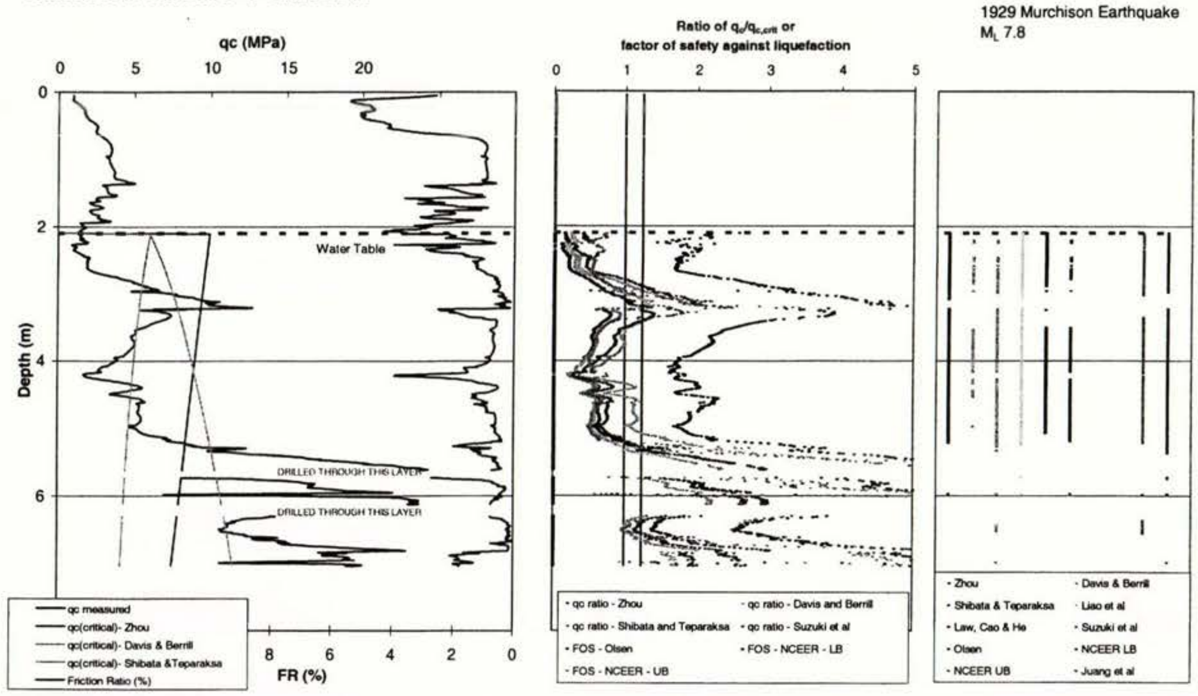
DEPTH (m)	Comments
0.00	
0.10	Brown Silty Topsoil
0.20	
0.30	Fine brown SILT
0.40	
0.50	
0.60	
0.70	
0.80	
0.90	Fine grey SAND
1.00	
1.10	
1.20	Medium grey SAND
1.30	
1.40	
1.50	
1.60	1.60m Encountered Gravel - BORE REFUSED
1.70	
1.80	
1.90	
2.00	
2.10	
2.20	
2.30	
2.40	
2.50	
2.60	
2.70	
2.80	
2.90	
3.00	NB. Did not encounter water table
3.10	
3.20	
3.30	
3.40	
3.50	
3.60	
3.70	
3.80	
3.90	
4.00	



Liquefaction Potential - CPT
SIM001.CPT/ SIM002.CPT/ SIM003.CPT

Prediction for Karamea
Simpsons Paddock

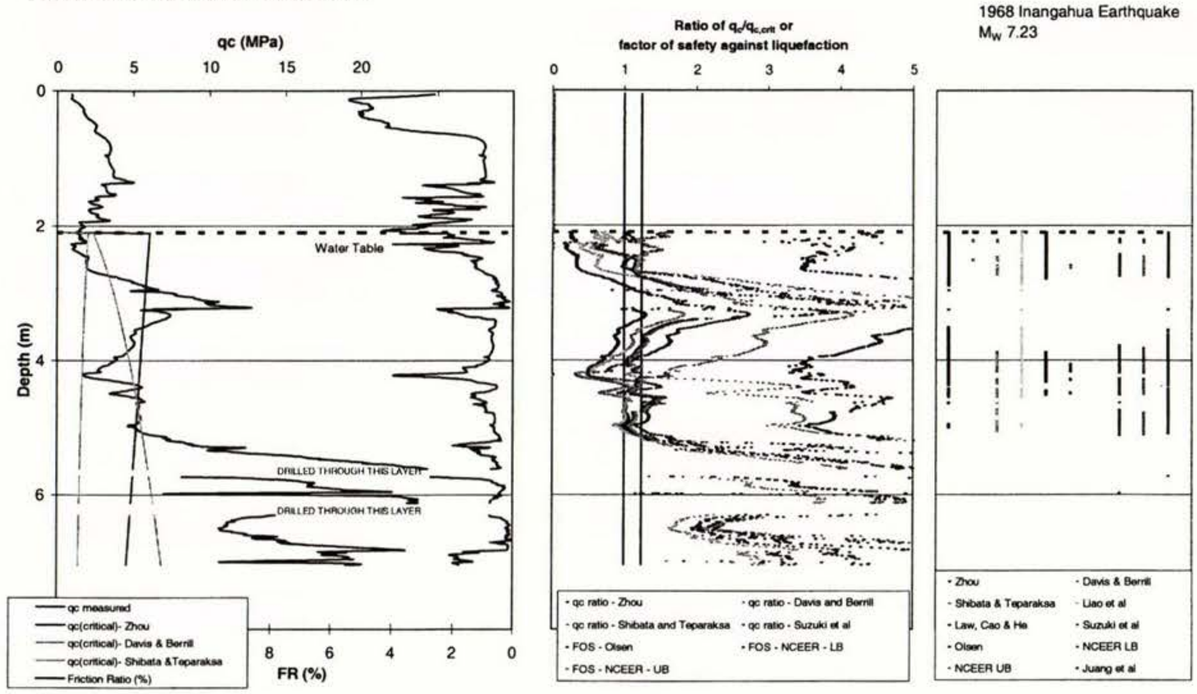
1929 Murchison Earthquake
M_L 7.8



Liquefaction Potential - CPT
SIM001.CPT/ SIM002.CPT/ SIM003.CPT

Prediction for Karamea
Simpsons Paddock

1968 Inangahua Earthquake
M_W 7.23



SIM004.CPT

Test Boring Log

Job	Simpsons Paddock
By	Kirsti
Date	22/02/2003
Type	Hand Auger
Boring Number	1

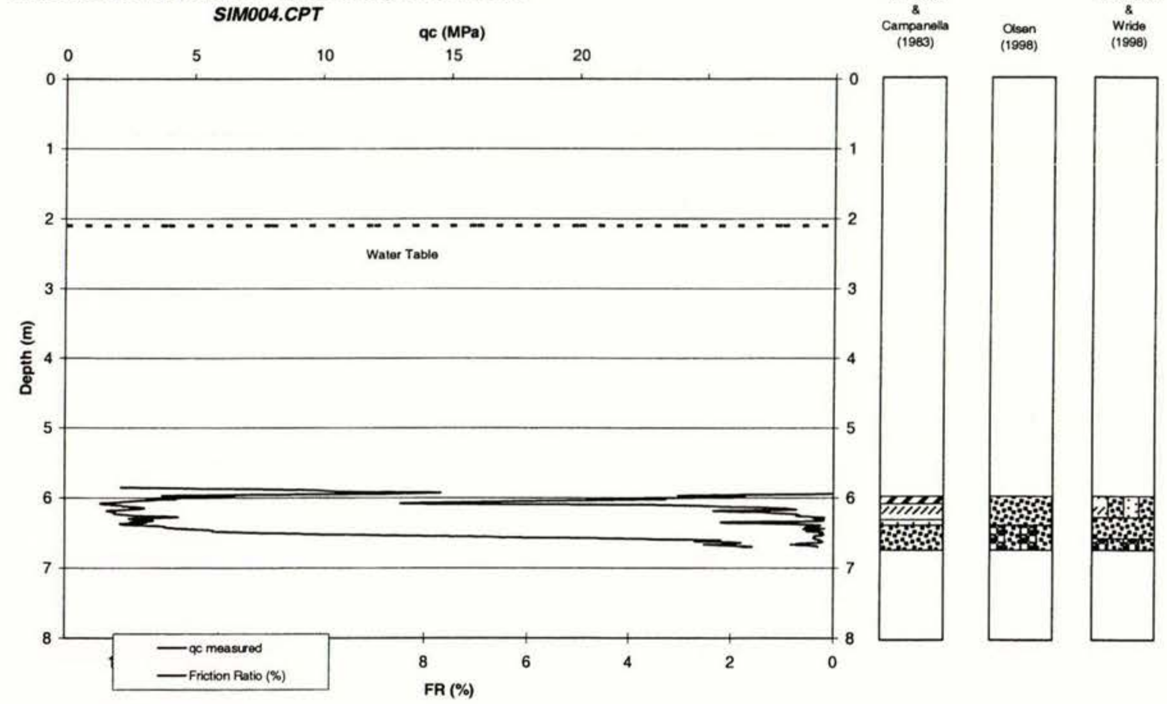
DEPTH (m)				Comments
0.00				
0.10				Brown Silty TOPSOIL
0.20				
0.30				Very stiff, very fine brown SILT
0.40				
0.50				
0.60				
0.70				
0.80				
0.90				0.90m Encountered Gravel - BORE REFUSED
1.00				
1.10				
1.20				
1.30				
1.40				
1.50				
1.60				
1.70				
1.80				
1.90				
2.00				
2.10				
2.20				
2.30				
2.40				
2.50				
2.60				
2.70				
2.80				
2.90				
3.00				NB. Did not encounter water table
3.10				
3.20				
3.30				
3.40				
3.50				
3.60				
3.70				
3.80				
3.90				
4.00				

Test Boring Log

Job	Simpsons Paddock
By	Kirsti
Date	22/02/2003
Type	Hand Auger
Boring Number	2

DEPTH (m)				Comments
0.00				
0.10				Brown Silty Topsoil
0.20				
0.30				Very fine brown SILT
0.40				
0.50				
0.60				
0.70				
0.80				0.80m Encountered Gravel - BORE REFUSED
0.90				
1.00				
1.10				
1.20				
1.30				
1.40				
1.50				
1.60				
1.70				
1.80				
1.90				
2.00				
2.10				
2.20				
2.30				
2.40				
2.50				
2.60				
2.70				
2.80				
2.90				
3.00				NB. Did not encounter water table
3.10				
3.20				
3.30				
3.40				
3.50				
3.60				
3.70				
3.80				
3.90				
4.00				

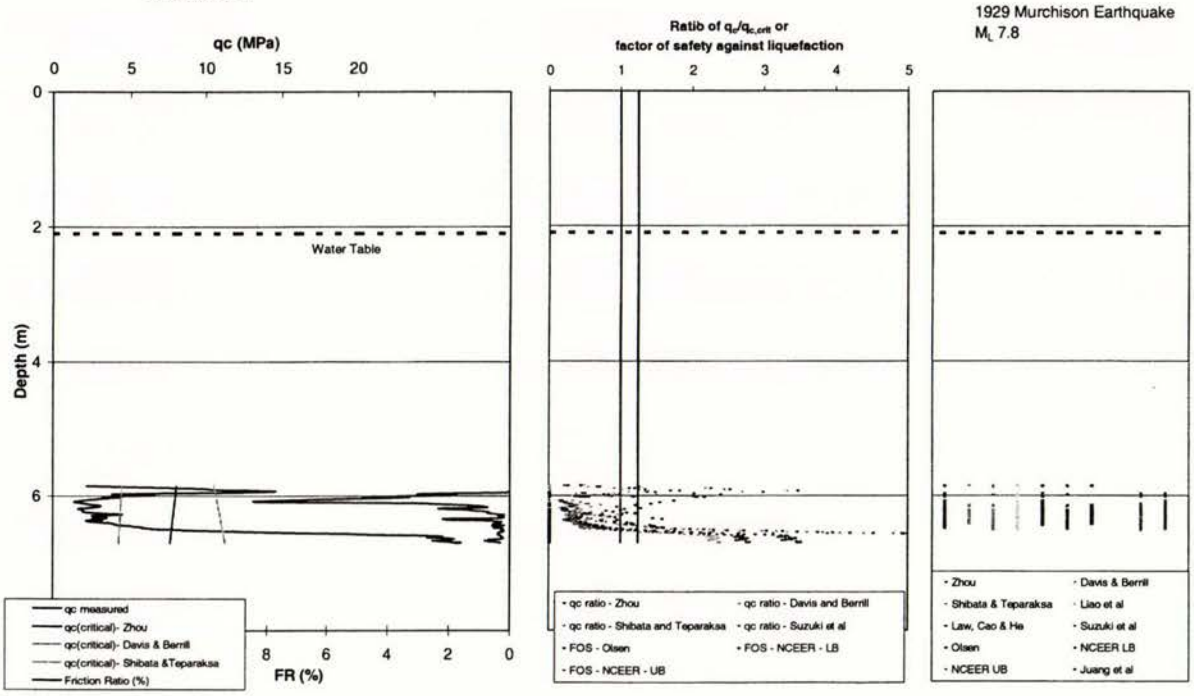
***Inferred Soil Profiles** - based on experimental data*
SIM004.CPT

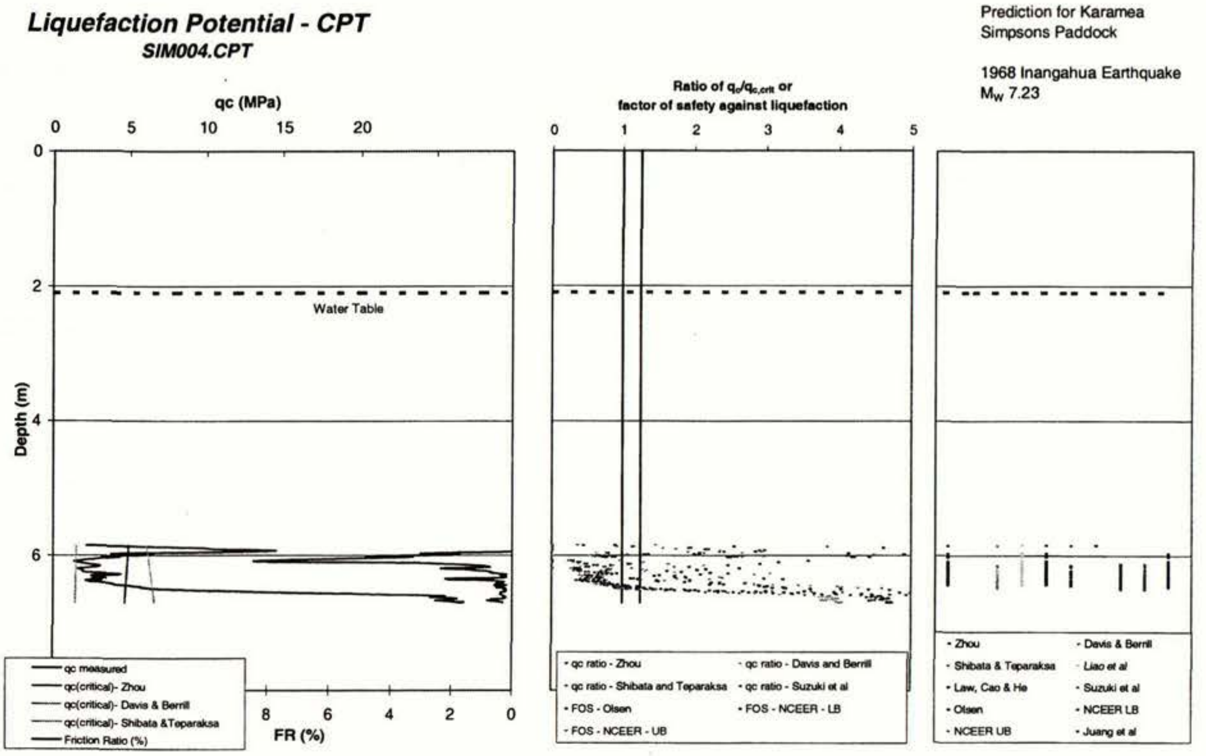


Liquefaction Potential - CPT
SIM004.CPT

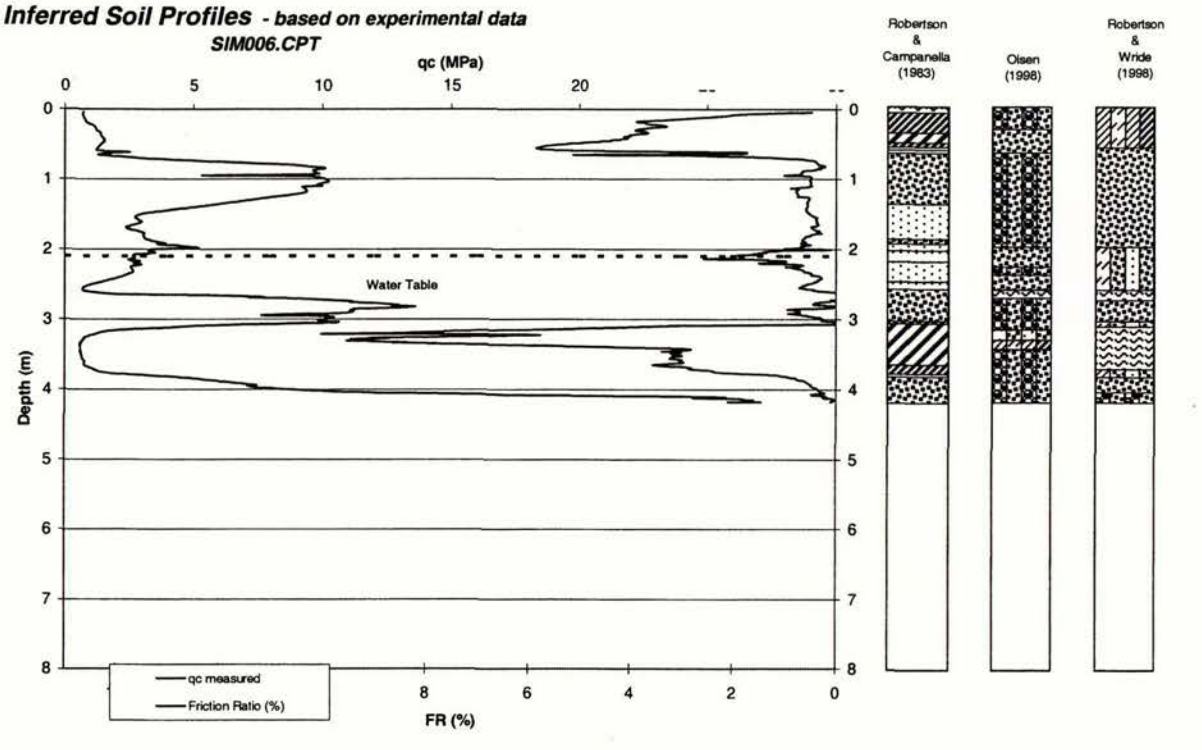
Prediction for Karamea
Simpsons Paddock

1929 Murchison Earthquake
M_L 7.8




Liquefaction Potential - CPT
SIM004.CPT
Prediction for Karamea
Simpsons Paddock
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
Water Table
Depth (m)
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

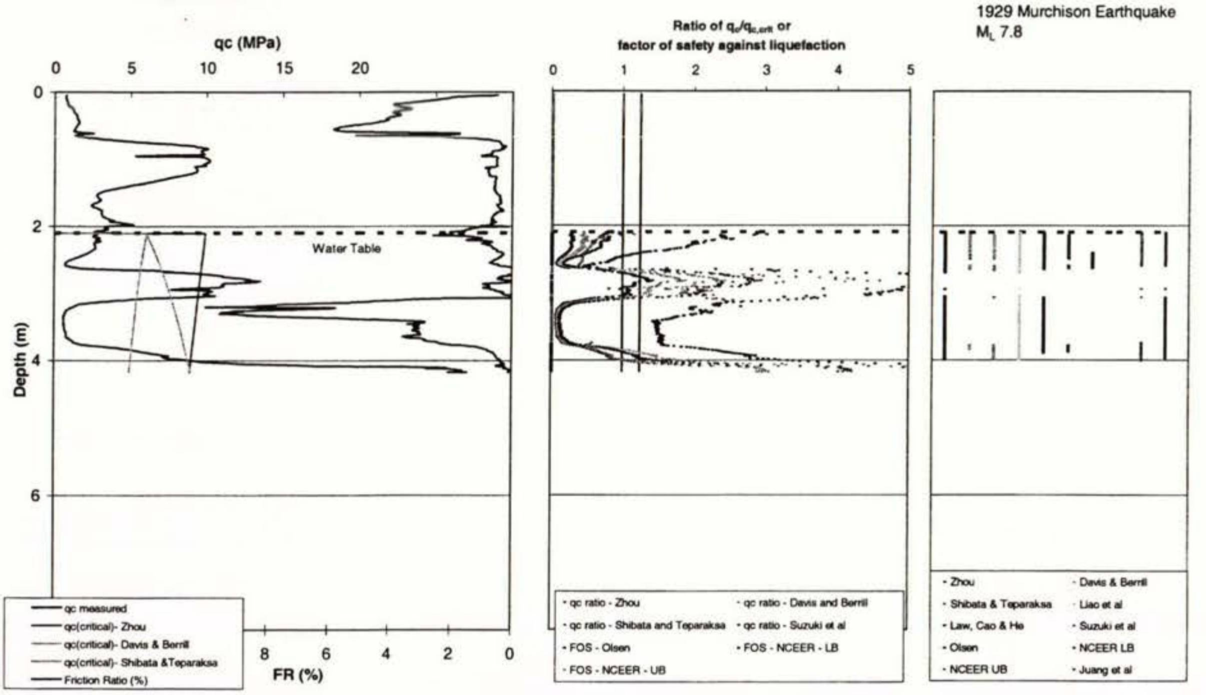
SIM006.CPT


Inferred Soil Profiles - based on experimental data
SIM006.CPT
qc (MPa)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)
Water Table
Depth (m)
qc measured
Friction Ratio (%)
FR (%)

Liquefaction Potential - CPT
SIM006.CPT

Prediction for Karamea
Simpsons Paddock

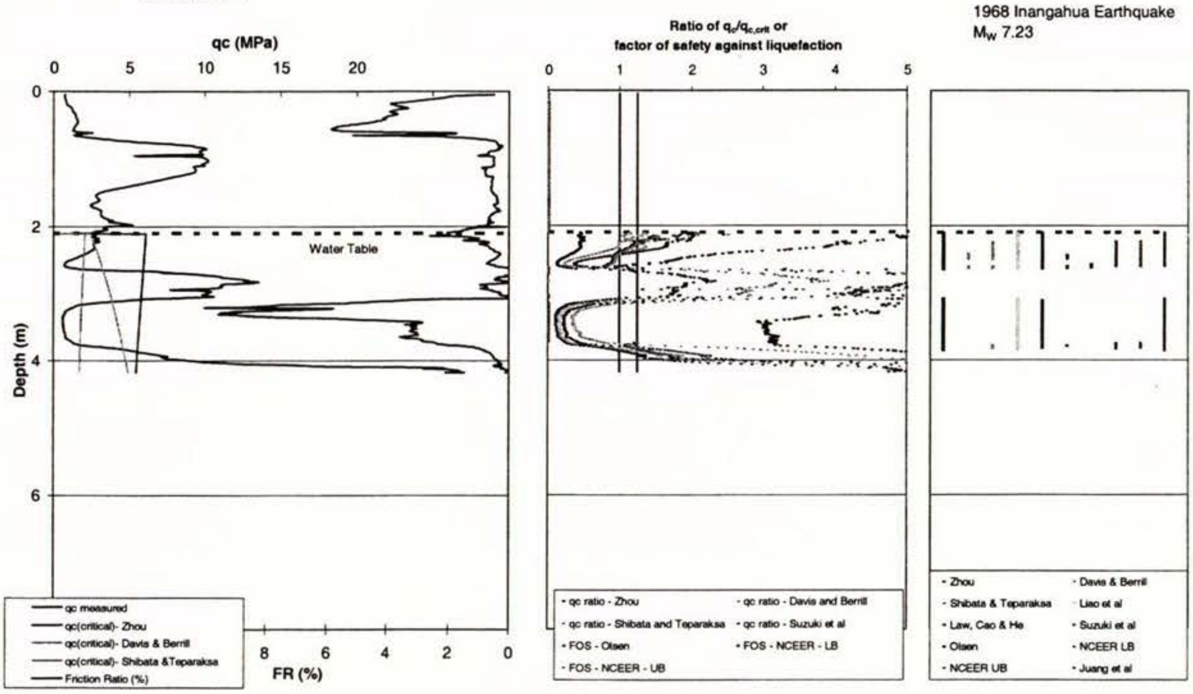
1929 Murchison Earthquake
M_L 7.8



Liquefaction Potential - CPT
SIM006.CPT

Prediction for Karamea
Simpsons Paddock

1968 Inangahua Earthquake
M_W 7.23



A1.3.4 Paddock adjacent the Wharf location

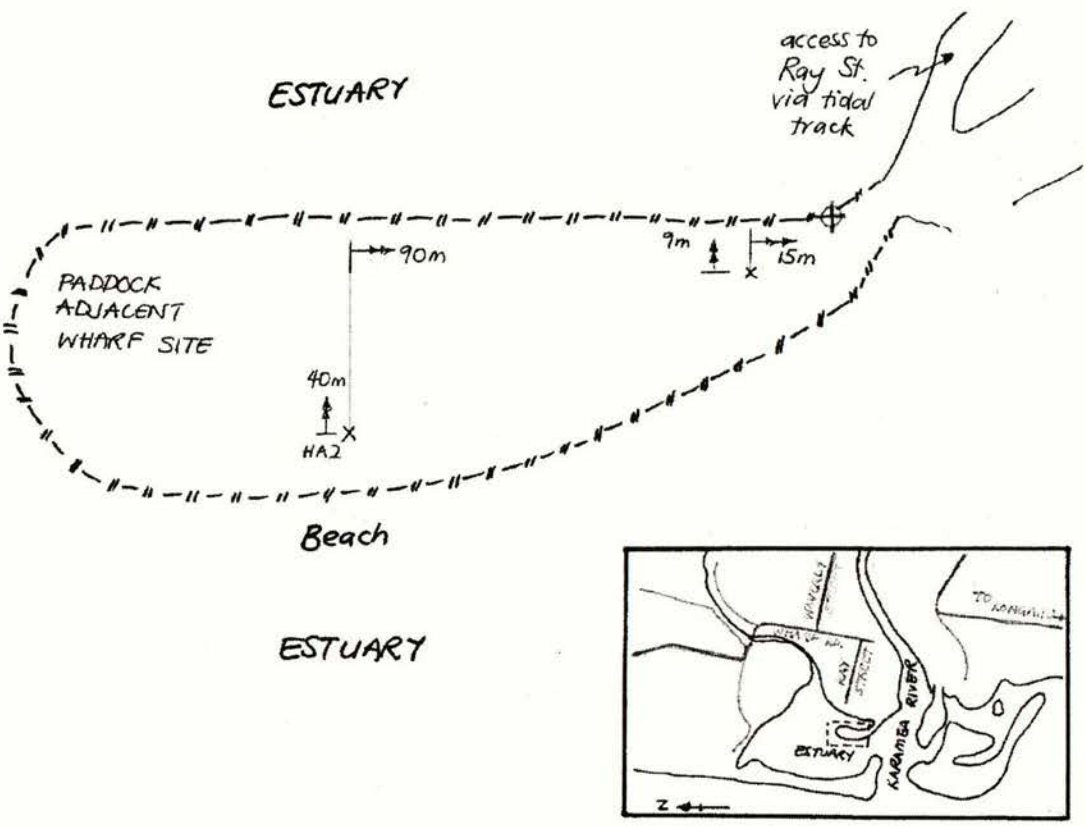
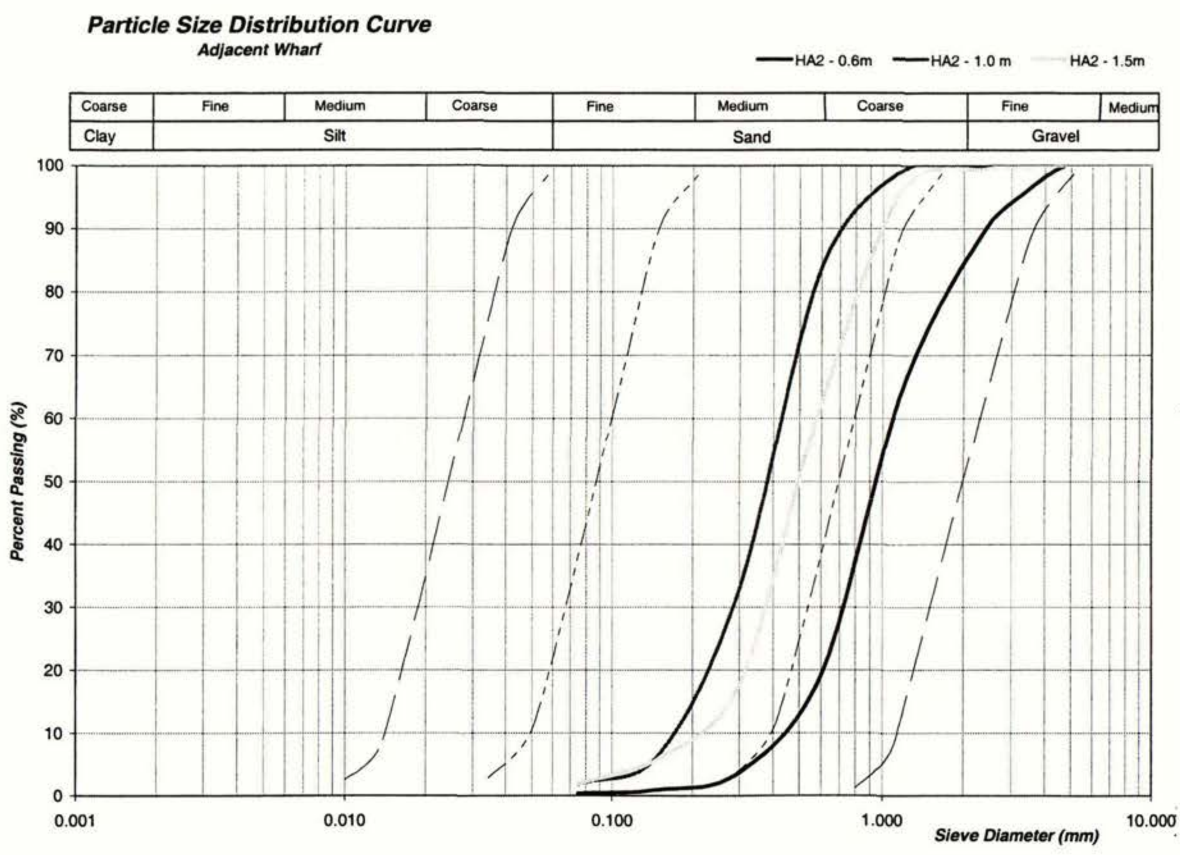


Figure A- 11. Site Diagram indicating locations of tests in the Paddock adjacent the Wharf Site, Karamea



Test Boring Log

Job: Paddock adjacent Wharf
By: Kirsti
Date: 22/02/2003

Type: Hand Auger
Boring Number: 2

DEPTH (m)	Comments
0.00	
0.10	organic material - sandy TOPSOIL
0.20	Very loose dry fine yellow SAND
0.30	
0.40	
0.50	moist medium-coarse SAND
0.60	hole caving in
0.70	
0.80	
0.90	
1.00	
1.10	
1.20	
1.30	
1.40	
1.50	Very Coarse SAND
1.60	
1.70	
1.80	
1.90	
2.00	Pebbly SAND
2.10	
2.20	2.1m Gravel encountered - BORE REFUSED
2.30	
2.40	
2.50	
2.60	
2.70	
2.80	
2.90	
3.00	NB. Did not encounter water table
3.10	
3.20	
3.30	
3.40	
3.50	
3.60	
3.70	
3.80	
3.90	
4.00	

Test Boring Log

Job: Paddock adjacent Wharf
By: Kirsti
Date: 22/02/2003

Type: Hand Auger
Boring Number: 1

DEPTH (m)	Comments
0.00	
0.10	organic material- TOPSOIL
0.20	Coarse yellow- white pebbly SAND
0.30	
0.40	
0.50	
0.60	
0.70	very consistent
0.80	
0.90	
1.00	
1.10	
1.20	
1.30	
1.40	
1.50	
1.60	
1.70	
1.80	
1.90	
2.00	1.90m hole caving in - BORE TERMINATED
2.10	
2.20	
2.30	
2.40	
2.50	
2.60	
2.70	
2.80	
2.90	
3.00	
3.10	
3.20	
3.30	
3.40	
3.50	
3.60	
3.70	
3.80	
3.90	
4.00	

A1.3.5 Paddock near Wharf

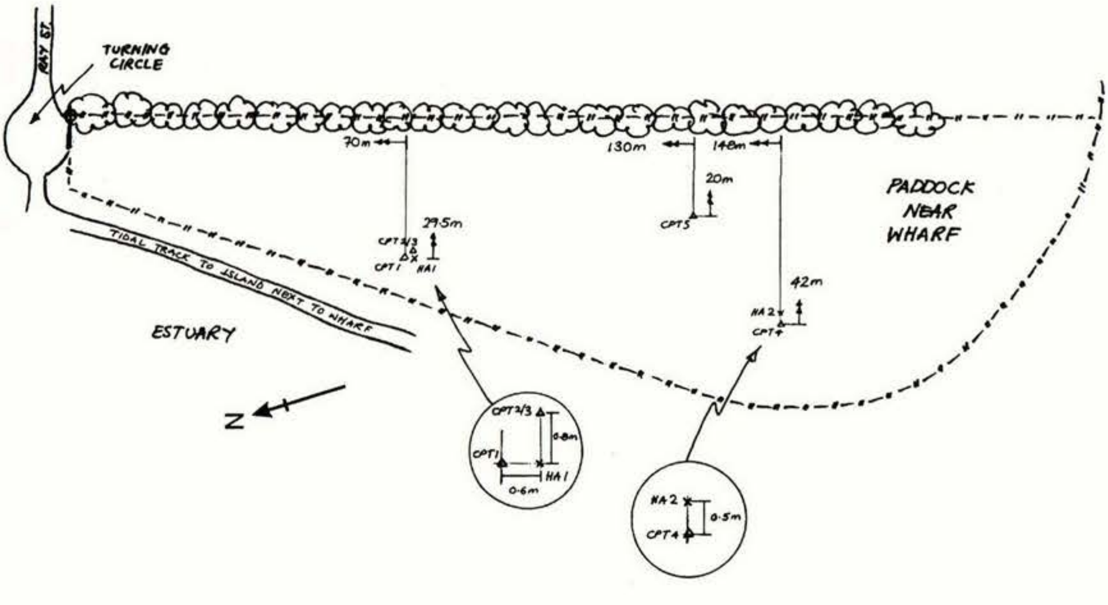
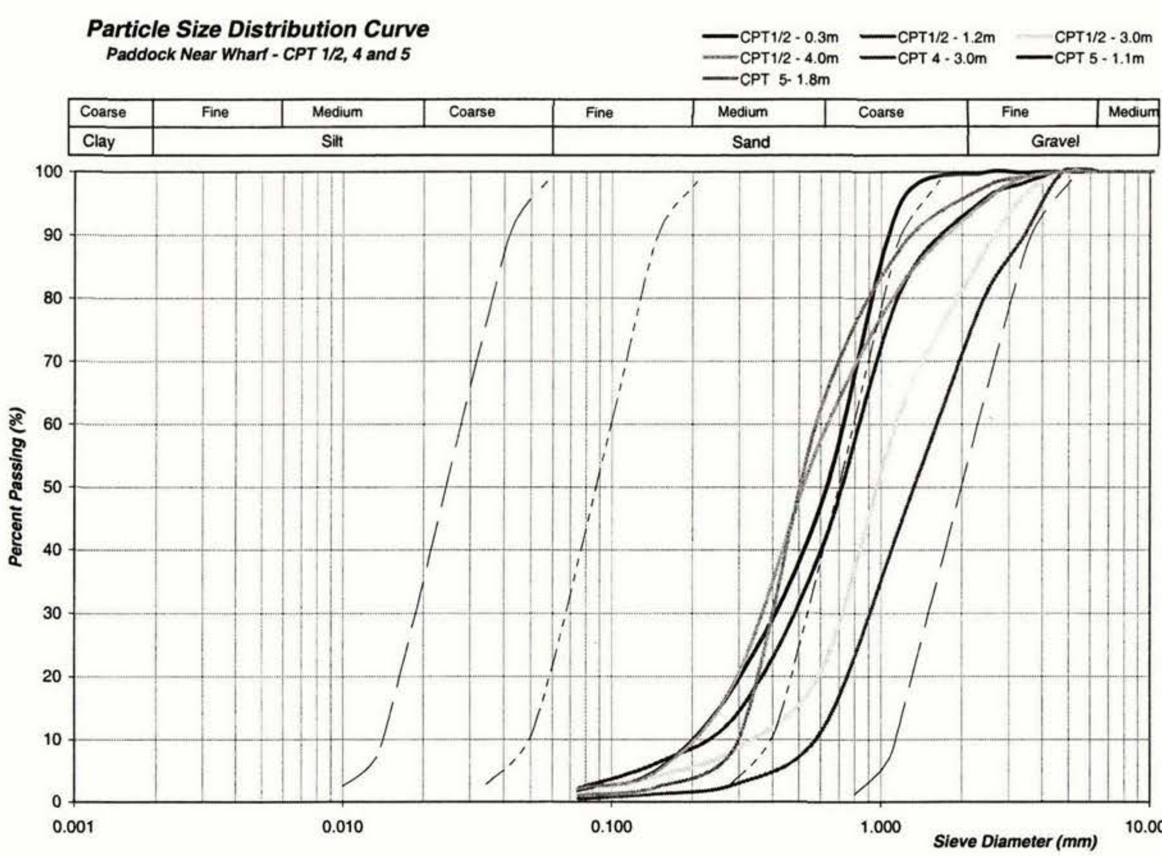


Figure A- 12. Site Diagram indicating locations of tests in the Paddock near the Wharf Site, Karamea

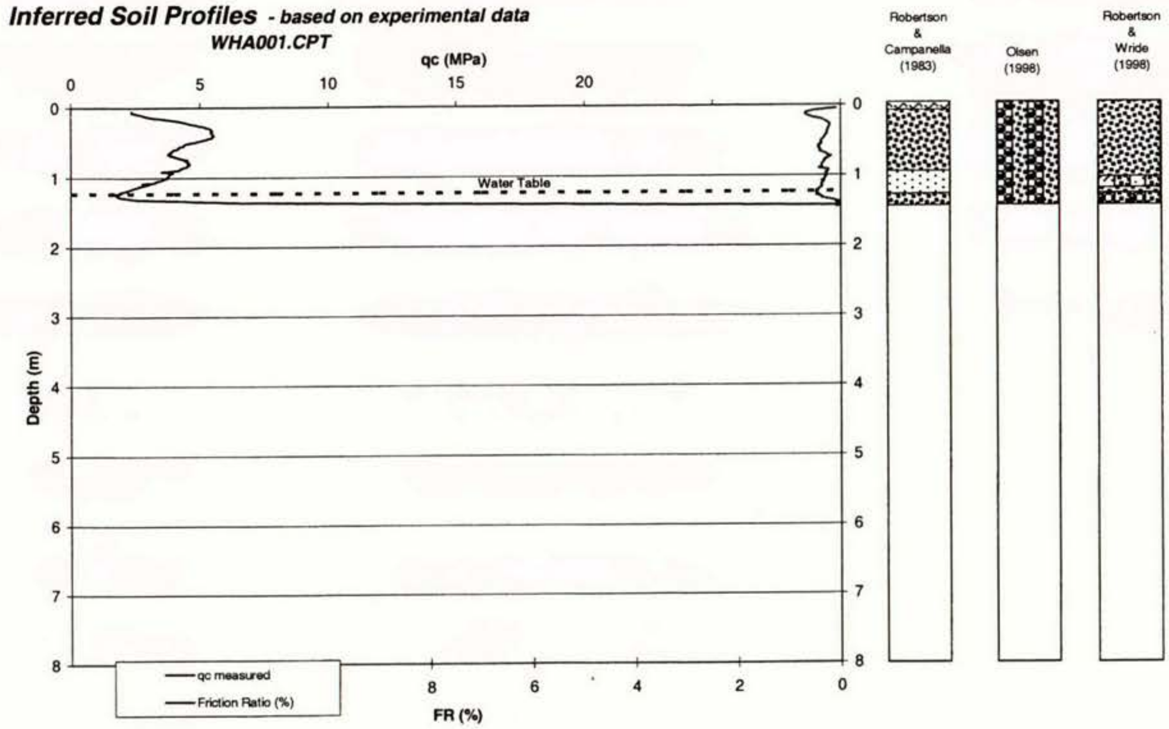


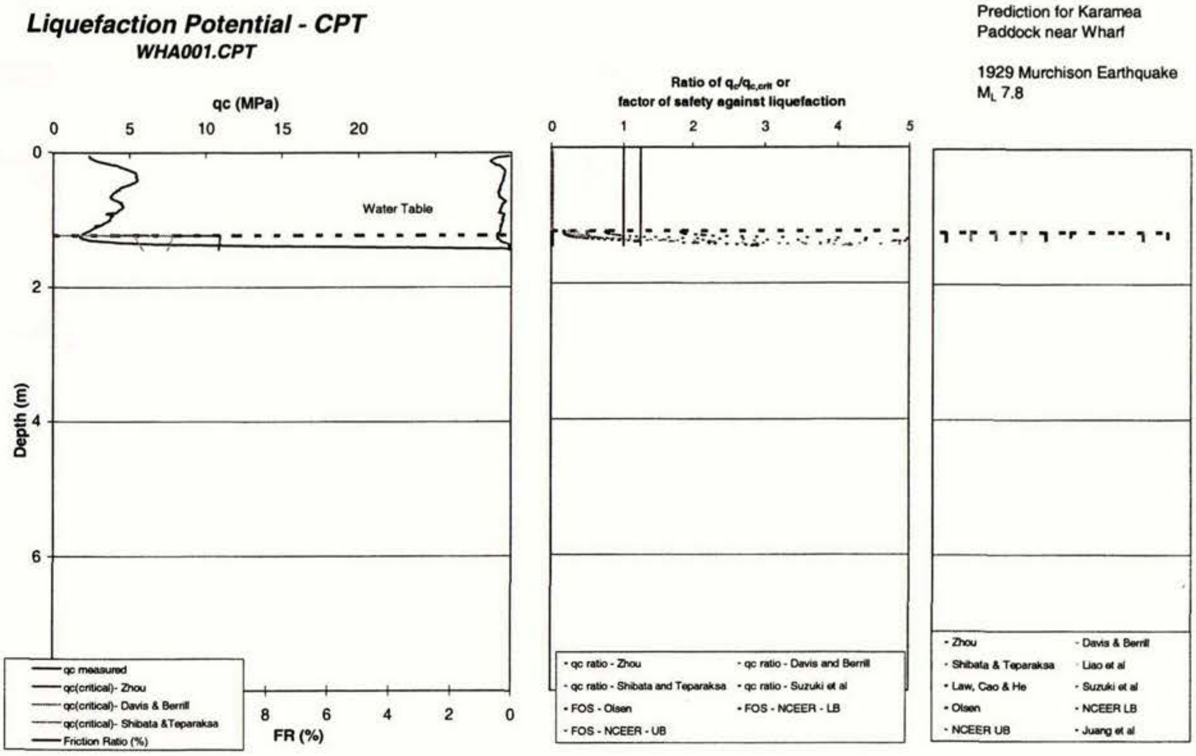
WHA001.CPT

Test Boring Log

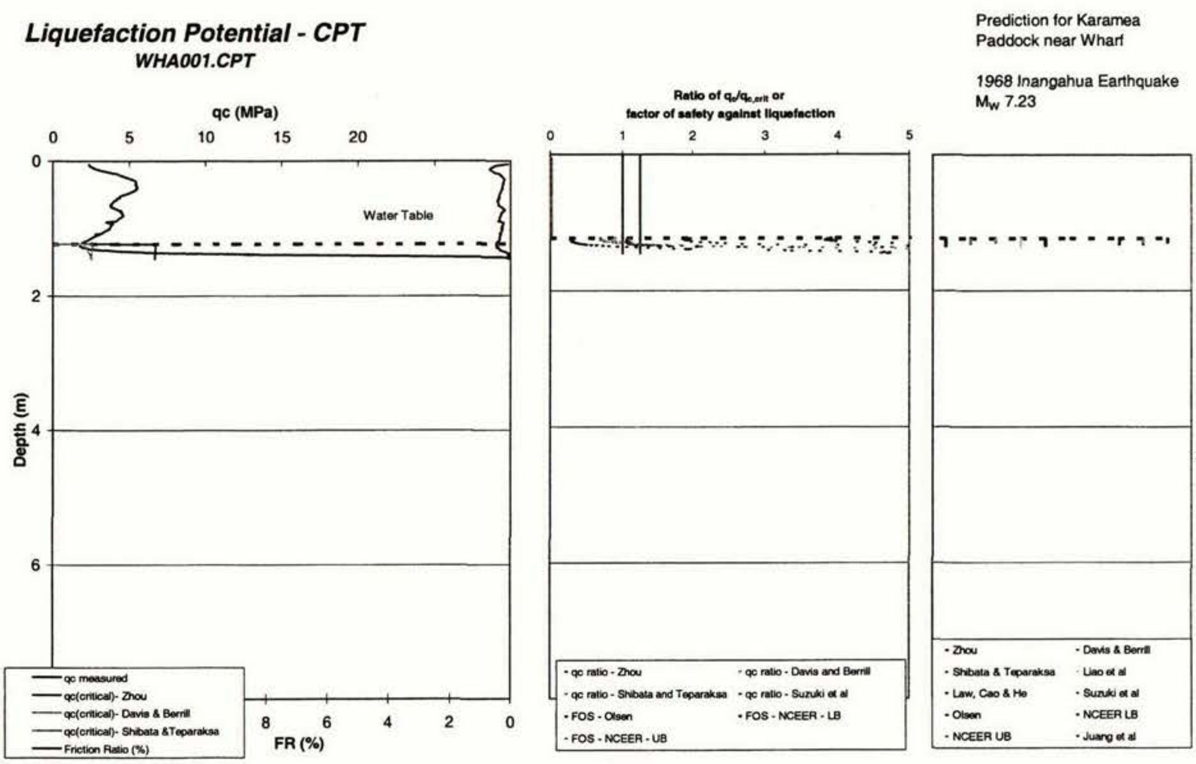
Job: Paddock near Wharf
By: Kirsti
Date: 22/02/2003 4:05pm
Type: Hand Auger
Boring Number: 1

DEPTH (m)	Comments
0.00	
0.10	Dark brown sandy TOPSOIL
0.20	Coarse Brown SAND
0.30	moist and contains silica
0.40	
0.50	
0.60	
0.70	
0.80	
0.90	
1.00	
1.10	
1.20	
1.30	
1.40	
1.50	
1.60	Pebbly SAND containing organic material ie. Wood
1.70	1.60m BORE REFUSED
1.80	
1.90	
2.00	
2.10	
2.20	
2.30	
2.40	
2.50	
2.60	
2.70	
2.80	
2.90	
3.00	
3.10	
3.20	NB. Sand left on ground surface
3.30	from flight auger was white
3.40	
3.50	
3.60	
3.70	
3.80	
3.90	
4.00	



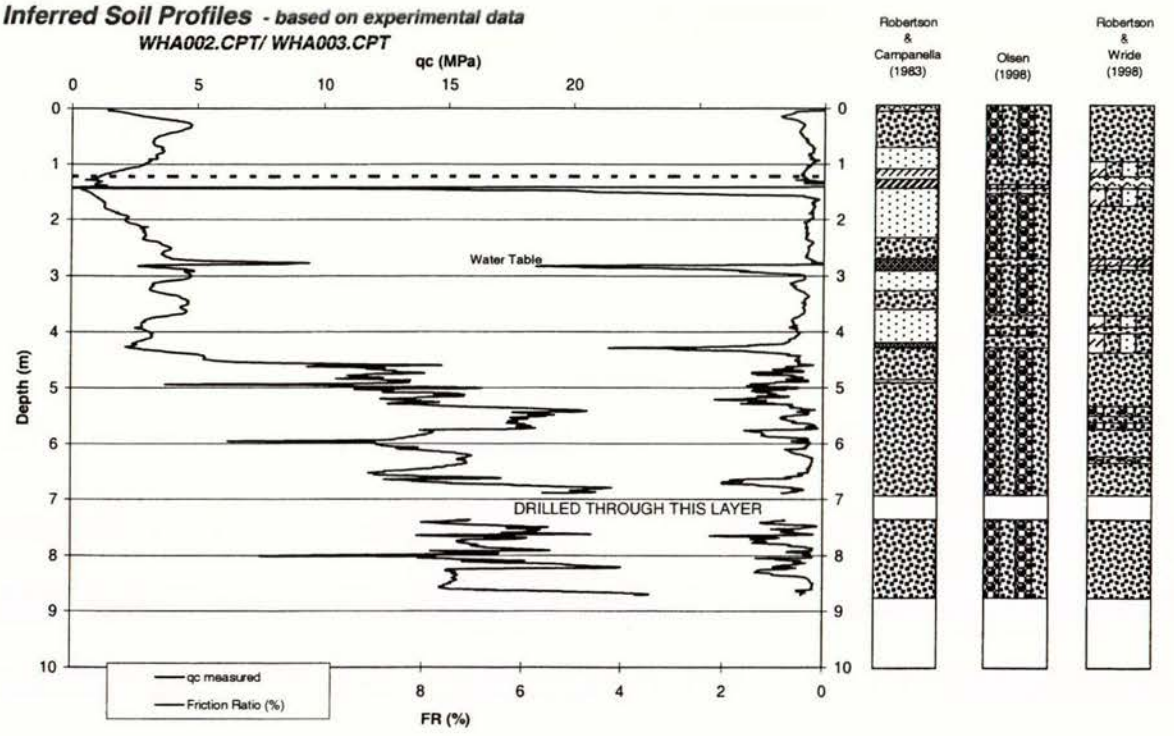


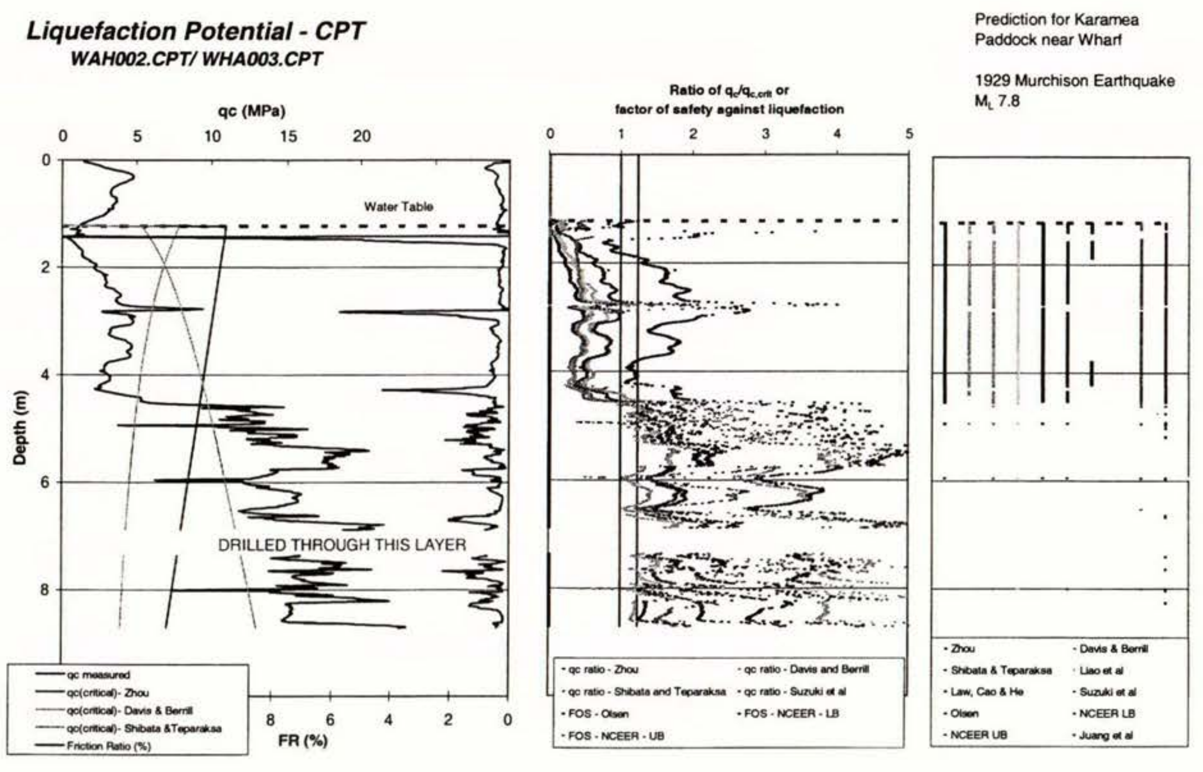
Liquefaction Potential - CPT
WHA001.CPT
Prediction for Karamea
Paddock near Wharf
1929 Murchison Earthquake
M_L 7.8
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
0 2 4 6
8 6 4 2 0
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

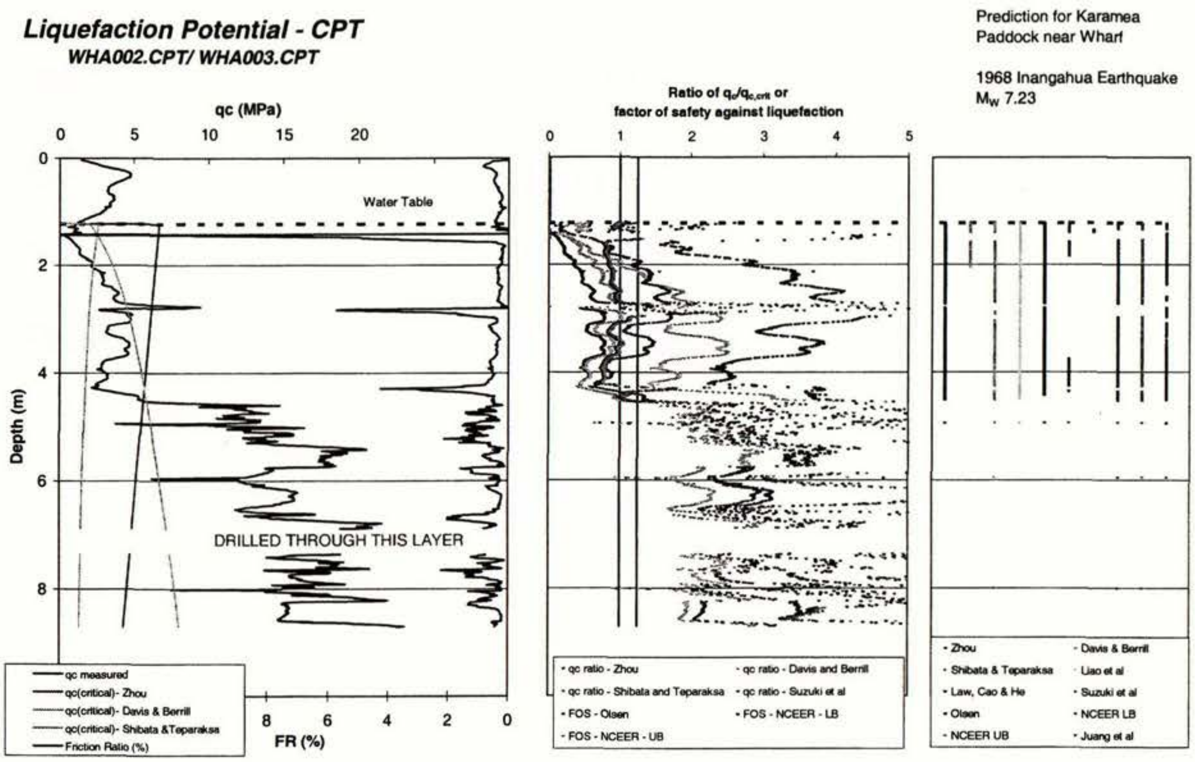


Liquefaction Potential - CPT
WHA001.CPT
Prediction for Karamea
Paddock near Wharf
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
0 2 4 6
8 6 4 2 0
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

WHA002.CPT / WHA003.CPT







WHA004.CPT

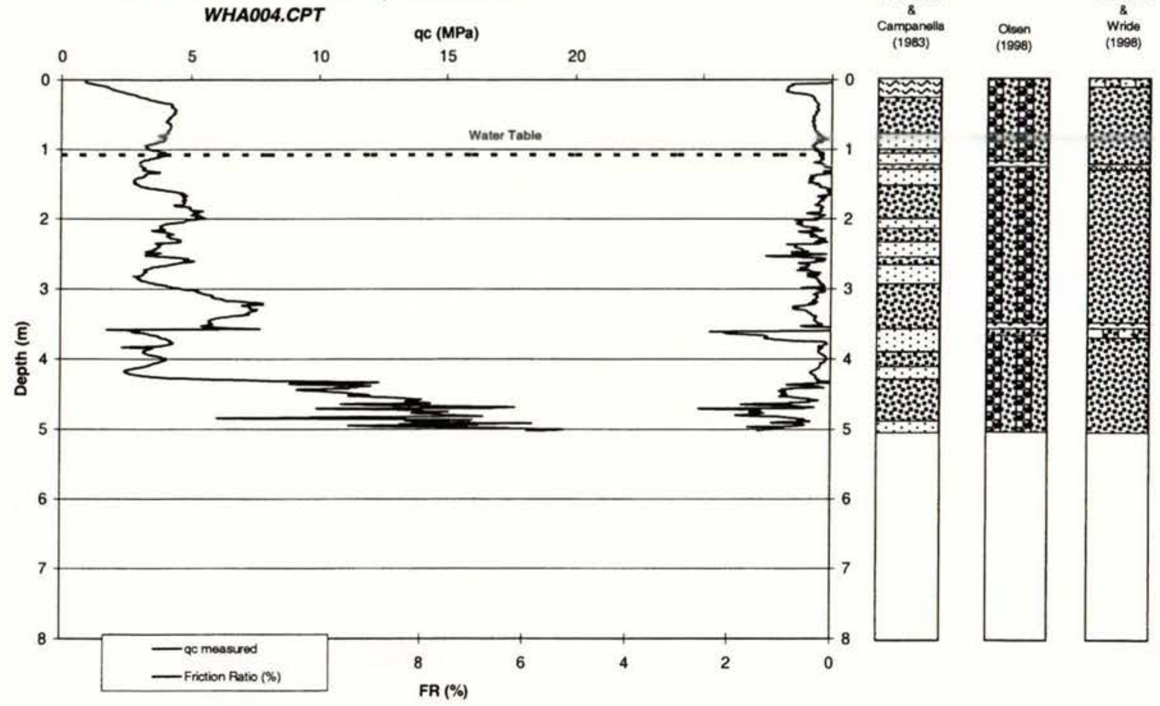
Test Boring Log

Job: Paddock near Wharf
By: Kirsti
Date: 22/02/2003

Type: Hand Auger
Boring Number: 2

DEPTH (m)	Comments
0.00 – 0.30	Dark brown sandy TOPSOIL Loose, dry fine yellow SAND
0.30 – 2.00	Moist, medium yellow SAND Coarse yellow SAND
2.00 – 2.10	Pebbly SAND
2.10 – 4.00	2.1m Encountered GRAVEL - BORE REFUSED NB. Did not encounter water table

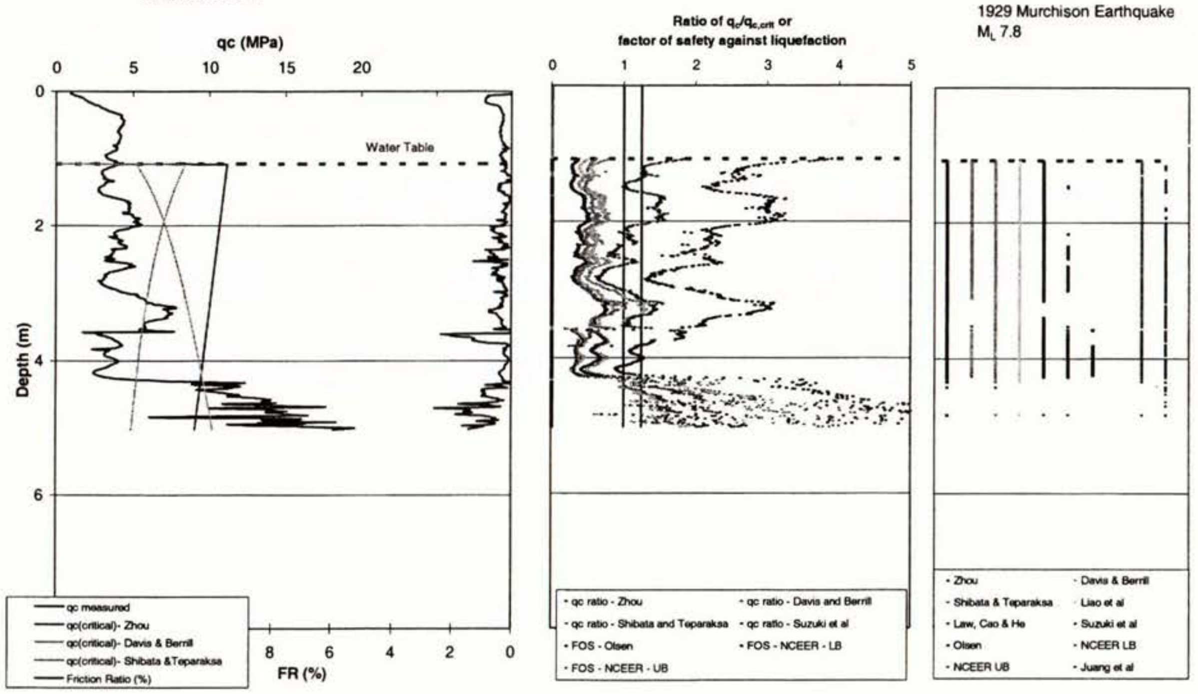
Inferred Soil Profiles - based on experimental data
WHA004.CPT

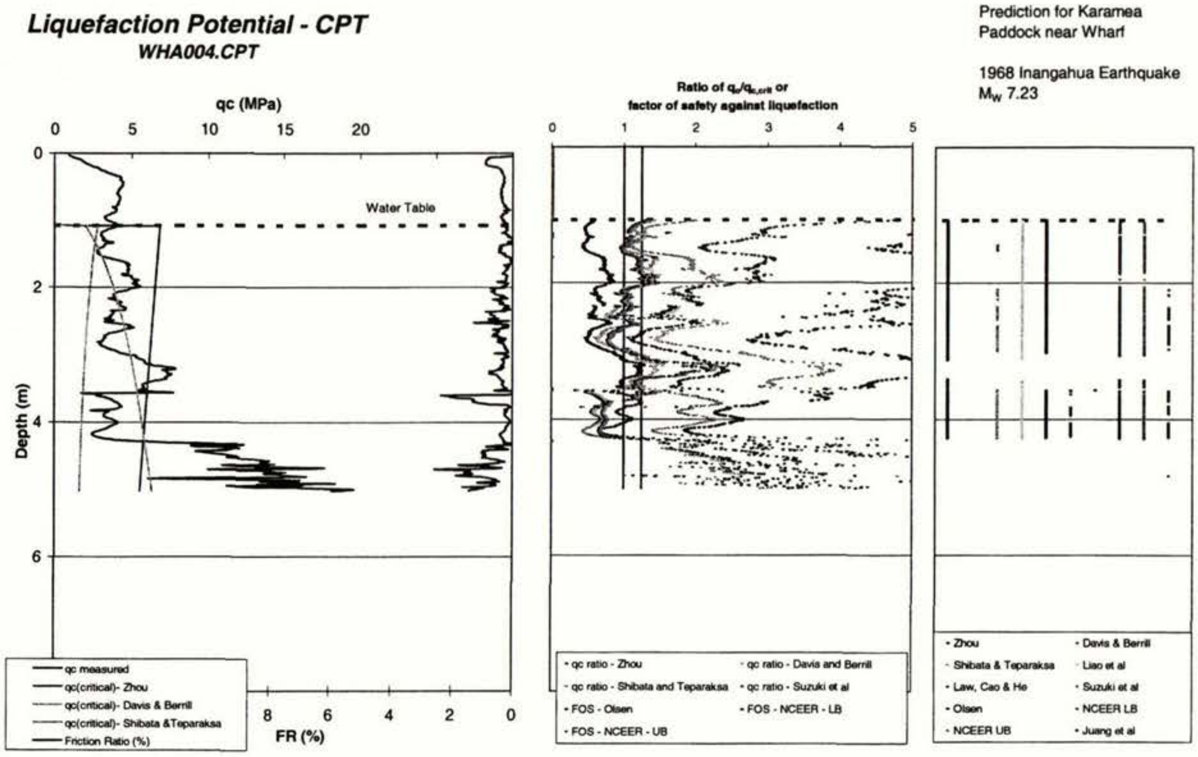


Liquefaction Potential - CPT
WHA004.CPT

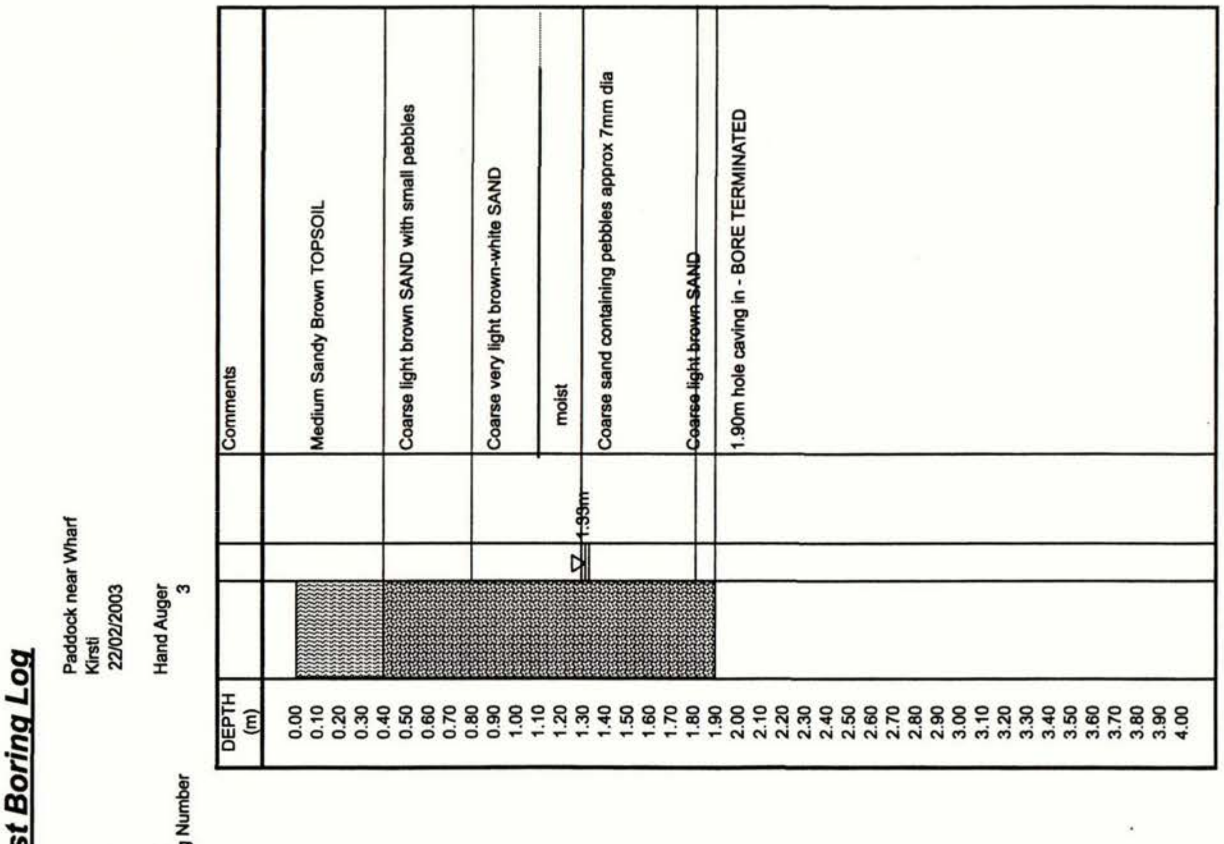
Prediction for Karamea Paddock near Wharf

1929 Murchison Earthquake M_L 7.8

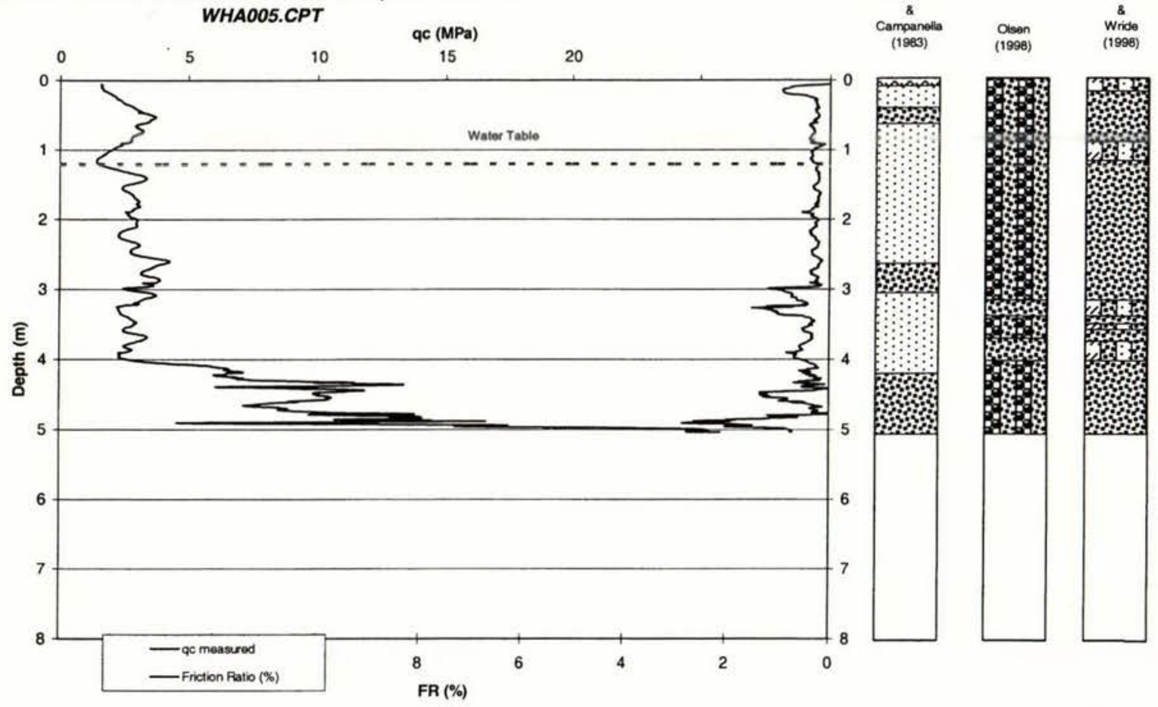



Liquefaction Potential - CPT
WHA004.CPT
Prediction for Karamea
Paddock near Wharf
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
Depth (m)
Water Table
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

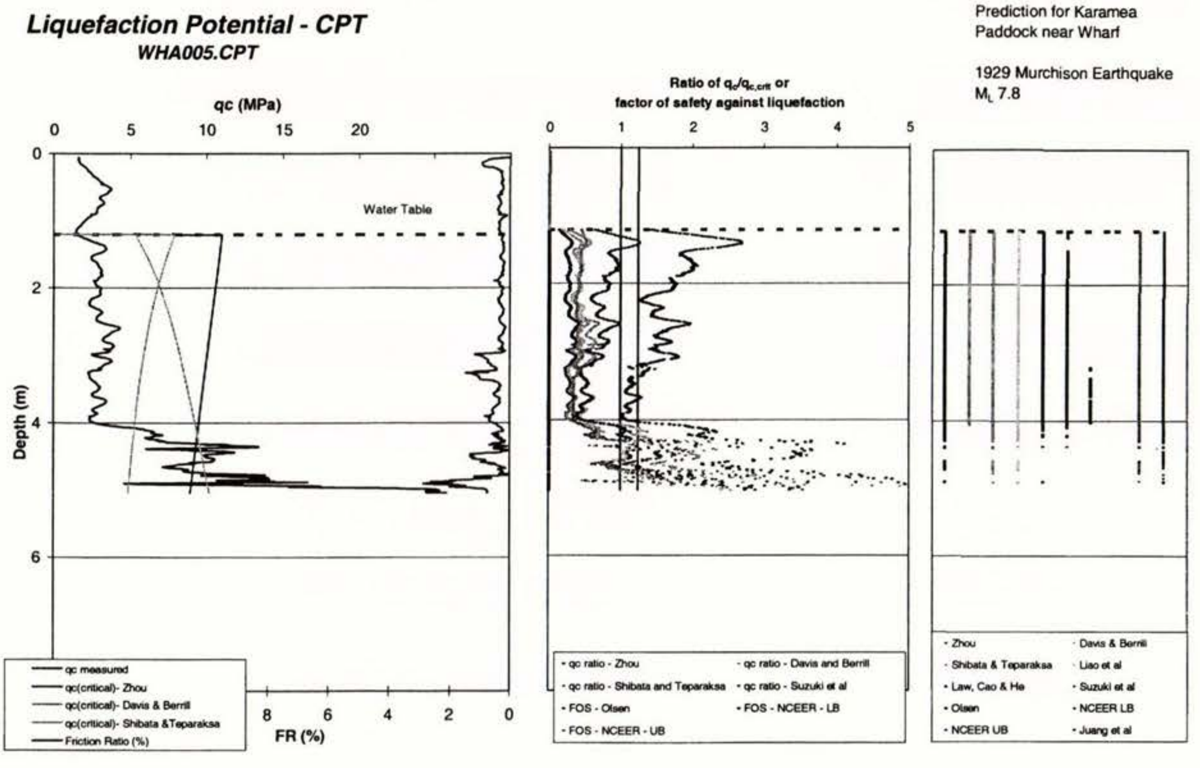
WHA005.CPT

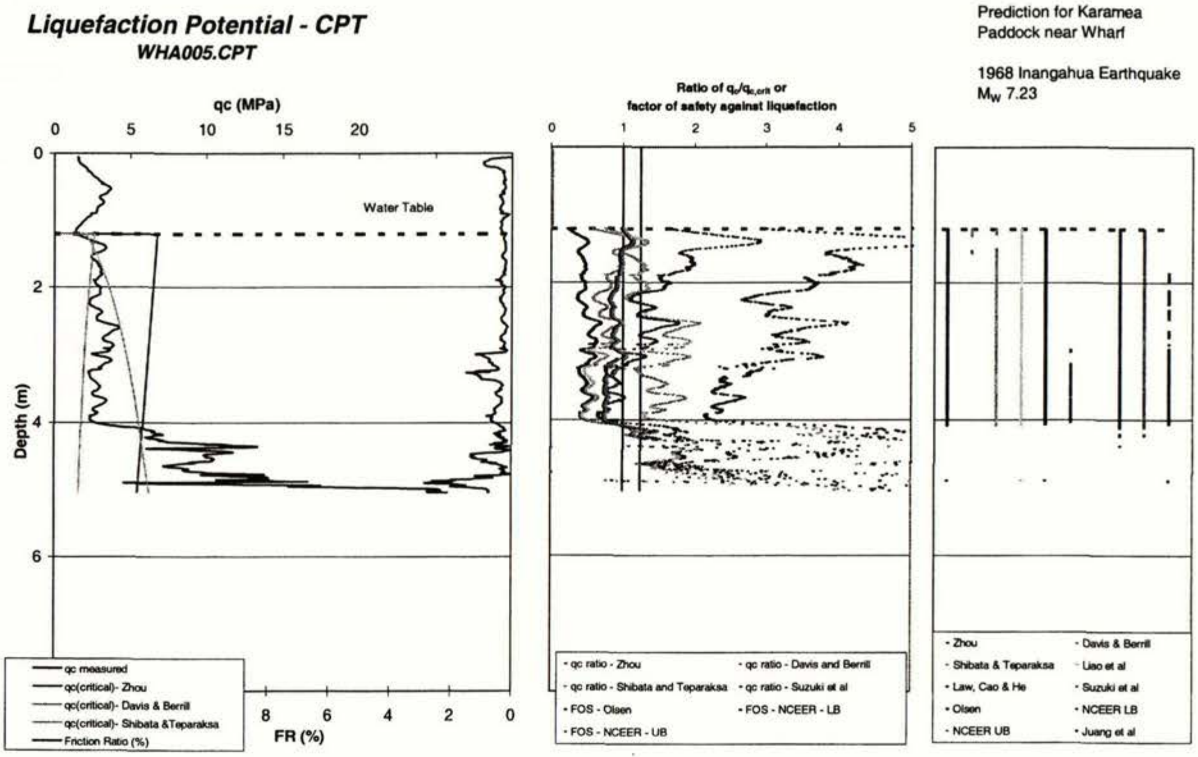

Test Boring Log
Job Paddock near Wharf
By Kirsti
Date 22/02/2003
Type Hand Auger
Boring Number 3
DEPTH (m)
Comments
Medium Sandy Brown TOPSOIL
Coarse light brown SAND with small pebbles
Coarse very light brown-white SAND
moist
Coarse sand containing pebbles approx 7mm dia
Coarse light brown SAND
1.90m hole caving in - BORE TERMINATED
1.33m

Inferred Soil Profiles - based on experimental data
WHA005.CPT



Liquefaction Potential - CPT
WHA005.CPT





A1.3.6 Oparara

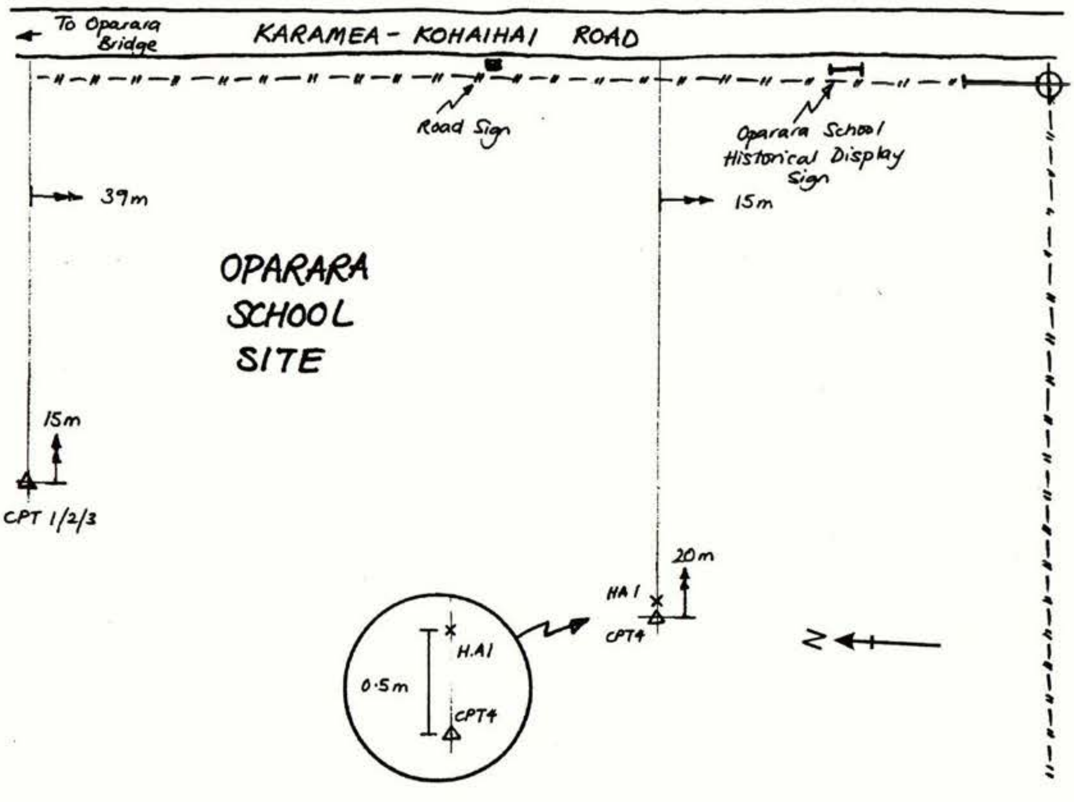
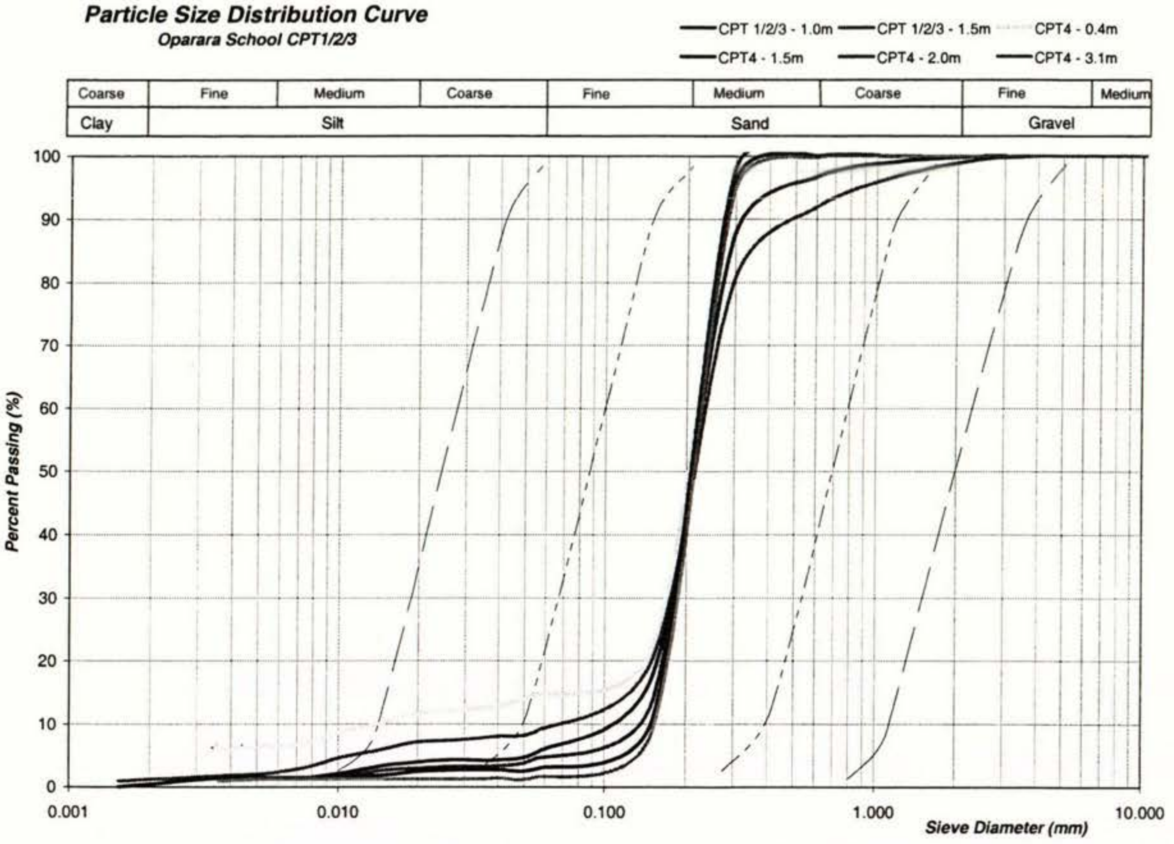
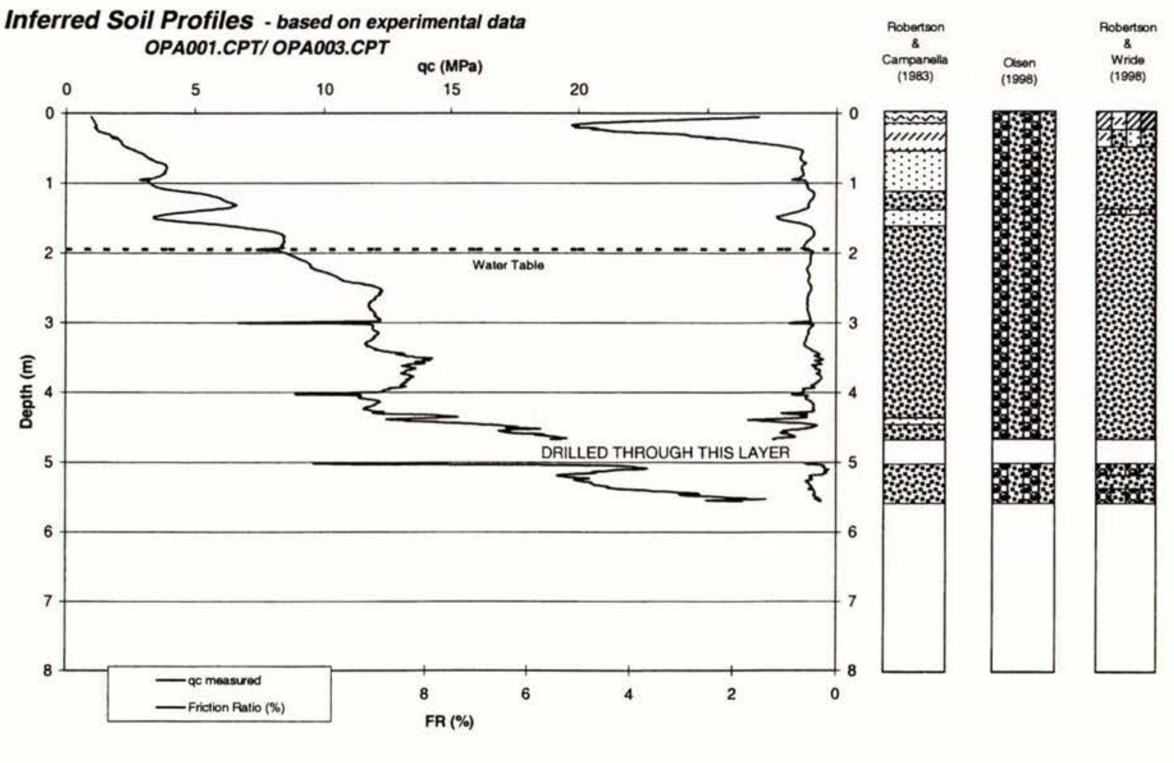
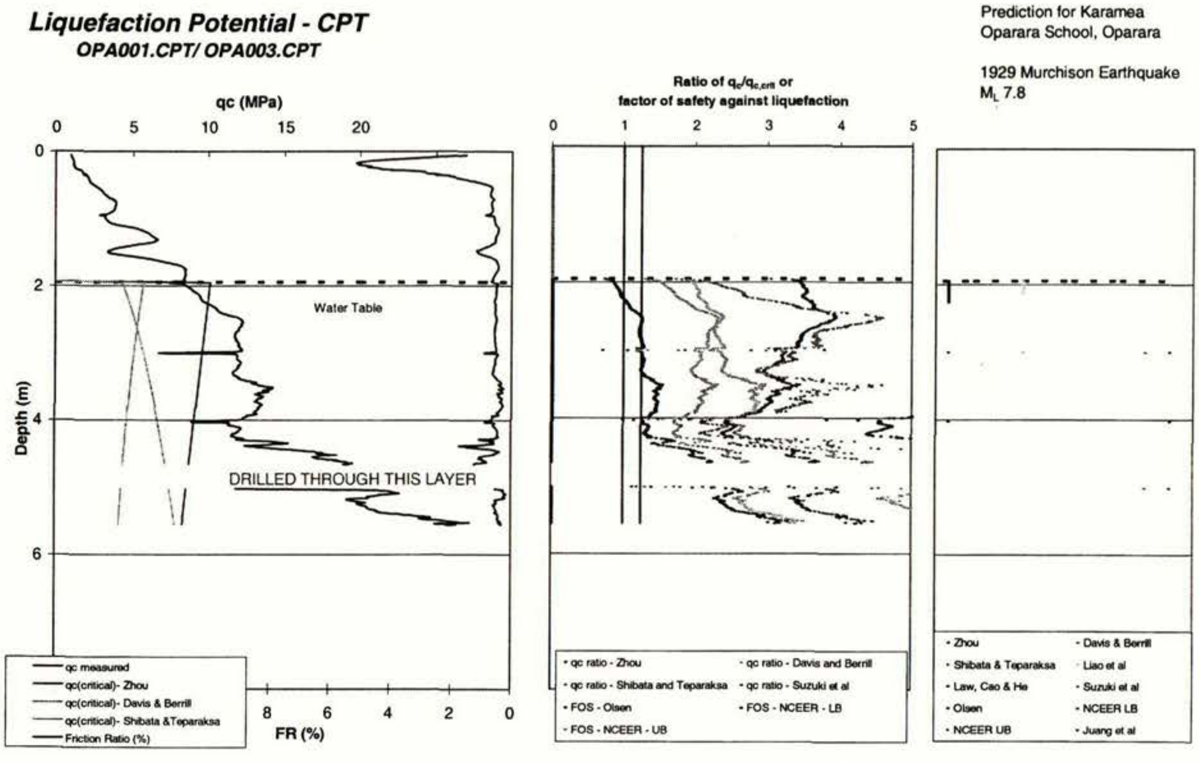


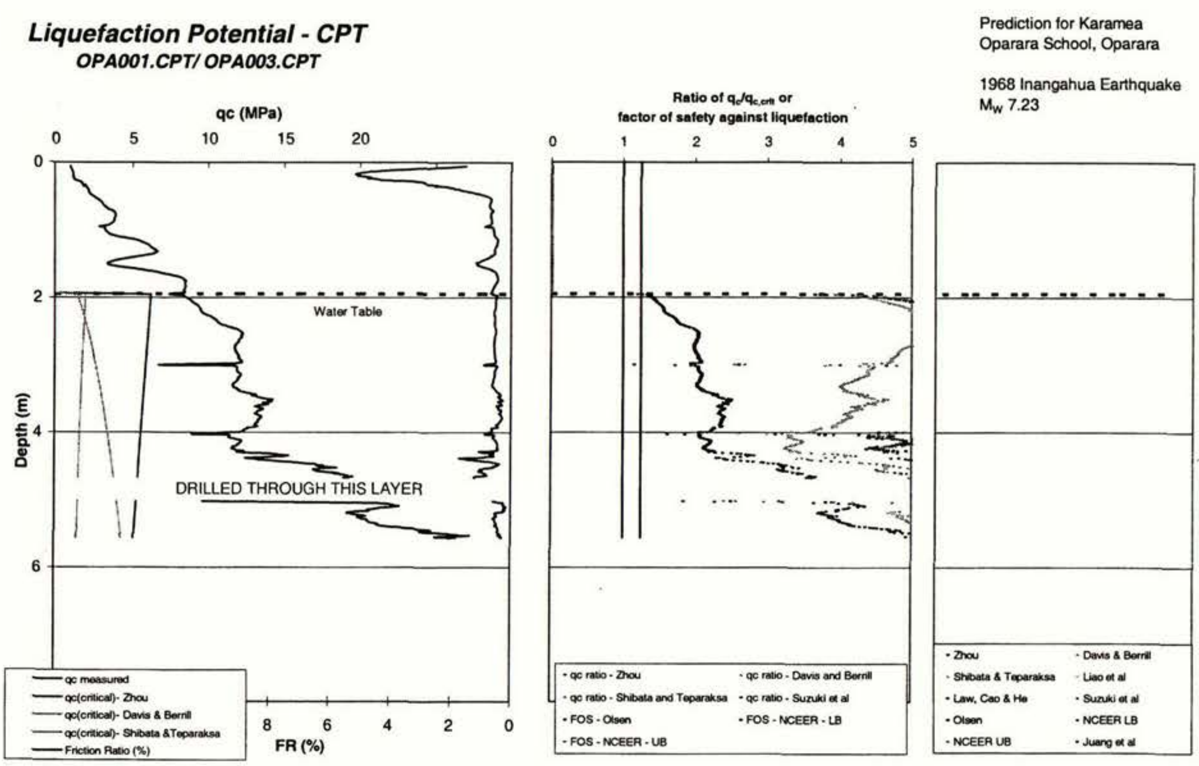
Figure A- 13. Site diagram indicating test locations at Oparara School



OPA001.CPT / OPA003.CPT




Liquefaction Potential - CPT
OPA001.CPT/ OPA003.CPT
Prediction for Karamea
Oparara School, Oparara
1929 Murchison Earthquake
M_L 7.8
qc (MPa)
0
5
10
15
20
Depth (m)
Water Table
DRILLED THROUGH THIS LAYER
FR (%)
8
6
4
2
0
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
- qc ratio - Zhou
- qc ratio - Davis and Berrill
- qc ratio - Shibata and Teparaksa
- qc ratio - Suzuki et al
- FOS - Olsen
- FOS - NCEER - LB
- FOS - NCEER - UB
- Zhou
- Davis & Berrill
- Shibata & Teparaksa
- Liao et al
- Law, Cao & He
- Suzuki et al
- Olsen
- NCEER LB
- NCEER UB
- Juang et al

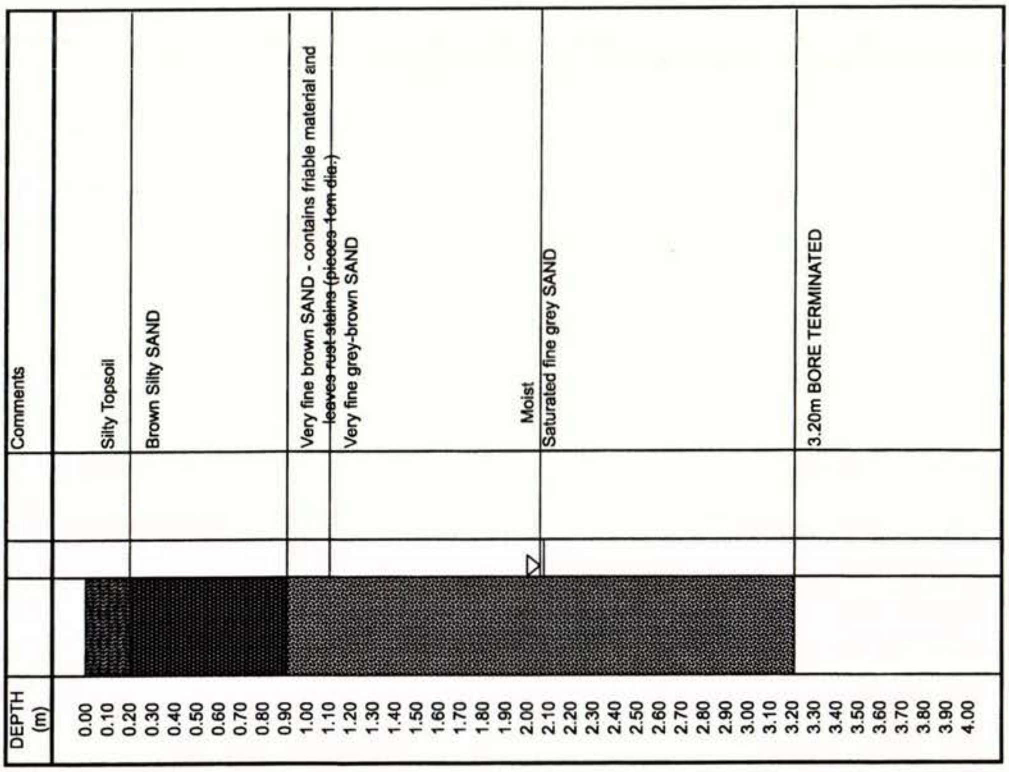

Liquefaction Potential - CPT
OPA001.CPT/ OPA003.CPT
Prediction for Karamea
Oparara School, Oparara
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0
5
10
15
20
Depth (m)
Water Table
DRILLED THROUGH THIS LAYER
FR (%)
8
6
4
2
0
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
- qc ratio - Zhou
- qc ratio - Davis and Berrill
- qc ratio - Shibata and Teparaksa
- qc ratio - Suzuki et al
- FOS - Olsen
- FOS - NCEER - LB
- FOS - NCEER - UB
- Zhou
- Davis & Berrill
- Shibata & Teparaksa
- Liao et al
- Law, Cao & He
- Suzuki et al
- Olsen
- NCEER LB
- NCEER UB
- Juang et al

OPA004.CPT

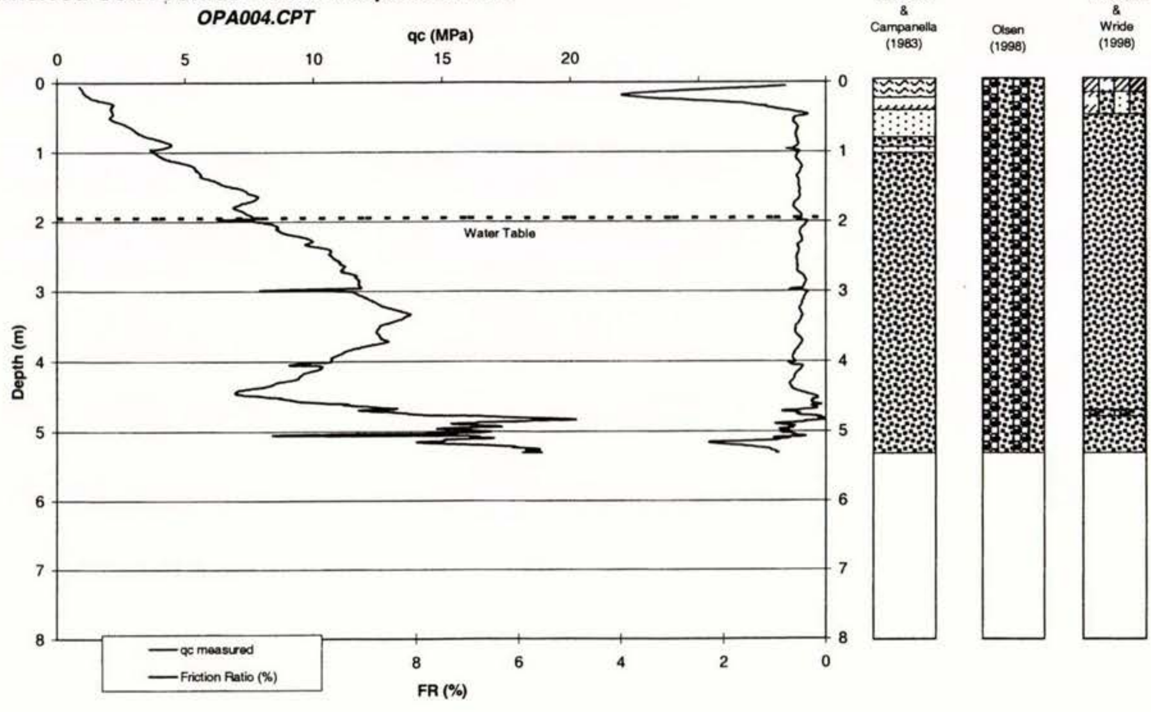
Test Boring Log

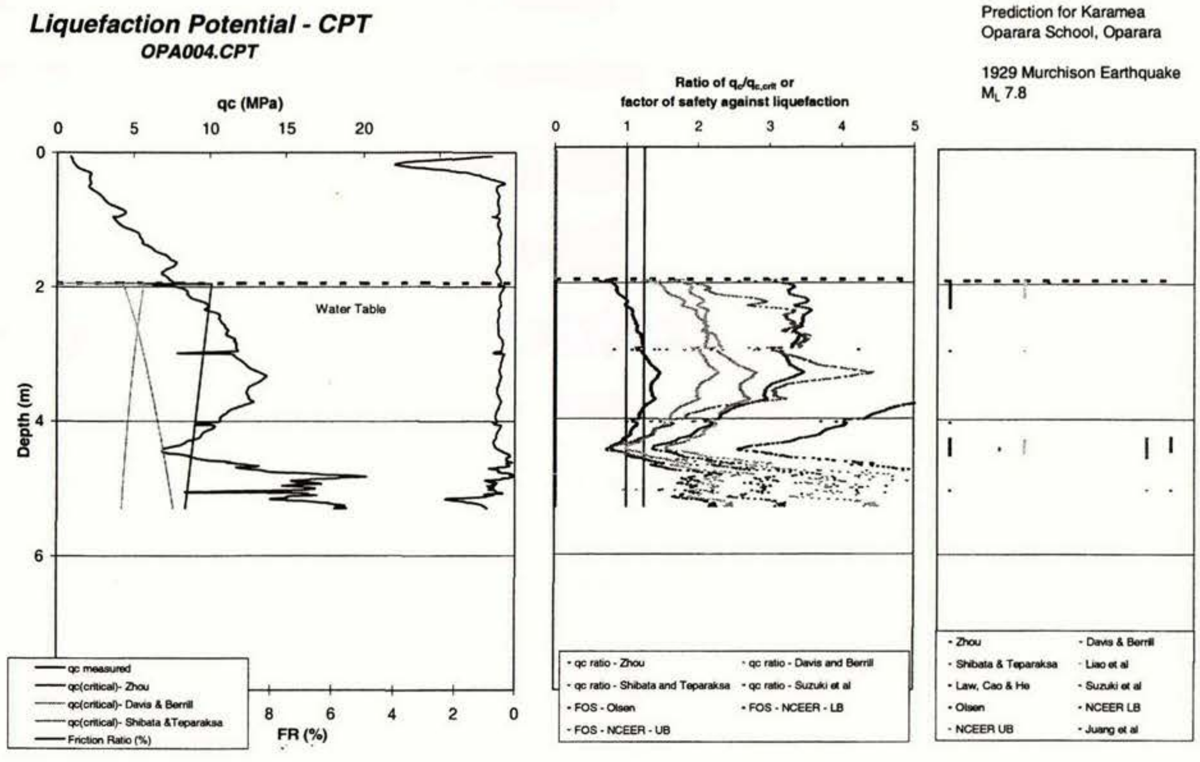
Job Oparara
By Kirsti, Alex
Date 22/02/2003

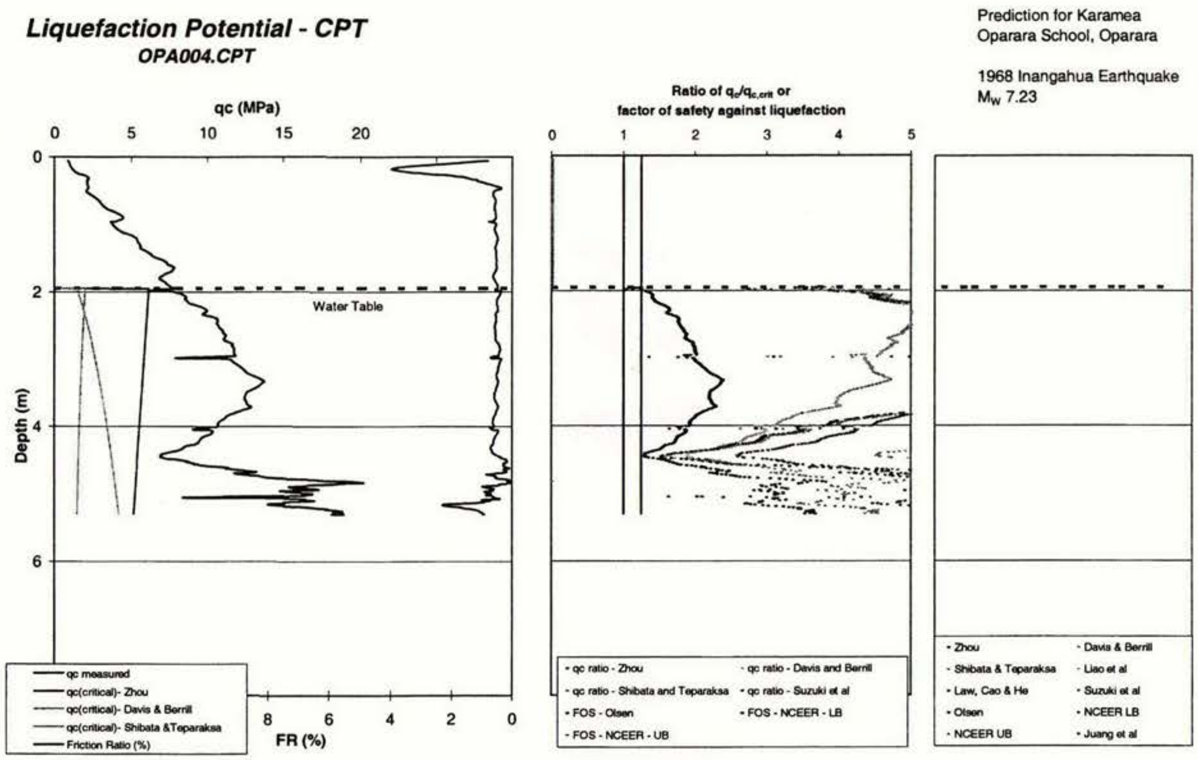
Type Hand Auger
Boring Number 1



***Inferred Soil Profiles** - based on experimental data*
OPA004.CPT



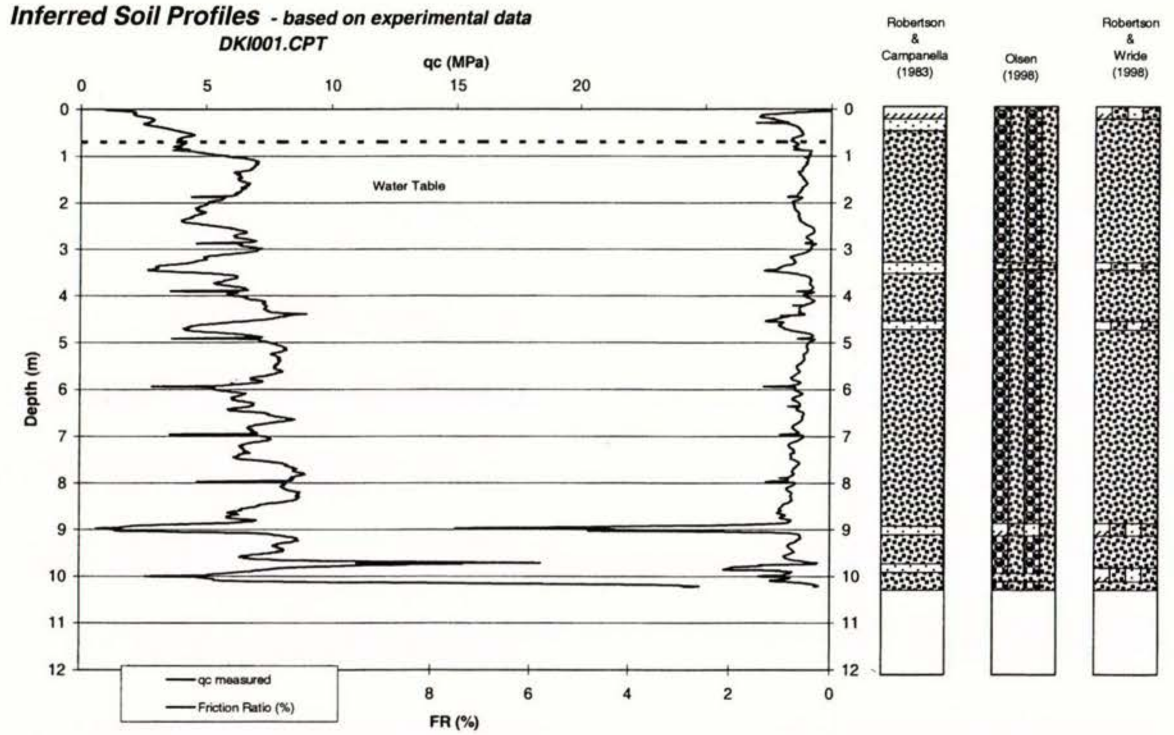

Liquefaction Potential - CPT
OPA004.CPT
Prediction for Karamea
Oparara School, Oparara
1929 Murchison Earthquake
M_L 7.8
qc (MPa)
0
5
10
15
20
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0
1
2
3
4
5
Depth (m)
0
2
4
6
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8
6
4
2
0
FR (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

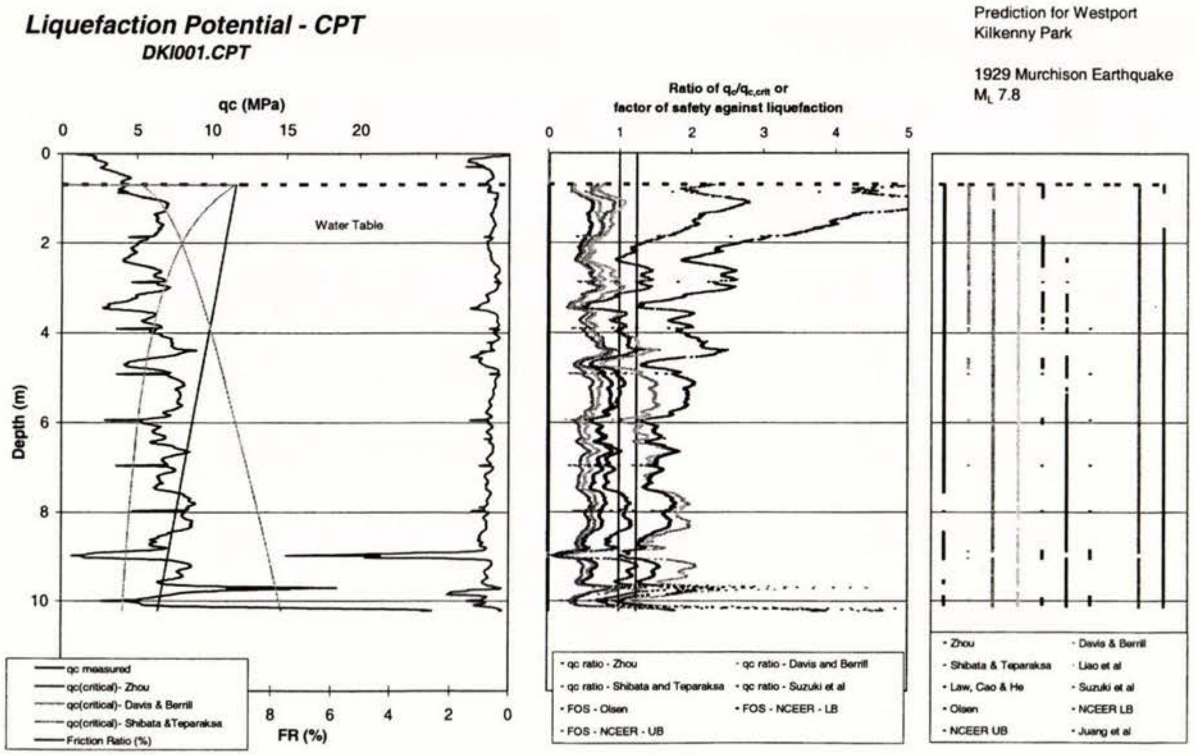

Liquefaction Potential - CPT
OPA004.CPT
Prediction for Karamea
Oparara School, Oparara
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0
5
10
15
20
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0
1
2
3
4
5
Depth (m)
0
2
4
6
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8
6
4
2
0
FR (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

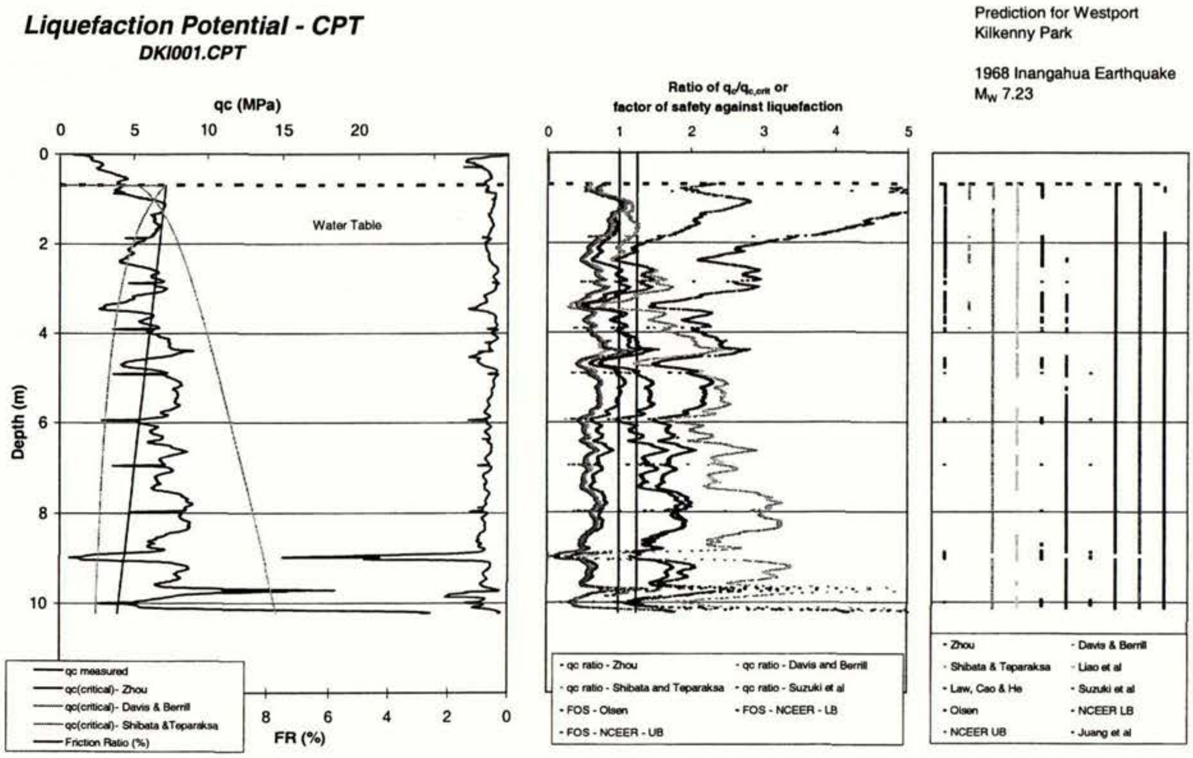
Appendix B – Analysis of CPT data from Dou (1992)

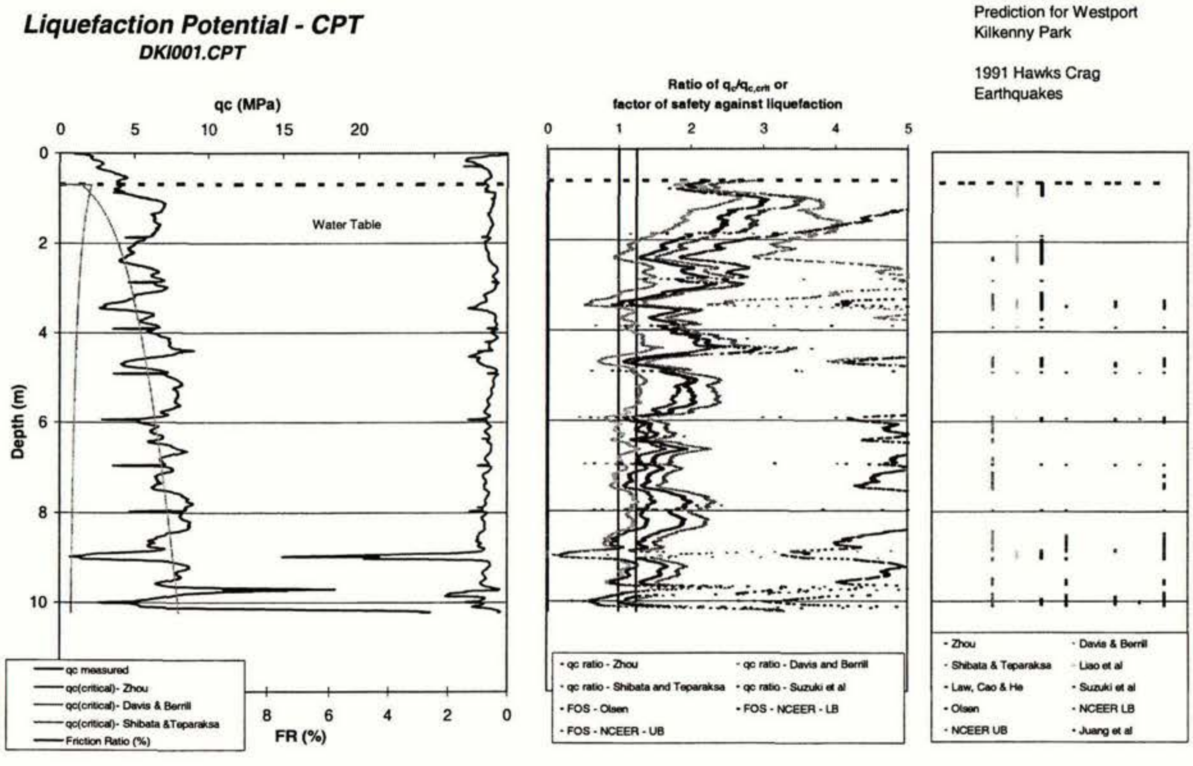
B1.1 Westport- Kilkenny Park

KIL001.CPT

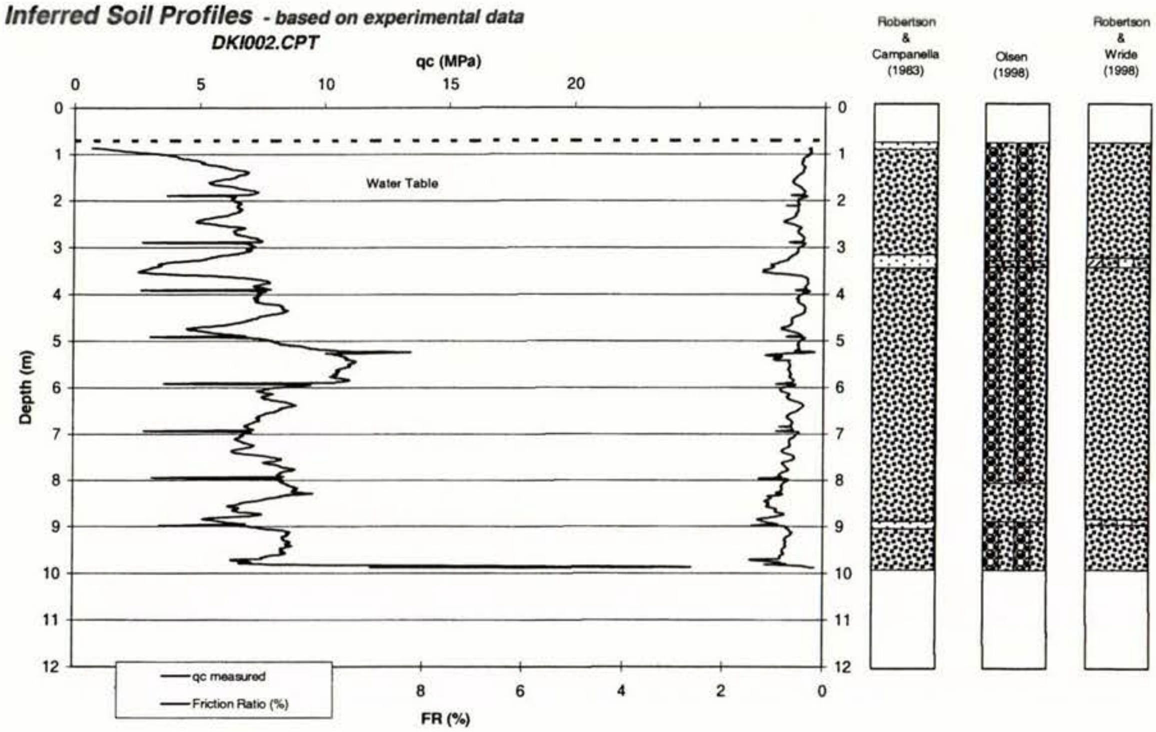




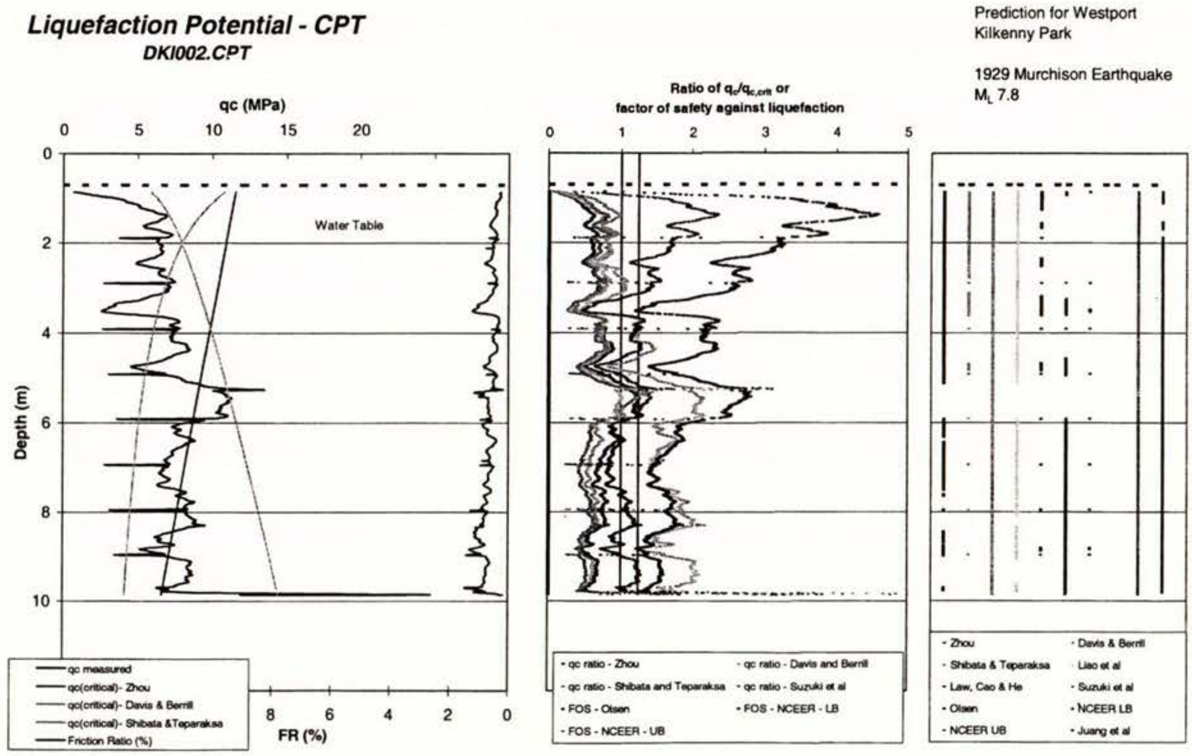

Liquefaction Potential - CPT
DKI001.CPT
Prediction for Westport
Kilkenny Park
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
Depth (m)
Water Table
FR (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al


Liquefaction Potential - CPT
DKI001.CPT
Prediction for Westport
Kilkenny Park
1991 Hawks Crag
Earthquakes
qc (MPa)
Depth (m)
Water Table
FR (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

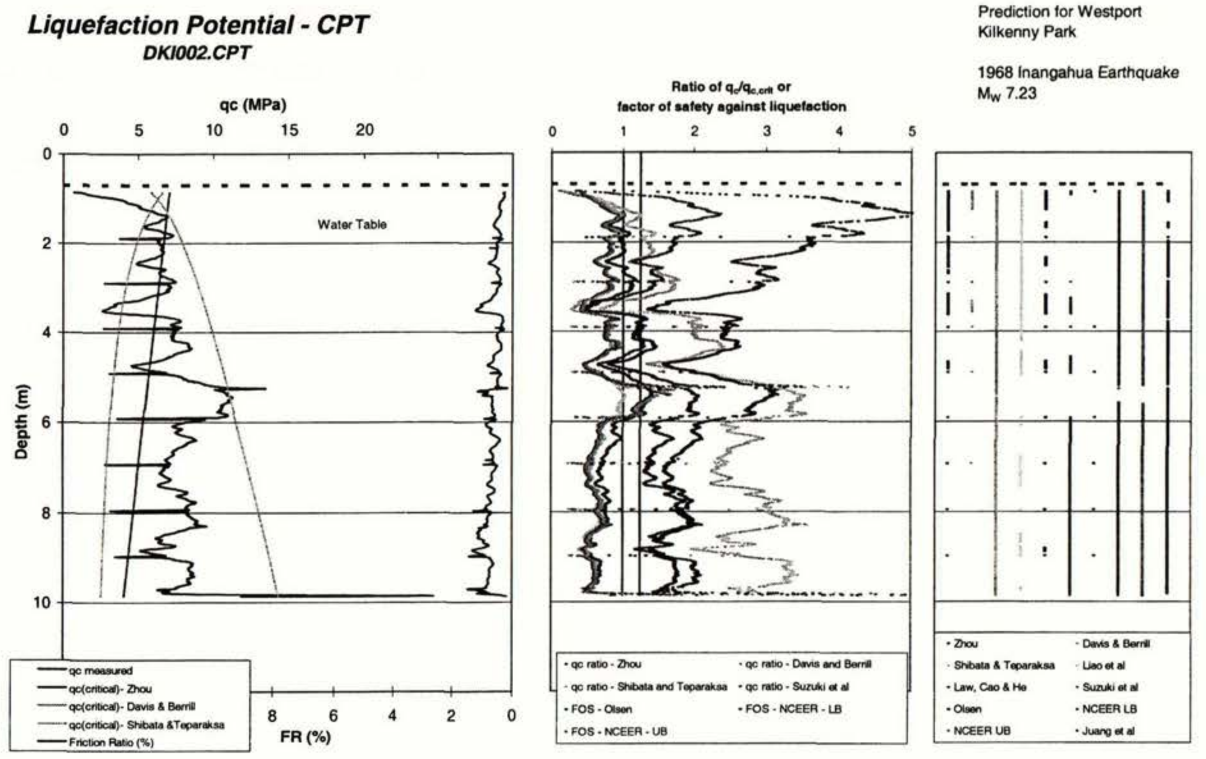
KIL002.CPT

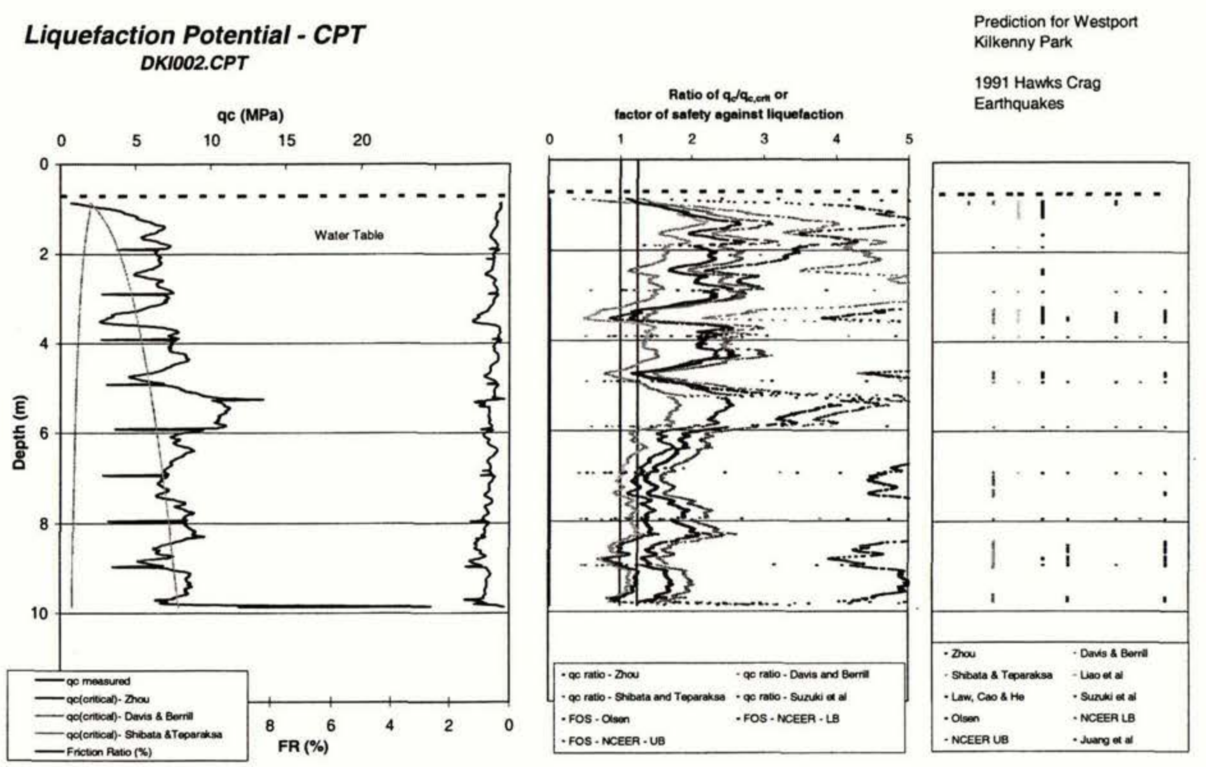


Inferred Soil Profiles - based on experimental data
DKI002.CPT
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
0 1 2 3 4 5 6 7 8 9 10 11 12
qc measured
Friction Ratio (%)
8 6 4 2 0
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)

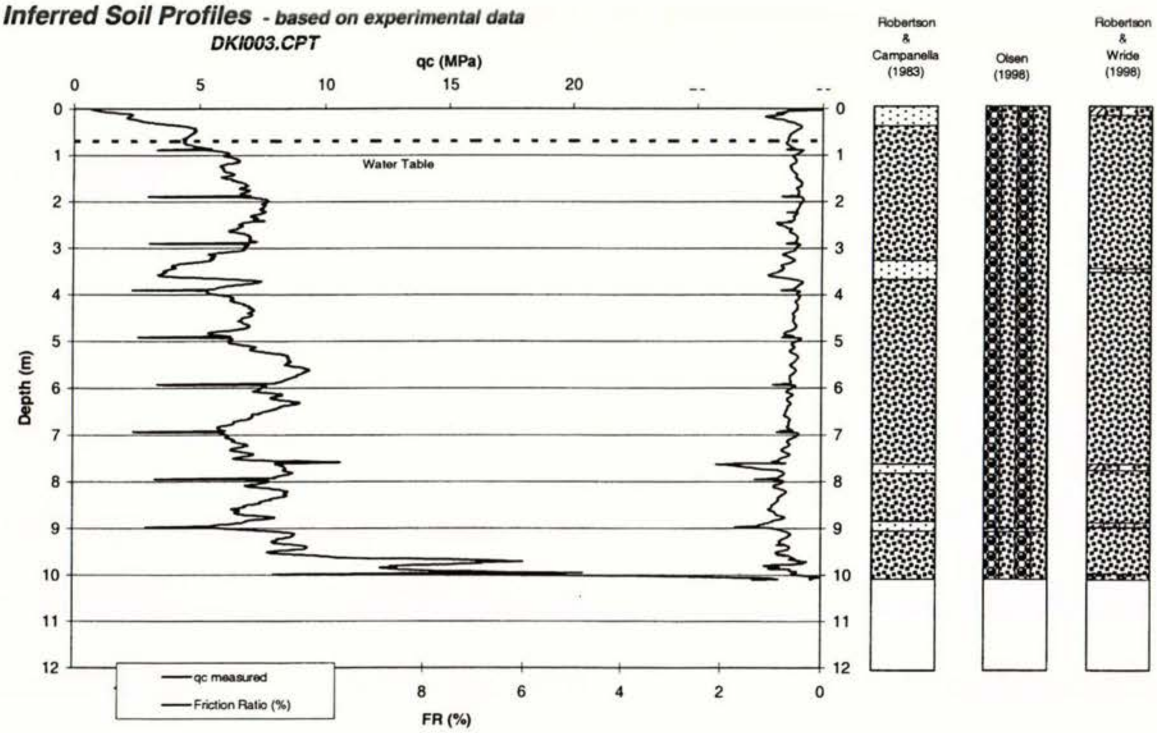


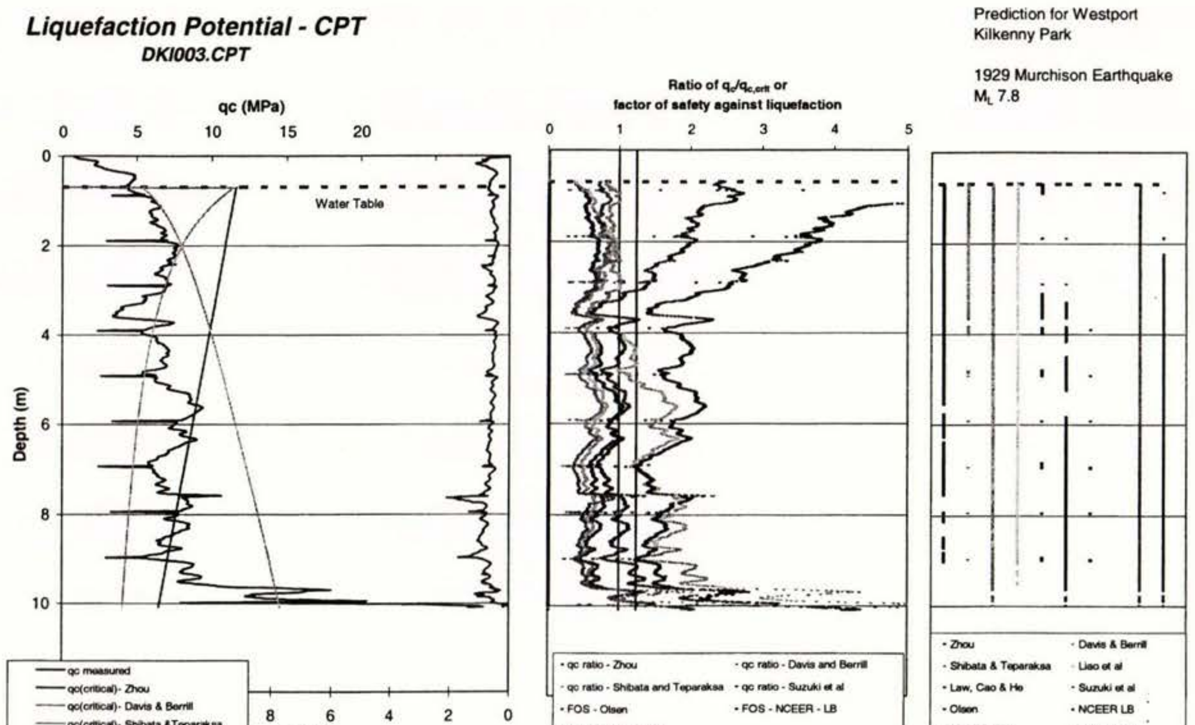
Liquefaction Potential - CPT
DKI002.CPT
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
Ratio of qc/qc,crit or factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Prediction for Westport
Kilkenny Park
1929 Murchison Earthquake
M_L 7.8
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al


Liquefaction Potential - CPT
DKI002.CPT
Prediction for Westport
Kilkenny Park
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
Water Table
Depth (m)
FR (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al


Liquefaction Potential - CPT
DKI002.CPT
Prediction for Westport
Kilkenny Park
1991 Hawks Crag
Earthquakes
qc (MPa)
Water Table
Depth (m)
FR (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

KIL003.CPT

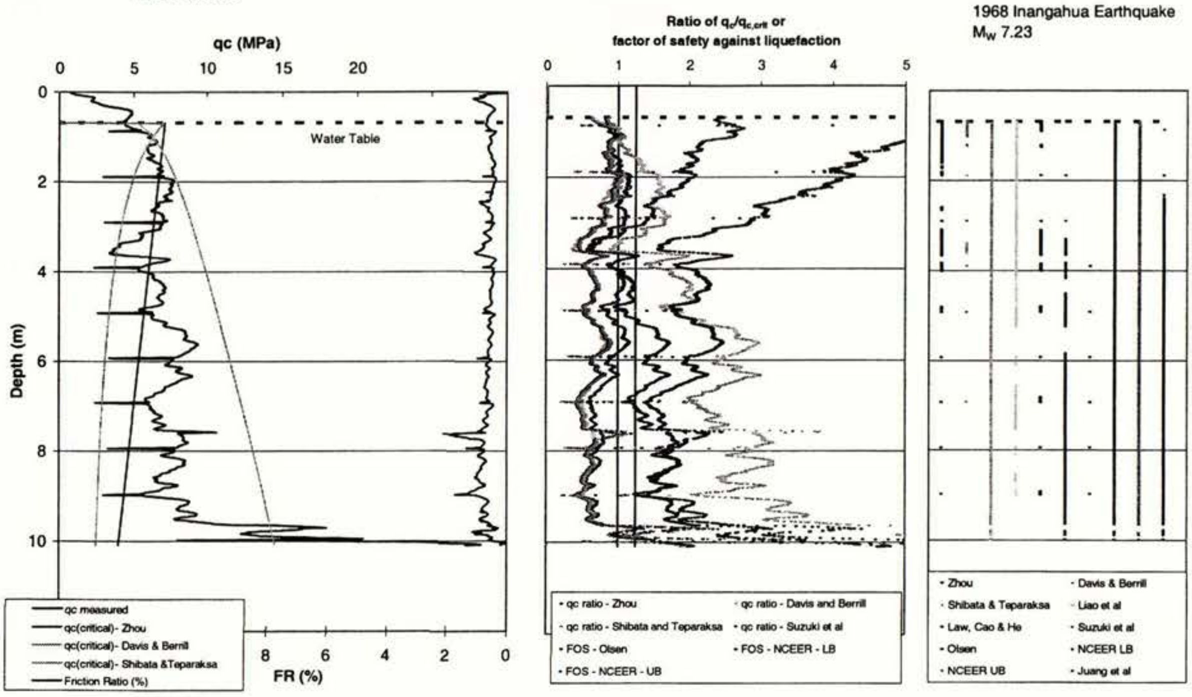

Inferred Soil Profiles - based on experimental data
DKI003.CPT
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
qc measured
Friction Ratio (%)
8 6 4 2 0
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)


Liquefaction Potential - CPT
DKI003.CPT
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Prediction for Westport
Kilkenny Park
1929 Murchison Earthquake
M_L 7.8
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

Liquefaction Potential - CPT
DKI003.CPT

Prediction for Westport
Kilkenny Park

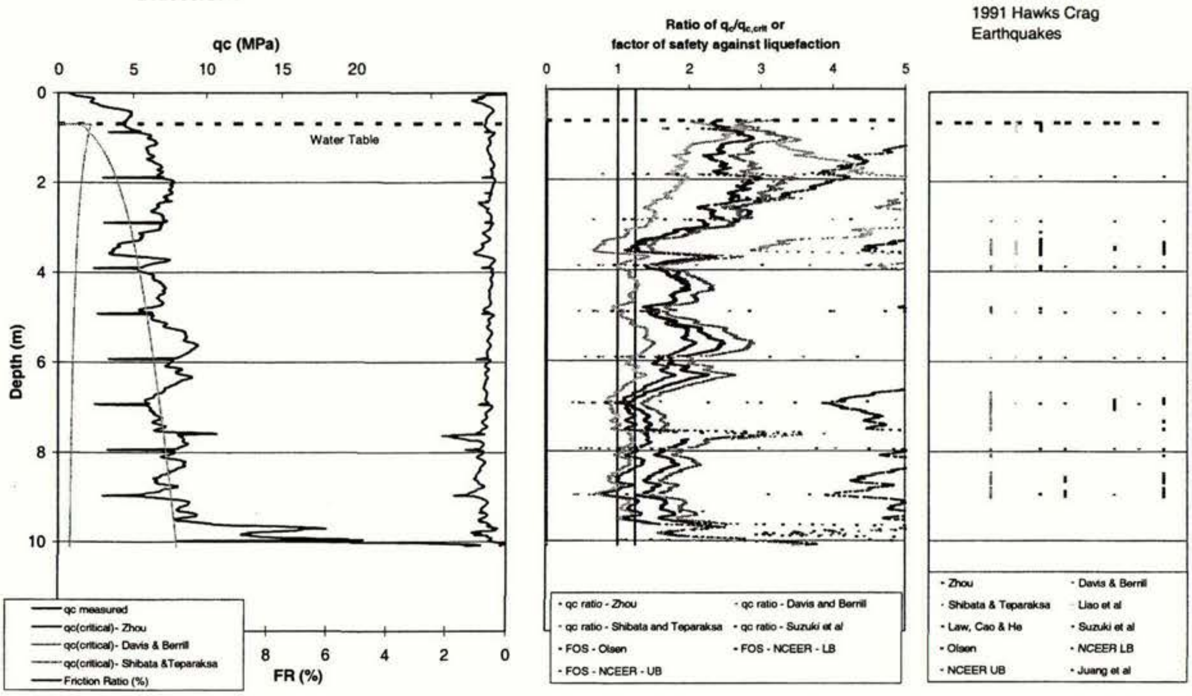
1968 Inangahua Earthquake
M_W 7.23



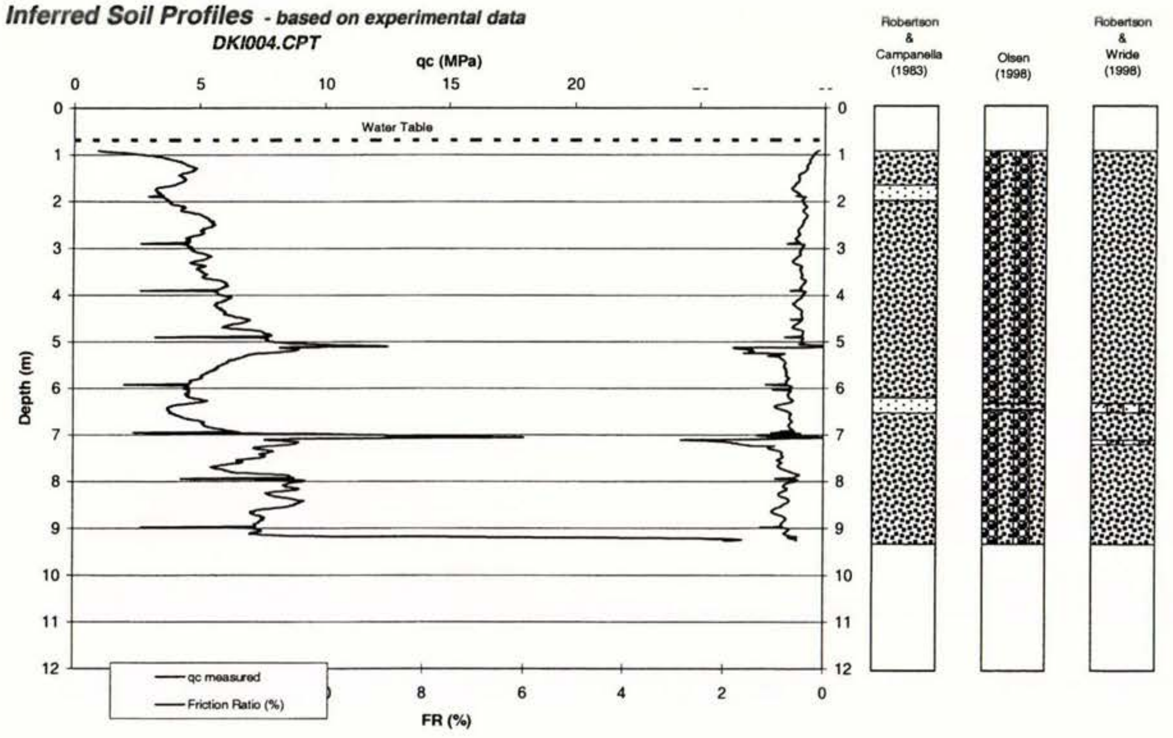
Liquefaction Potential - CPT
DKI003.CPT

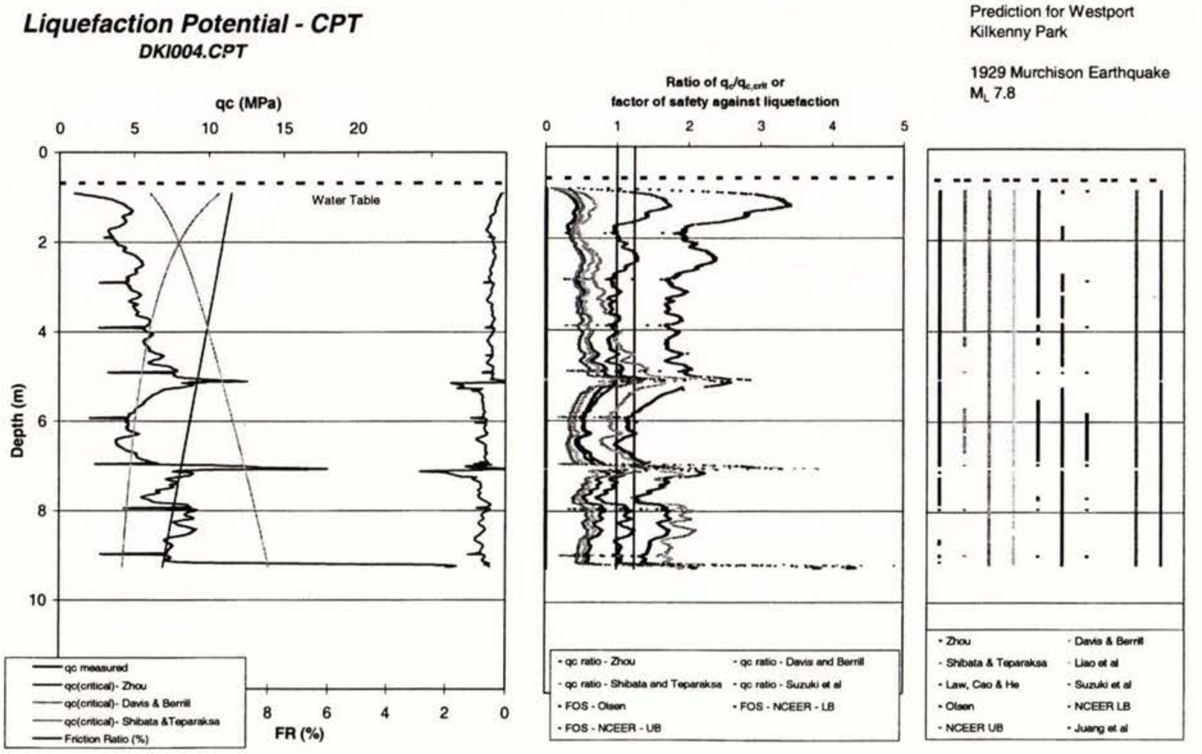
Prediction for Westport
Kilkenny Park

1991 Hawks Crag
Earthquakes



KIL004.CPT

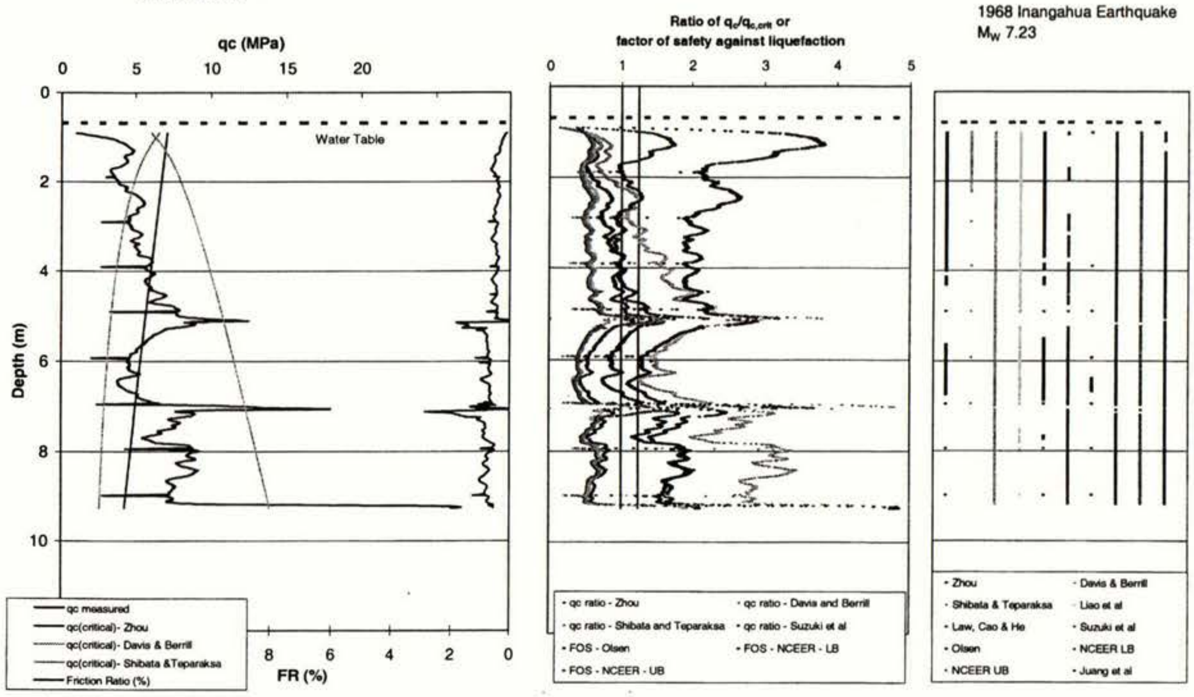

Inferred Soil Profiles - based on experimental data
DKI004.CPT
qc (MPa)
0
5
10
15
20
Water Table
Depth (m)
FR (%)
8
6
4
2
0
qc measured
Friction Ratio (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)


Liquefaction Potential - CPT
DKI004.CPT
qc (MPa)
Water Table
Depth (m)
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
- qc ratio - Zhou
- qc ratio - Davis and Berrill
- qc ratio - Shibata and Teparaksa
- qc ratio - Suzuki et al
- FOS - Olsen
- FOS - NCEER - LB
- FOS - NCEER - UB
Prediction for Westport
Kilkenny Park
1929 Murchison Earthquake
M_L 7.8
- Zhou
- Davis & Berrill
- Shibata & Teparaksa
- Liao et al
- Law, Cao & He
- Suzuki et al
- Olsen
- NCEER LB
- NCEER UB
- Juang et al

Liquefaction Potential - CPT
DKI004.CPT

Prediction for Westport
Kilkenny Park

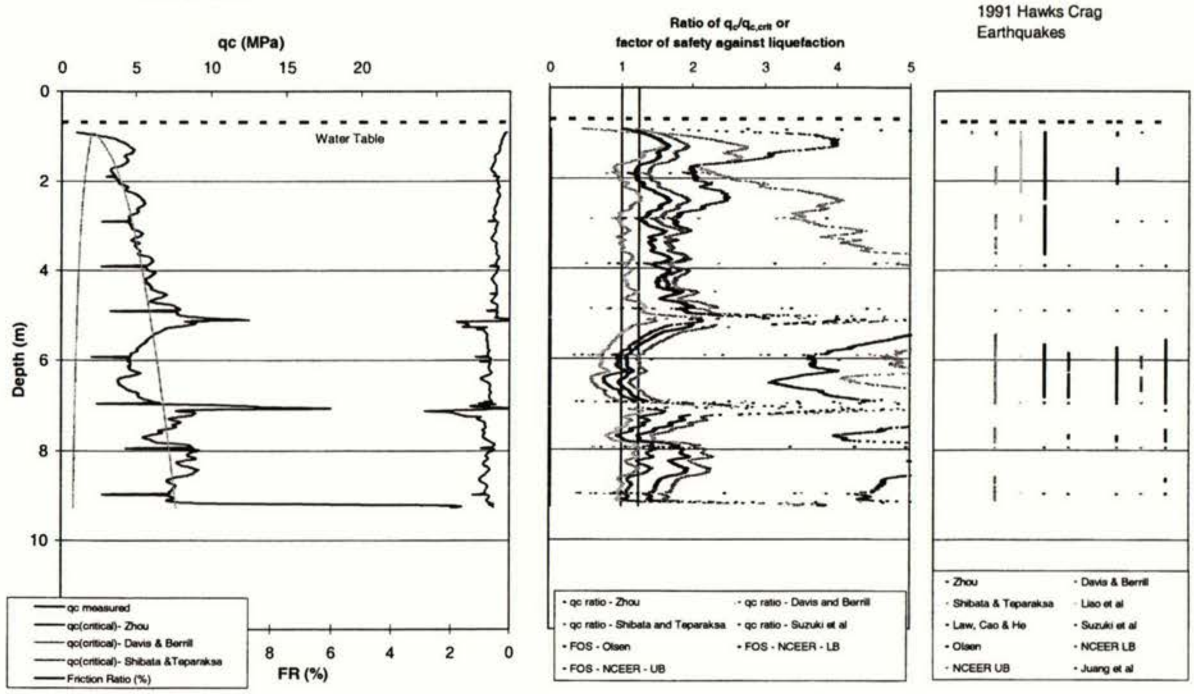
1968 Inangahua Earthquake
M_W 7.23



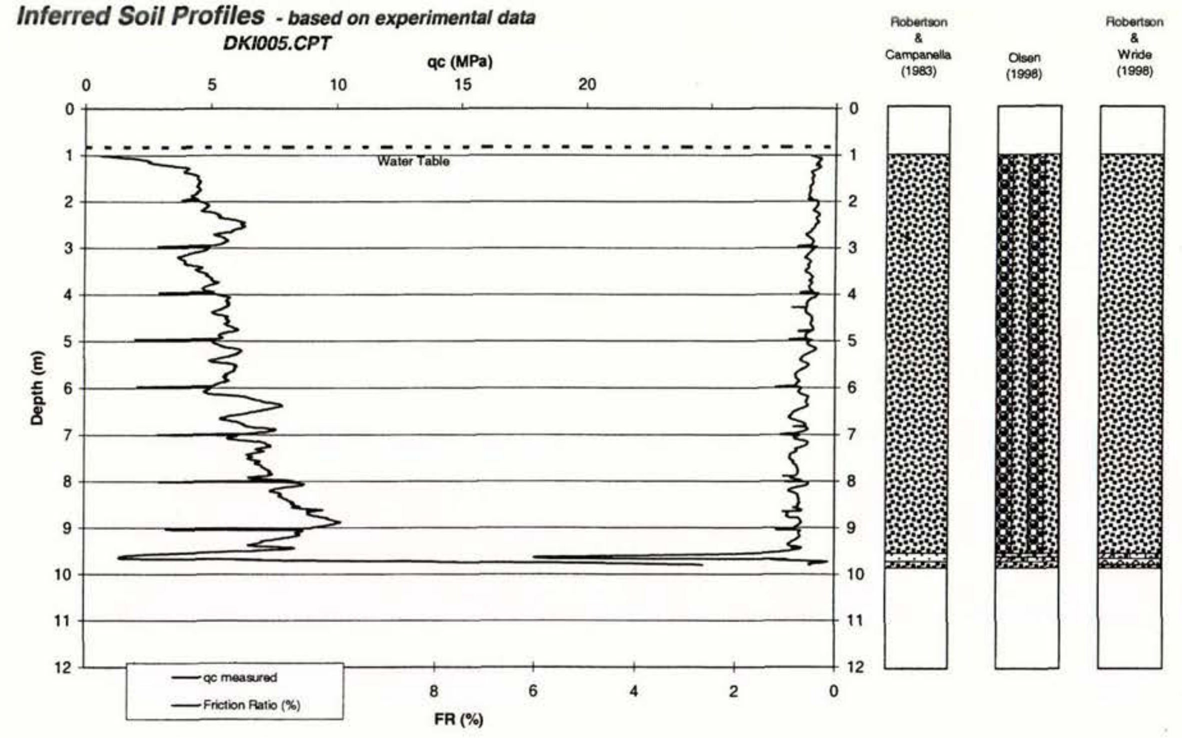
Liquefaction Potential - CPT
DKI004.CPT

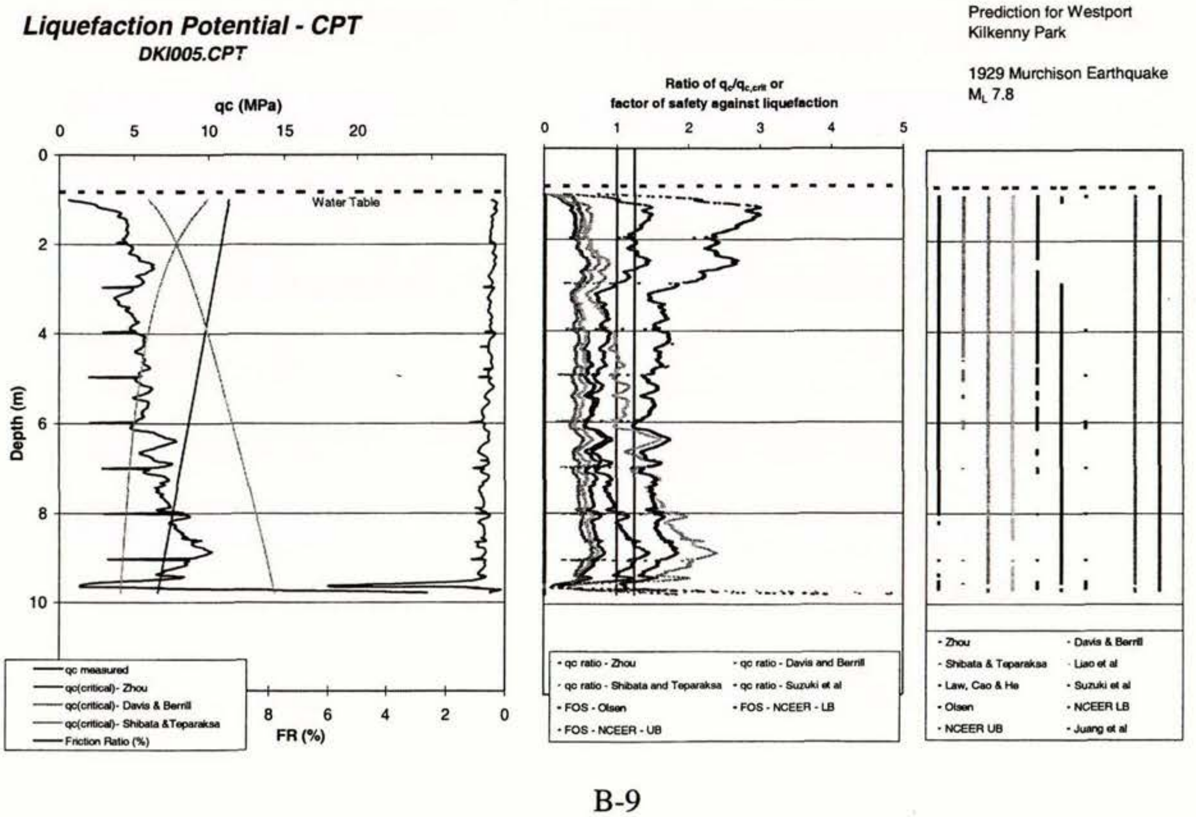
Prediction for Westport
Kilkenny Park

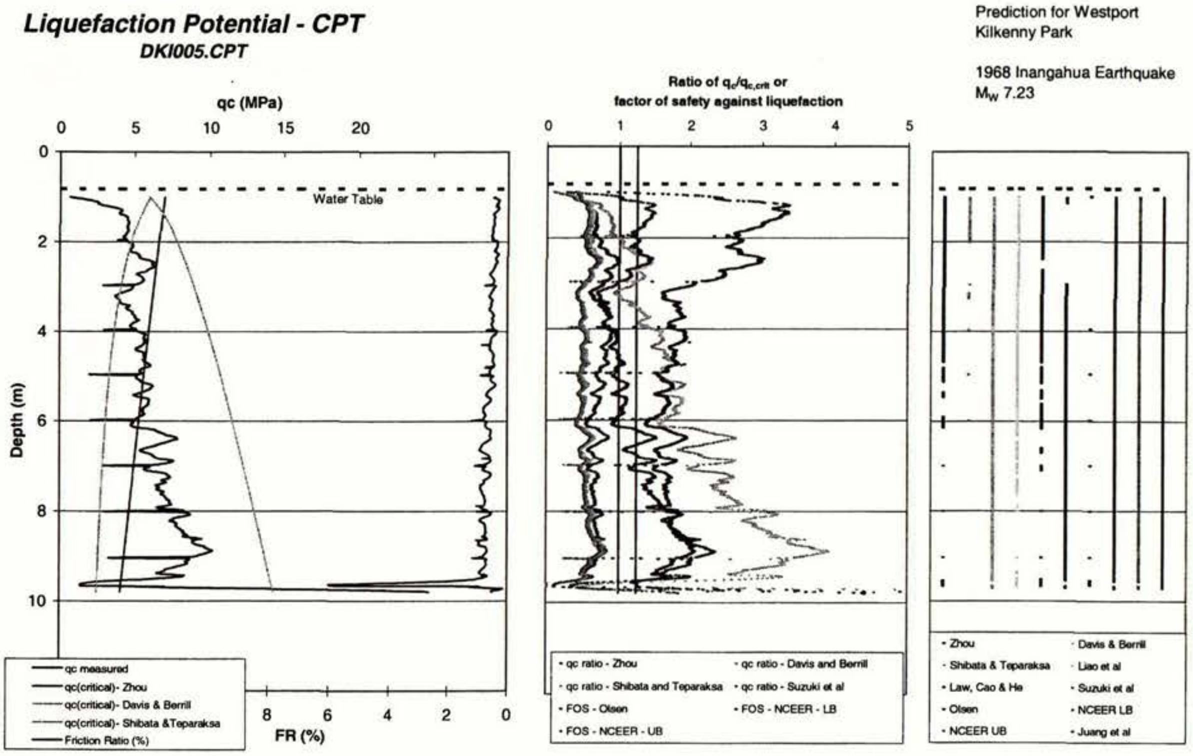
1991 Hawks Crag
Earthquakes

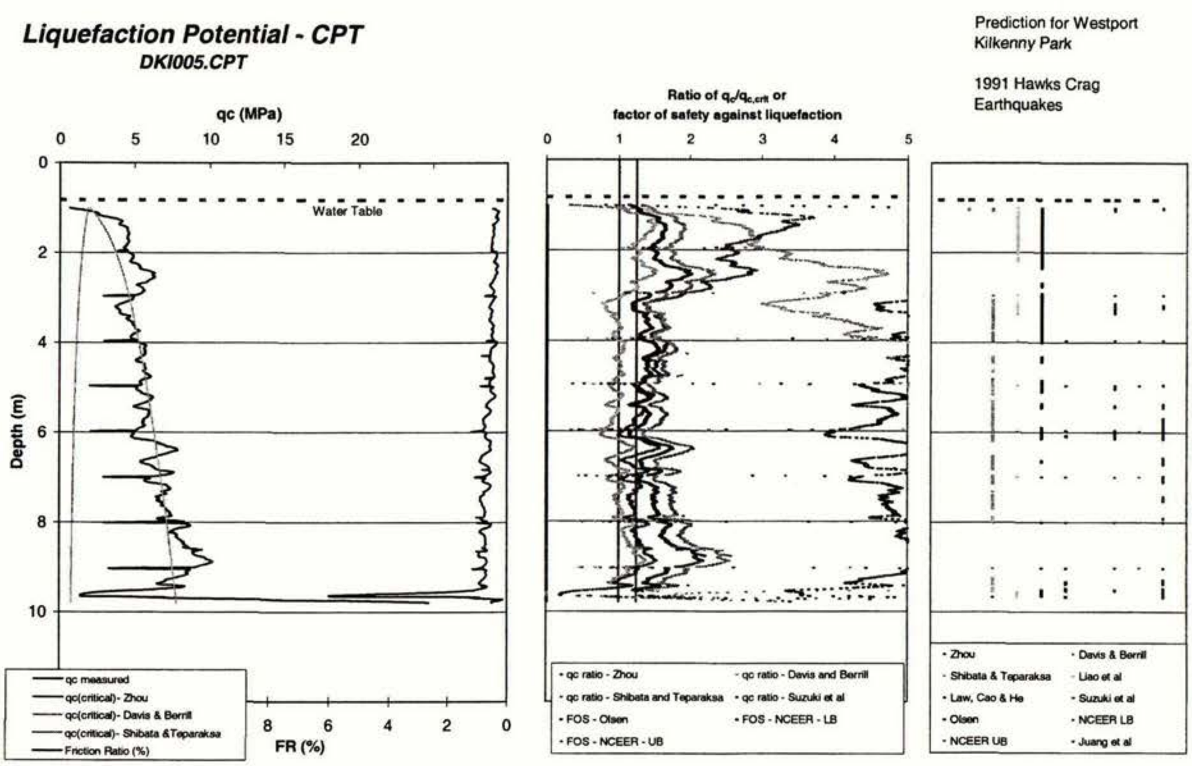


KIL005.CPT

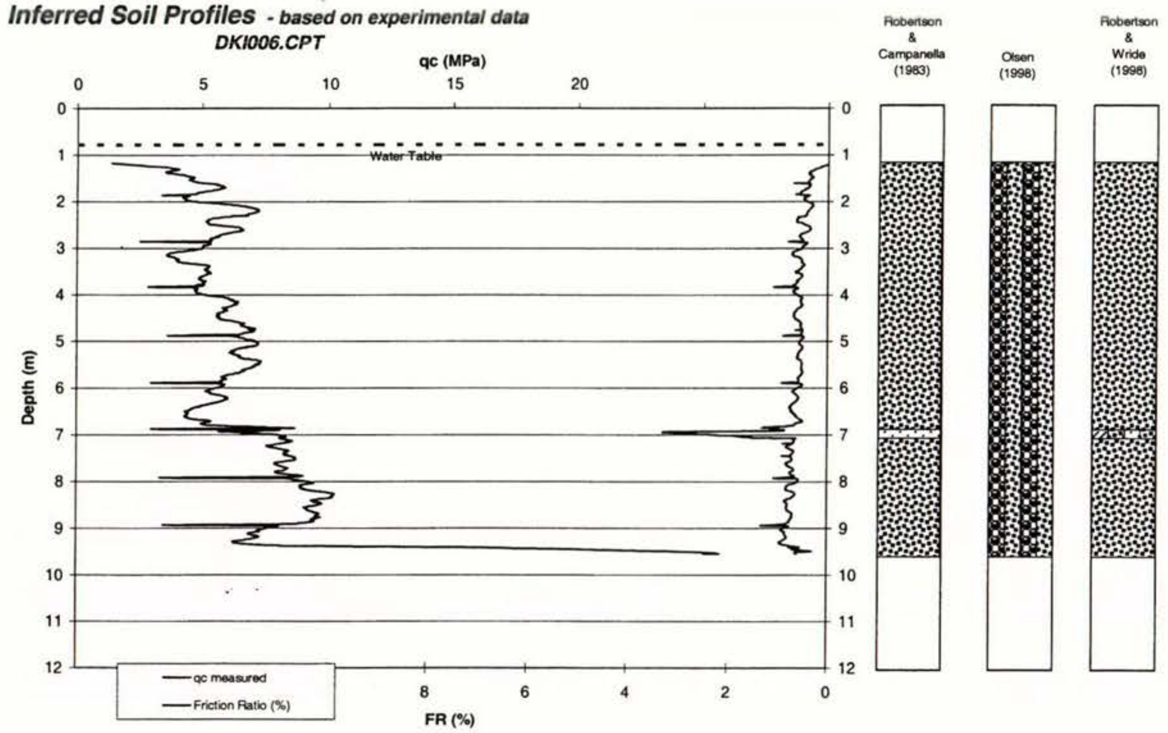

Inferred Soil Profiles - based on experimental data
DKI005.CPT
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
0 1 2 3 4 5 6 7 8 9 10 11 12
qc measured
Friction Ratio (%)
8 6 4 2 0
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)


Liquefaction Potential - CPT
DKI005.CPT
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
0 2 4 6 8 10
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
0 1 2 3 4 5
- qc ratio - Zhou
- qc ratio - Davis and Berrill
- qc ratio - Shibata and Teparaksa
- qc ratio - Suzuki et al
- FOS - Olsen
- FOS - NCEER - LB
- FOS - NCEER - UB
Prediction for Westport
Kilkenny Park
1929 Murchison Earthquake
M_L 7.8
- Zhou
- Davis & Berrill
- Shibata & Teparaksa
- Liao et al
- Law, Cao & He
- Suzuki et al
- Olsen
- NCEER LB
- NCEER UB
- Juang et al

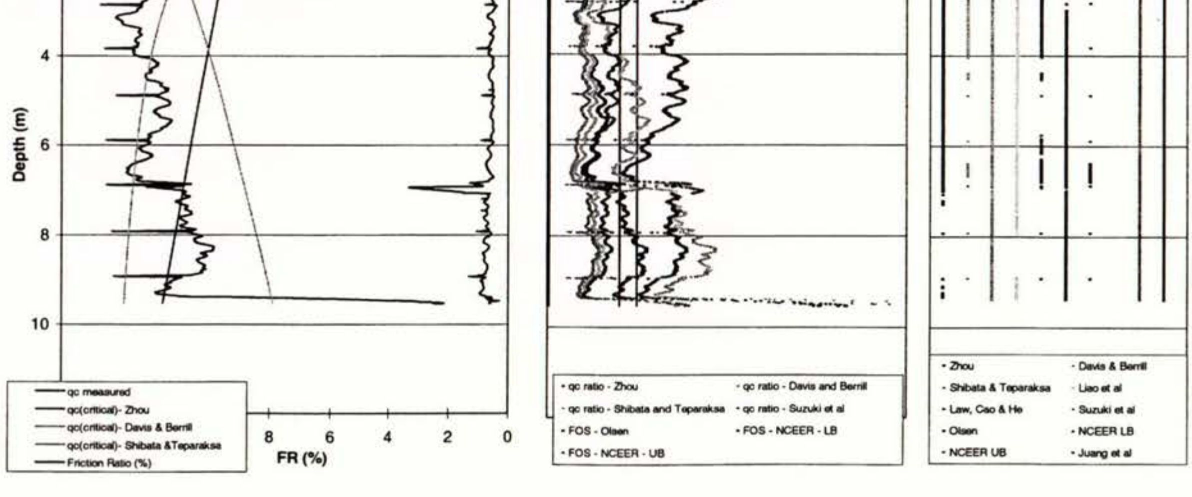

Liquefaction Potential - CPT
DKI005.CPT
Prediction for Westport
Kilkenny Park
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
FR (%)
8 6 4 2 0
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al


Liquefaction Potential - CPT
DKI005.CPT
Prediction for Westport
Kilkenny Park
1991 Hawks Crag
Earthquakes
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
FR (%)
8 6 4 2 0
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

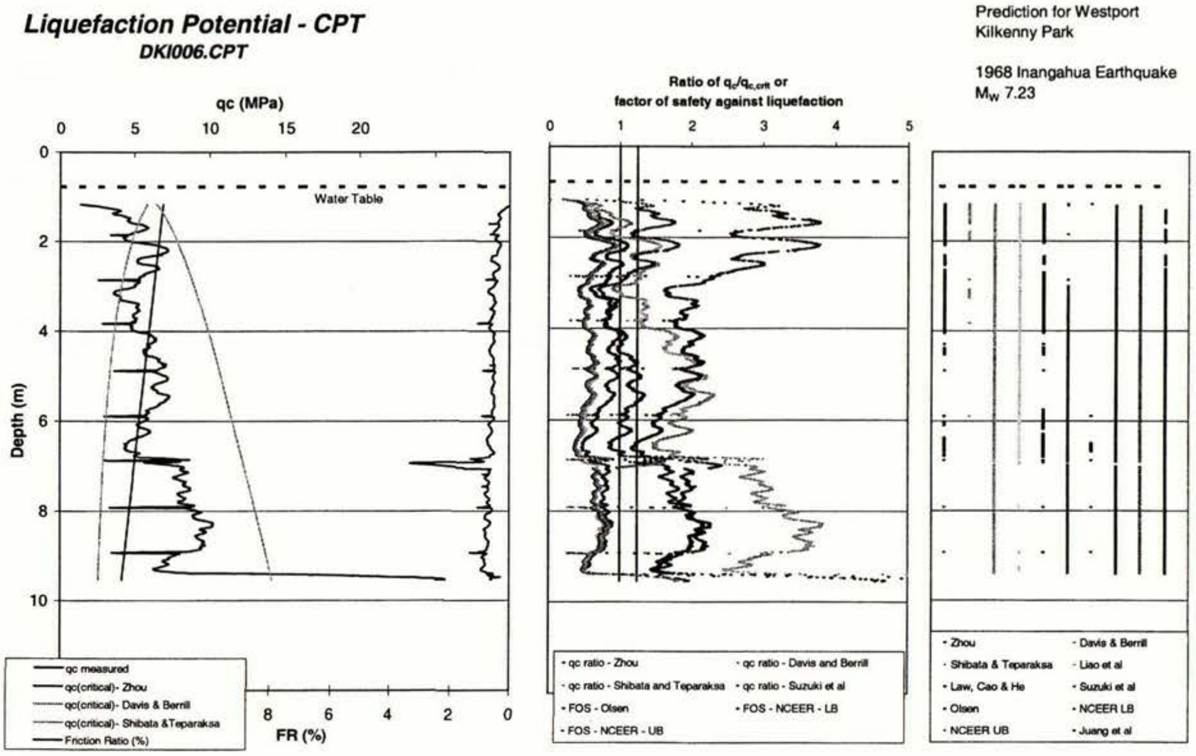
KIL006.CPT

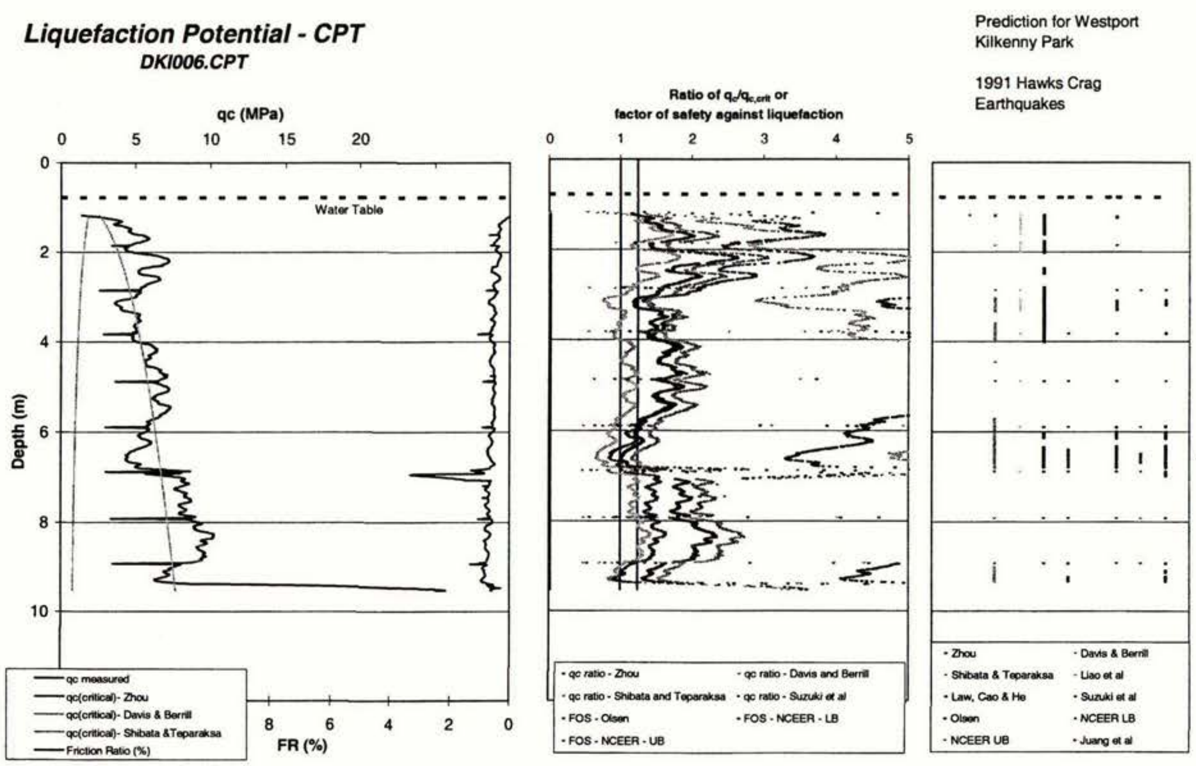


Inferred Soil Profiles - based on experimental data
DKI006.CPT
qc (MPa)
0
5
10
15
20
Depth (m)
Water Table
qc measured
Friction Ratio (%)
FR (%)
8
6
4
2
0
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)



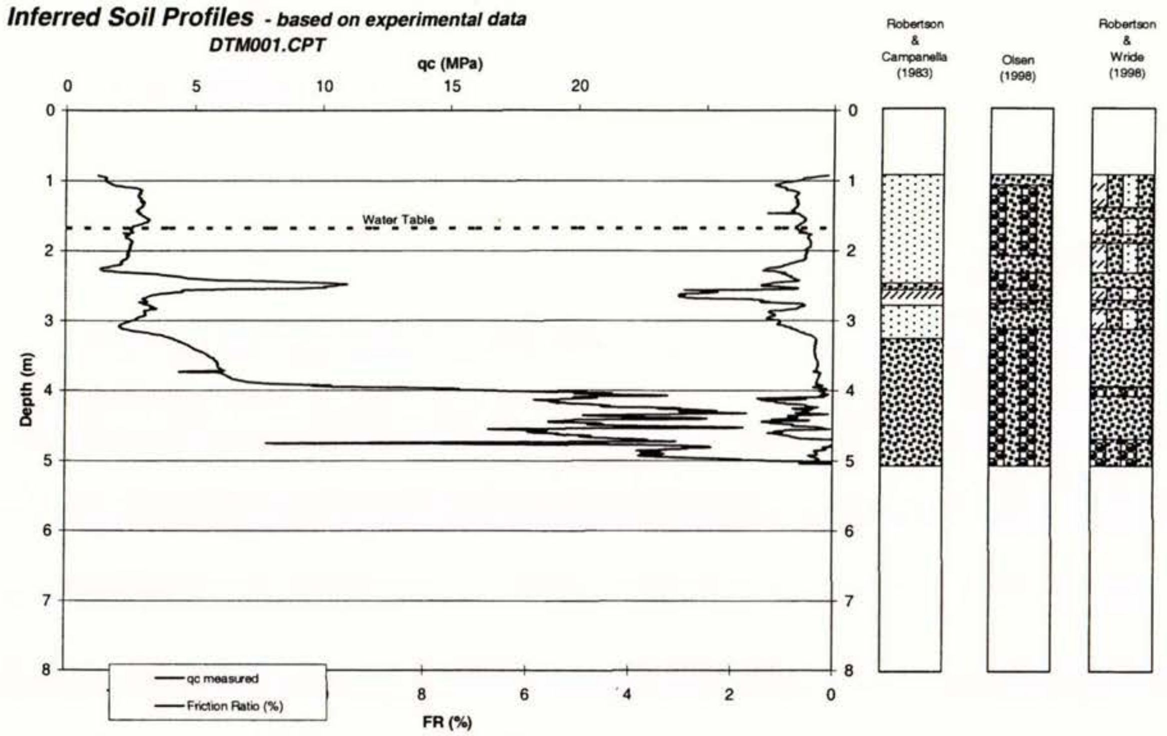
Liquefaction Potential - CPT
DKI006.CPT
qc (MPa)
Depth (m)
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
- qc ratio - Zhou
- qc ratio - Davis and Berrill
- qc ratio - Shibata and Teparaksa
- qc ratio - Suzuki et al
- FOS - Olsen
- FOS - NCEER - LB
- FOS - NCEER - UB
Prediction for Westport Kilkenny Park
1929 Murchison Earthquake M_L 7.8
- Zhou
- Davis & Berrill
- Shibata & Teparaksa
- Liao et al
- Law, Cao & He
- Suzuki et al
- Olsen
- NCEER LB
- NCEER UB
- Juang et al

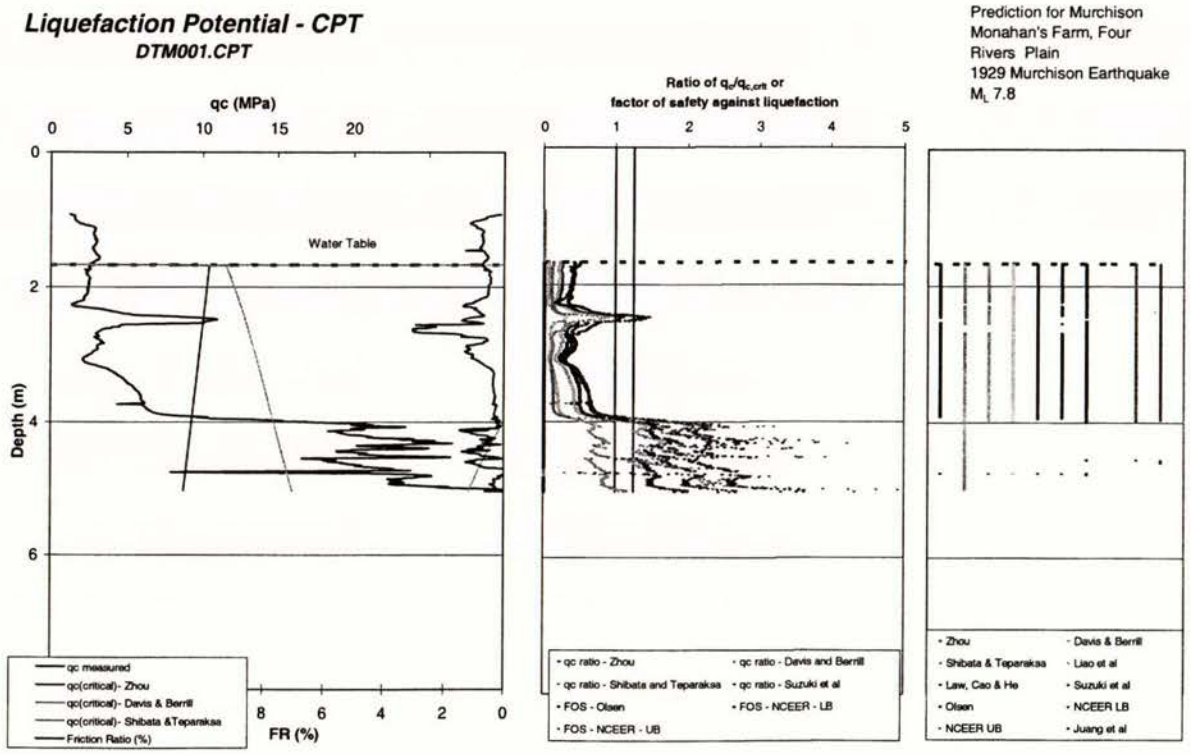

Liquefaction Potential - CPT
DKI006.CPT
Prediction for Westport
Kilkenny Park
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0 5 10 15 20
Depth (m)
0 2 4 6 8 10
Water Table
FR (%)
8 6 4 2 0
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

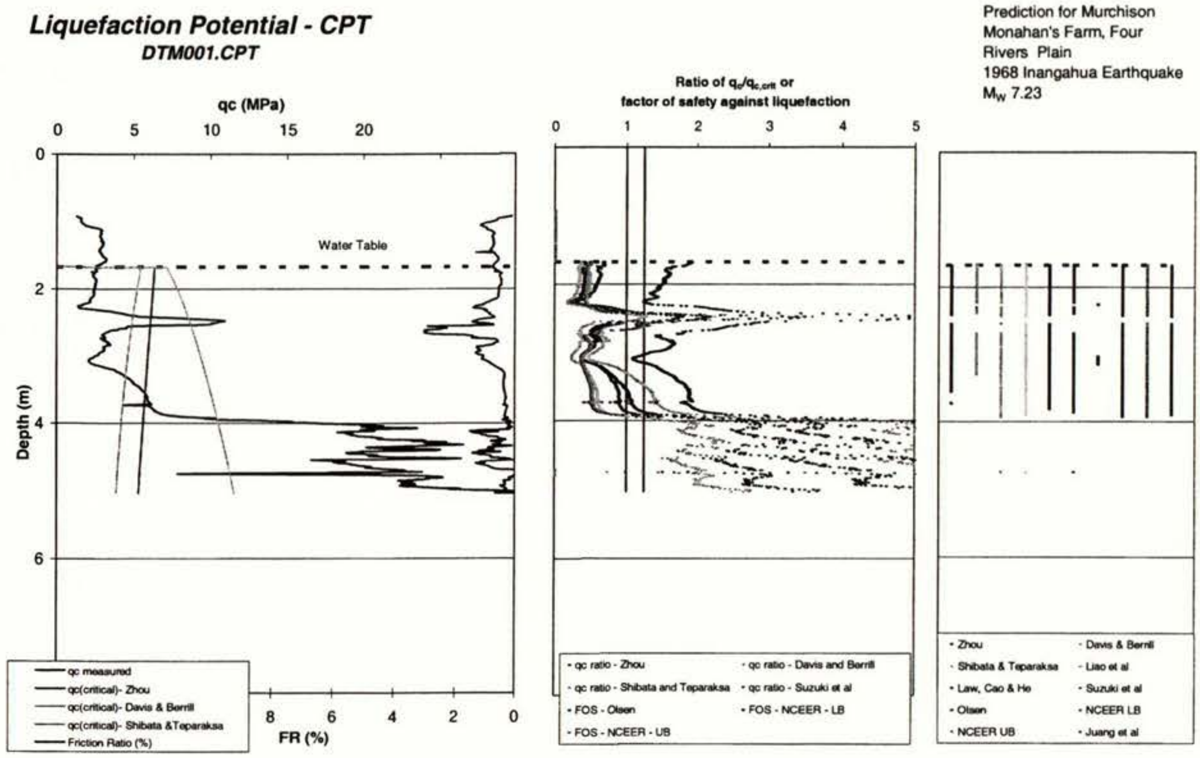

Liquefaction Potential - CPT
DKI006.CPT
Prediction for Westport
Kilkenny Park
1991 Hawks Crag
Earthquakes
qc (MPa)
0 5 10 15 20
Depth (m)
0 2 4 6 8 10
Water Table
FR (%)
8 6 4 2 0
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

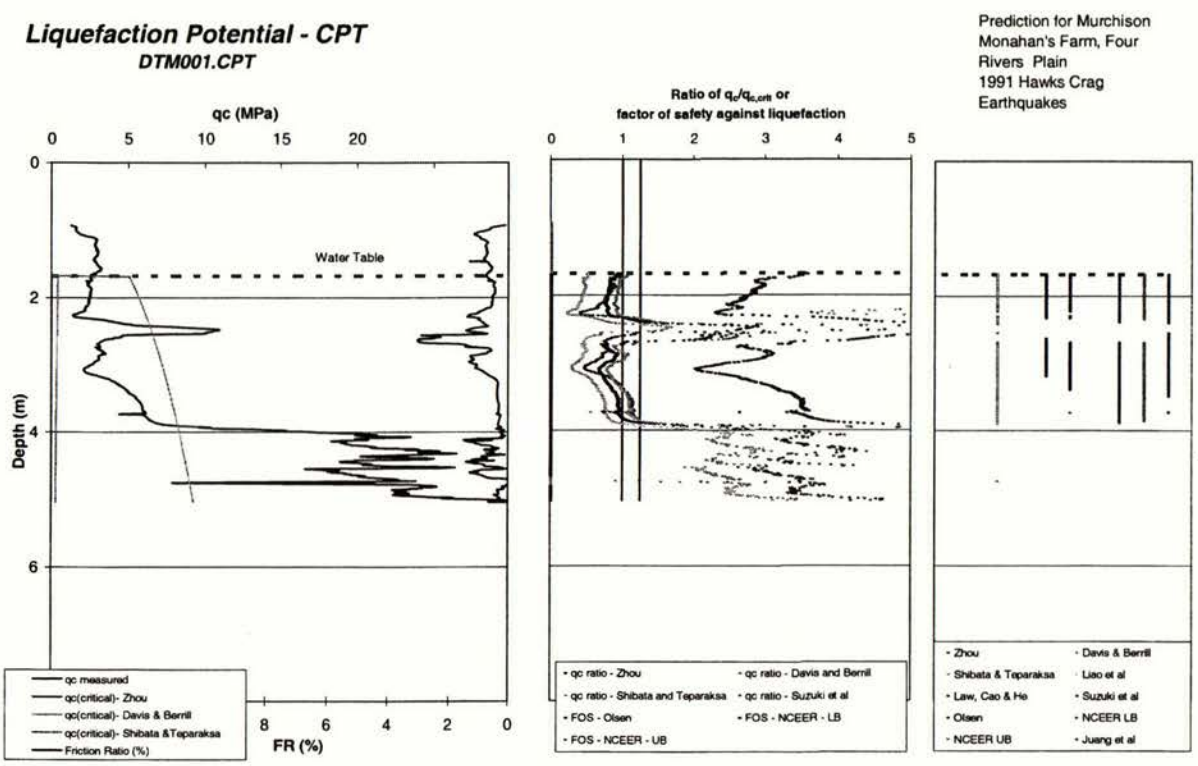
B1.2 **Murchison – Monahan's Farm, Four Rivers Plain**

MON001.CPT





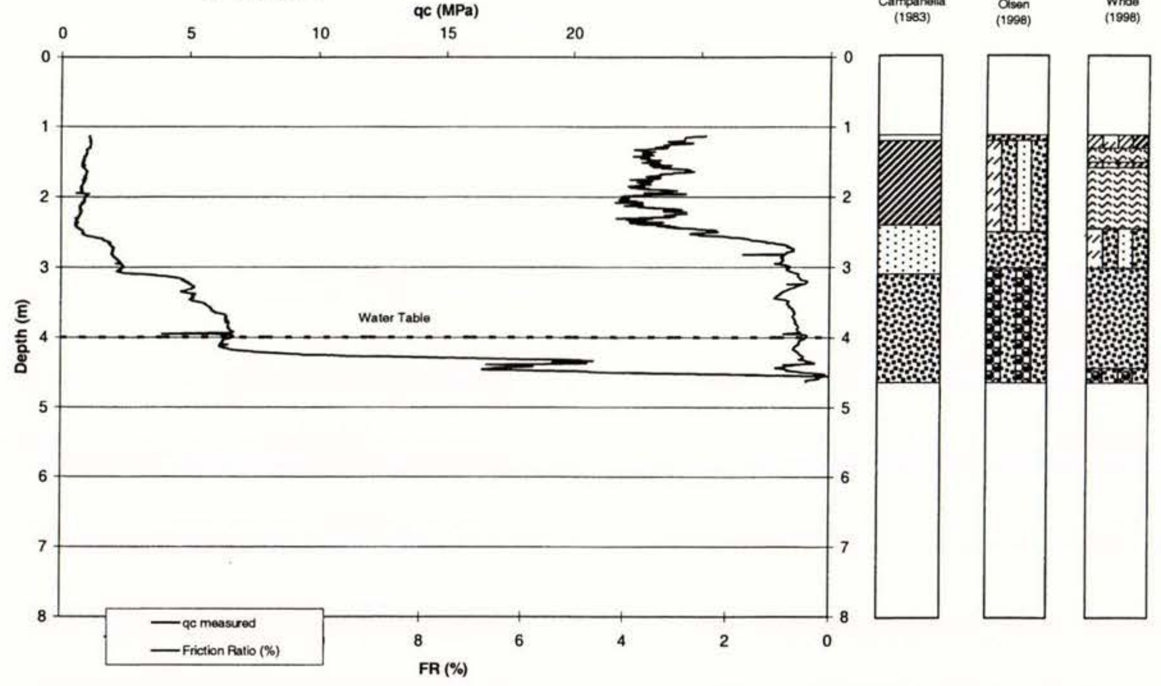

Liquefaction Potential - CPT
DTM001.CPT
Prediction for Murchison
Monahan's Farm, Four Rivers Plain
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0
5
10
15
20
Depth (m)
0
2
4
6
Water Table
8
6
4
2
0
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
0
1
2
3
4
5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al


Liquefaction Potential - CPT
DTM001.CPT
Prediction for Murchison
Monahan's Farm, Four Rivers Plain
1991 Hawks Crag Earthquakes
qc (MPa)
0
5
10
15
20
Depth (m)
0
2
4
6
Water Table
8
6
4
2
0
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
0
1
2
3
4
5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

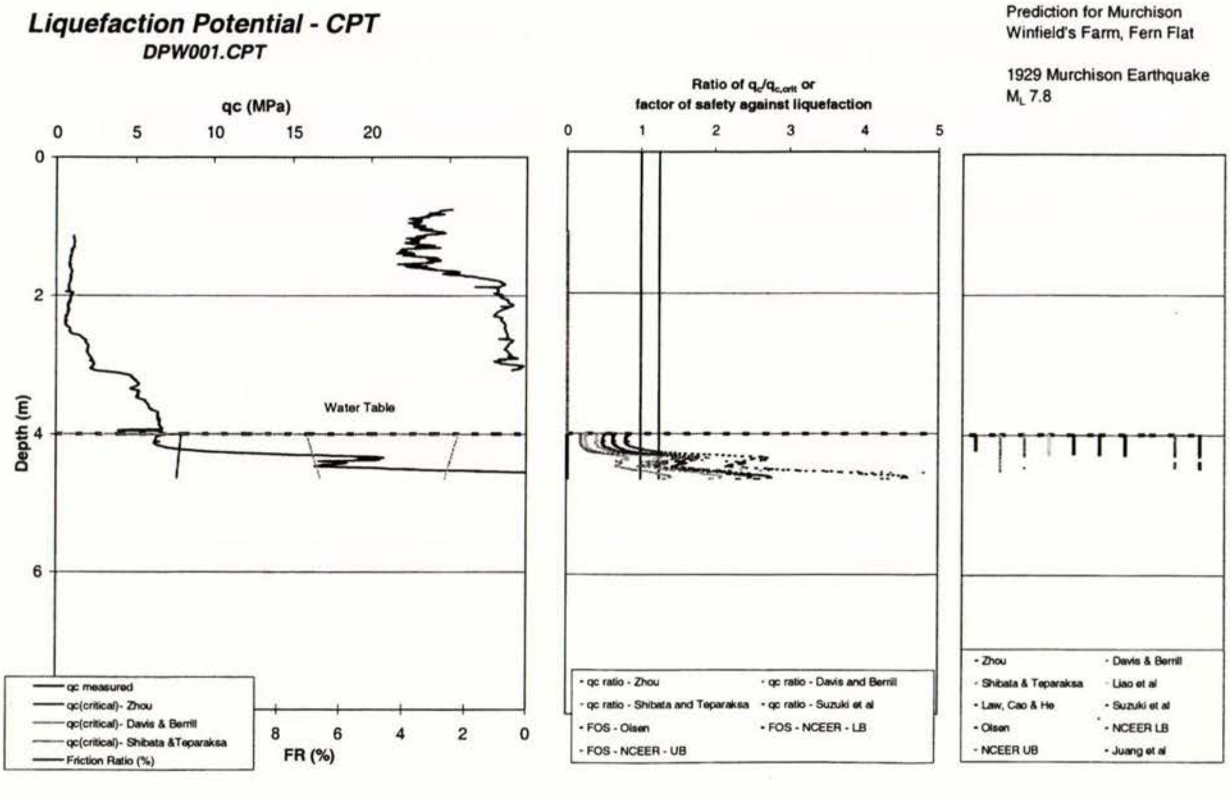
B1.3 Murchison - Winfield's Farm, Fern Flat

WIN001.CPT

***Inferred Soil Profiles** - based on experimental data*
DPW001.CPT



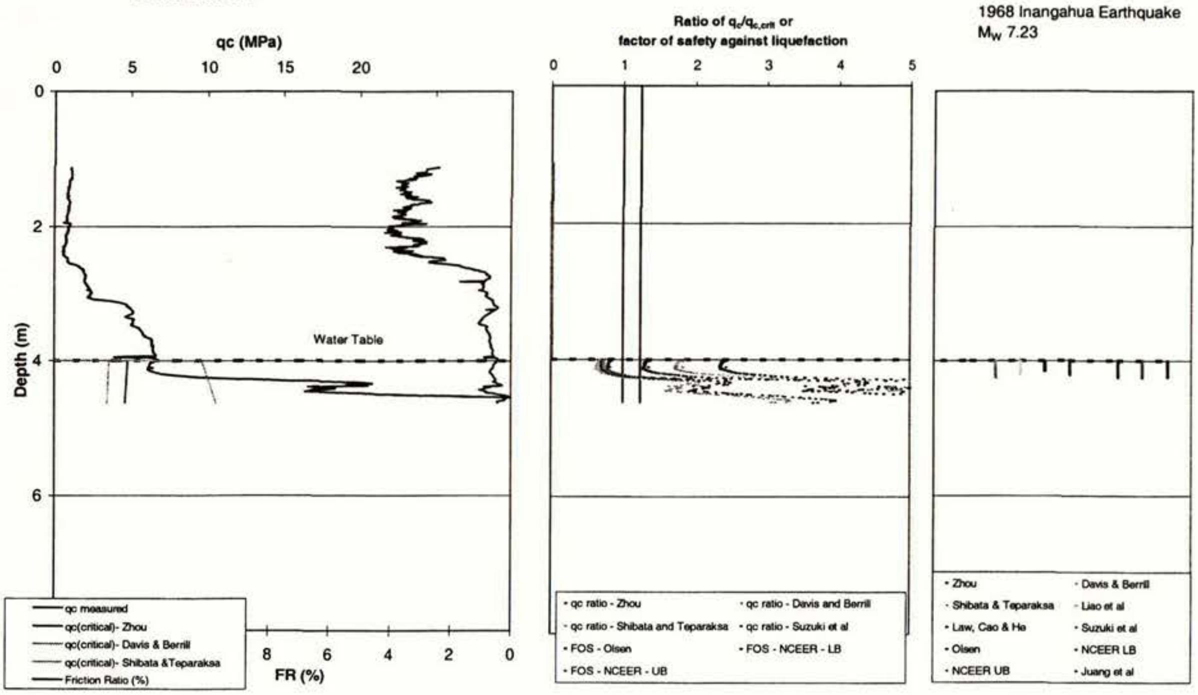
Liquefaction Potential - CPT
DPW001.CPT



Liquefaction Potential - CPT
DPW001.CPT

Prediction for Murchison
Winfield's Farm, Fern Flat

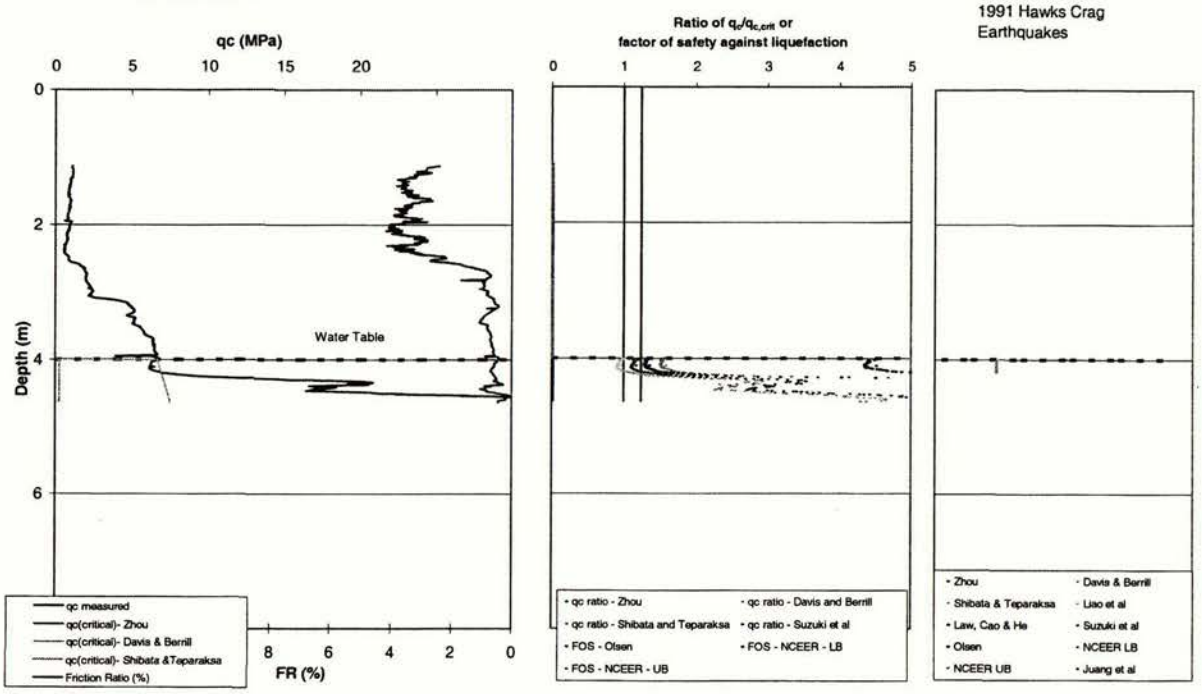
1968 Inangahua Earthquake
M_W 7.23



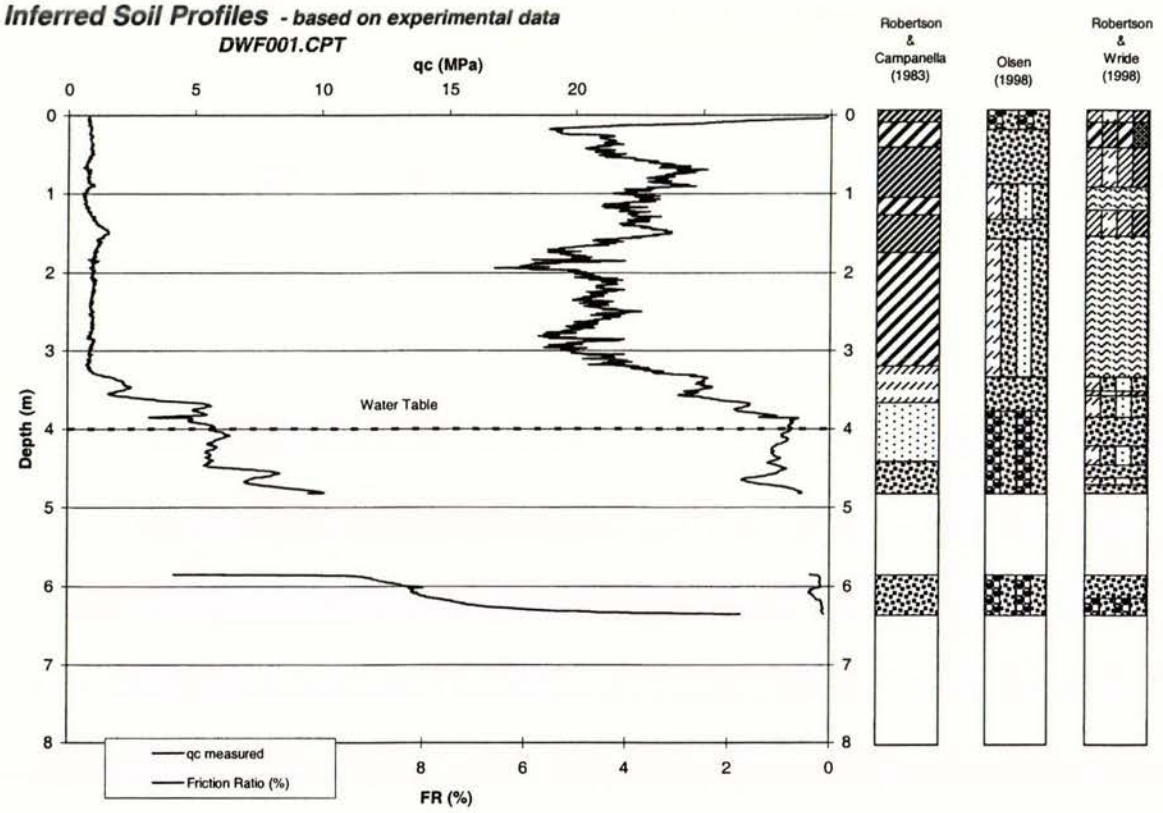
Liquefaction Potential - CPT
DPW001.CPT

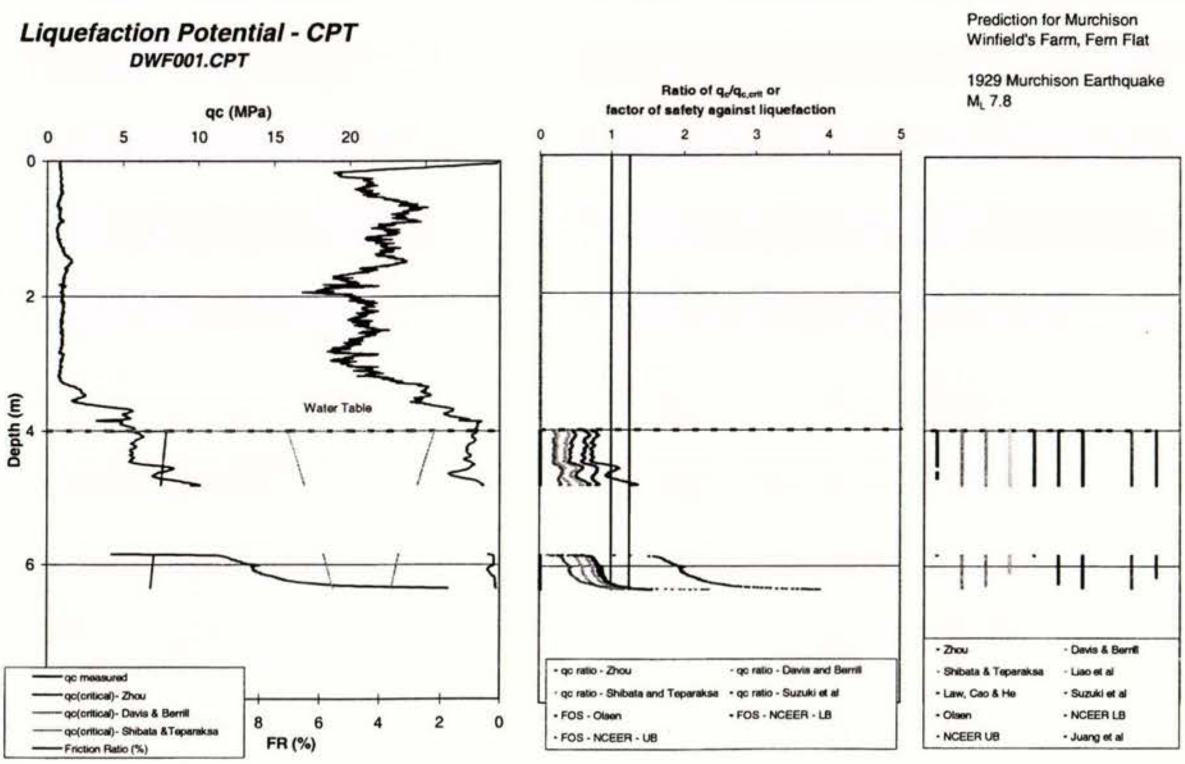
Prediction for Murchison
Winfield's Farm, Fern Flat

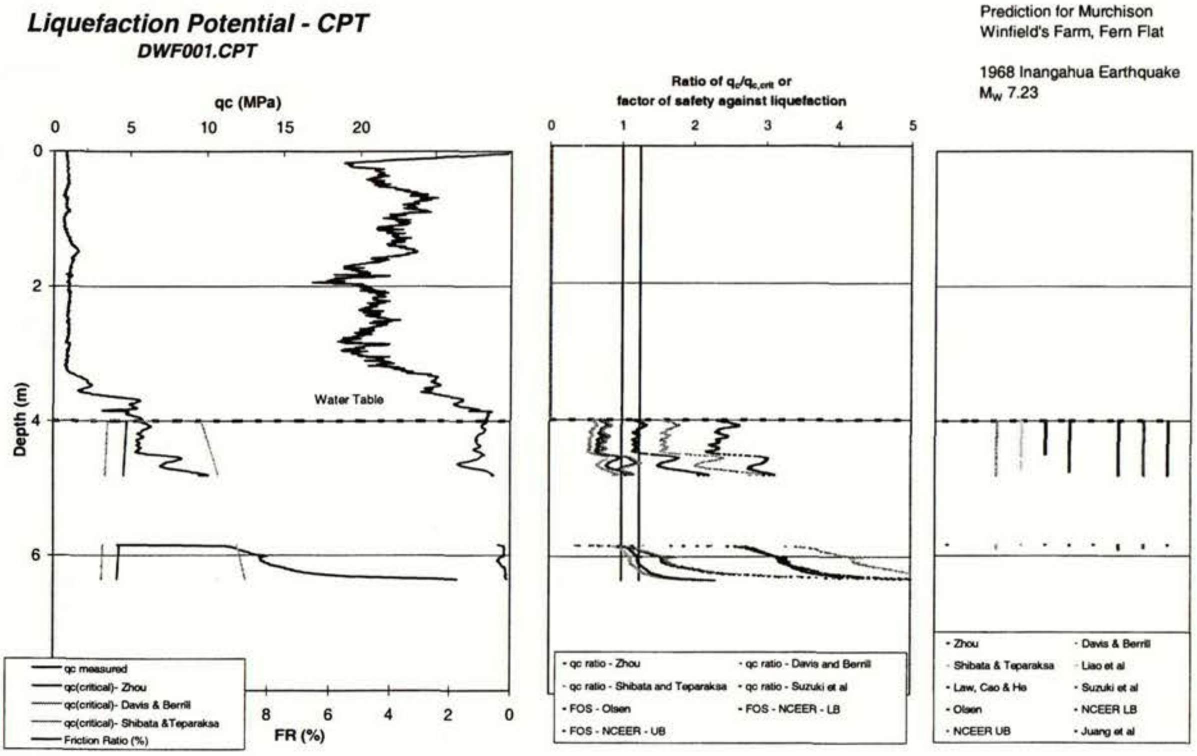
1991 Hawks Crag
Earthquakes

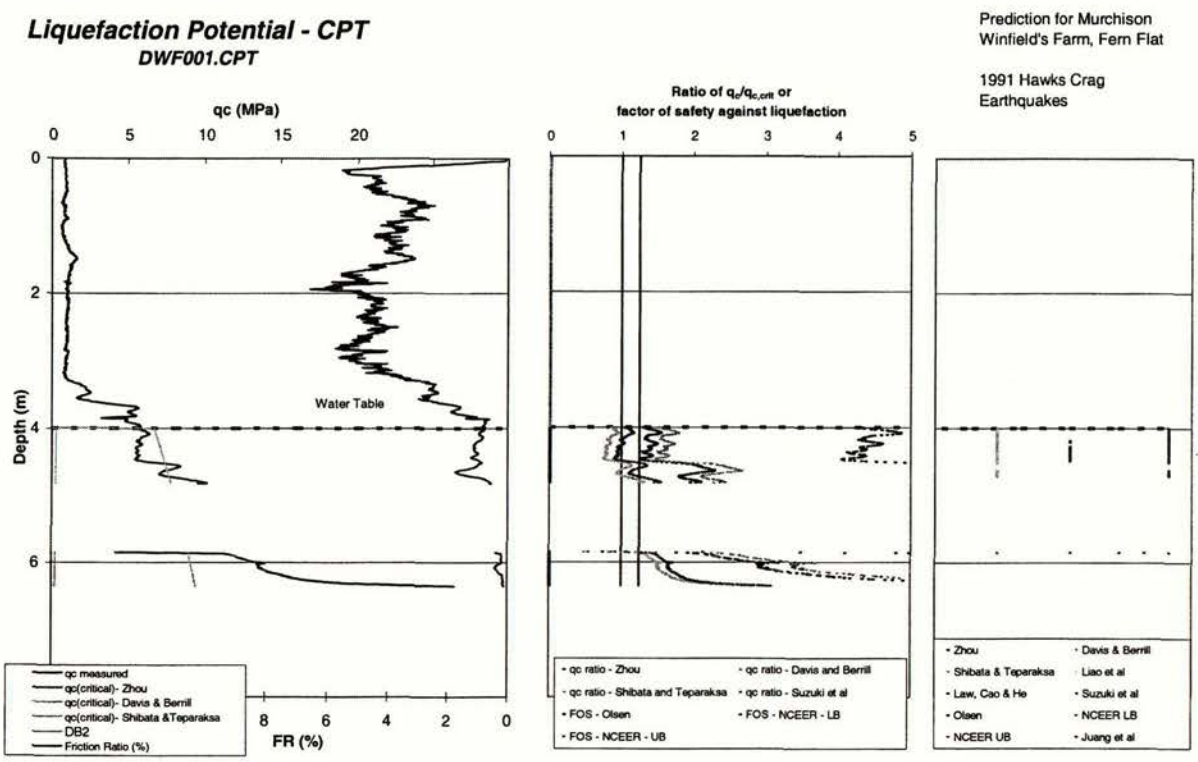


WIN002.CPT

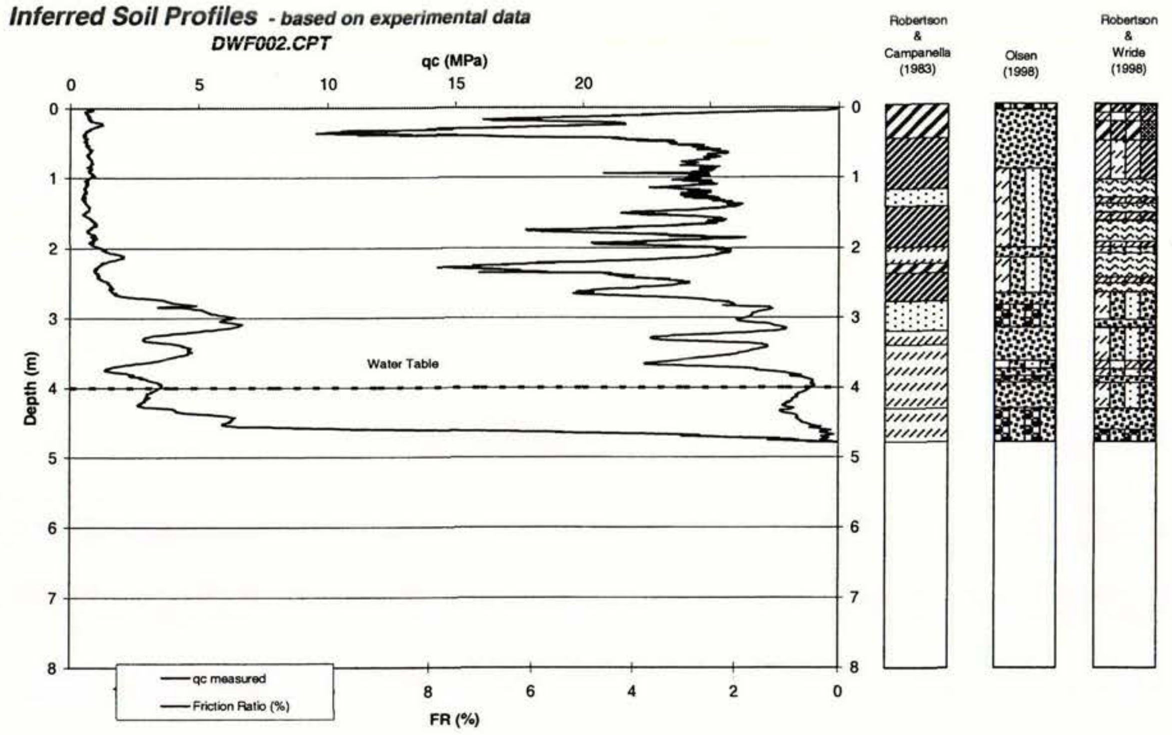

Inferred Soil Profiles - based on experimental data
DWF001.CPT
qc (MPa)
0 5 10 15 20
Depth (m)
Water Table
qc measured
Friction Ratio (%)
8 6 4 2 0
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)

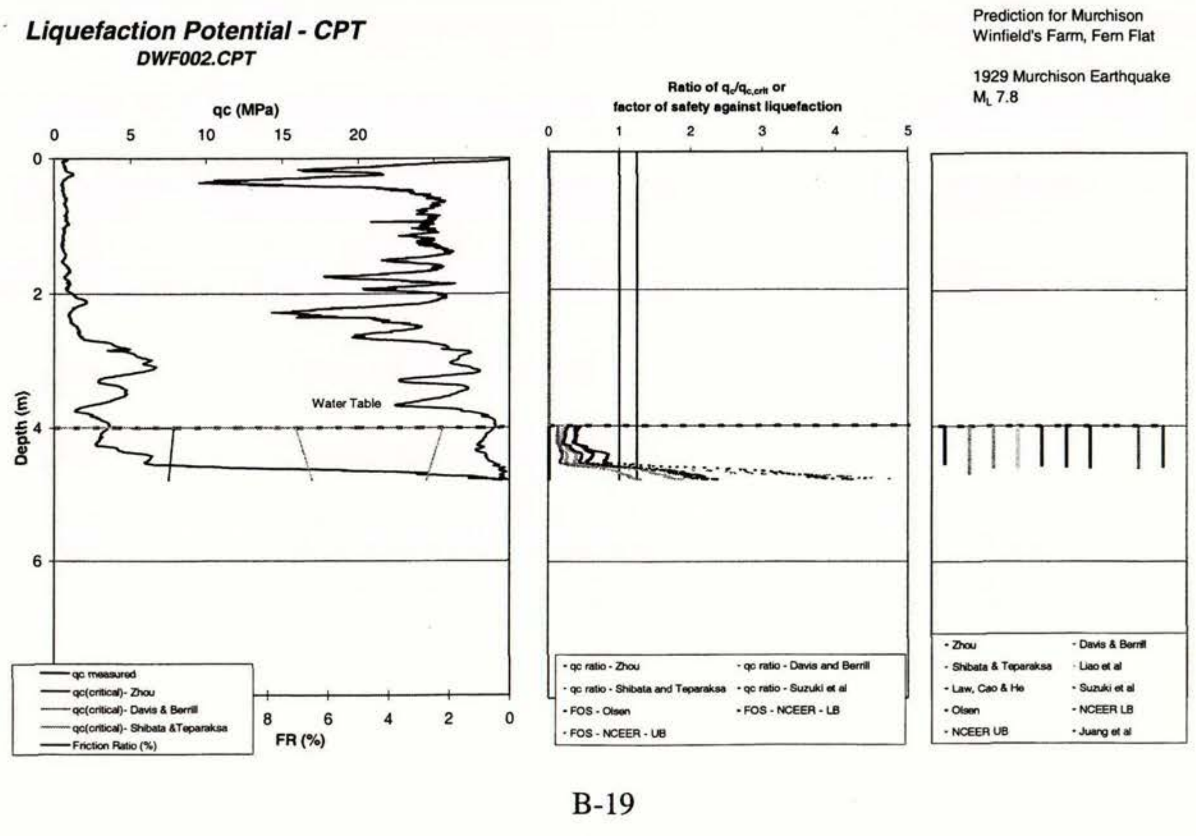

Liquefaction Potential - CPT
DWF001.CPT
qc (MPa)
0 5 10 15 20
Depth (m)
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Prediction for Murchison
Winfield's Farm, Fern Flat
1929 Murchison Earthquake
M_L 7.8
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

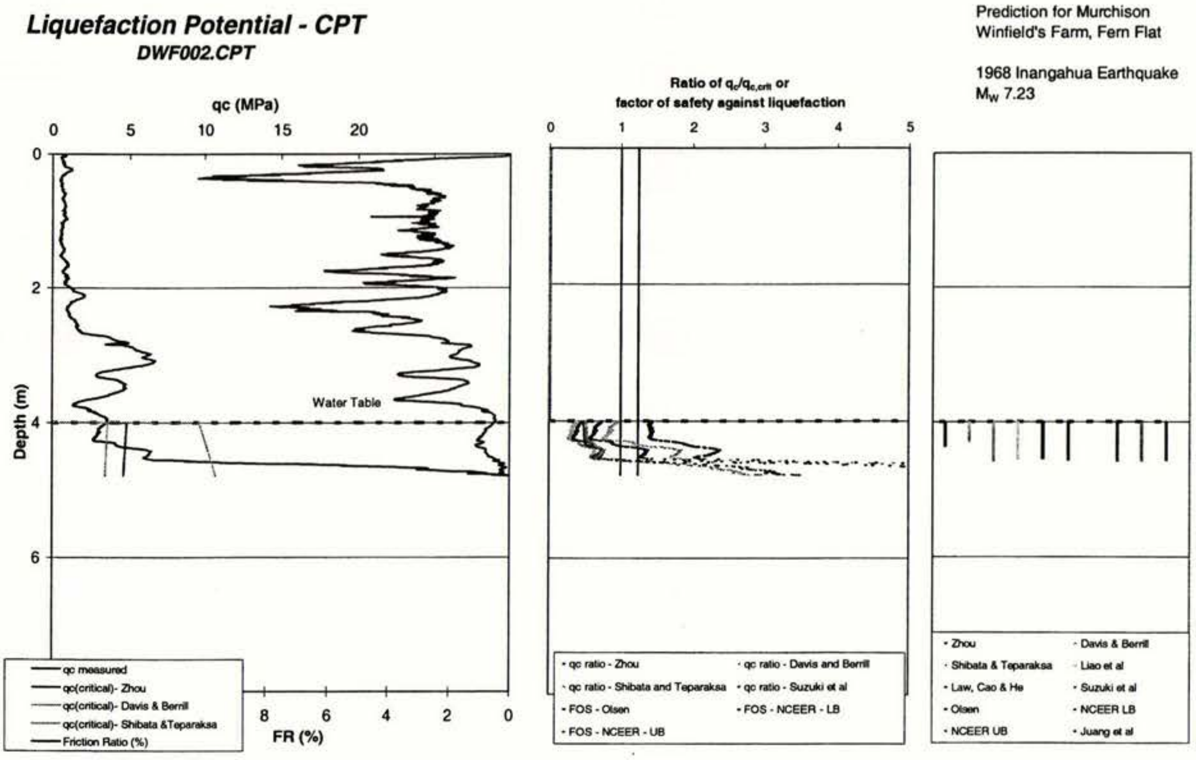

Liquefaction Potential - CPT
DWF001.CPT
Prediction for Murchison
Winfield's Farm, Fern Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0 5 10 15 20
Depth (m)
Water Table
FR (%)
8 6 4 2 0
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
- qc ratio - Zhou
- qc ratio - Davis and Berrill
- qc ratio - Shibata and Teparaksa
- qc ratio - Suzuki et al
- FOS - Olsen
- FOS - NCEER - LB
- FOS - NCEER - UB
- Zhou
- Davis & Berrill
- Shibata & Teparaksa
- Liao et al
- Law, Cao & He
- Suzuki et al
- Olsen
- NCEER LB
- NCEER UB
- Juang et al

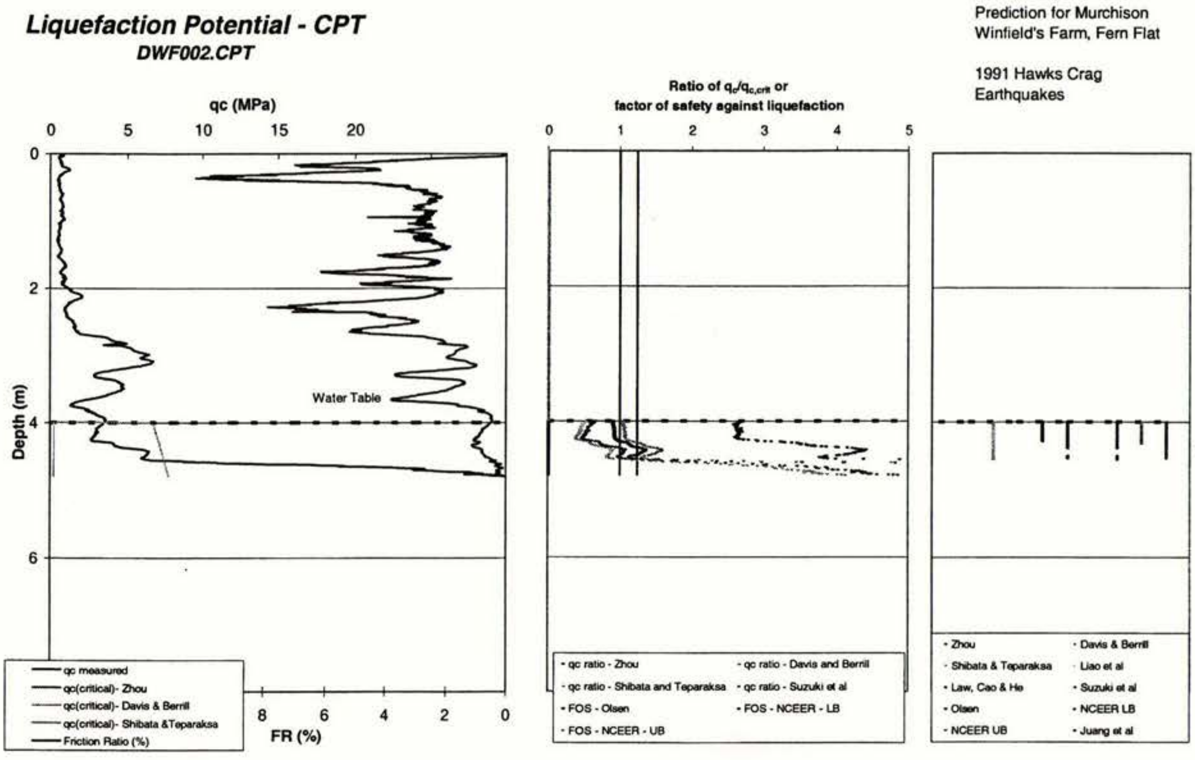

Liquefaction Potential - CPT
DWF001.CPT
Prediction for Murchison
Winfield's Farm, Fern Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
0 5 10 15 20
Depth (m)
Water Table
FR (%)
8 6 4 2 0
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
DB2
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
- qc ratio - Zhou
- qc ratio - Davis and Berrill
- qc ratio - Shibata and Teparaksa
- qc ratio - Suzuki et al
- FOS - Olsen
- FOS - NCEER - LB
- FOS - NCEER - UB
- Zhou
- Davis & Berrill
- Shibata & Teparaksa
- Liao et al
- Law, Cao & He
- Suzuki et al
- Olsen
- NCEER LB
- NCEER UB
- Juang et al

WIN003.CPT

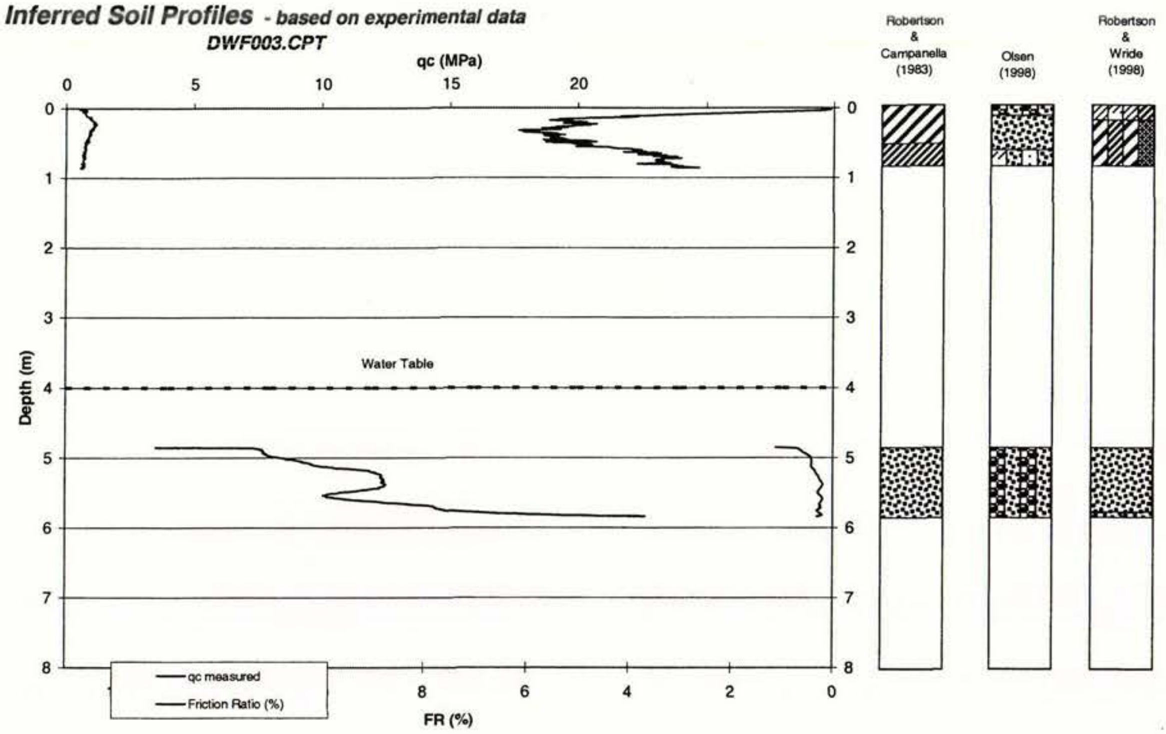

Inferred Soil Profiles - based on experimental data
DWF002.CPT
qc (MPa)
Depth (m)
Water Table
qc measured
Friction Ratio (%)
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)

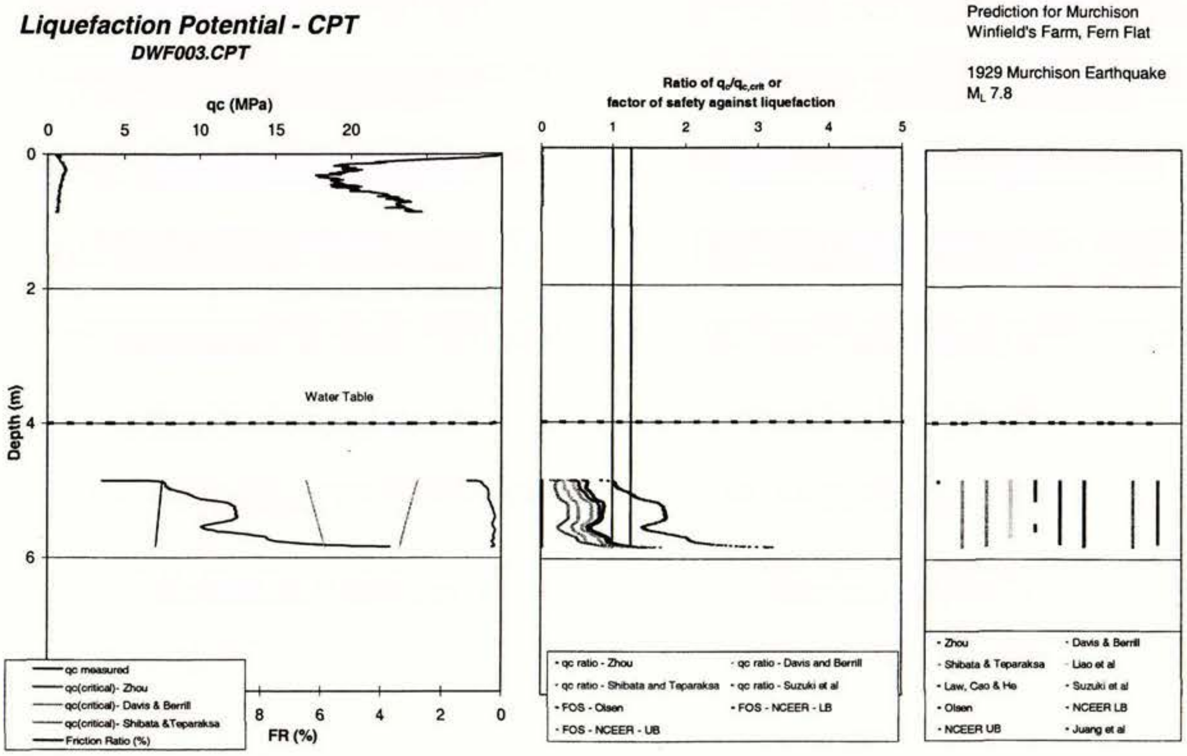

Liquefaction Potential - CPT
DWF002.CPT
Prediction for Murchison
Winfield's Farm, Fern Flat
1929 Murchison Earthquake
M_L 7.8
qc (MPa)
Depth (m)
Water Table
Ratio of qc/qc,crit or factor of safety against liquefaction
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

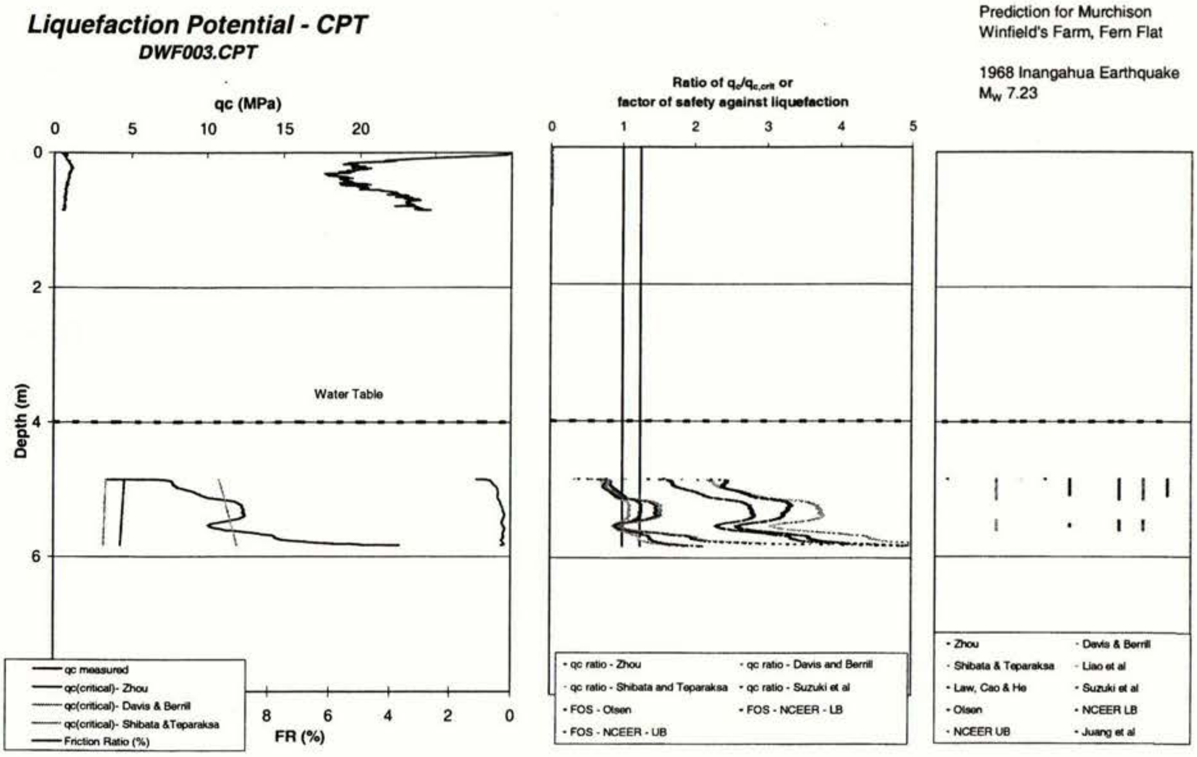

Liquefaction Potential - CPT
DWF002.CPT
Prediction for Murchison
Winfield's Farm, Fern Flat
1968 Inangahua Earthquake
M$_W$ 7.23
qc (MPa)
0
5
10
15
20
Depth (m)
Water Table
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

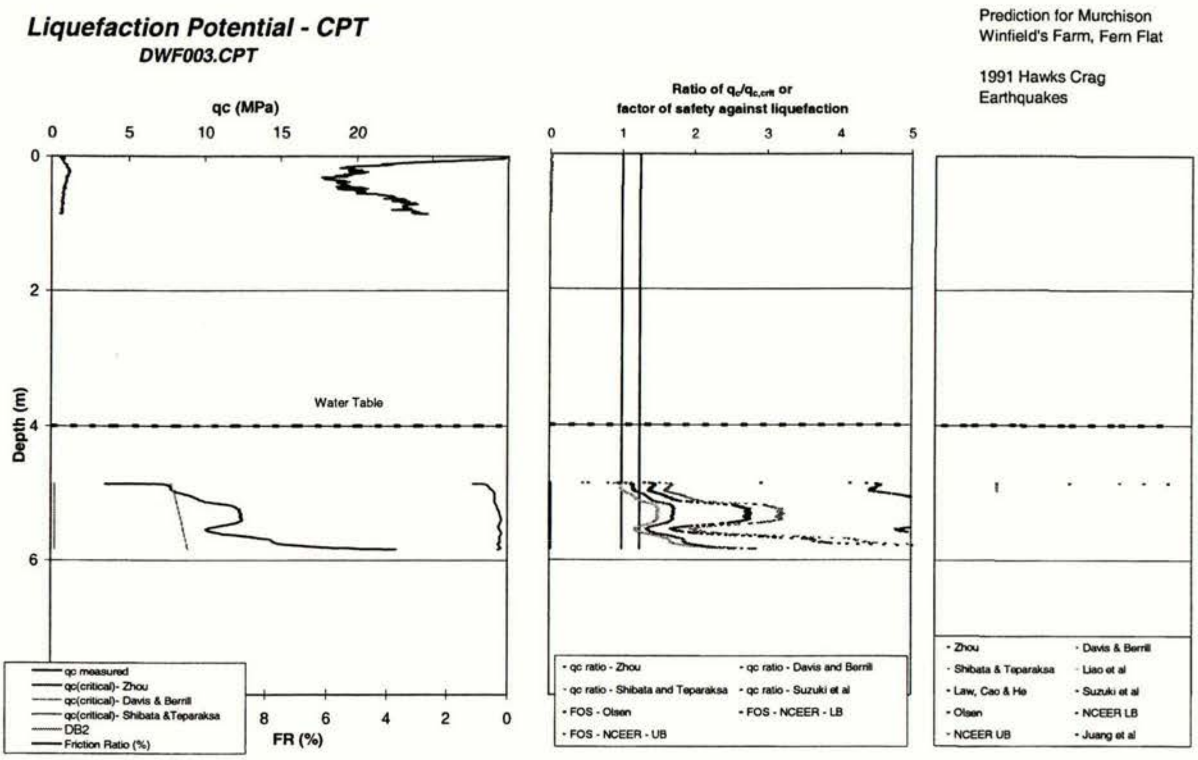

Liquefaction Potential - CPT
DWF002.CPT
Prediction for Murchison
Winfield's Farm, Fern Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
0
5
10
15
20
Depth (m)
Water Table
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

WIN004.CPT

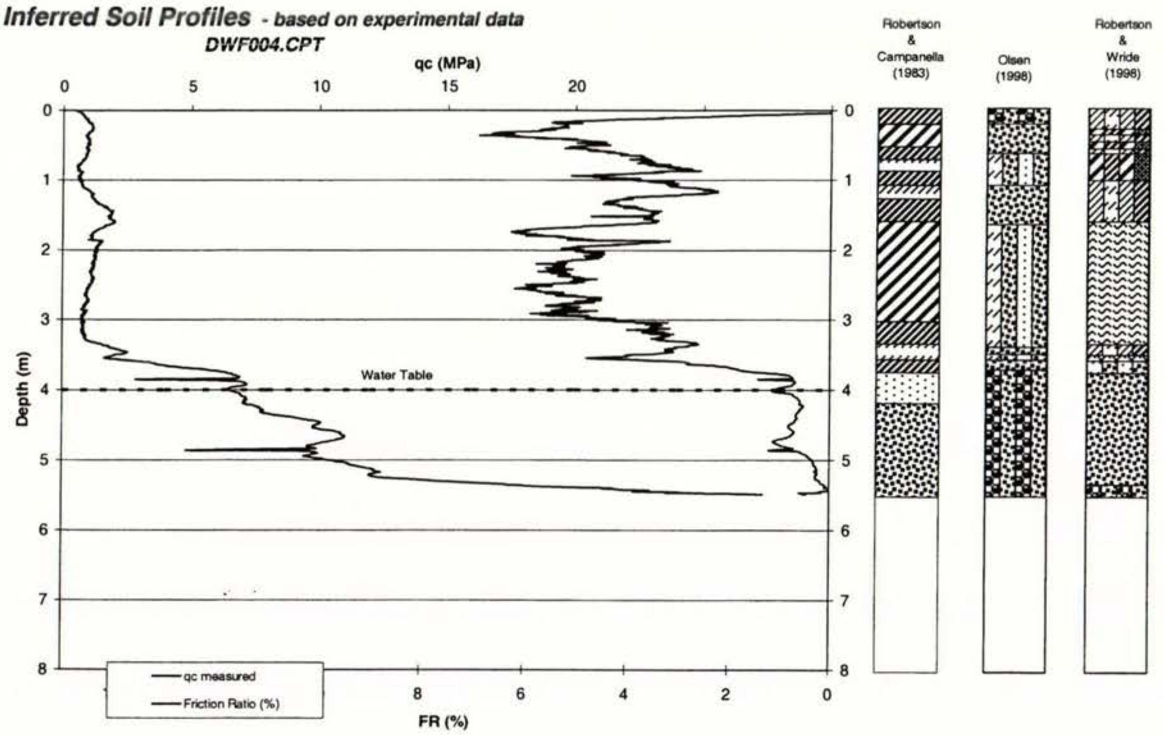

Inferred Soil Profiles - based on experimental data
DWF003.CPT
qc (MPa)
0 5 10 15 20
Depth (m)
Water Table
qc measured
Friction Ratio (%)
FR (%)
8 6 4 2 0
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)

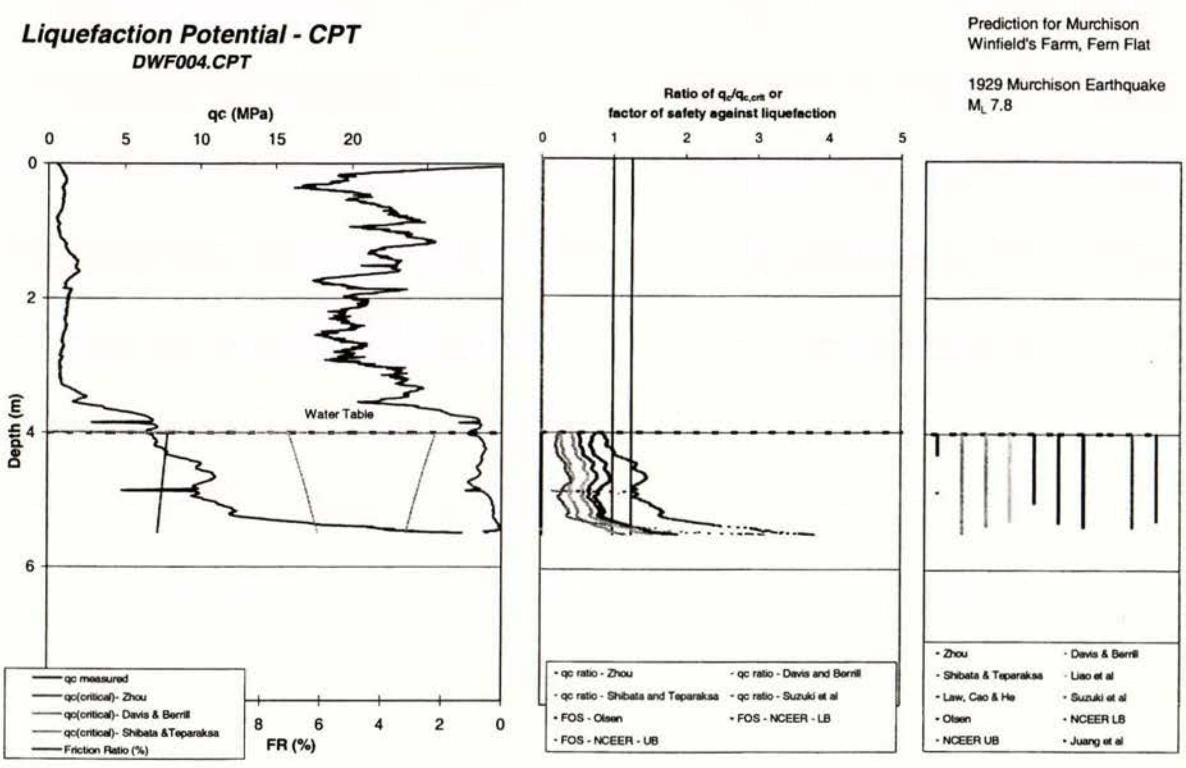

Liquefaction Potential - CPT
DWF003.CPT
qc (MPa)
0 5 10 15 20
Depth (m)
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
8 6 4 2 0
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Prediction for Murchison
Winfield's Farm, Fern Flat
1929 Murchison Earthquake
M_L 7.8
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

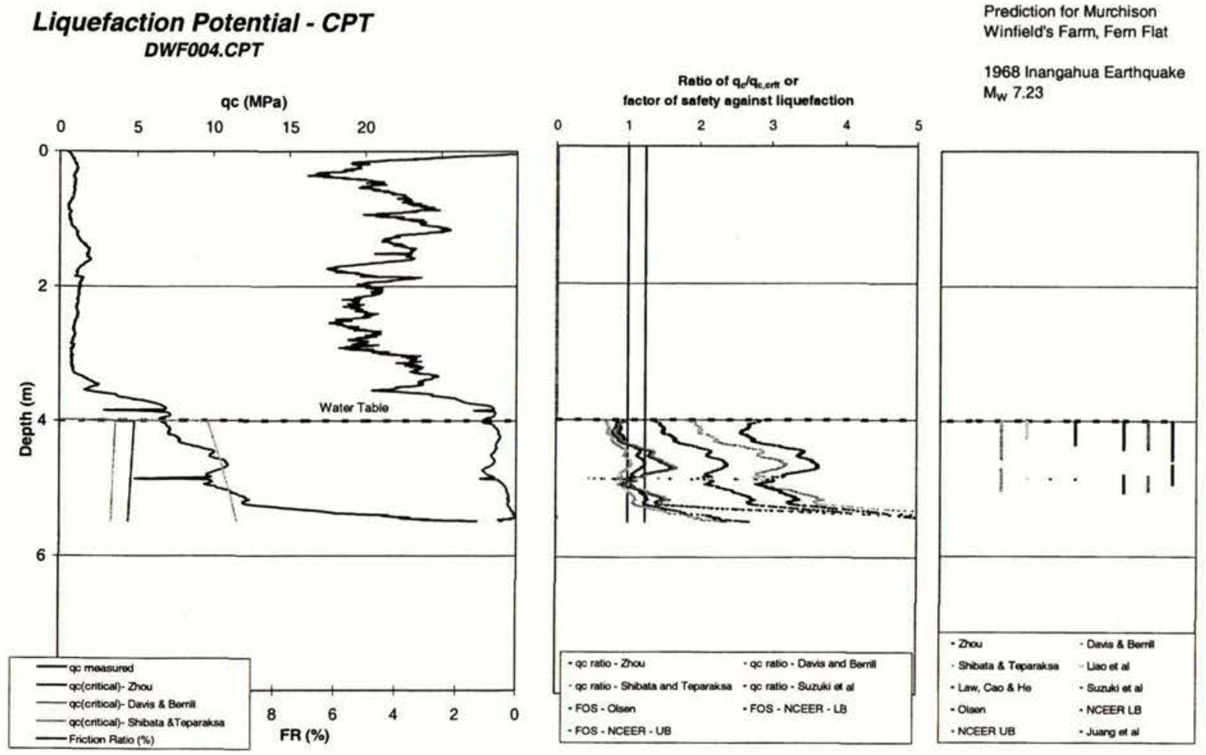

Liquefaction Potential - CPT
DWF003.CPT
Prediction for Murchison
Winfield's Farm, Fern Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
Depth (m)
Water Table
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

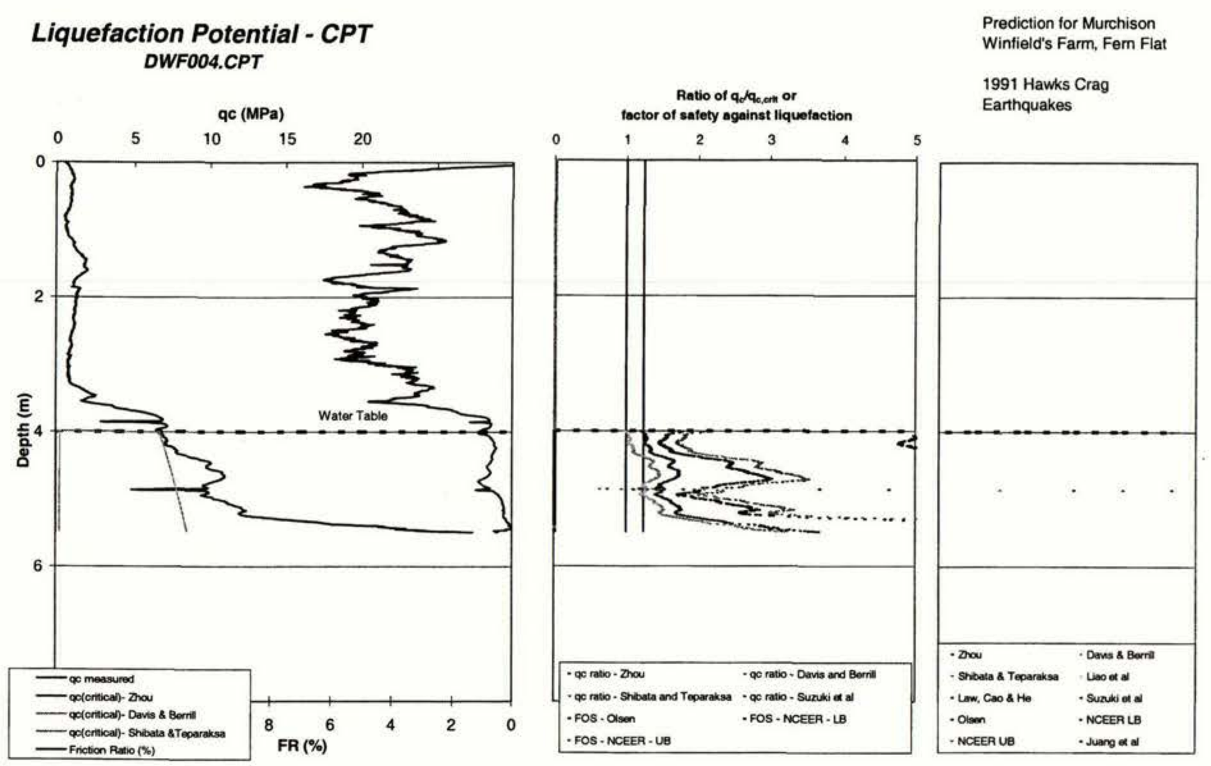

Liquefaction Potential - CPT
DWF003.CPT
Prediction for Murchison
Winfield's Farm, Fern Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
Depth (m)
Water Table
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
DB2
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

WIN005.CPT

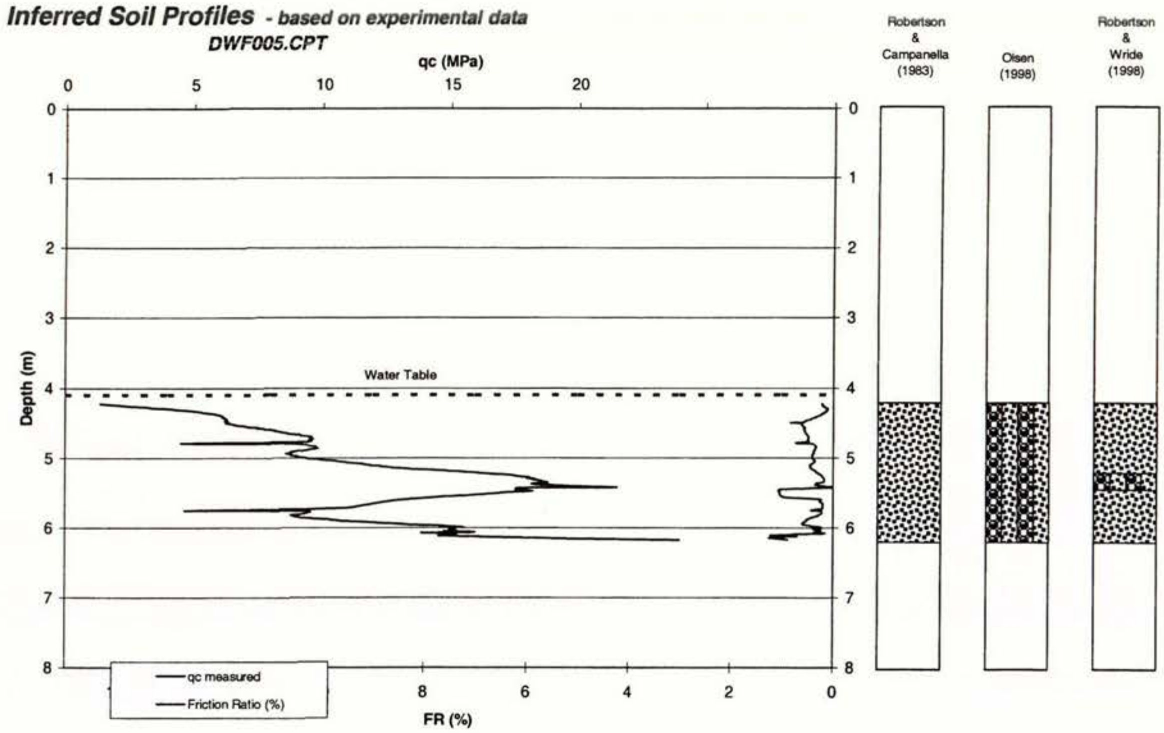

Inferred Soil Profiles - based on experimental data
DWF004.CPT
qc (MPa)
Depth (m)
Water Table
qc measured
Friction Ratio (%)
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)

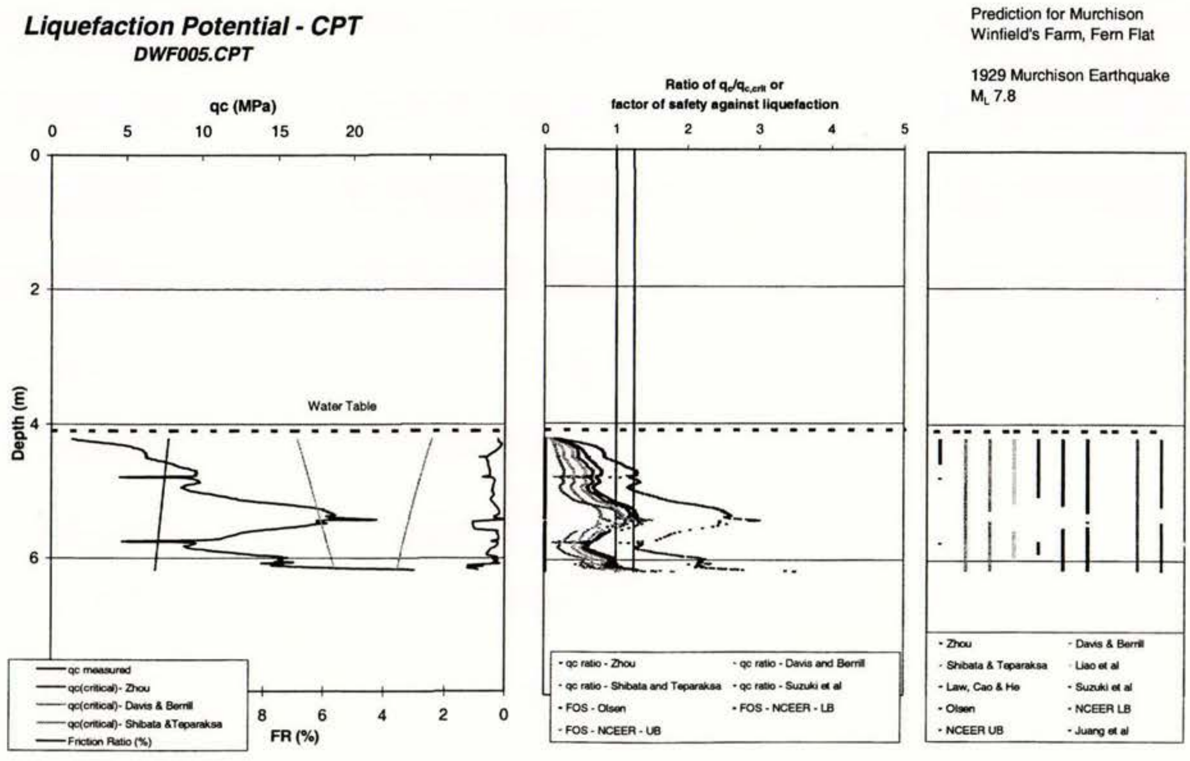

Liquefaction Potential - CPT
DWF004.CPT
qc (MPa)
Depth (m)
Water Table
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
- qc ratio - Zhou
- qc ratio - Davis and Berrill
- qc ratio - Shibata and Teparaksa
- qc ratio - Suzuki et al
- FOS - Olsen
- FOS - NCEER - LB
- FOS - NCEER - UB
Prediction for Murchison
Winfield's Farm, Fern Flat
1929 Murchison Earthquake
M_L 7.8
- Zhou
- Davis & Berrill
- Shibata & Teparaksa
- Liao et al
- Law, Cao & He
- Suzuki et al
- Olsen
- NCEER LB
- NCEER UB
- Juang et al

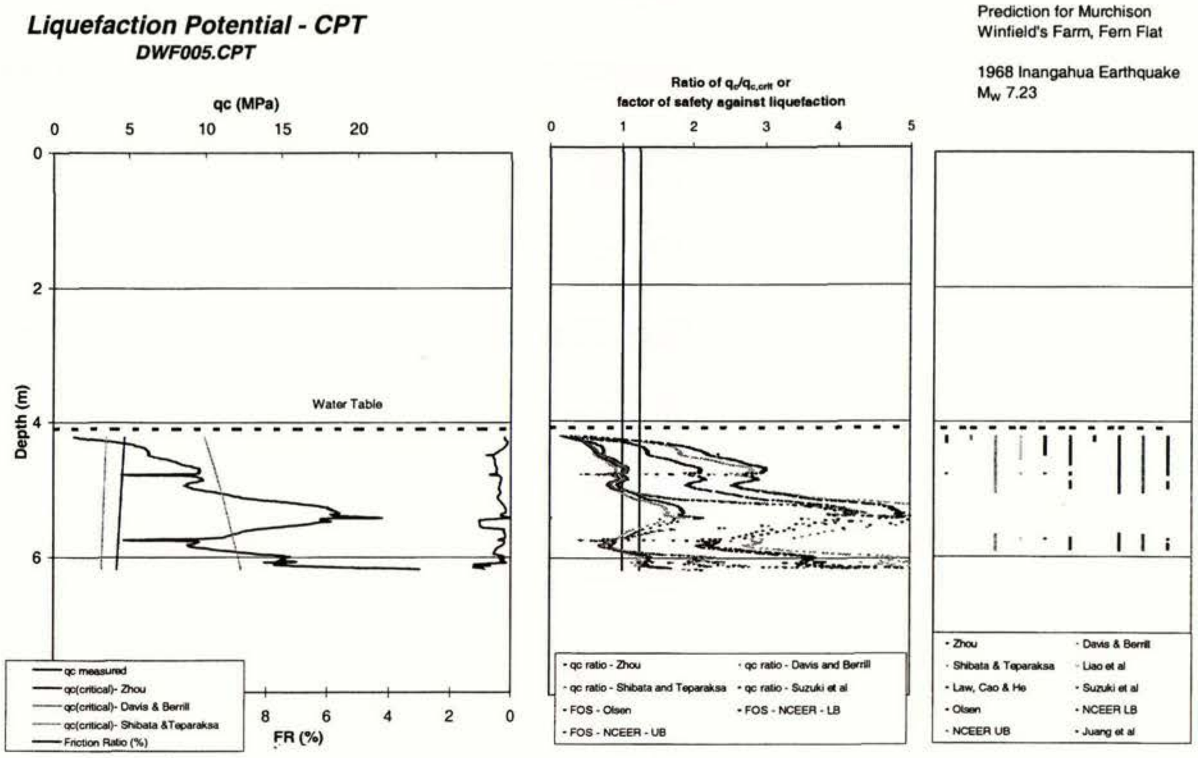

Liquefaction Potential - CPT
DWF004.CPT
Prediction for Murchison
Winfield's Farm, Fern Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
Depth (m)
Water Table
FR (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

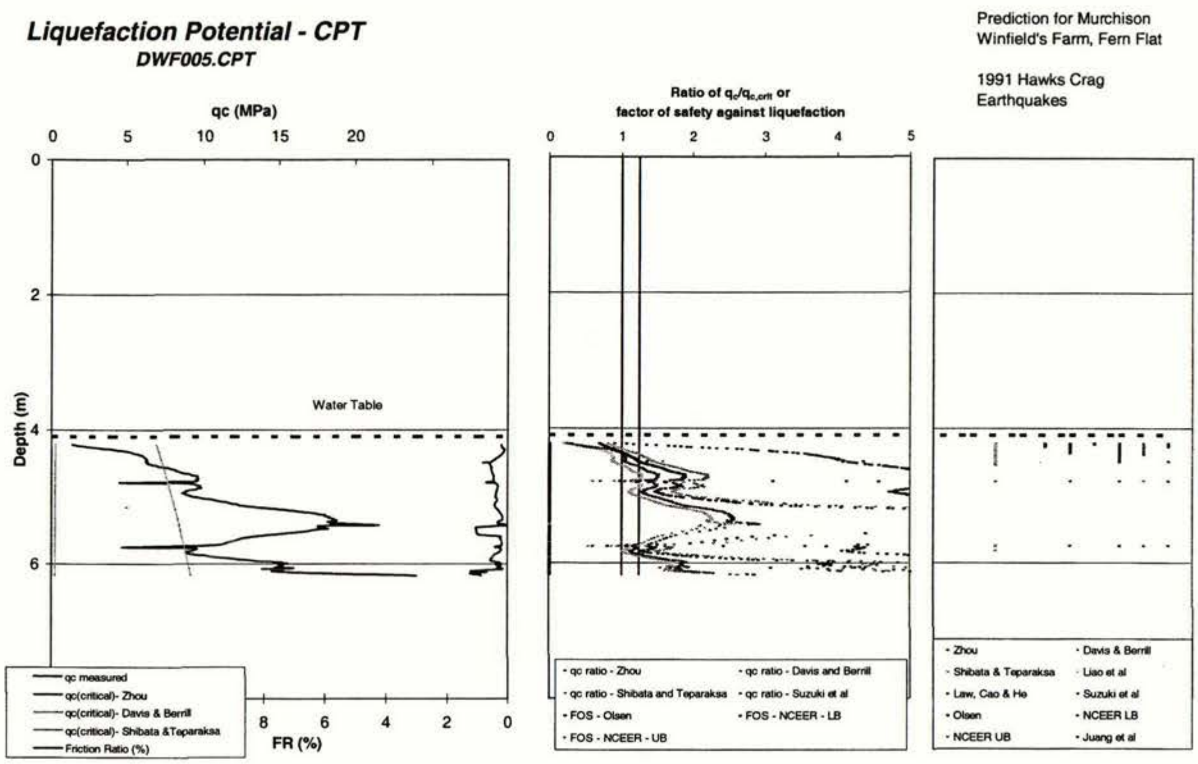

Liquefaction Potential - CPT
DWF004.CPT
Prediction for Murchison
Winfield's Farm, Fern Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
Depth (m)
Water Table
FR (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

WIN006.CPT

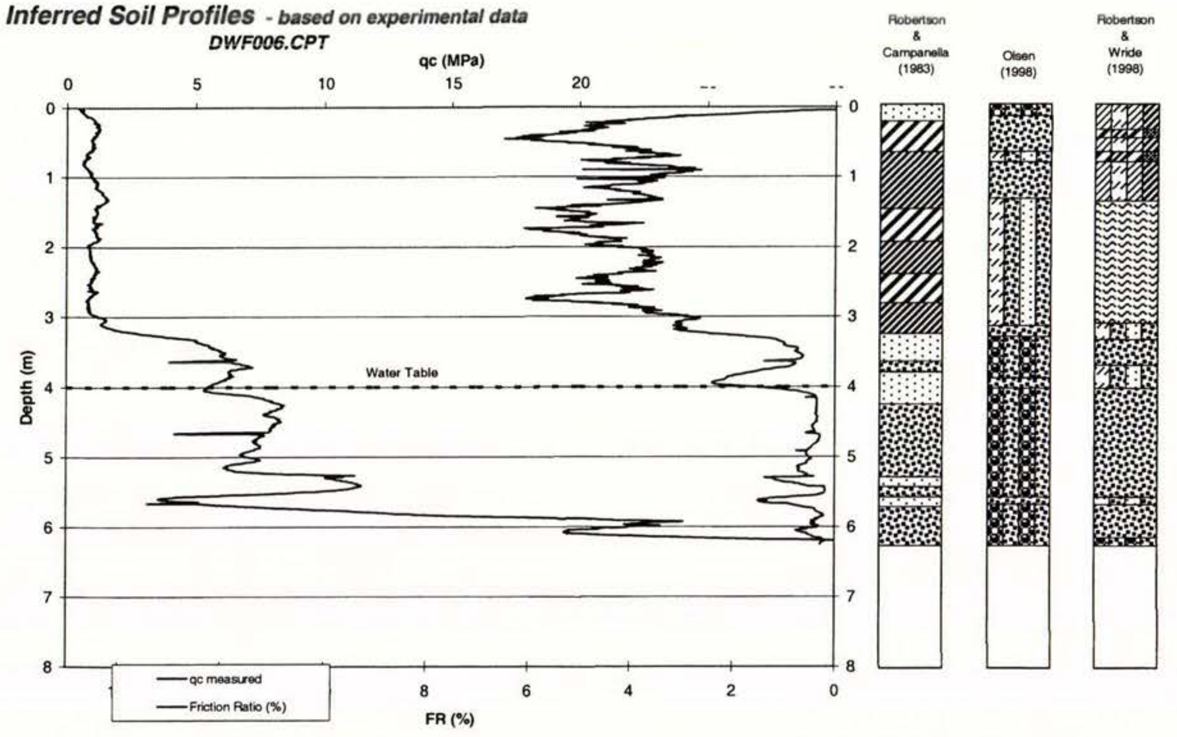

Inferred Soil Profiles - based on experimental data
DWF005.CPT
qc (MPa)
Depth (m)
Water Table
FR (%)
qc measured
Friction Ratio (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)

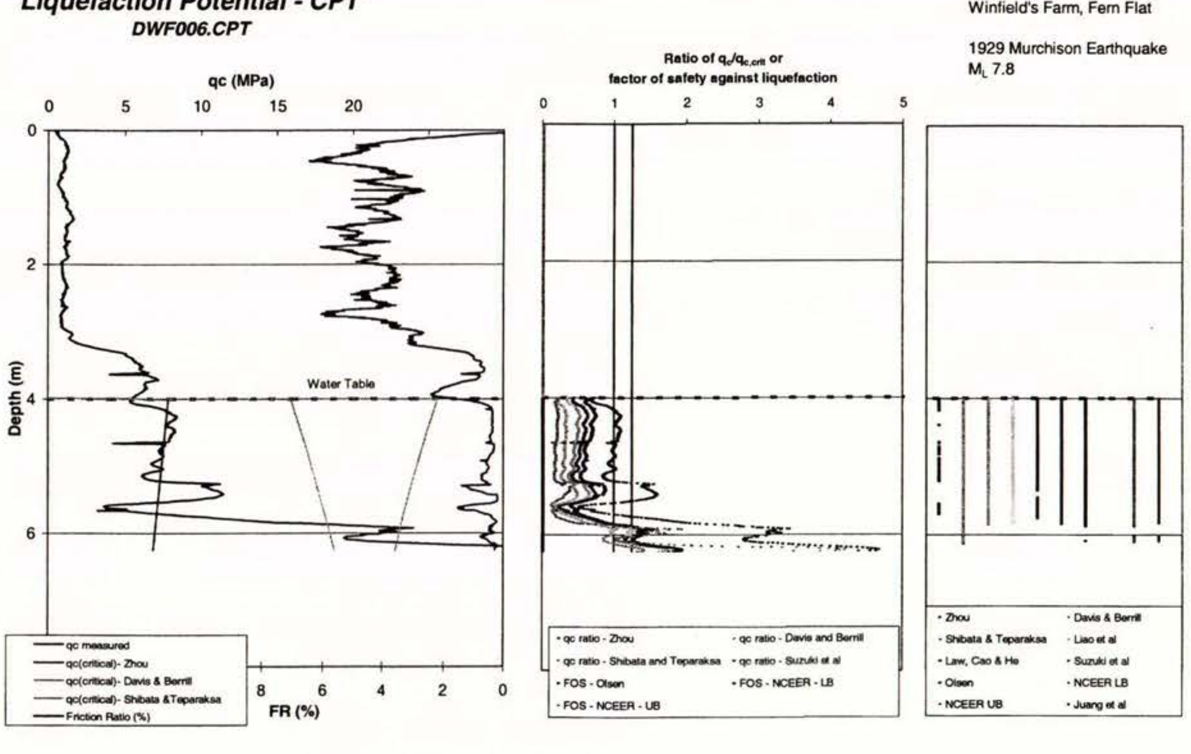

Liquefaction Potential - CPT
DWF005.CPT
Prediction for Murchison
Winfield's Farm, Fern Flat
1929 Murchison Earthquake
M_L 7.8
qc (MPa)
Depth (m)
Water Table
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
- qc ratio - Zhou
- qc ratio - Davis and Berrill
- qc ratio - Shibata and Teparaksa
- qc ratio - Suzuki et al
- FOS - Olsen
- FOS - NCEER - LB
- FOS - NCEER - UB
- Zhou
- Davis & Berrill
- Shibata & Teparaksa
- Liao et al
- Law, Cao & He
- Suzuki et al
- Olsen
- NCEER LB
- NCEER UB
- Juang et al

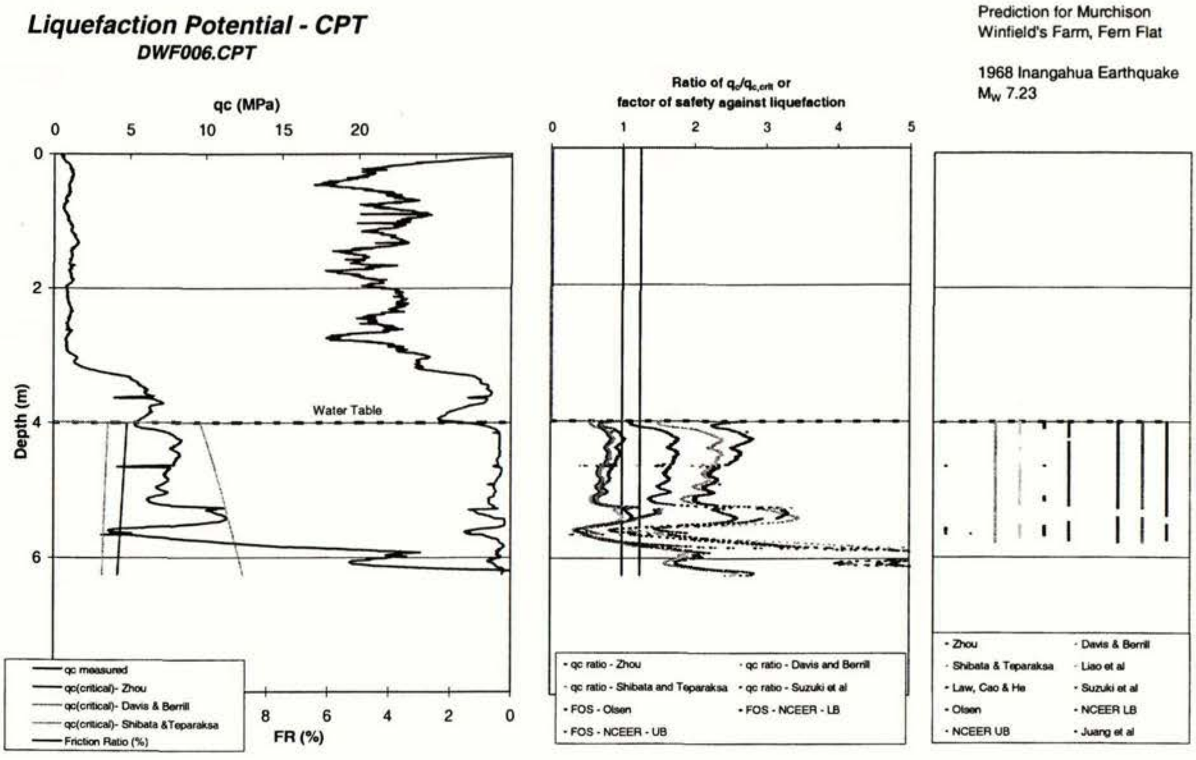

Liquefaction Potential - CPT
DWF005.CPT
Prediction for Murchison
Winfield's Farm, Fern Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0 5 10 15 20
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
0 1 2 3 4 5
Depth (m)
0 2 4 6
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

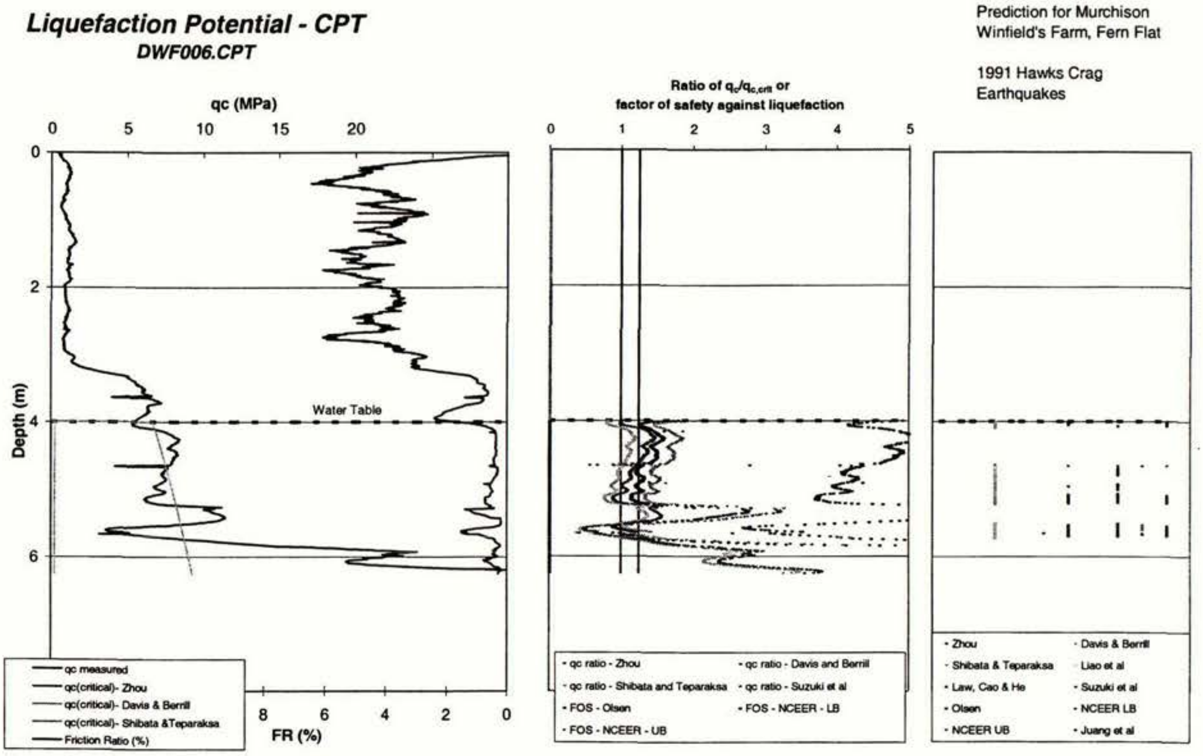

Liquefaction Potential - CPT
DWF005.CPT
Prediction for Murchison
Winfield's Farm, Fern Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
0 5 10 15 20
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
0 1 2 3 4 5
Depth (m)
0 2 4 6
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

WIN007.CPT

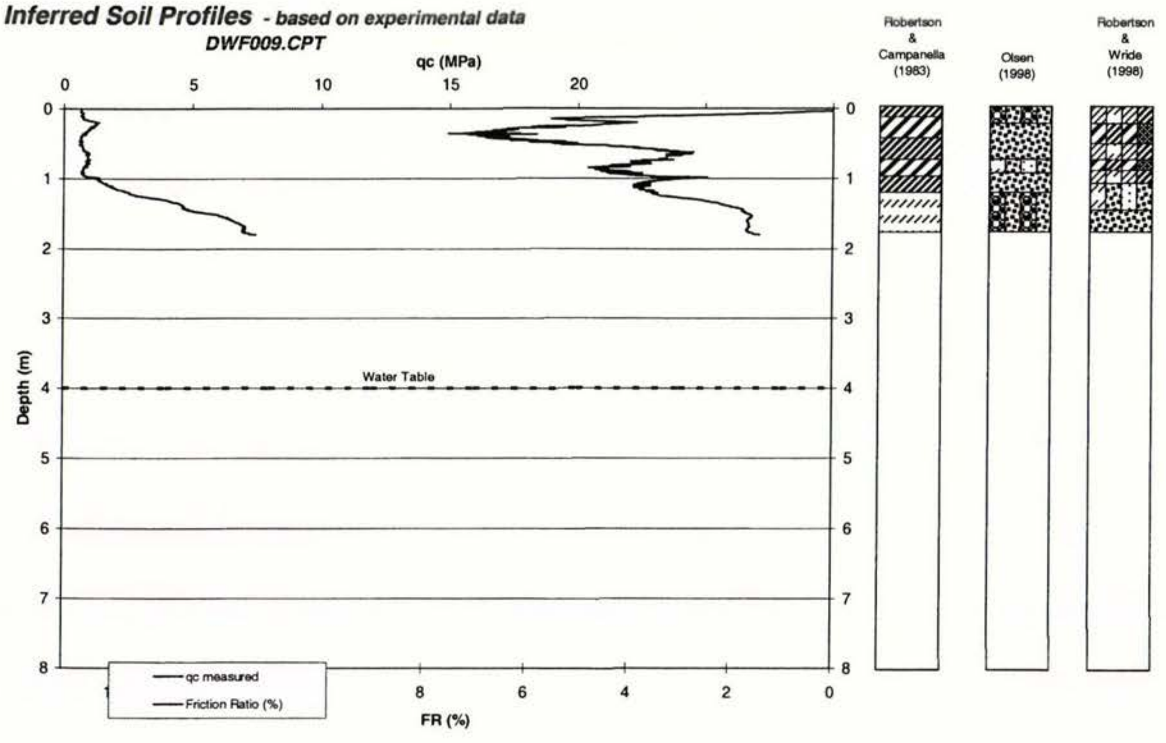

Inferred Soil Profiles - based on experimental data
DWF006.CPT
qc (MPa)
Depth (m)
Water Table
qc measured
Friction Ratio (%)
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)


Liquefaction Potential - CPT
DWF006.CPT
qc (MPa)
Depth (m)
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Prediction for Murchison
Winfield's Farm, Fern Flat
1929 Murchison Earthquake
M_L 7.8
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

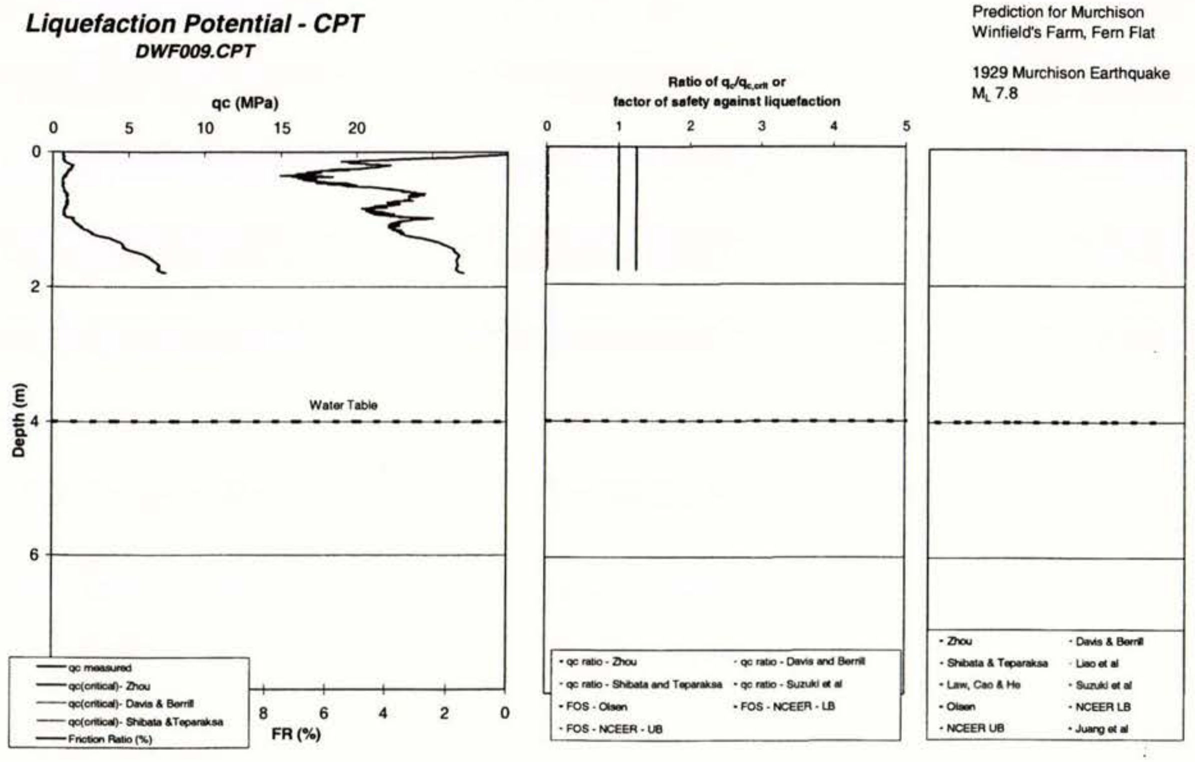

Liquefaction Potential - CPT
DWF006.CPT
Prediction for Murchison
Winfield's Farm, Fern Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0 5 10 15 20
Depth (m)
0 2 4 6
Water Table
8 6 4 2 0
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al


Liquefaction Potential - CPT
DWF006.CPT
Prediction for Murchison
Winfield's Farm, Fern Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
0 5 10 15 20
Depth (m)
0 2 4 6
Water Table
8 6 4 2 0
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

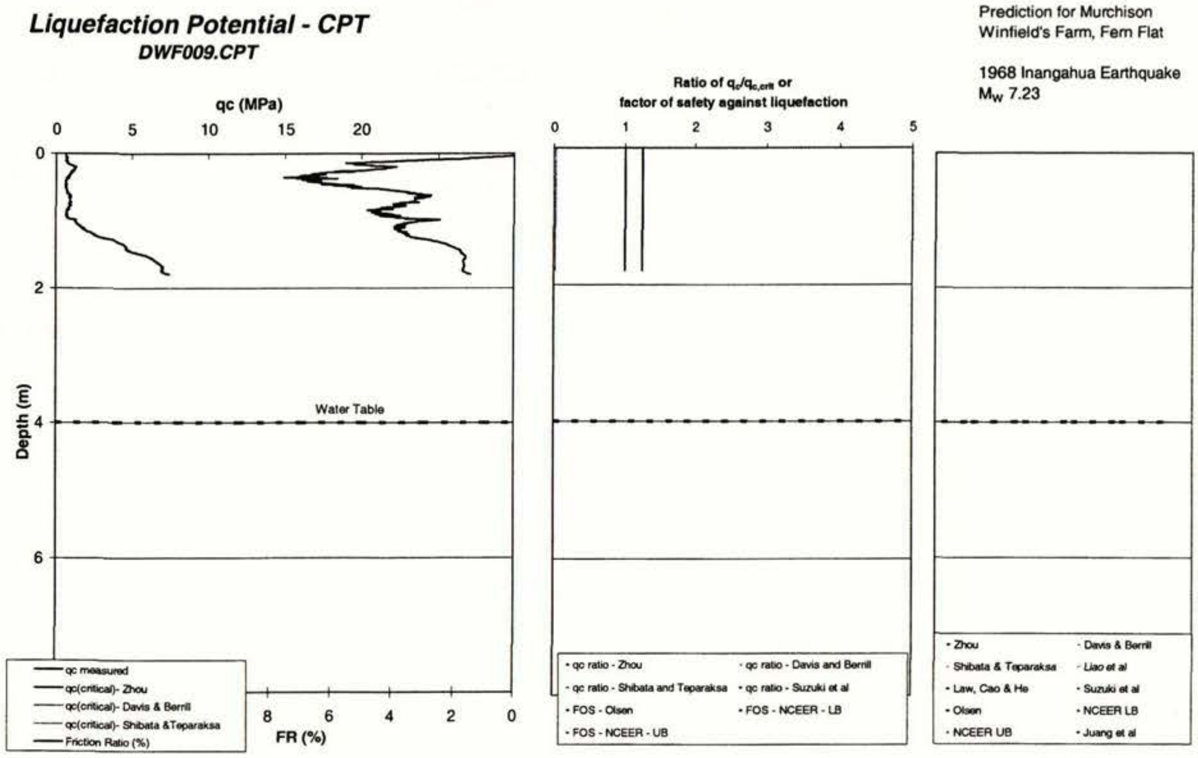
WIN008.CPT

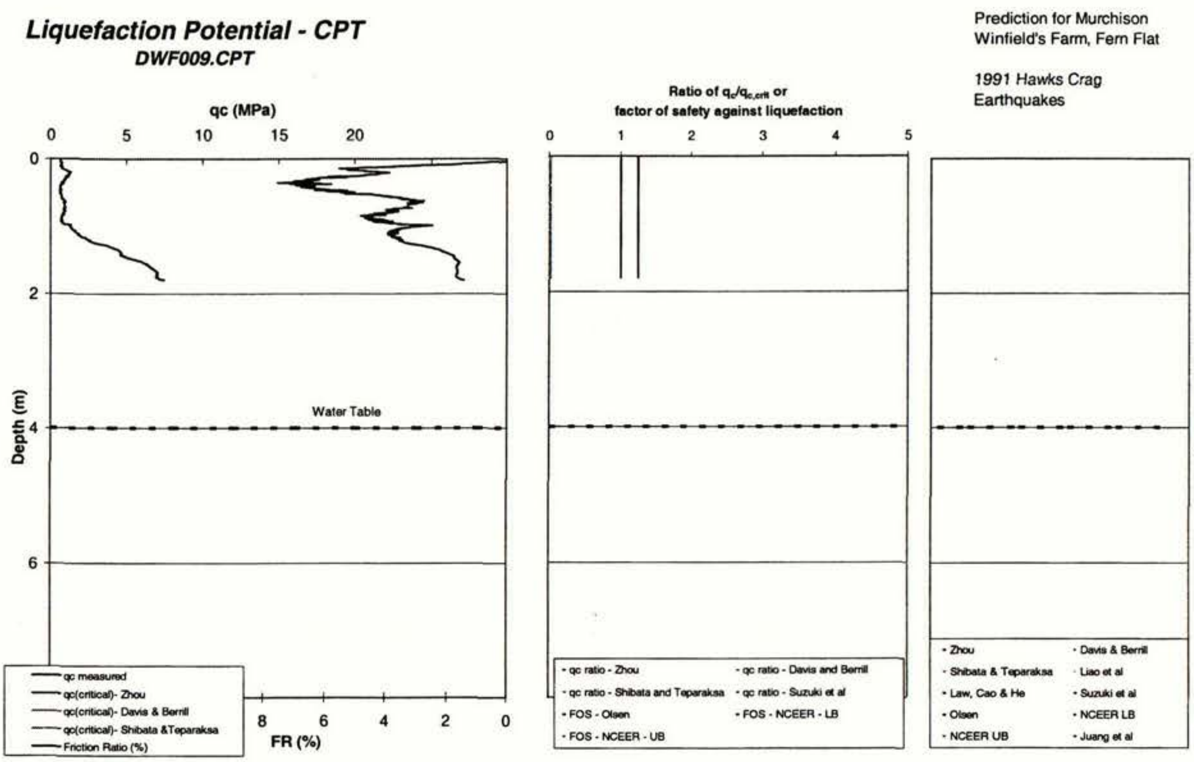


Inferred Soil Profiles - based on experimental data
DWF009.CPT
qc (MPa)
Depth (m)
Water Table
qc measured
Friction Ratio (%)
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)



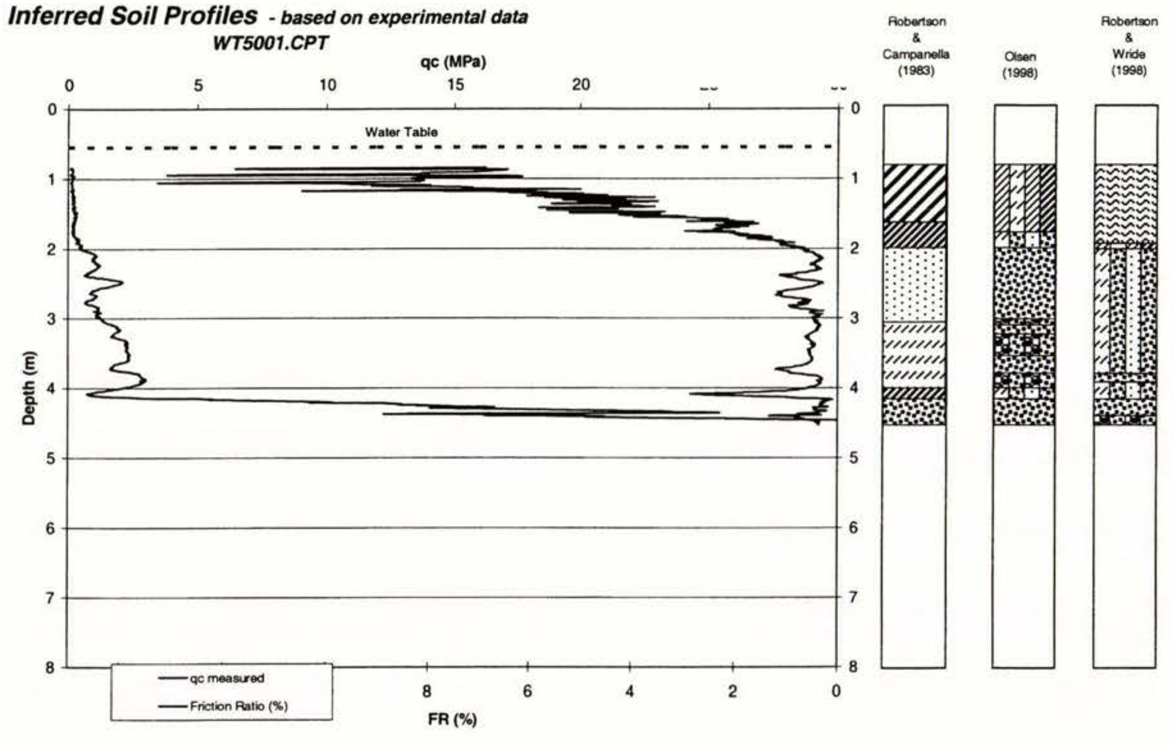
Liquefaction Potential - CPT
DWF009.CPT
qc (MPa)
Depth (m)
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Prediction for Murchison
Winfield's Farm, Fern Flat
1929 Murchison Earthquake
M_L 7.8
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

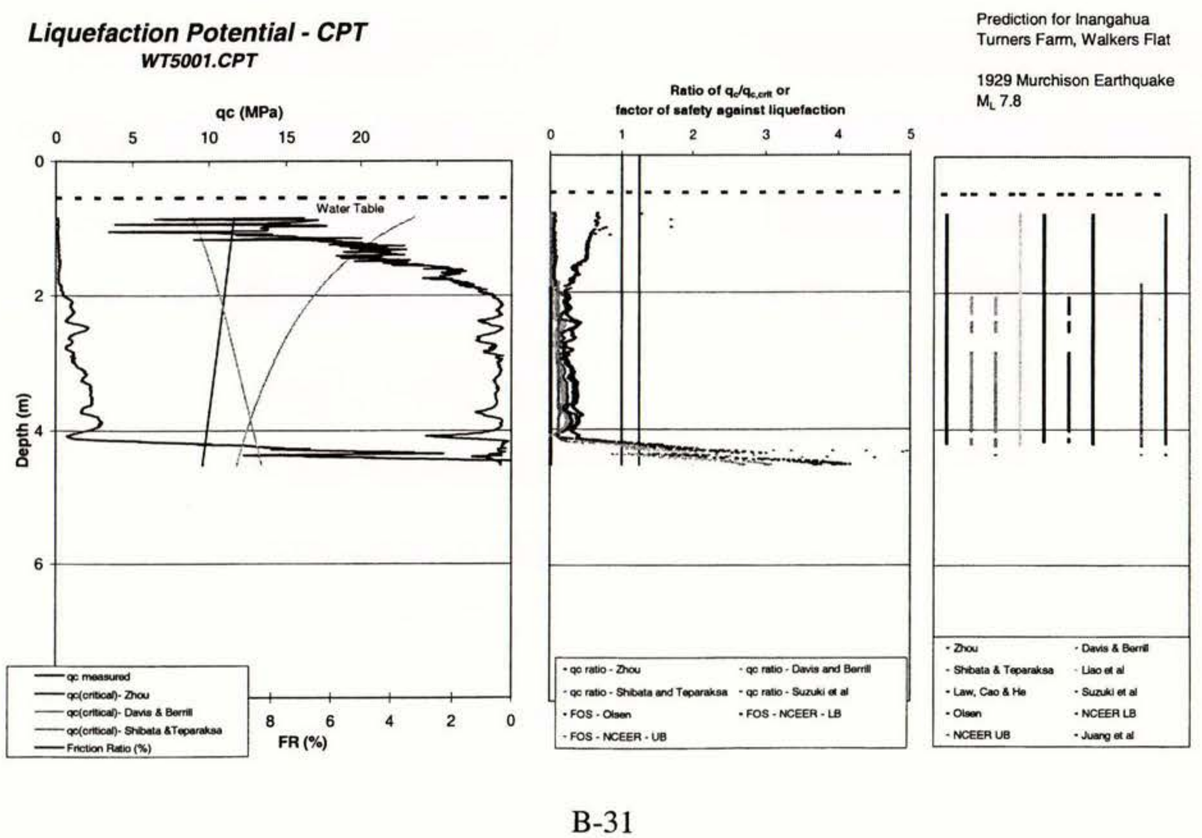

Liquefaction Potential - CPT
DWF009.CPT
Prediction for Murchison
Winfield's Farm, Fern Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0 5 10 15 20
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
Depth (m)
0 2 4 6
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

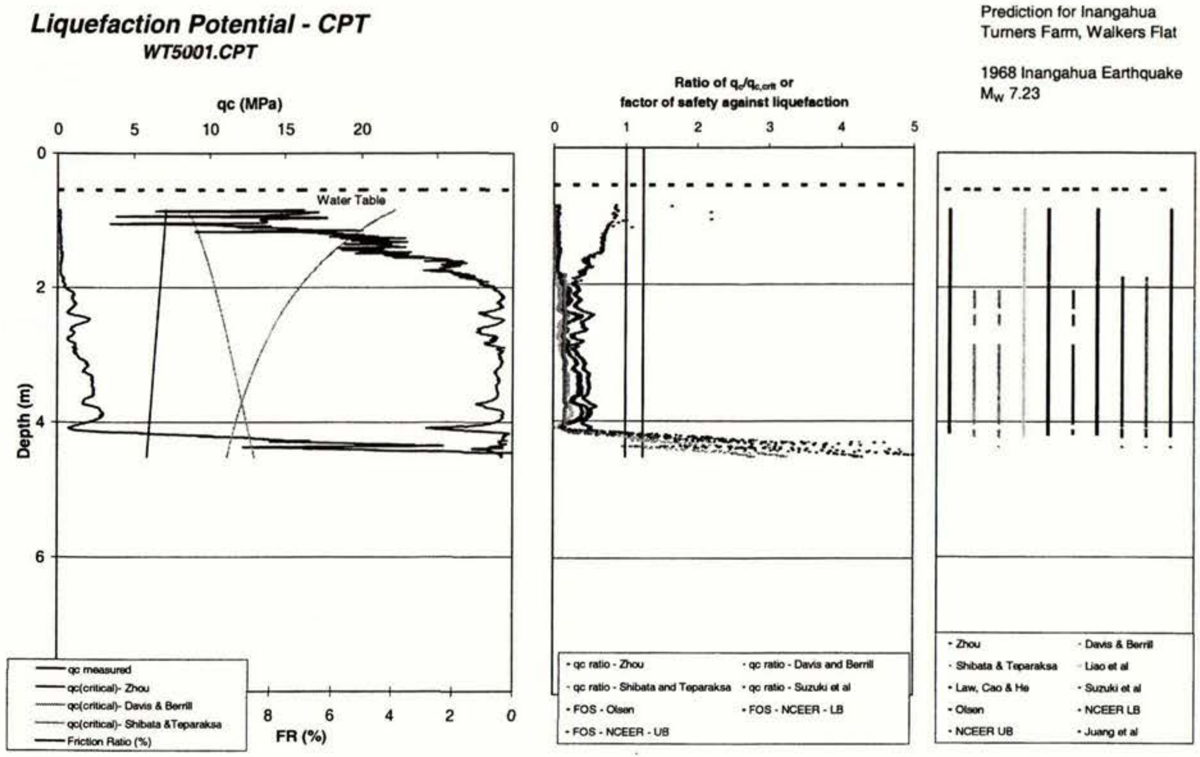

Liquefaction Potential - CPT
DWF009.CPT
Prediction for Murchison
Winfield's Farm, Fern Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
0 5 10 15 20
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
Depth (m)
0 2 4 6
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

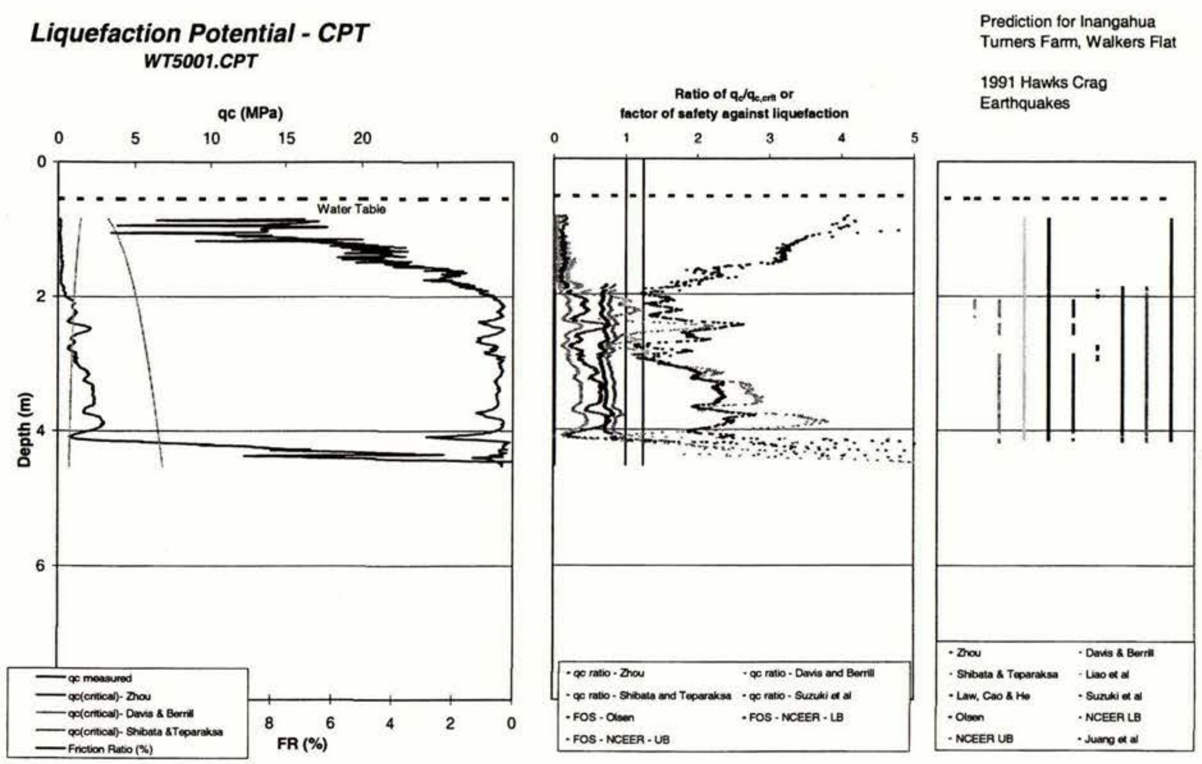
B1.1 Inangahua- Turner's Farm, Walkers Flat

TUR001.CPT

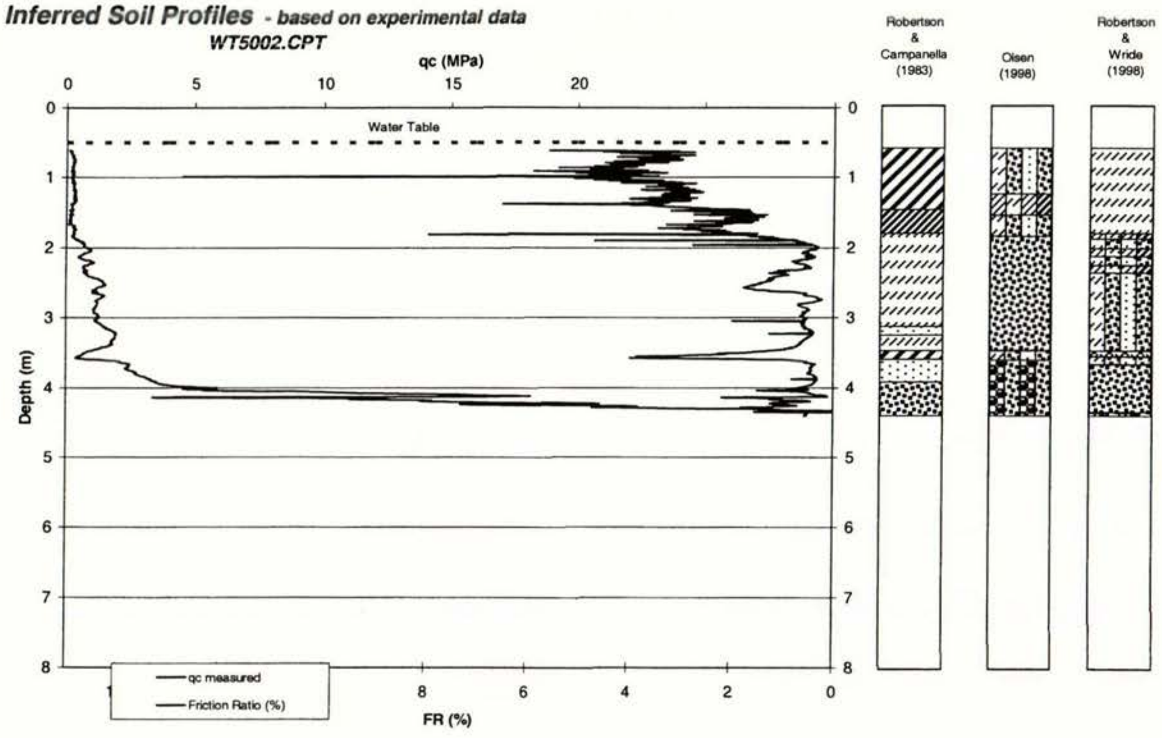


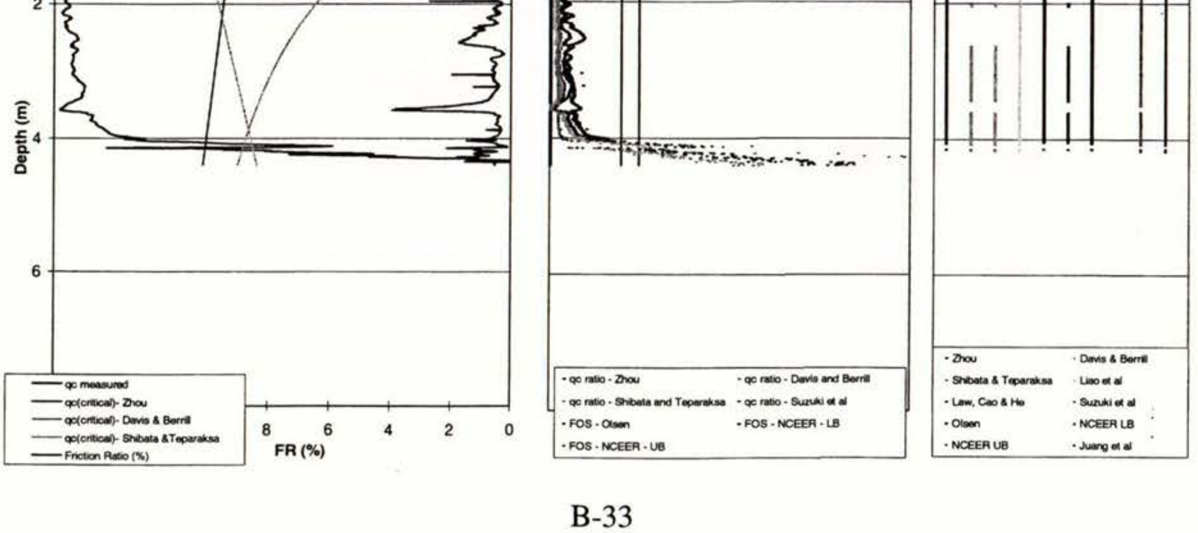


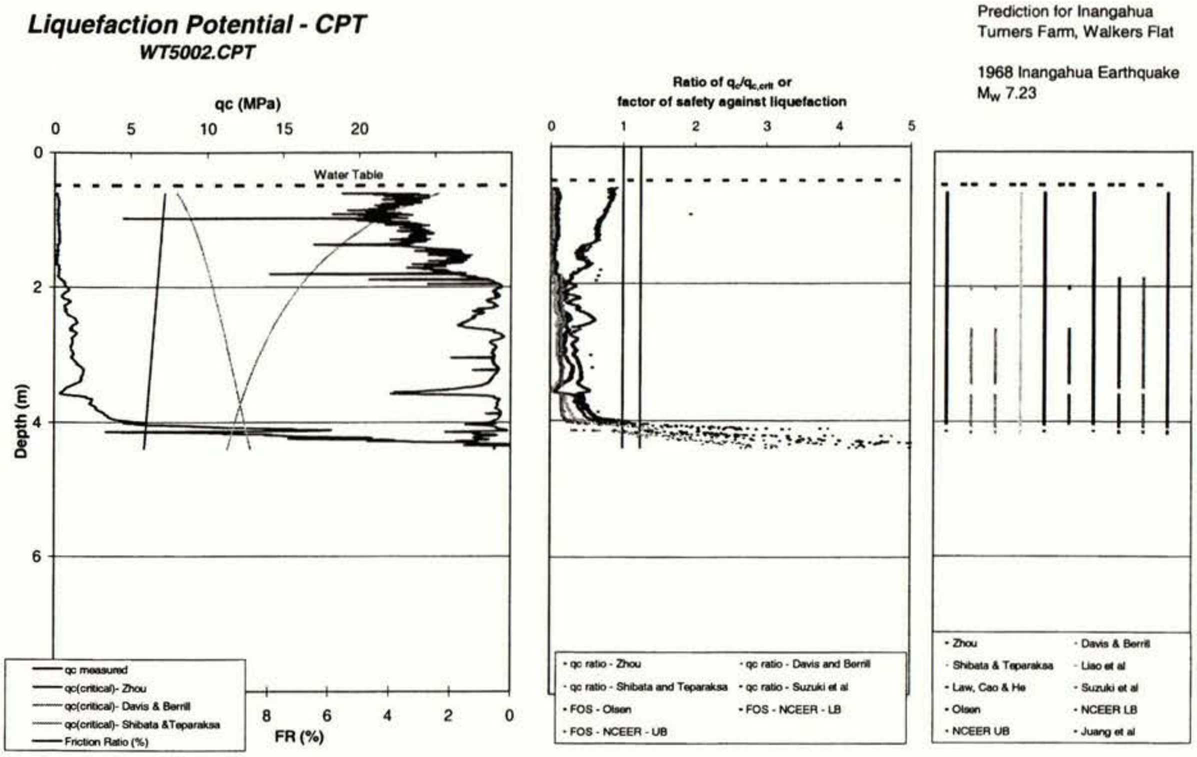

Liquefaction Potential - CPT
WT5001.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0 5 10 15 20
Depth (m)
0 2 4 6
Water Table
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

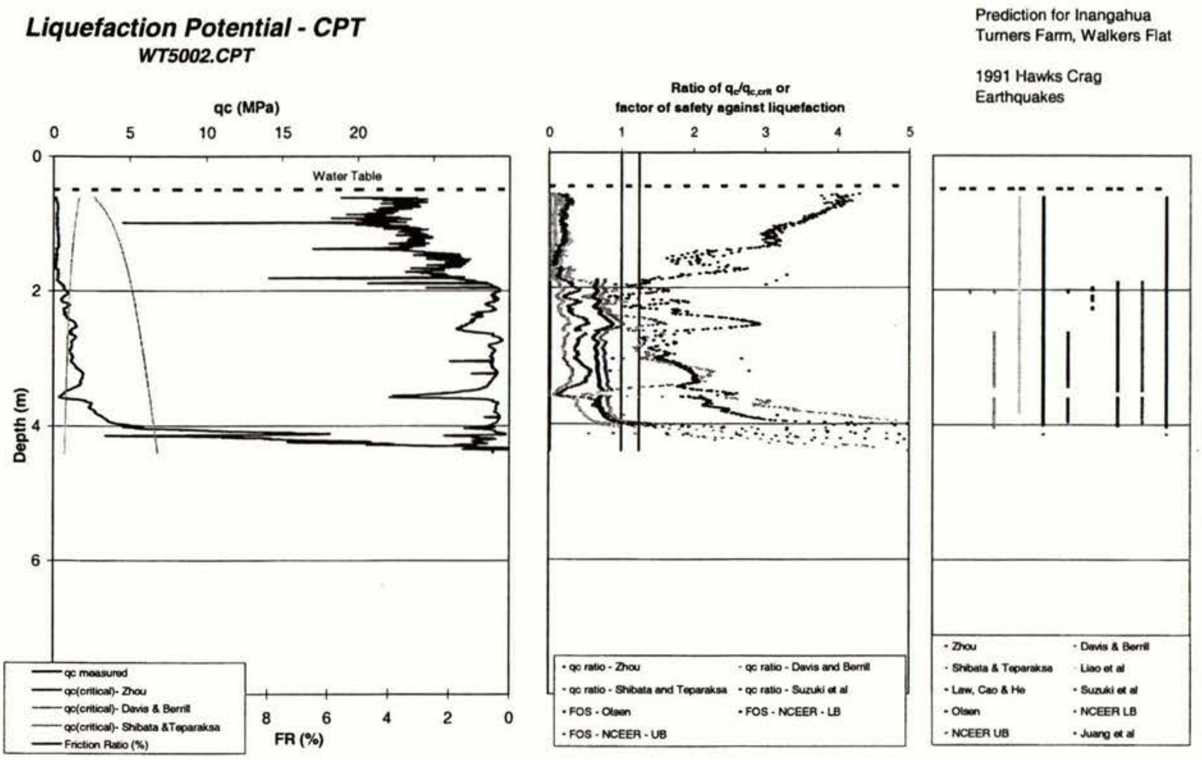

Liquefaction Potential - CPT
WT5001.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
0 5 10 15 20
Depth (m)
0 2 4 6
Water Table
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

TUR002.CPT

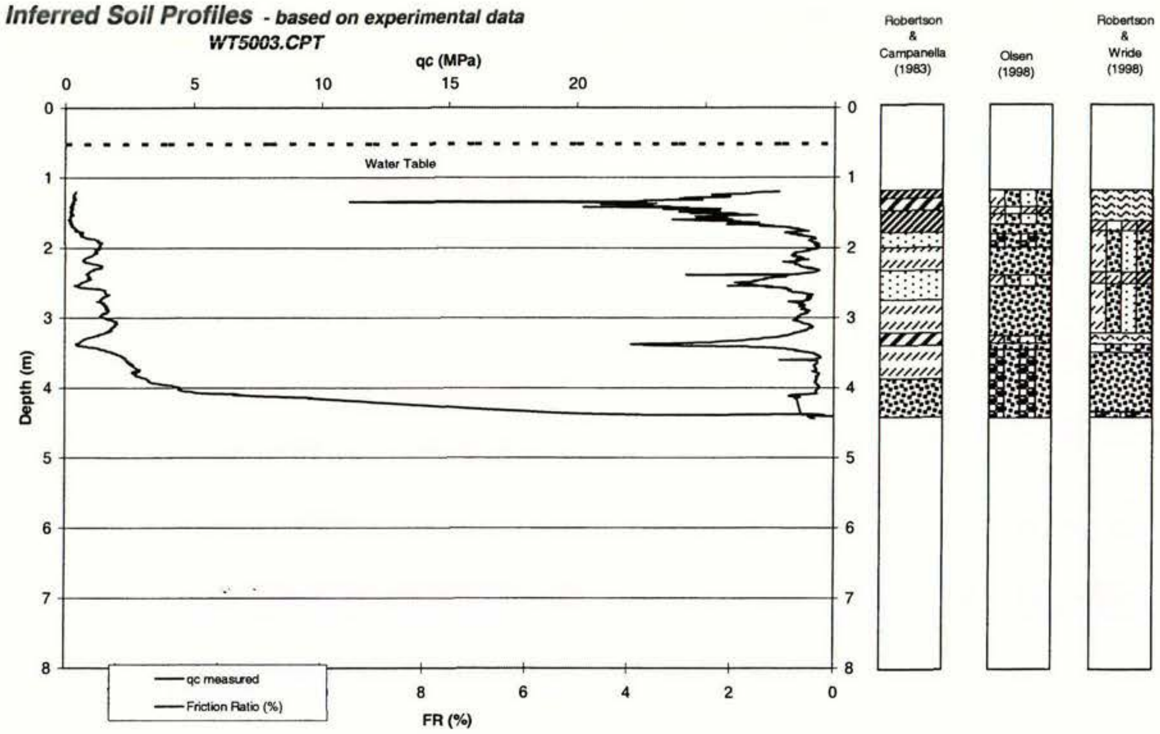

Inferred Soil Profiles - based on experimental data
WT5002.CPT
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
0 1 2 3 4 5 6 7 8
qc measured
Friction Ratio (%)
8 6 4 2 0
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)

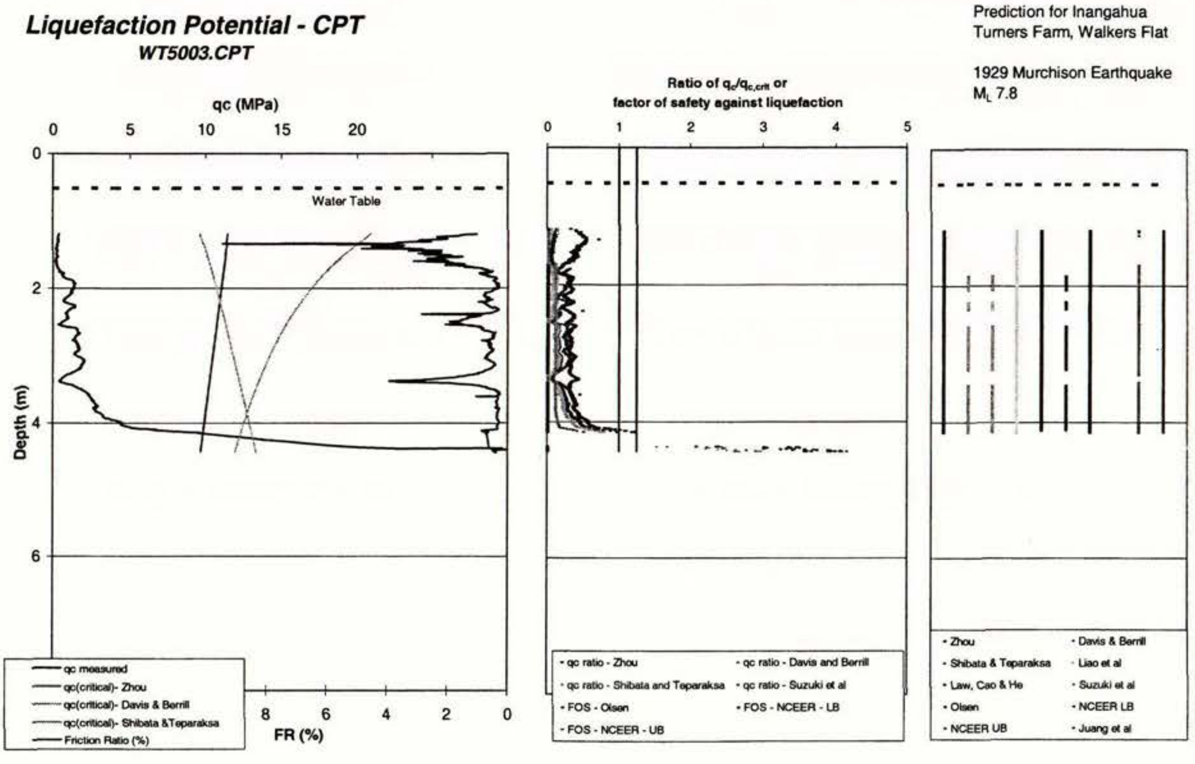

Liquefaction Potential - CPT
WT5002.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1929 Murchison Earthquake
M_L 7.8
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
0 2 4 6
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
Ratio of qc/qc,crit or
factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al


Liquefaction Potential - CPT
WT5002.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0
5
10
15
20
Water Table
Depth (m)
FR (%)
8
6
4
2
0
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al


Liquefaction Potential - CPT
WT5002.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
0
5
10
15
20
Water Table
Depth (m)
FR (%)
8
6
4
2
0
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

TUR003.CPT

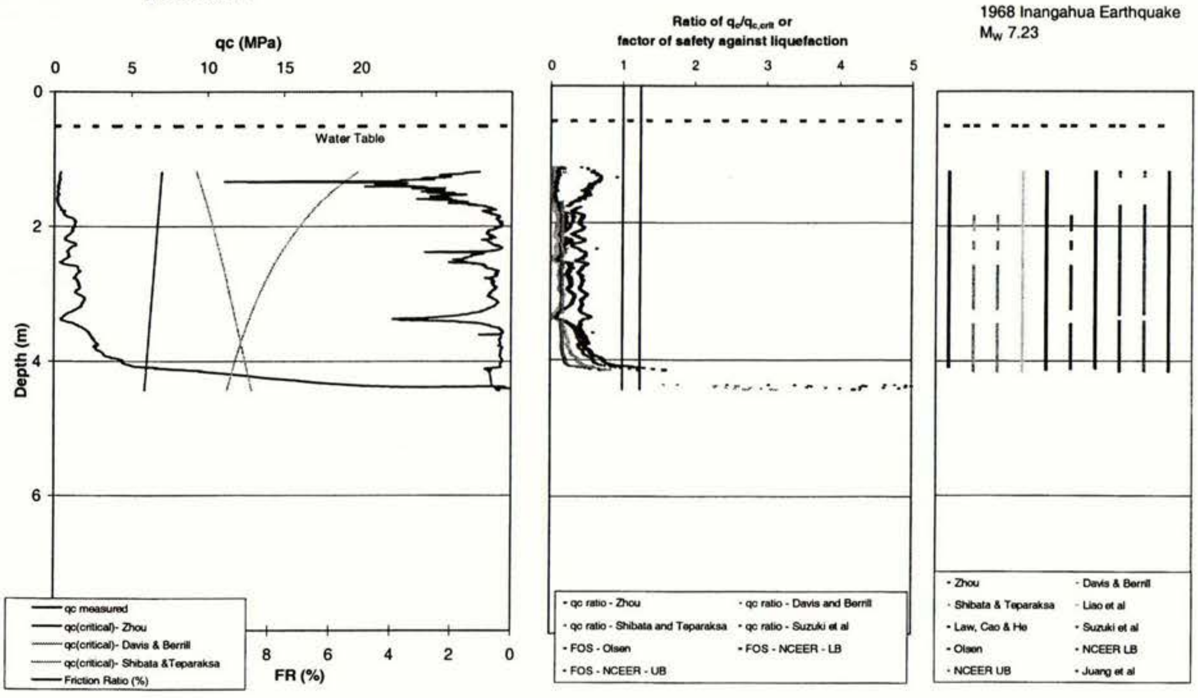

Inferred Soil Profiles - based on experimental data
WT5003.CPT
qc (MPa)
0
5
10
15
20
Water Table
Depth (m)
FR (%)
8
6
4
2
0
qc measured
Friction Ratio (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)


Liquefaction Potential - CPT
WT5003.CPT
qc (MPa)
Water Table
Depth (m)
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Prediction for Inangahua
Turners Farm, Walkers Flat
1929 Murchison Earthquake
M_L 7.8
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

Liquefaction Potential - CPT
WT5003.CPT

Prediction for Inangahua
Turners Farm, Walkers Flat

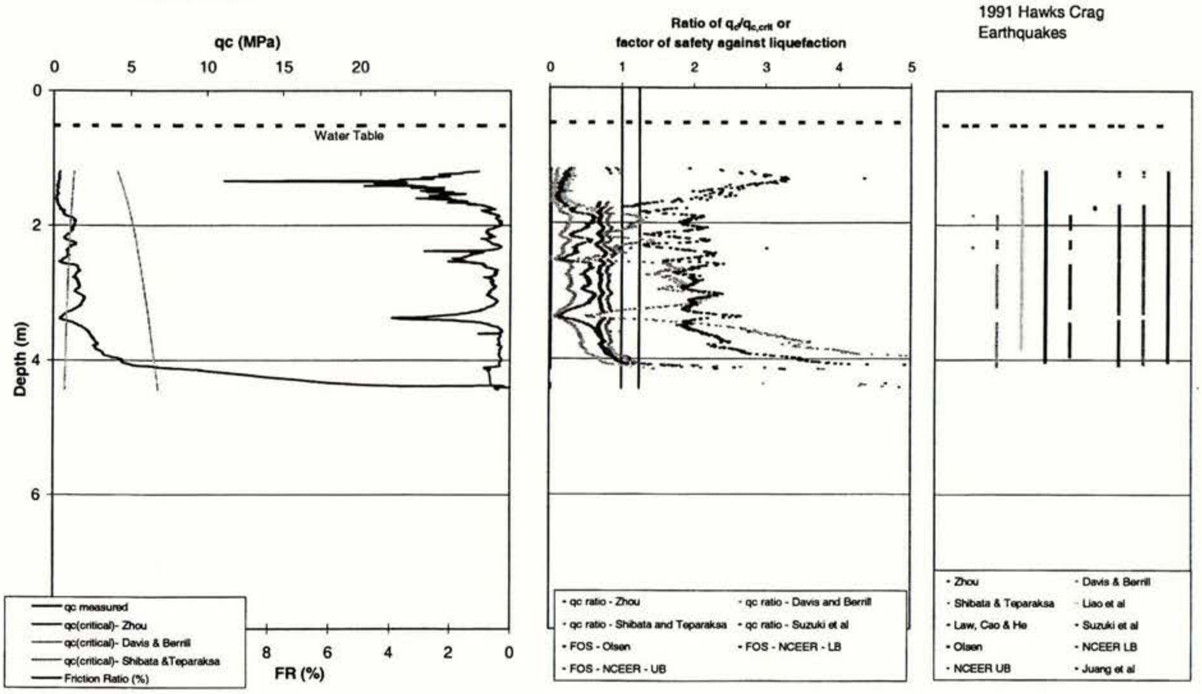
1968 Inangahua Earthquake
M_w 7.23



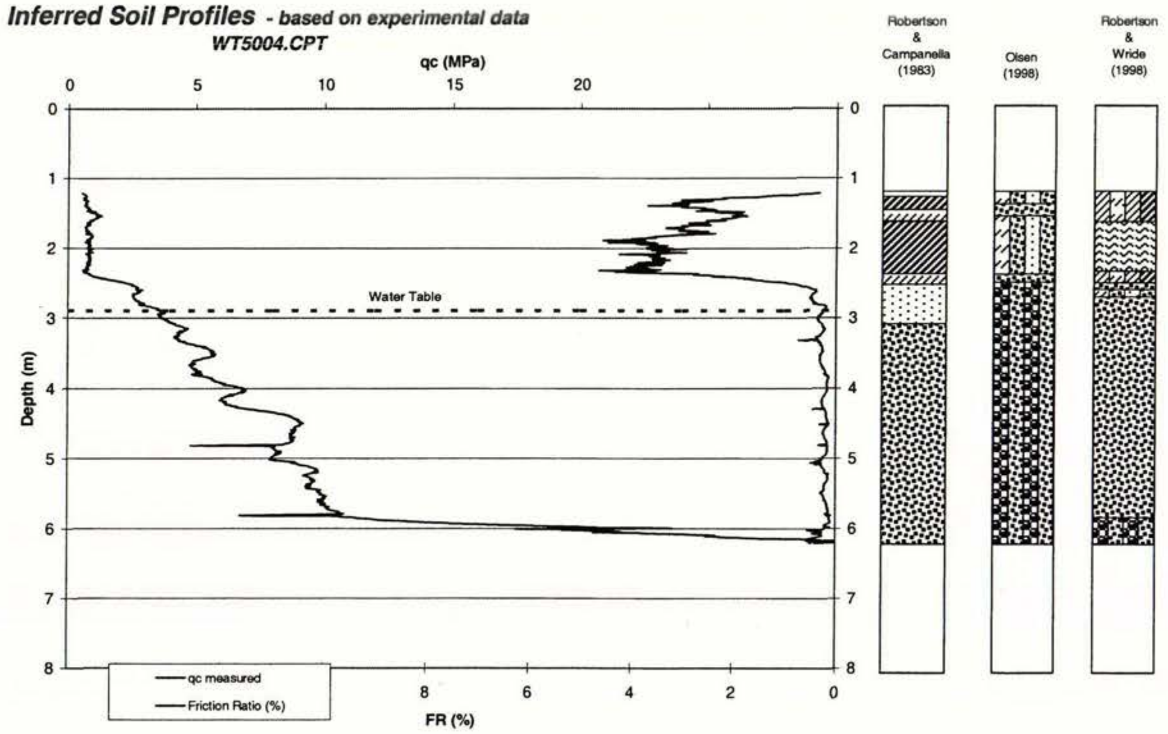
Liquefaction Potential - CPT
WT5003.CPT

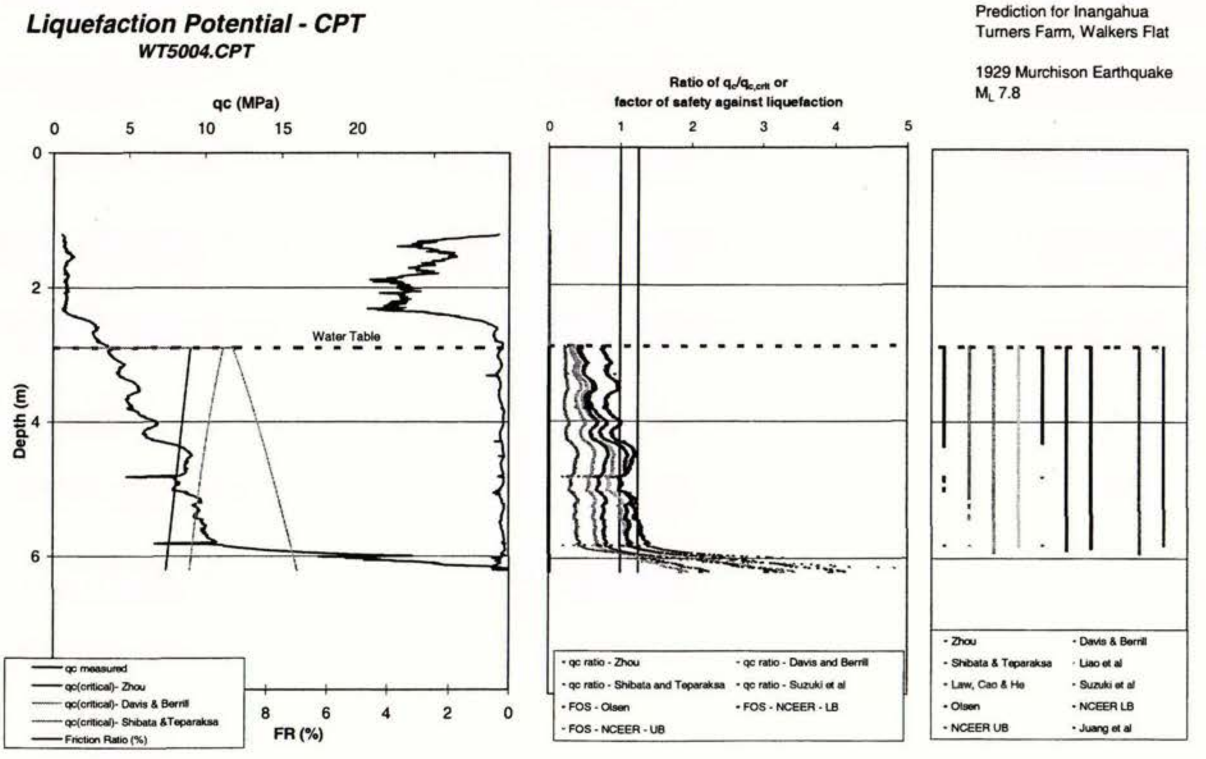
Prediction for Inangahua
Turners Farm, Walkers Flat

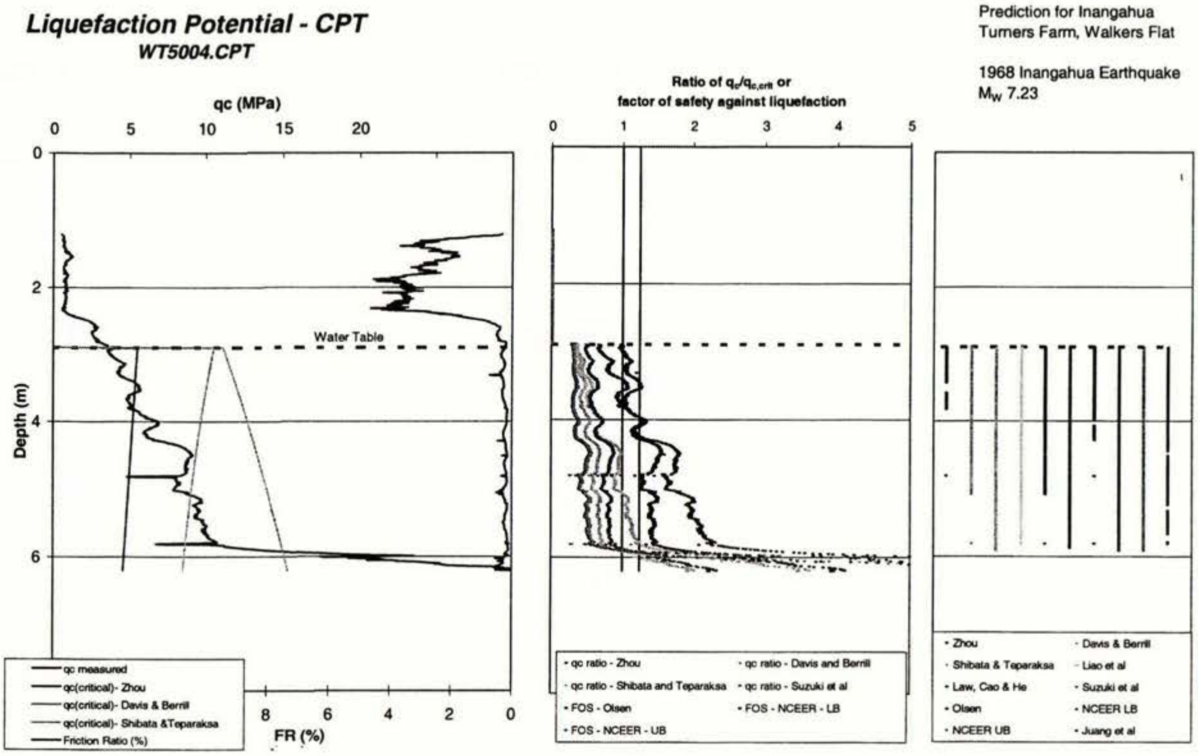
1991 Hawks Crag
Earthquakes

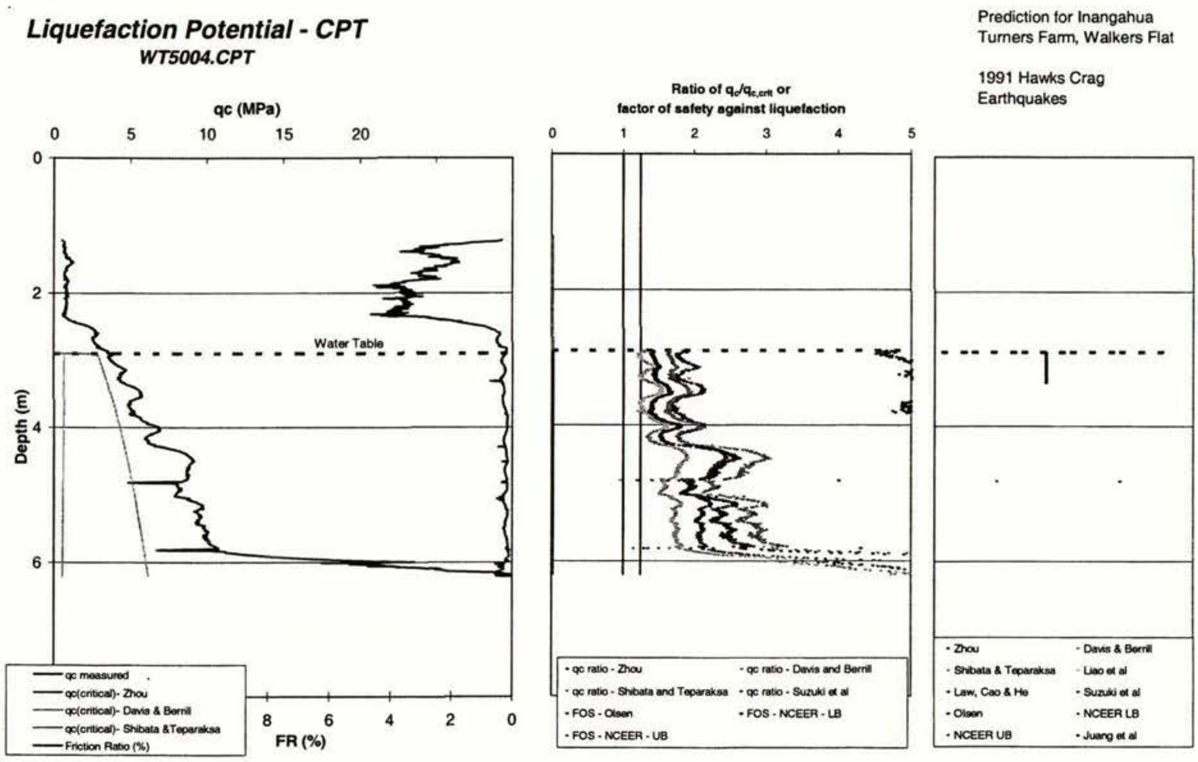


TUR004.CPT


Inferred Soil Profiles - based on experimental data
WT5004.CPT
qc (MPa)
0 5 10 15 20
Depth (m)
Water Table
qc measured
Friction Ratio (%)
8 6 4 2 0
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)

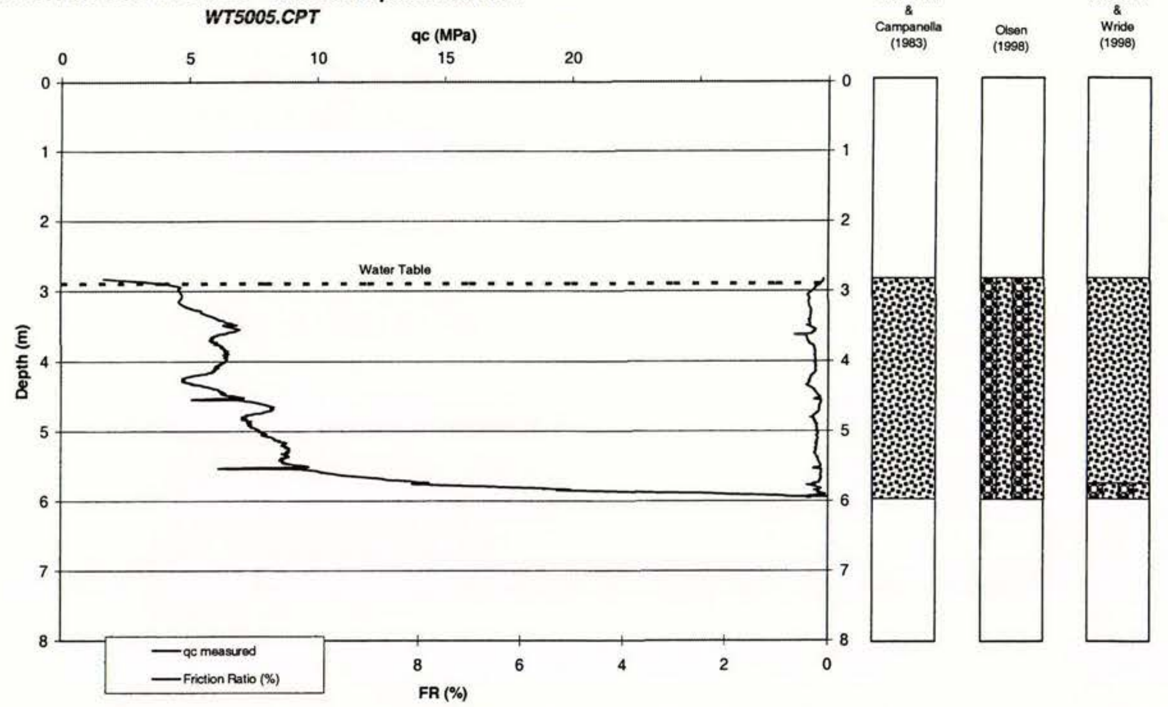

Liquefaction Potential - CPT
WT5004.CPT
qc (MPa)
0 5 10 15 20
Depth (m)
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
0 1 2 3 4 5
- qc ratio - Zhou
- qc ratio - Davis and Berrill
- qc ratio - Shibata and Teparaksa
- qc ratio - Suzuki et al
- FOS - Olsen
- FOS - NCEER - LB
- FOS - NCEER - UB
Prediction for Inangahua
Turners Farm, Walkers Flat
1929 Murchison Earthquake
M_L 7.8
- Zhou
- Davis & Berrill
- Shibata & Teparaksa
- Liao et al
- Law, Cao & He
- Suzuki et al
- Olsen
- NCEER LB
- NCEER UB
- Juang et al


Liquefaction Potential - CPT
WT5004.CPT
Prediction for Inangahua Turners Farm, Walkers Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
Water Table
Depth (m)
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al


Liquefaction Potential - CPT
WT5004.CPT
Prediction for Inangahua Turners Farm, Walkers Flat
1991 Hawks Crag Earthquakes
qc (MPa)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
Water Table
Depth (m)
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

TUR005.CPT

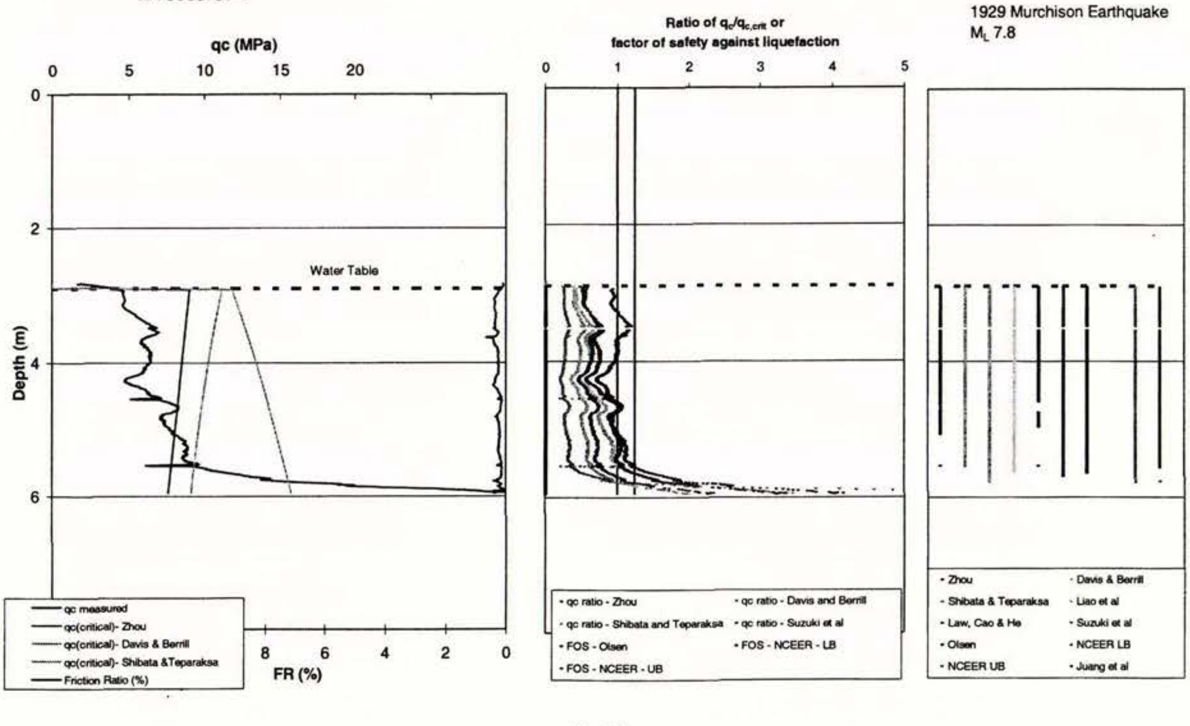
Inferred Soil Profiles - *based on experimental data*
WT5005.CPT

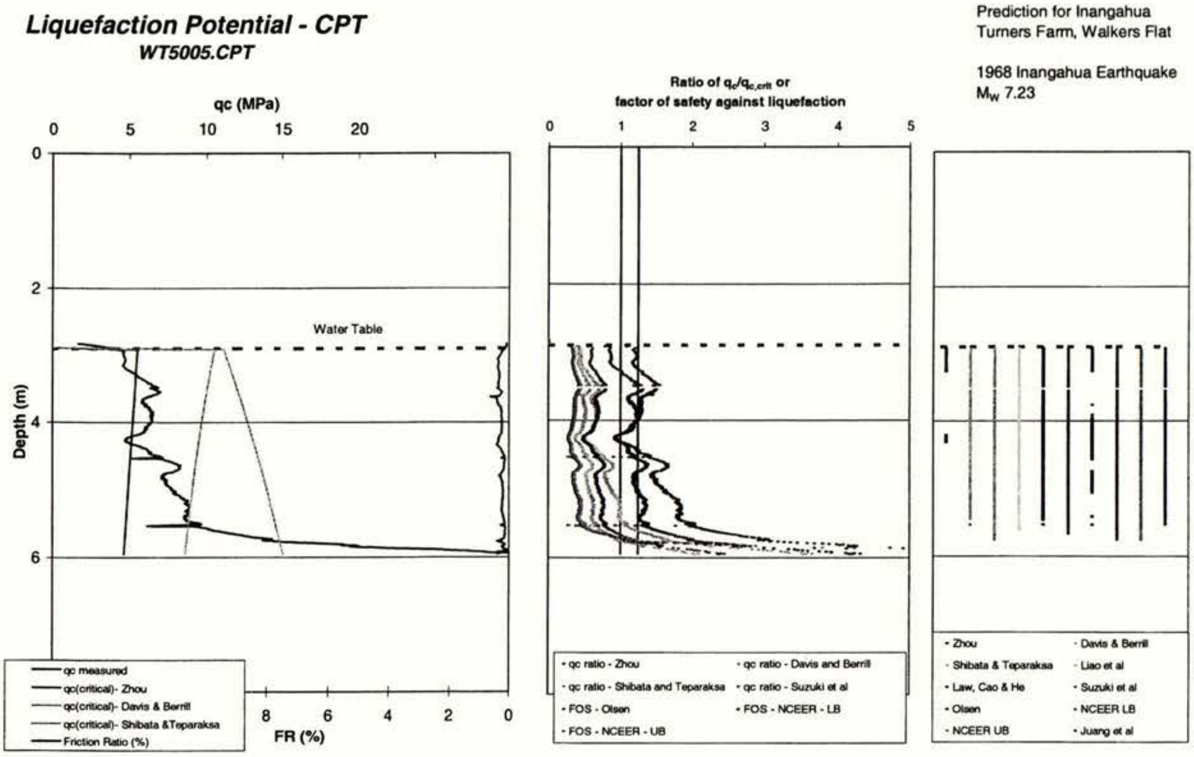


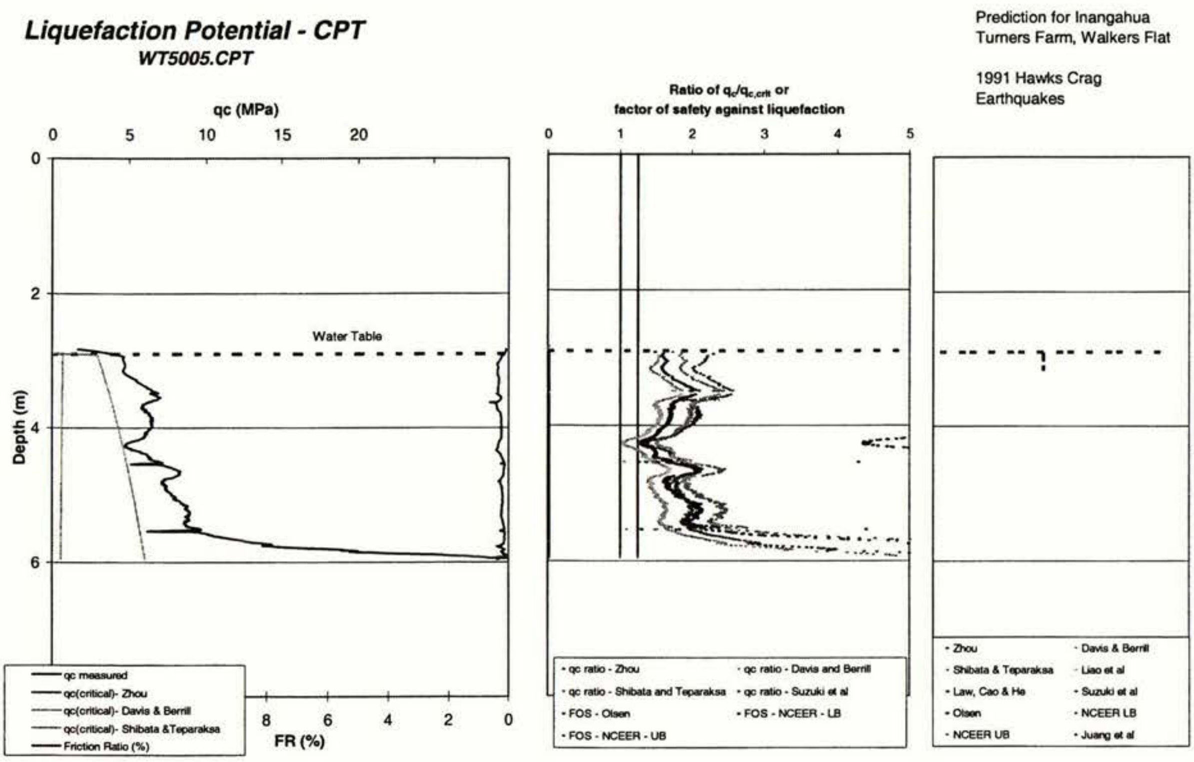
Liquefaction Potential - CPT
WT5005.CPT

Prediction for Inangahua
Turners Farm, Walkers Flat

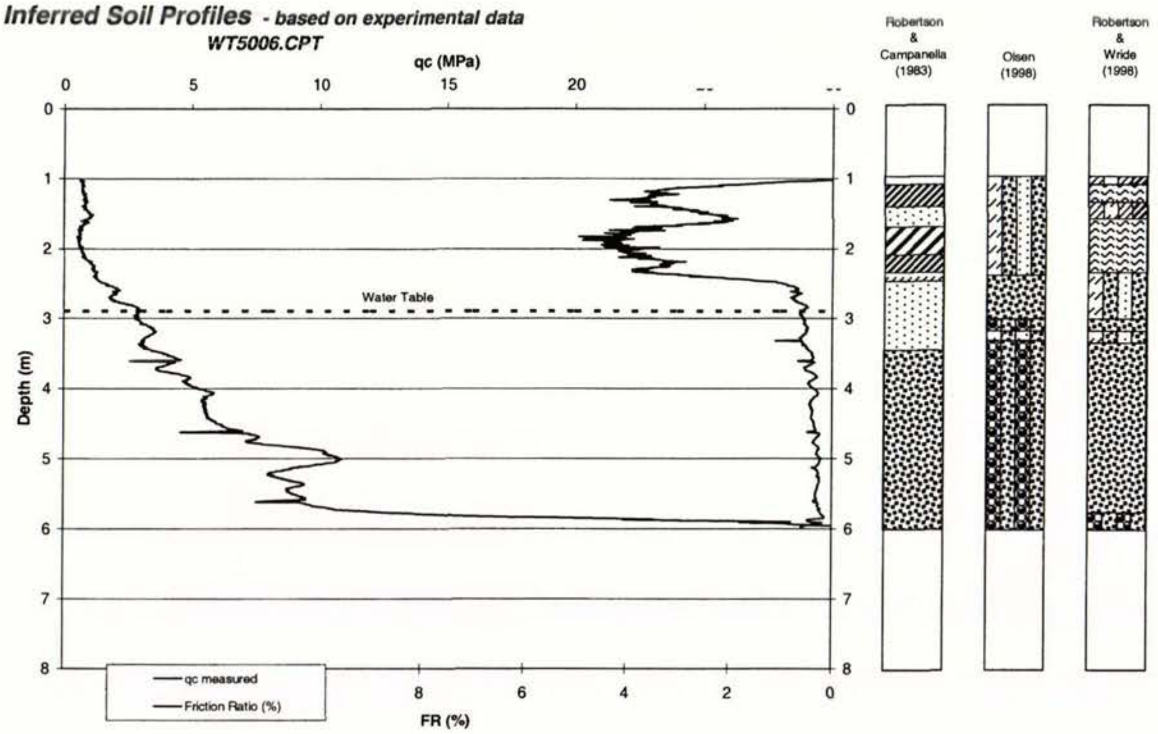
1929 Murchison Earthquake
M_L 7.8

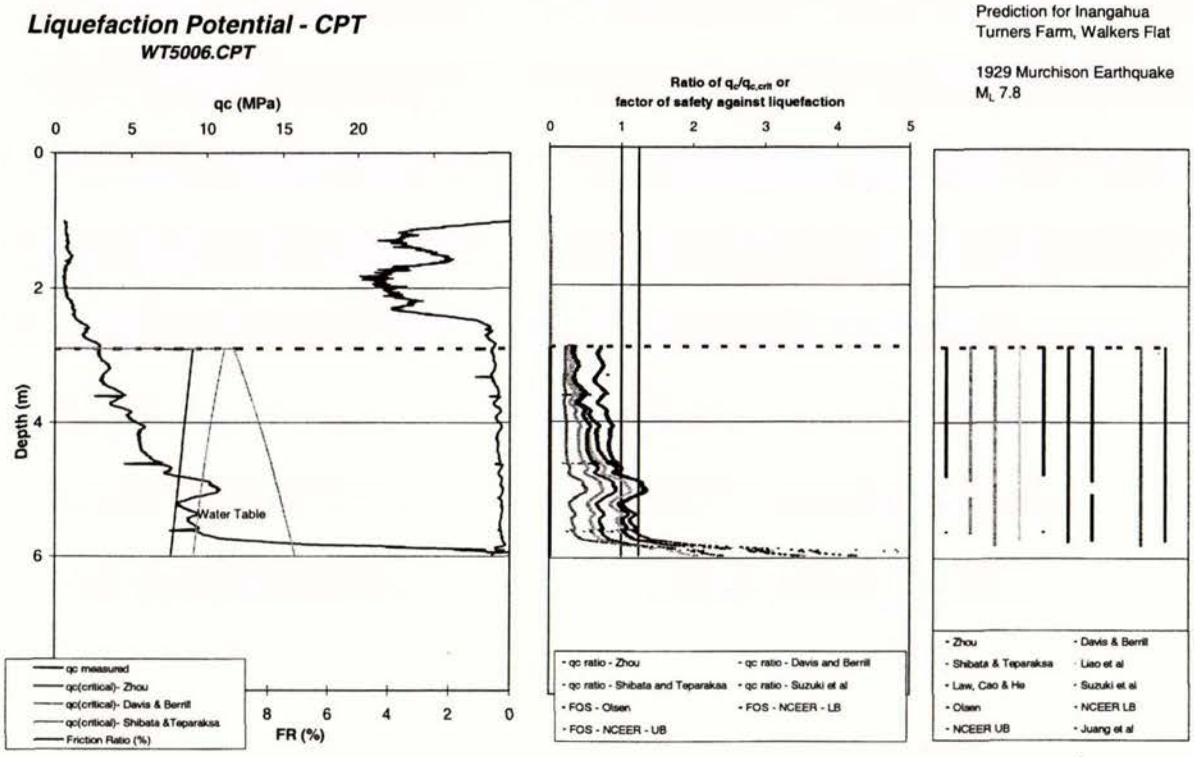


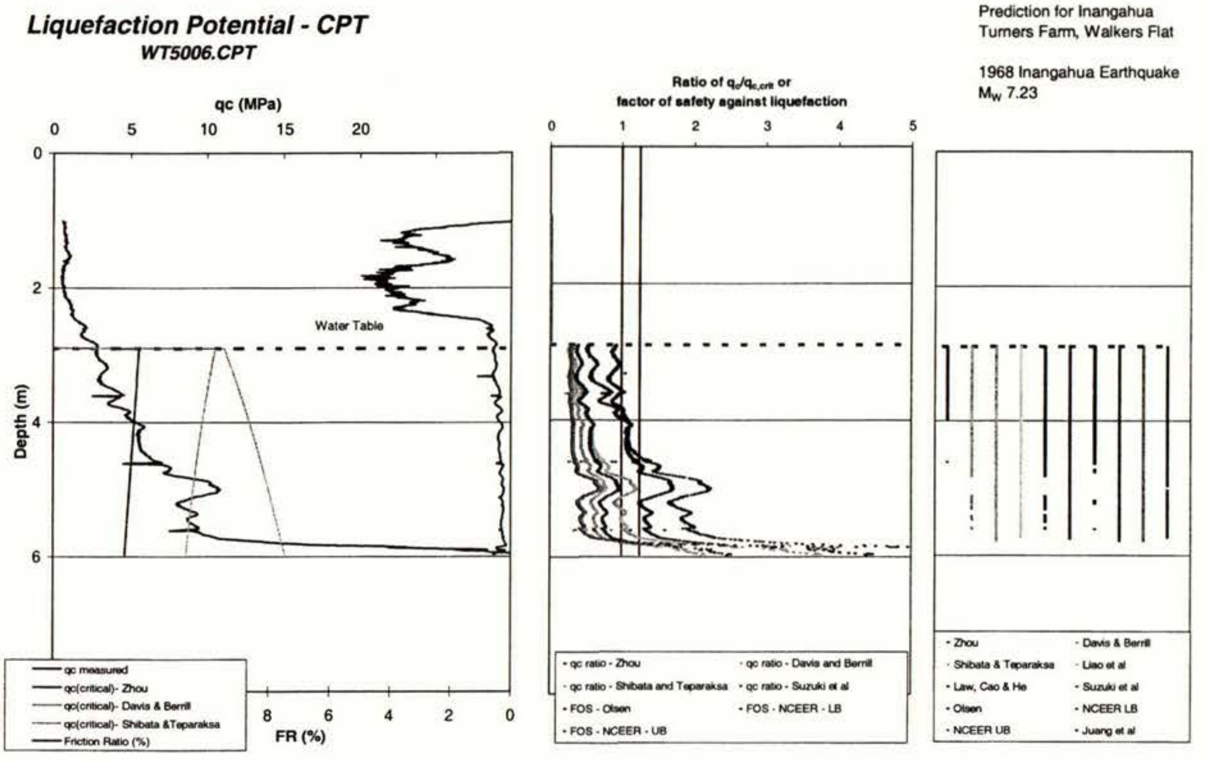

Liquefaction Potential - CPT
WT5005.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
Water Table
Depth (m)
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

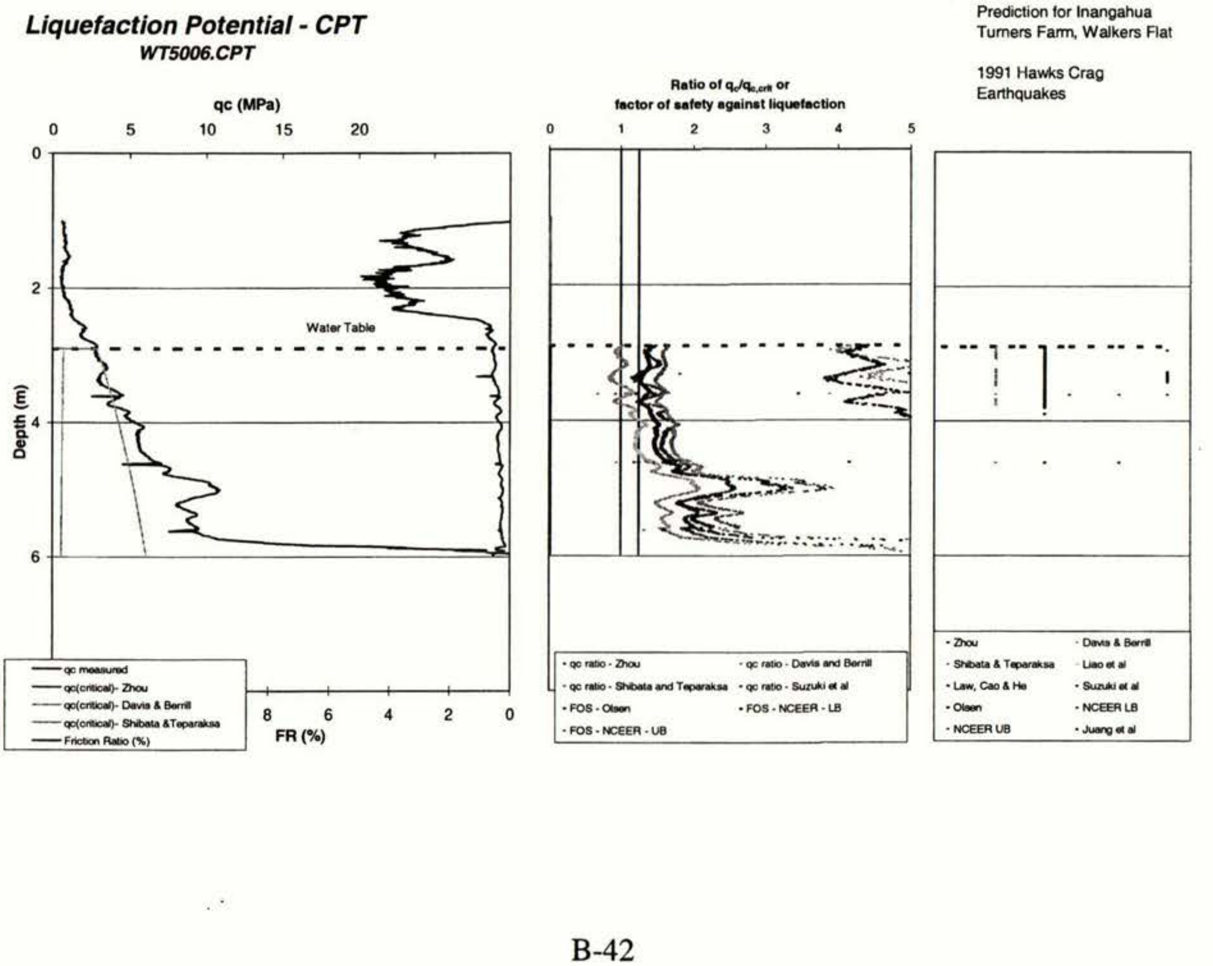

Liquefaction Potential - CPT
WT5005.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
Water Table
Depth (m)
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

TUR006.CPT

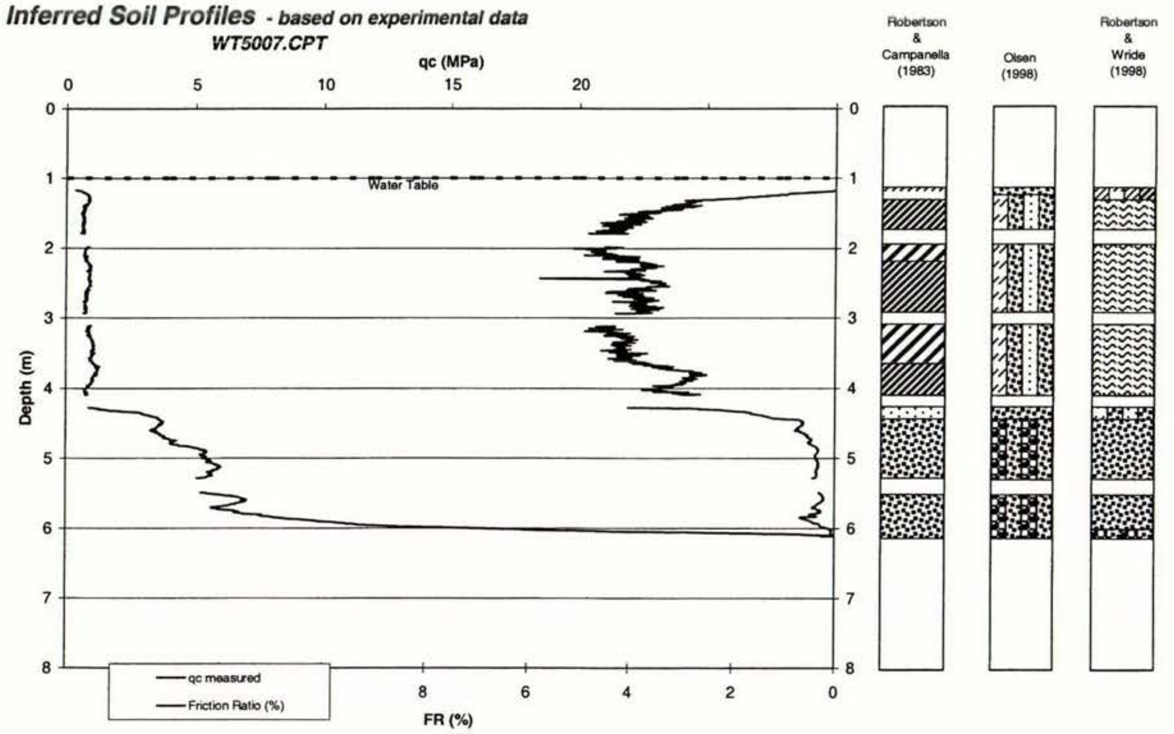

Inferred Soil Profiles - based on experimental data
WT5006.CPT
qc (MPa)
Water Table
Depth (m)
qc measured
Friction Ratio (%)
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)

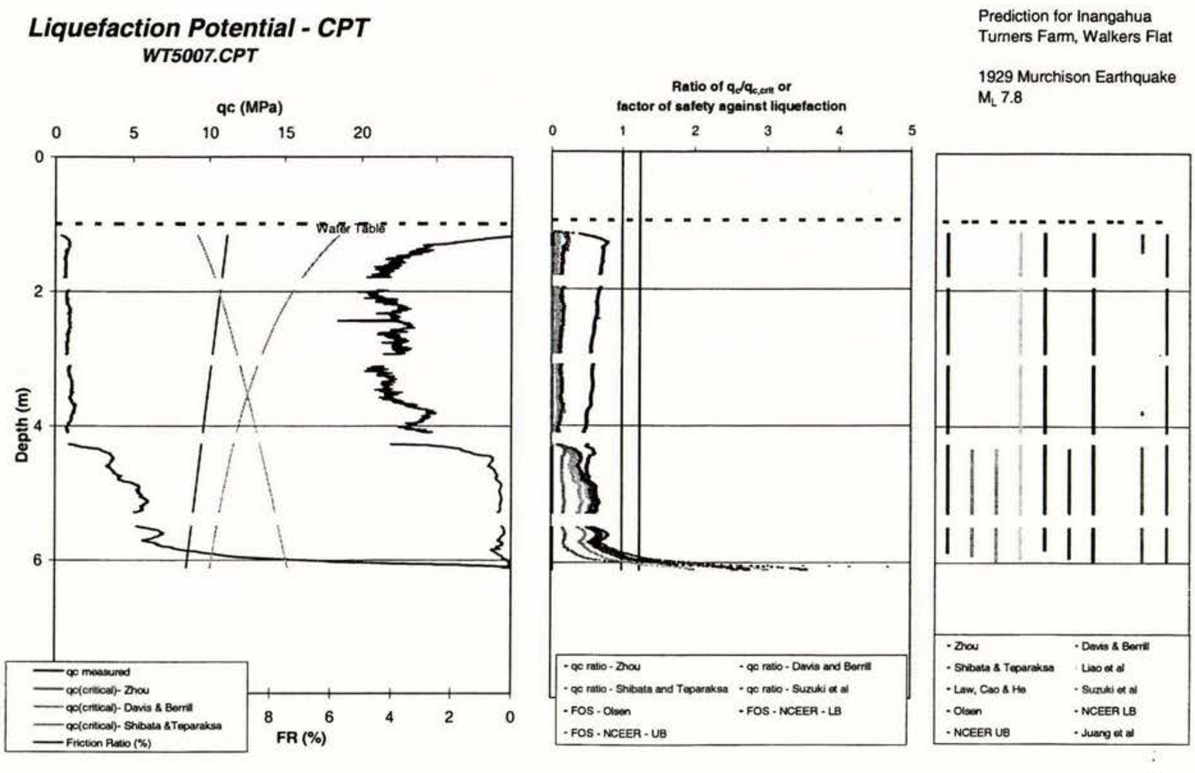

Liquefaction Potential - CPT
WT5006.CPT
qc (MPa)
Depth (m)
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
factor of safety against liquefaction
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Prediction for Inangahua
Turners Farm, Walkers Flat
1929 Murchison Earthquake
M_L 7.8
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

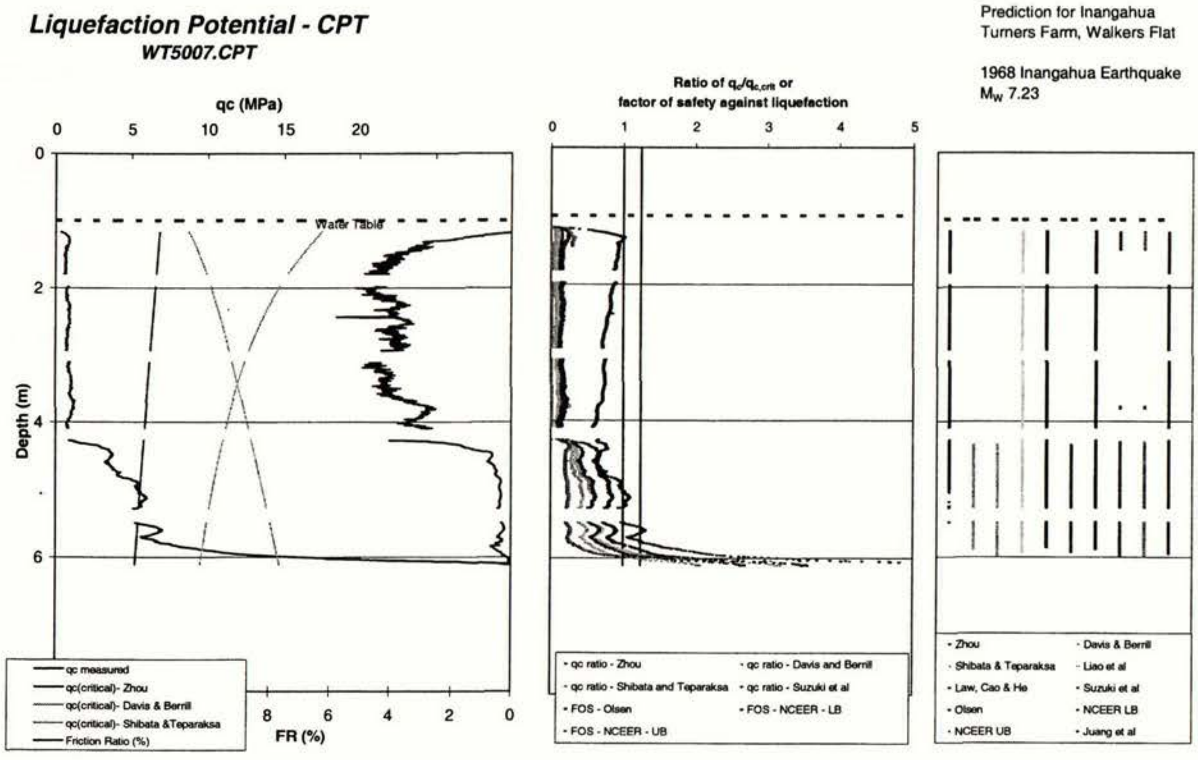

Liquefaction Potential - CPT
WT5006.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0 5 10 15 20
Depth (m)
0 2 4 6
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

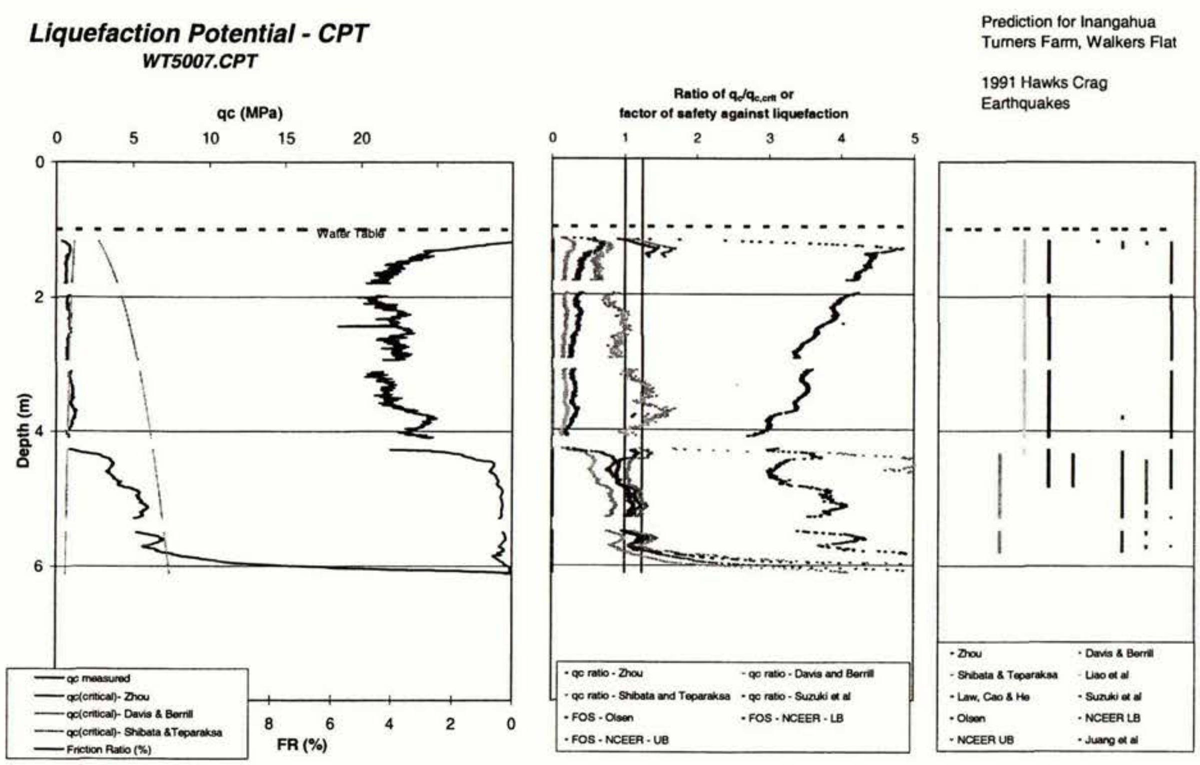

Liquefaction Potential - CPT
WT5006.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
0 5 10 15 20
Depth (m)
0 2 4 6
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

TUR007.CPT

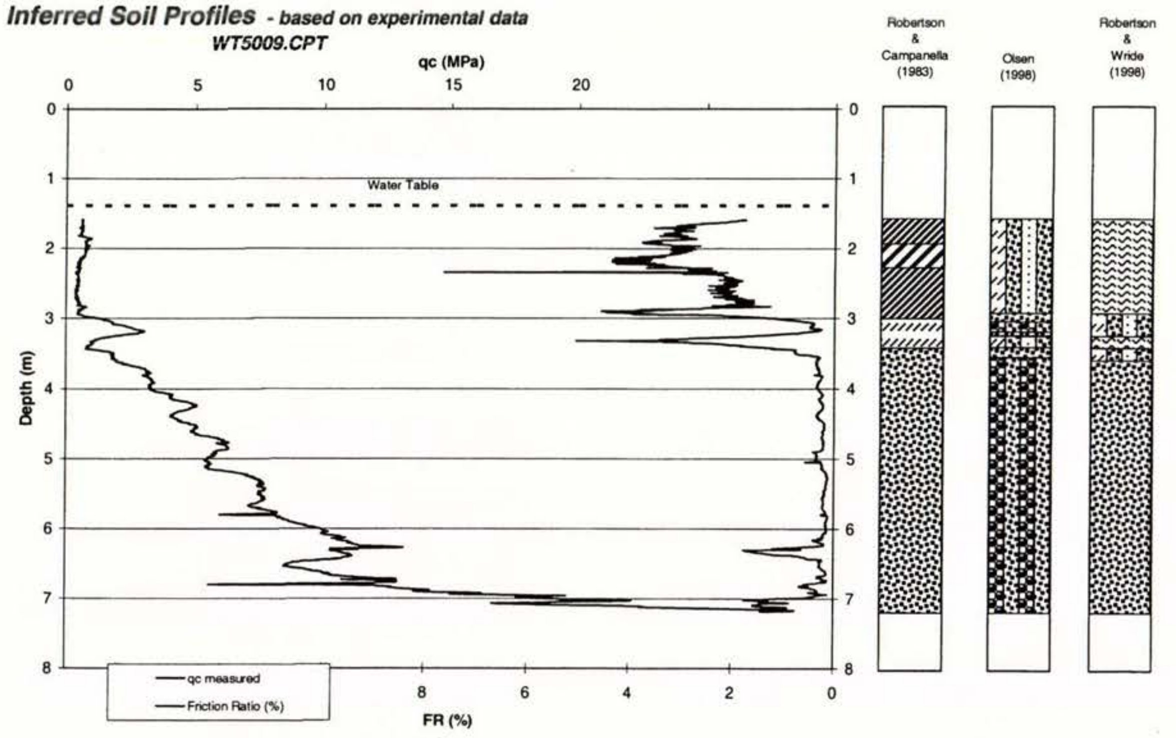

Inferred Soil Profiles - based on experimental data
WT5007.CPT
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
qc measured
Friction Ratio (%)
8 6 4 2 0
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)

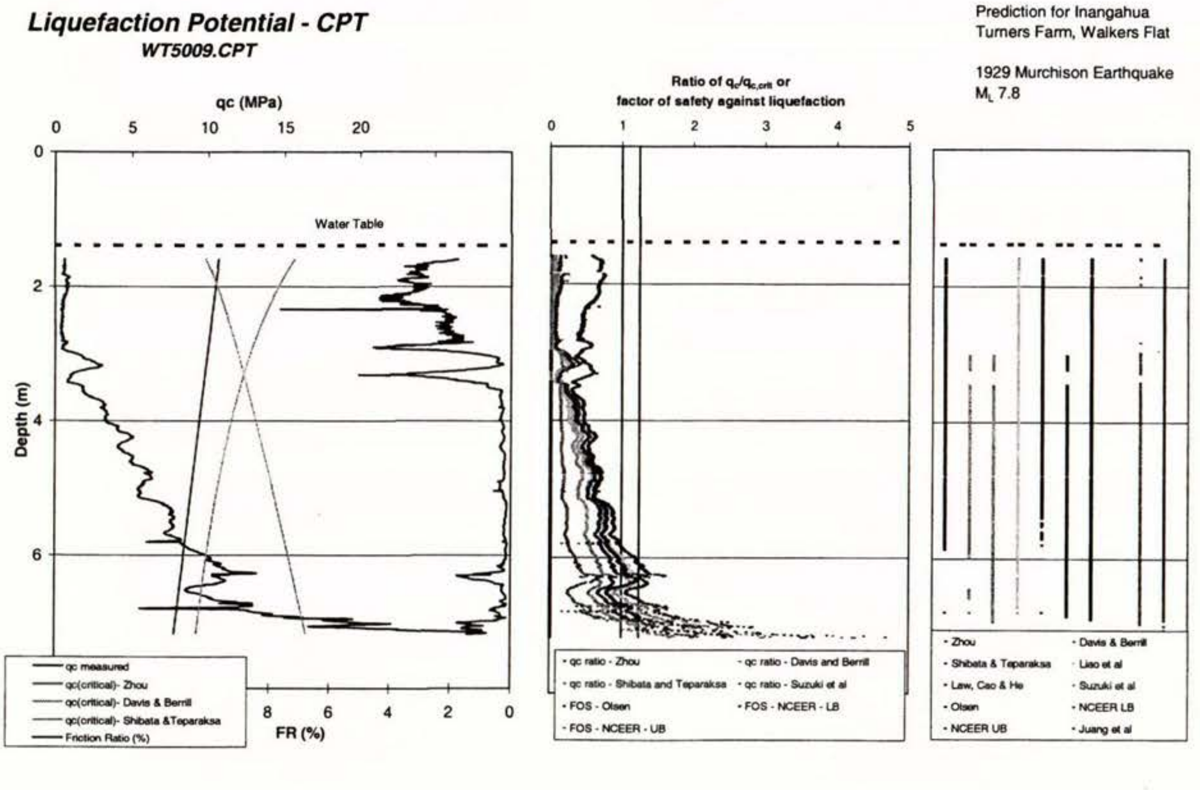

Liquefaction Potential - CPT
WT5007.CPT
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
0 1 2 3 4 5
- qc ratio - Zhou
- qc ratio - Davis and Berrill
- qc ratio - Shibata and Teparaksa
- qc ratio - Suzuki et al
- FOS - Olsen
- FOS - NCEER - LB
- FOS - NCEER - UB
Prediction for Inangahua Turners Farm, Walkers Flat
1929 Murchison Earthquake M_L 7.8
- Zhou
- Davis & Berrill
- Shibata & Teparaksa
Liao et al
- Law, Cao & He
- Suzuki et al
- Olsen
- NCEER LB
- NCEER UB
- Juang et al

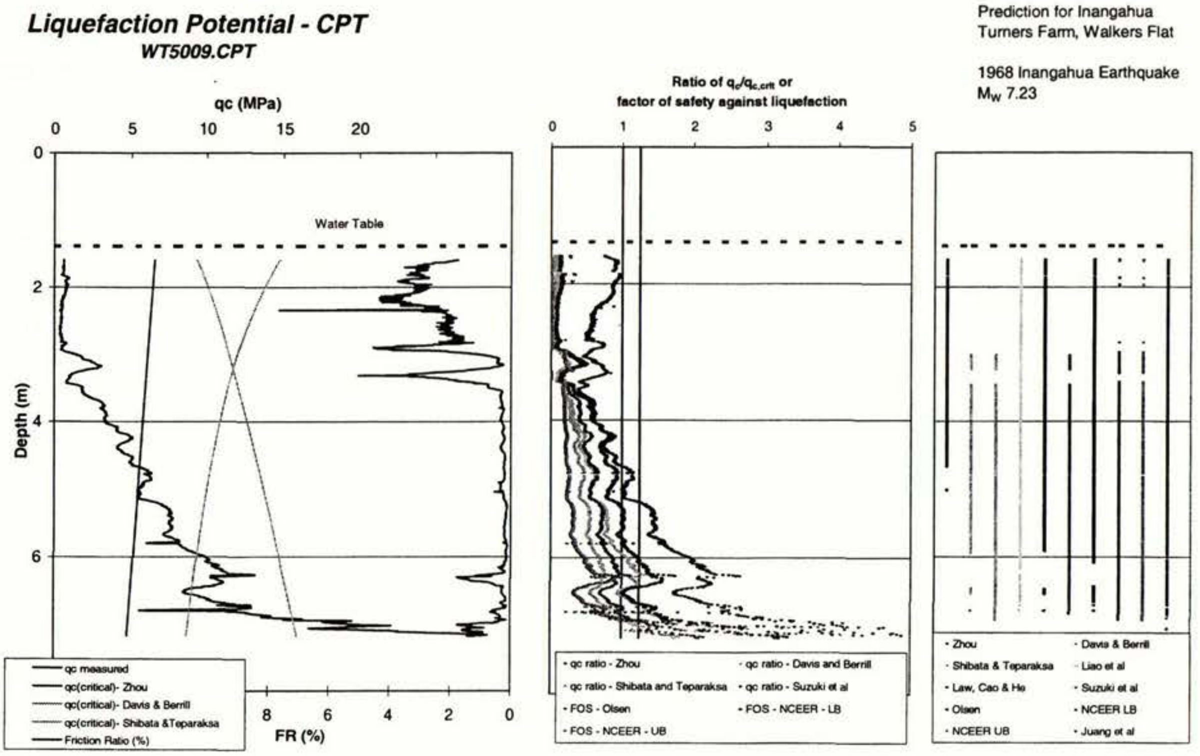

Liquefaction Potential - CPT
WT5007.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
Depth (m)
Water Table
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

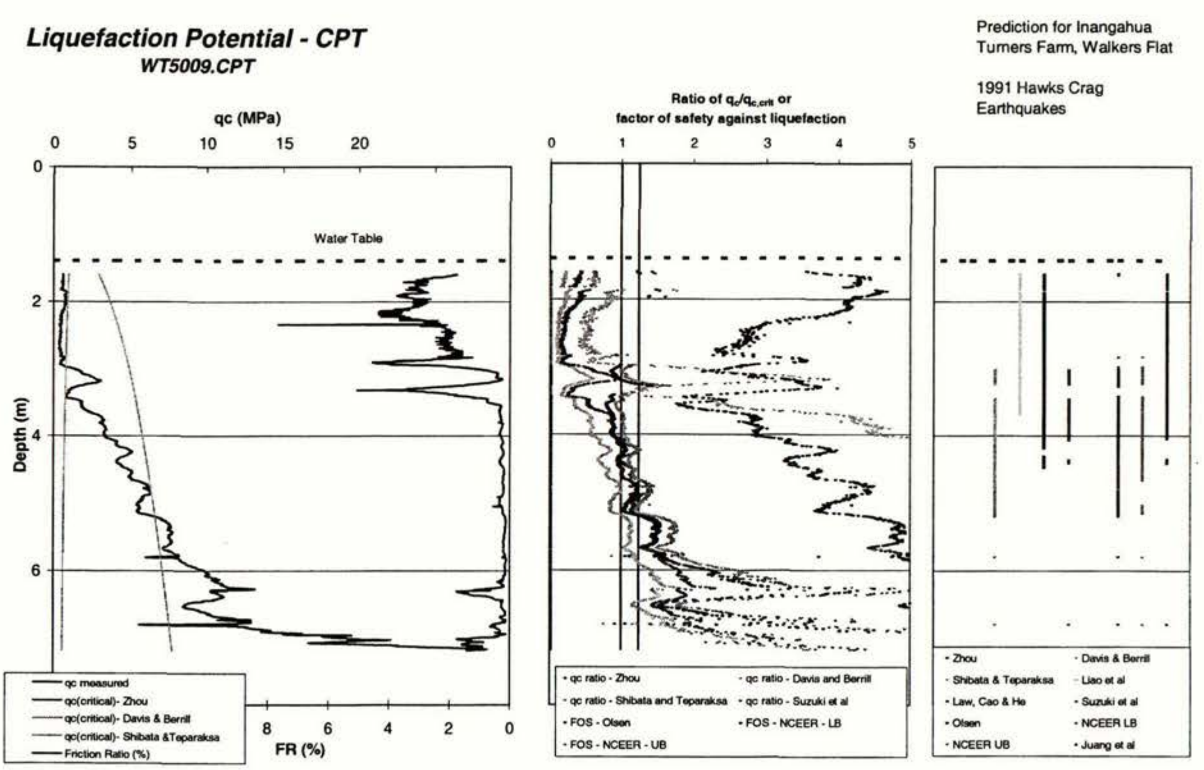

Liquefaction Potential - CPT
WT5007.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
Depth (m)
Water Table
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

TUR008.CPT

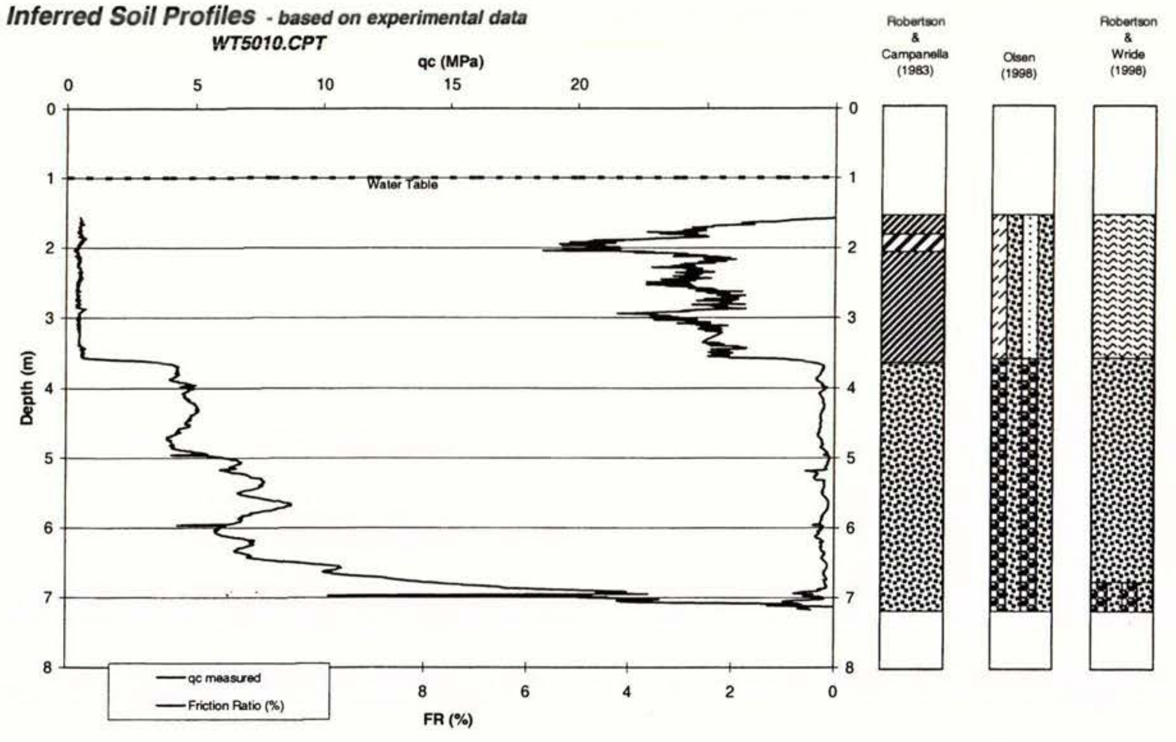

Inferred Soil Profiles - based on experimental data
WT5009.CPT
qc (MPa)
Depth (m)
Water Table
qc measured
Friction Ratio (%)
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)

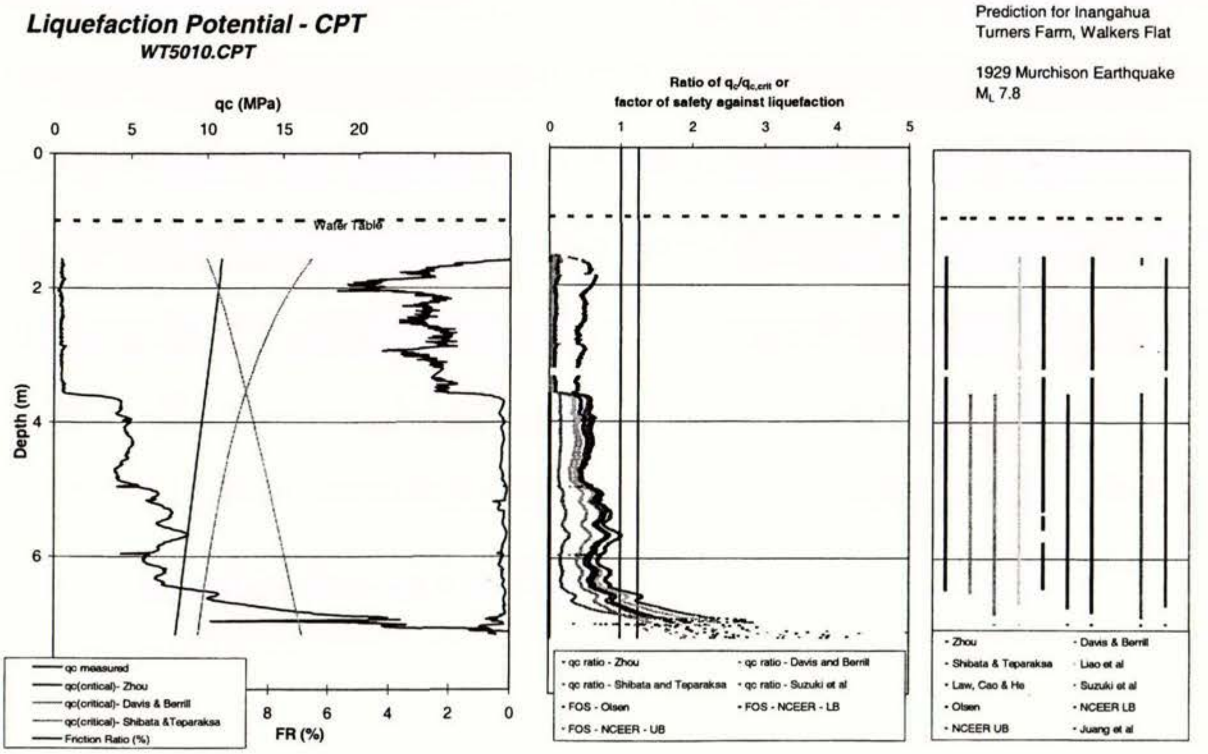

Liquefaction Potential - CPT
WT5009.CPT
qc (MPa)
Depth (m)
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
- qc ratio - Zhou
- qc ratio - Davis and Berrill
- qc ratio - Shibata and Teparaksa
- qc ratio - Suzuki et al
- FOS - Olsen
- FOS - NCEER - LB
- FOS - NCEER - UB
Prediction for Inangahua
Turners Farm, Walkers Flat
1929 Murchison Earthquake
M_L 7.8
- Zhou
- Davis & Berrill
- Shibata & Teparaksa
- Liao et al
- Law, Cao & He
- Suzuki et al
- Olsen
- NCEER LB
- NCEER UB
- Juang et al

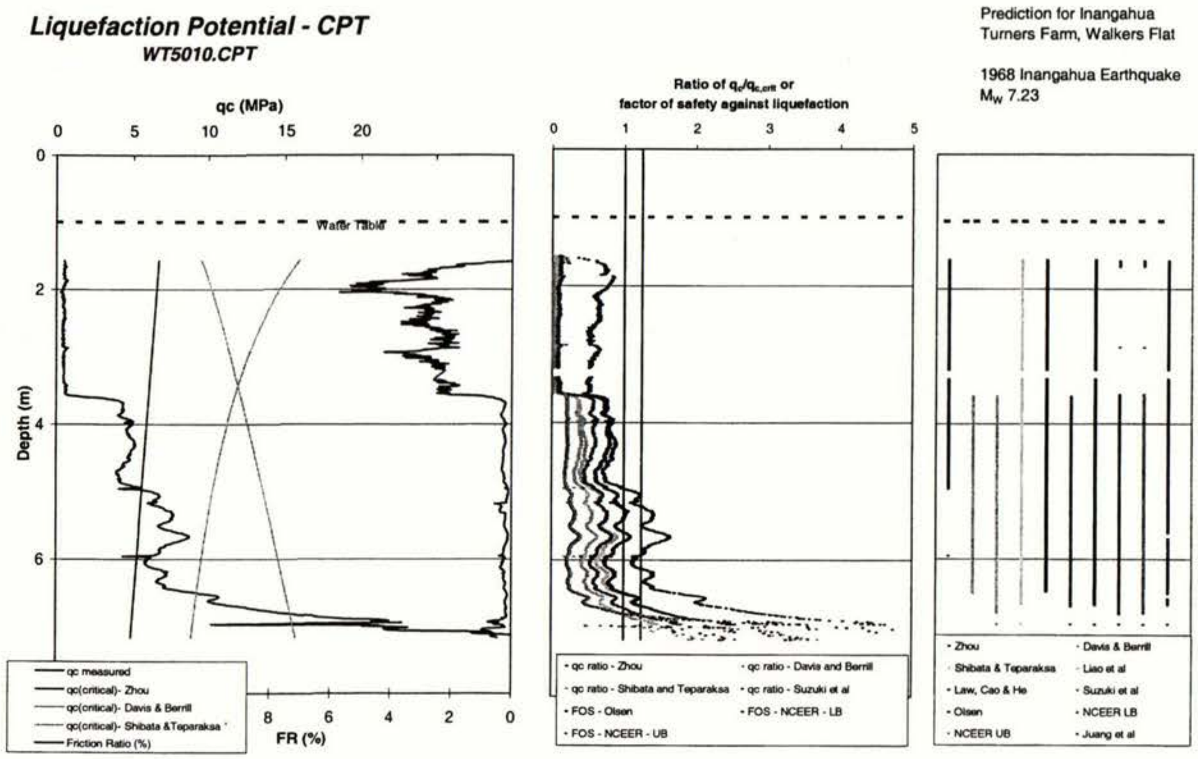

Liquefaction Potential - CPT
WT5009.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0
5
10
15
20
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0
1
2
3
4
5
Water Table
Depth (m)
0
2
4
6
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8
6
4
2
0
FR (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

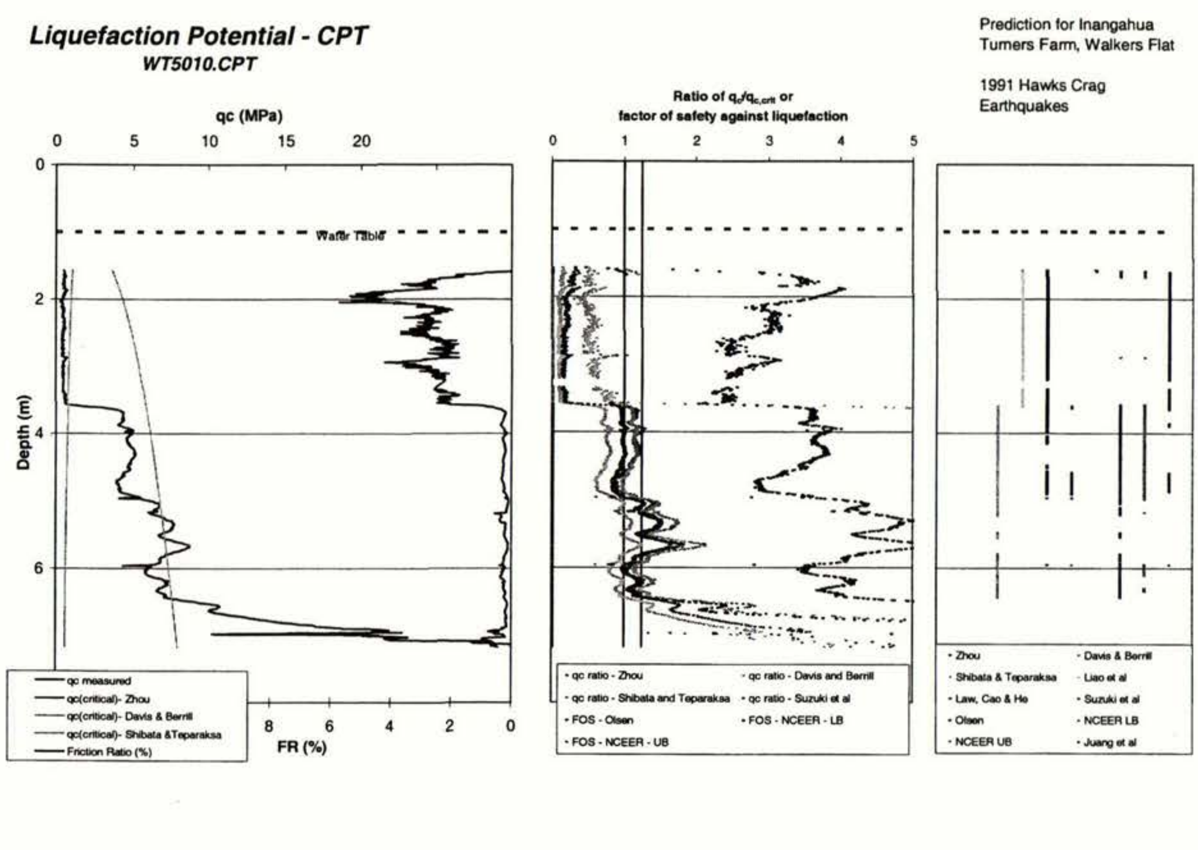

Liquefaction Potential - CPT
WT5009.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
0
5
10
15
20
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0
1
2
3
4
5
Water Table
Depth (m)
0
2
4
6
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8
6
4
2
0
FR (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

TUR009.CPT

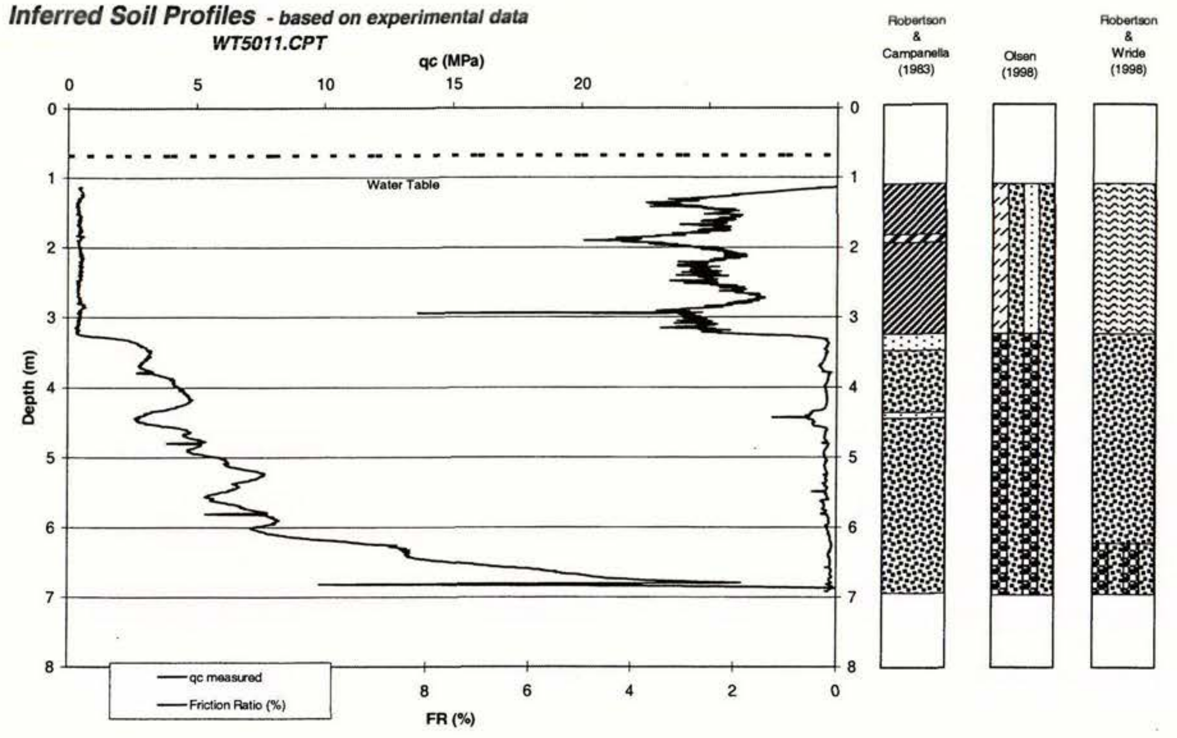

Inferred Soil Profiles - based on experimental data
WT5010.CPT
qc (MPa)
0
5
10
15
20
Water Table
Depth (m)
qc measured
Friction Ratio (%)
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)


Liquefaction Potential - CPT
WT5010.CPT
qc (MPa)
Water Table
Depth (m)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
- qc ratio - Zhou
- qc ratio - Davis and Berrill
- qc ratio - Shibata and Teparaksa
- qc ratio - Suzuki et al
- FOS - Olsen
- FOS - NCEER - LB
- FOS - NCEER - UB
Prediction for Inangahua
Turners Farm, Walkers Flat
1929 Murchison Earthquake
M_L 7.8
- Zhou
- Davis & Berrill
- Shibata & Teparaksa
- Liao et al
- Law, Cao & He
- Suzuki et al
- Olsen
- NCEER LB
- NCEER UB
- Juang et al

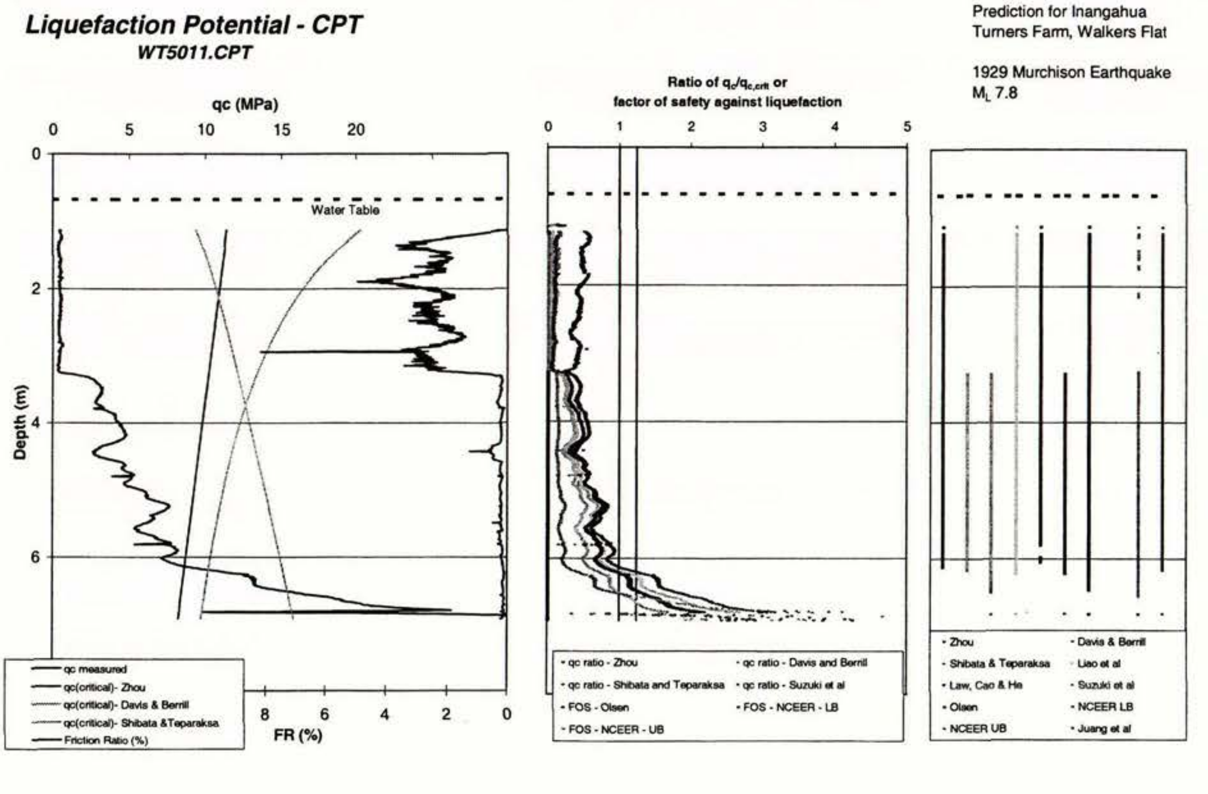

Liquefaction Potential - CPT
WT5010.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0 5 10 15 20
Depth (m)
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
8 6 4 2 0
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al


Liquefaction Potential - CPT
WT5010.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
0 5 10 15 20
Depth (m)
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
8 6 4 2 0
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

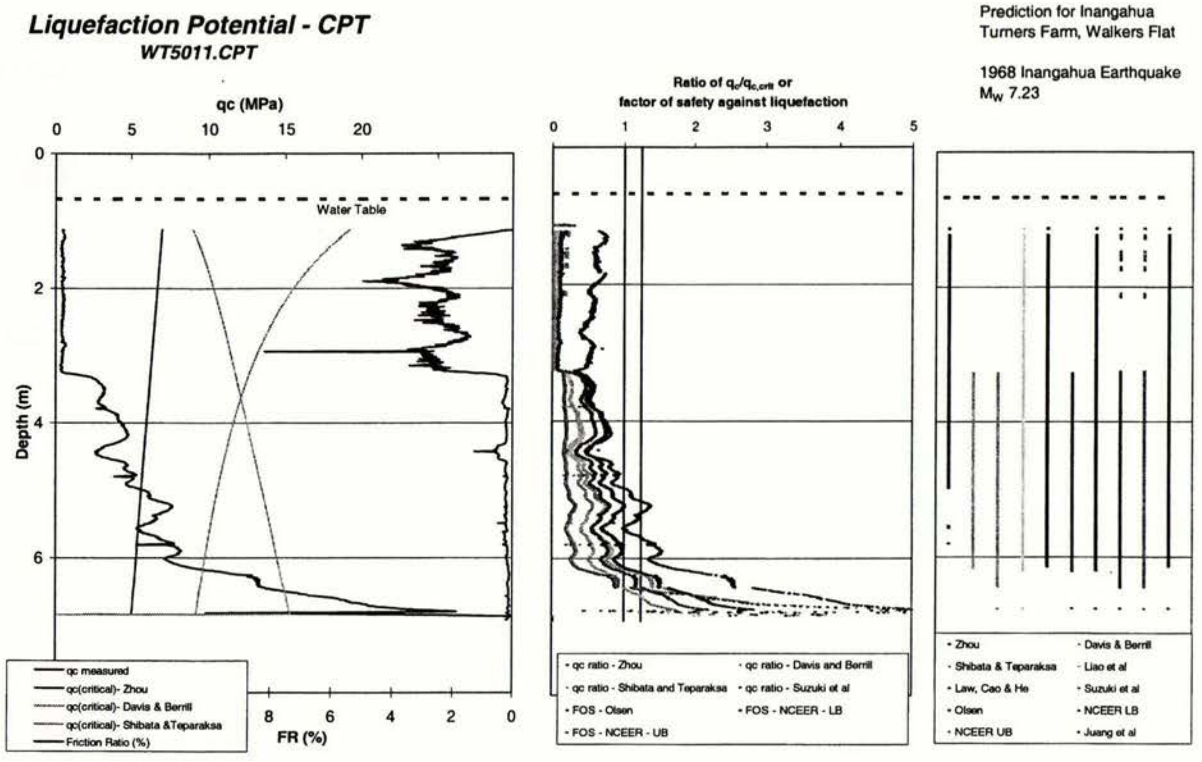
TUR010.CPT



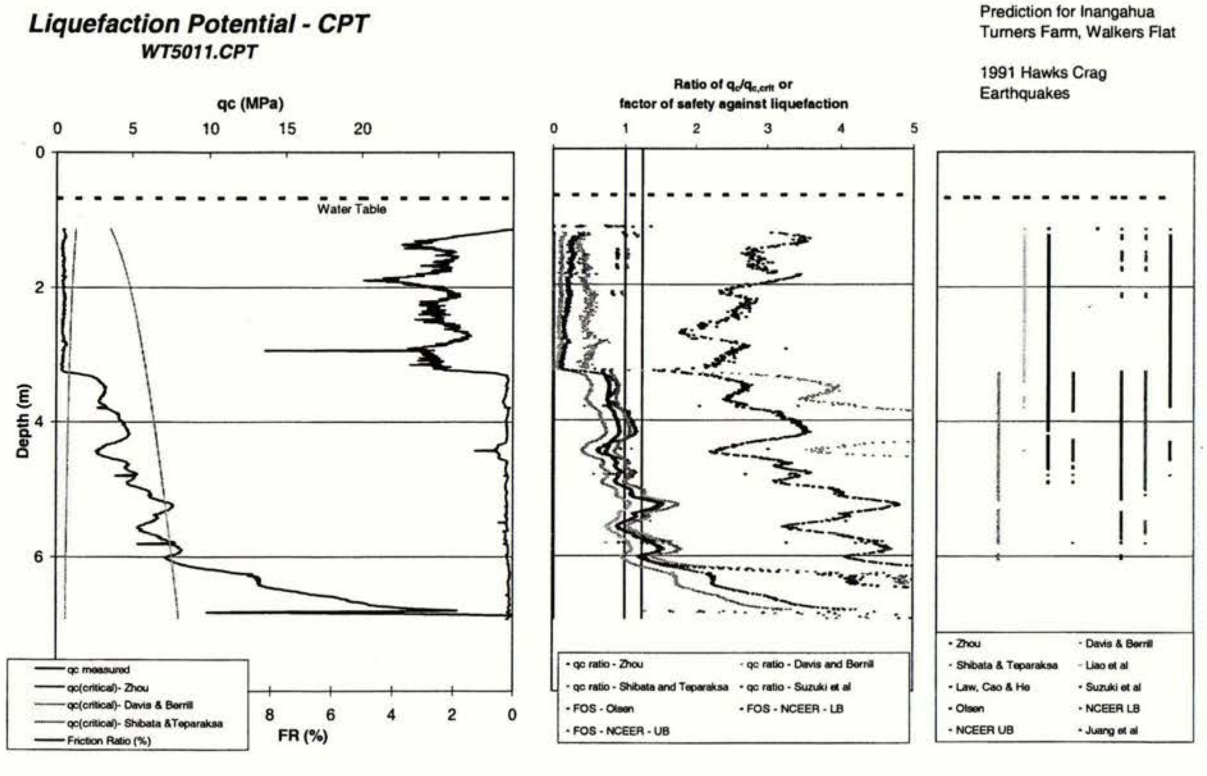
Inferred Soil Profiles - based on experimental data
WT5011.CPT
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
qc measured
Friction Ratio (%)
FR (%)
8 6 4 2 0
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)



Liquefaction Potential - CPT
WT5011.CPT
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
8 6 4 2 0
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Prediction for Inangahua Turners Farm, Walkers Flat
1929 Murchison Earthquake
M_L 7.8
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

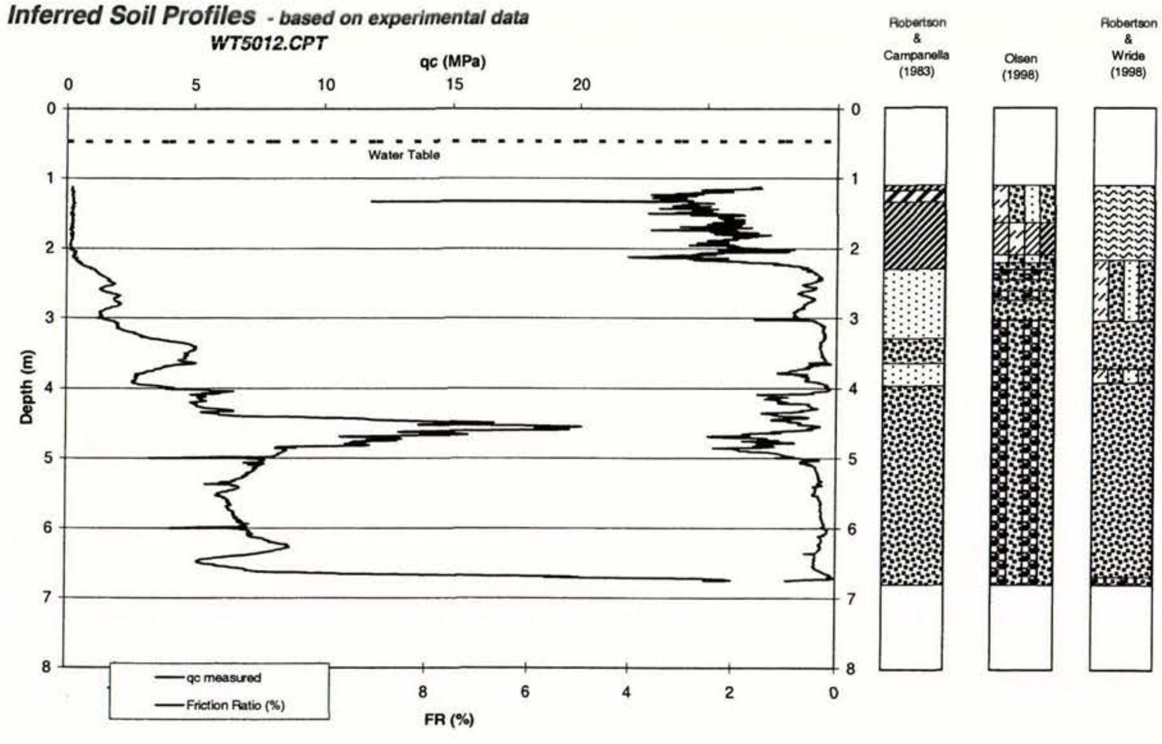


Liquefaction Potential - CPT
WT5011.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
Depth (m)
Water Table
FR (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

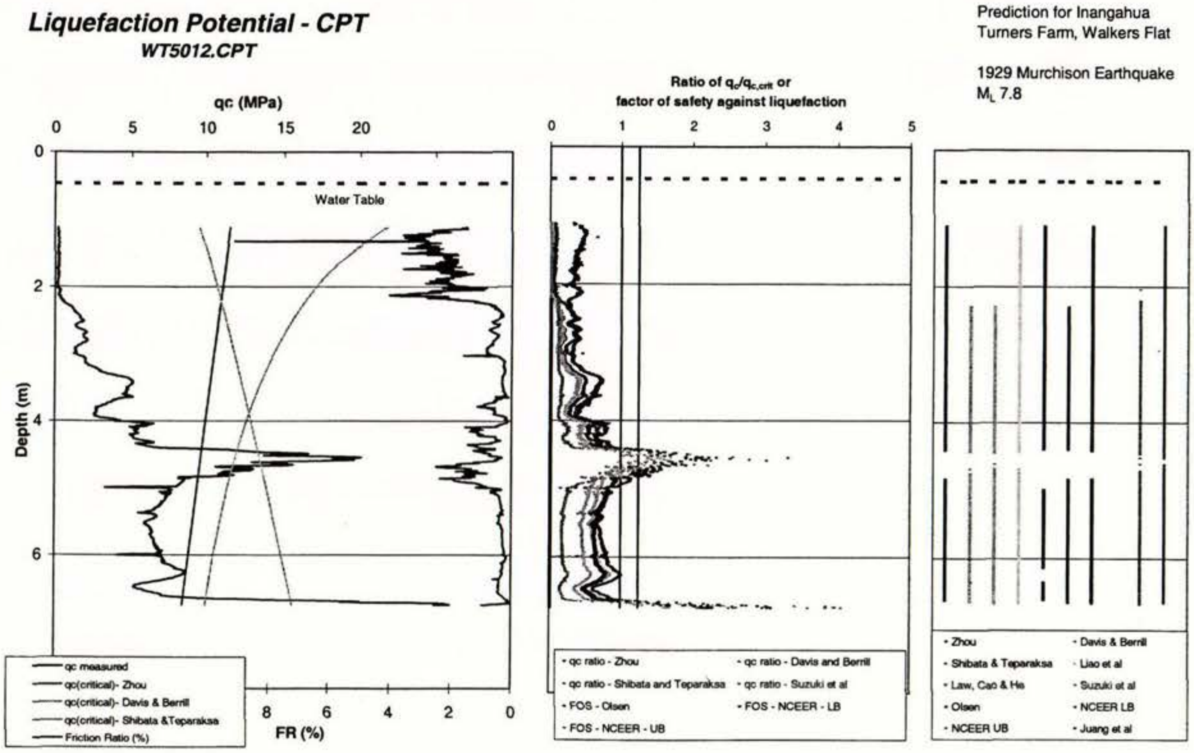


Liquefaction Potential - CPT
WT5011.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
Depth (m)
Water Table
FR (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

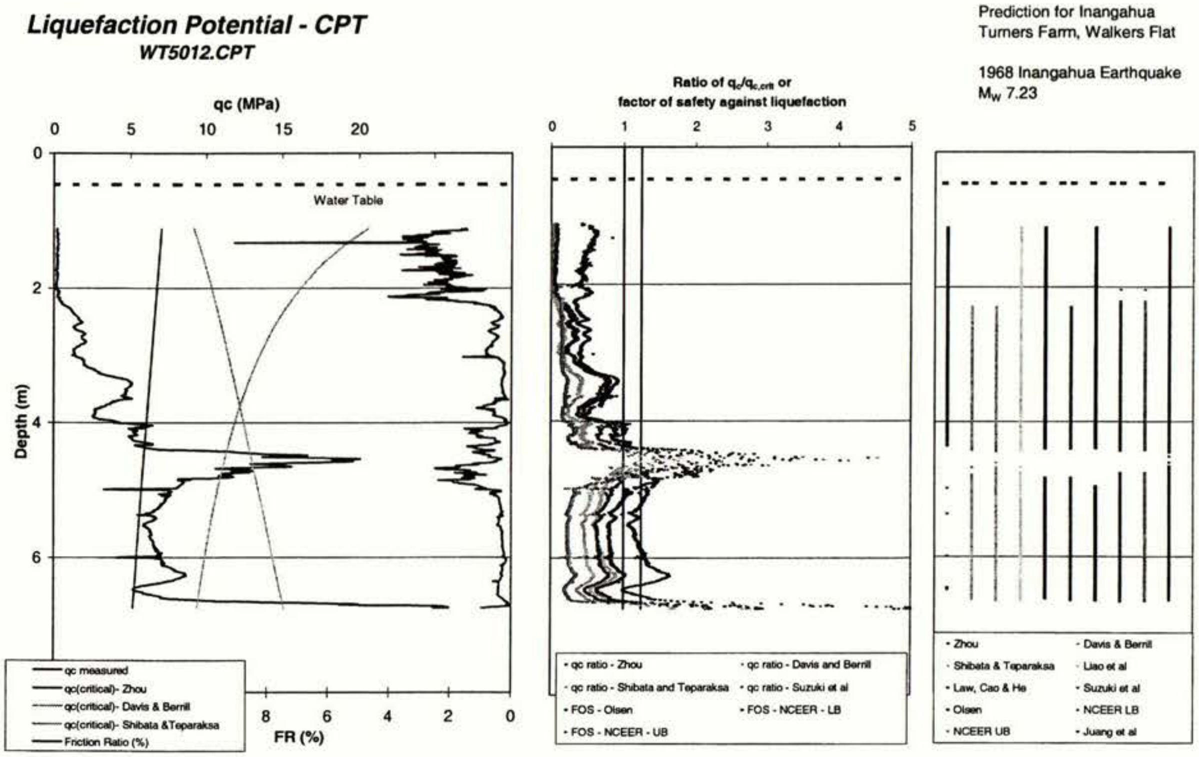
TUR011.CPT

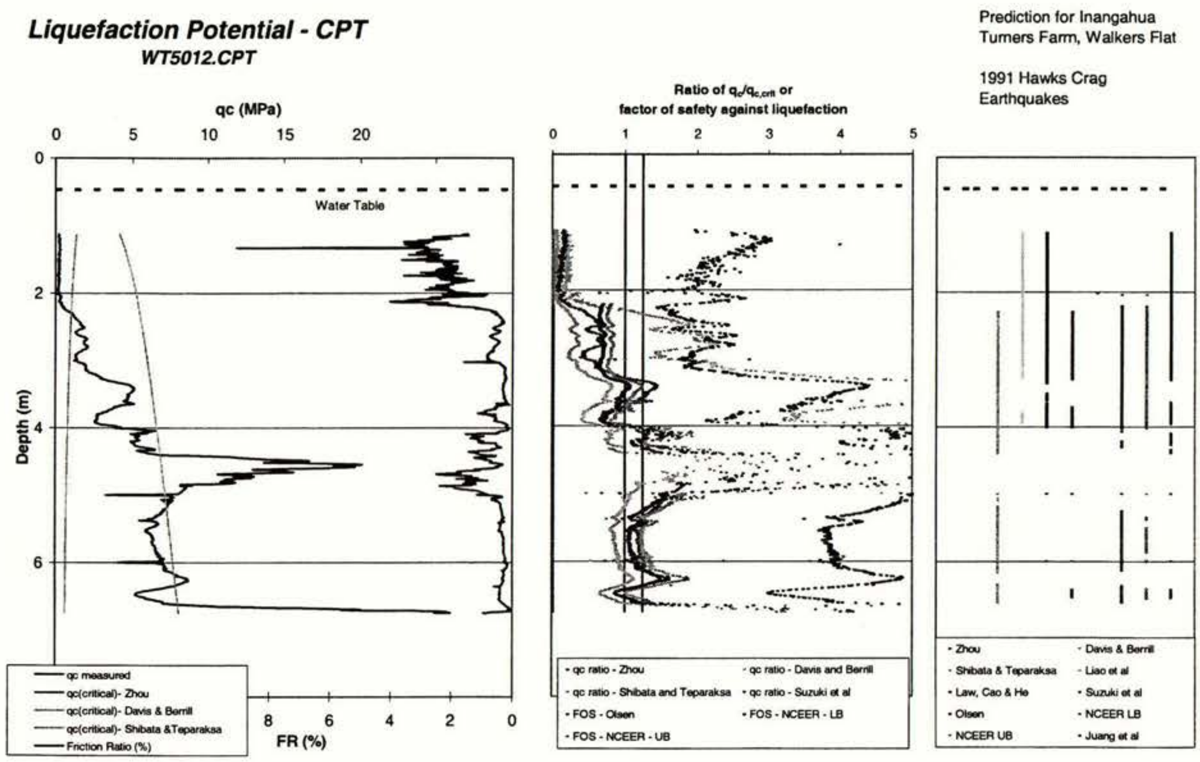


Inferred Soil Profiles - based on experimental data
WT5012.CPT
qc (MPa)
0
5
10
15
20
Water Table
Depth (m)
FR (%)
8
6
4
2
0
qc measured
Friction Ratio (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)

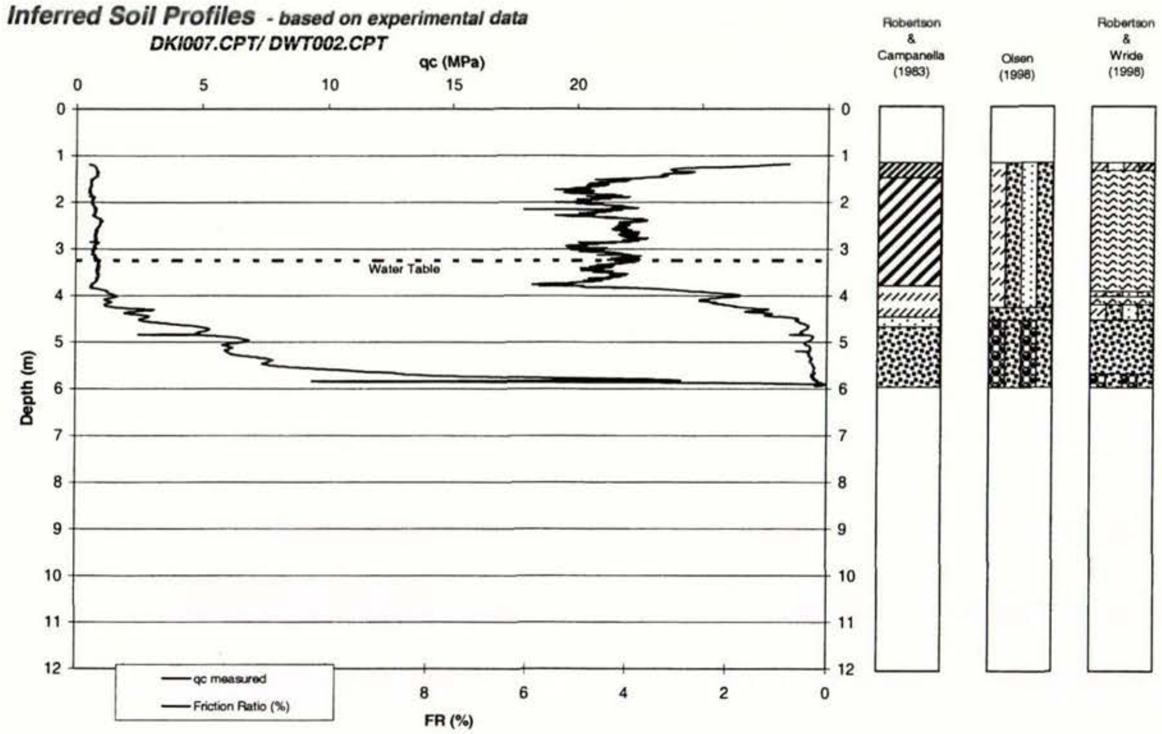


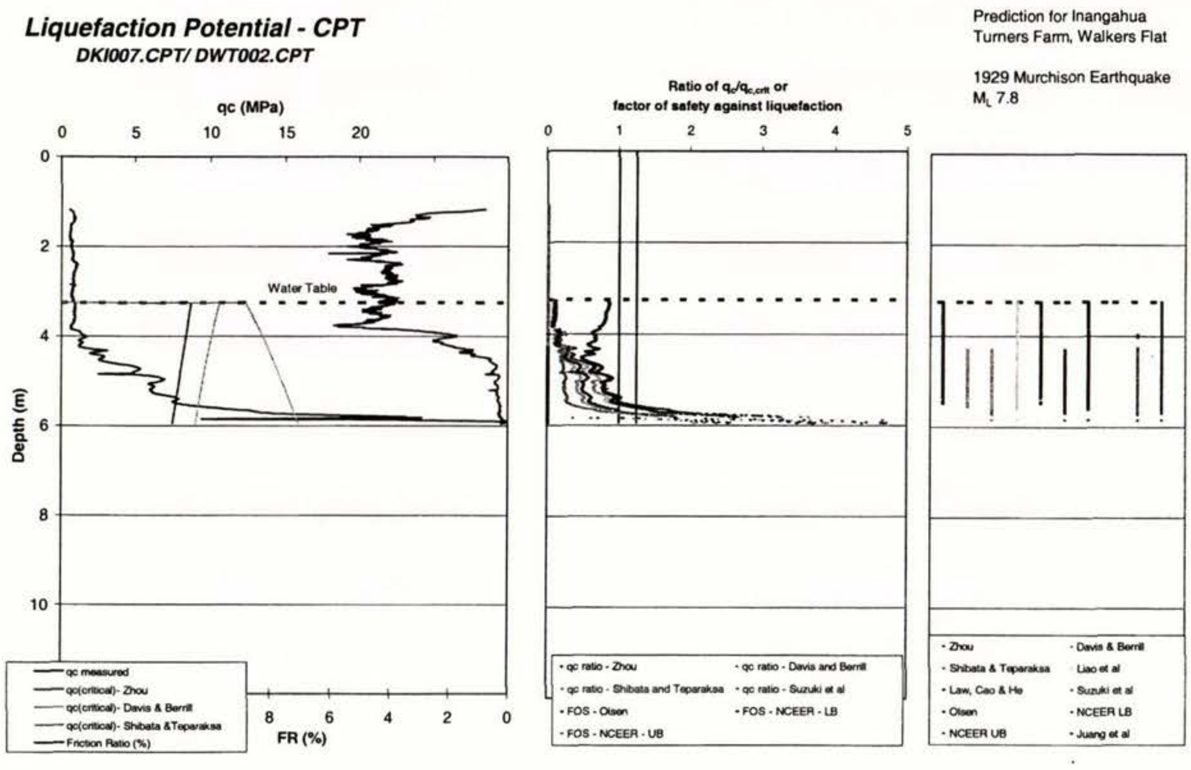
Liquefaction Potential - CPT
WT5012.CPT
qc (MPa)
Water Table
Depth (m)
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
- qc ratio - Zhou
- qc ratio - Davis and Berrill
- qc ratio - Shibata and Teparaksa
- qc ratio - Suzuki et al
- FOS - Olsen
- FOS - NCEER - LB
- FOS - NCEER - UB
Prediction for Inangahua
Turners Farm, Walkers Flat
1929 Murchison Earthquake
M_L 7.8
- Zhou
- Davis & Berrill
- Shibata & Teparaksa
- Liao et al
- Law, Cao & He
- Suzuki et al
- Olsen
- NCEER LB
- NCEER UB
- Juang et al

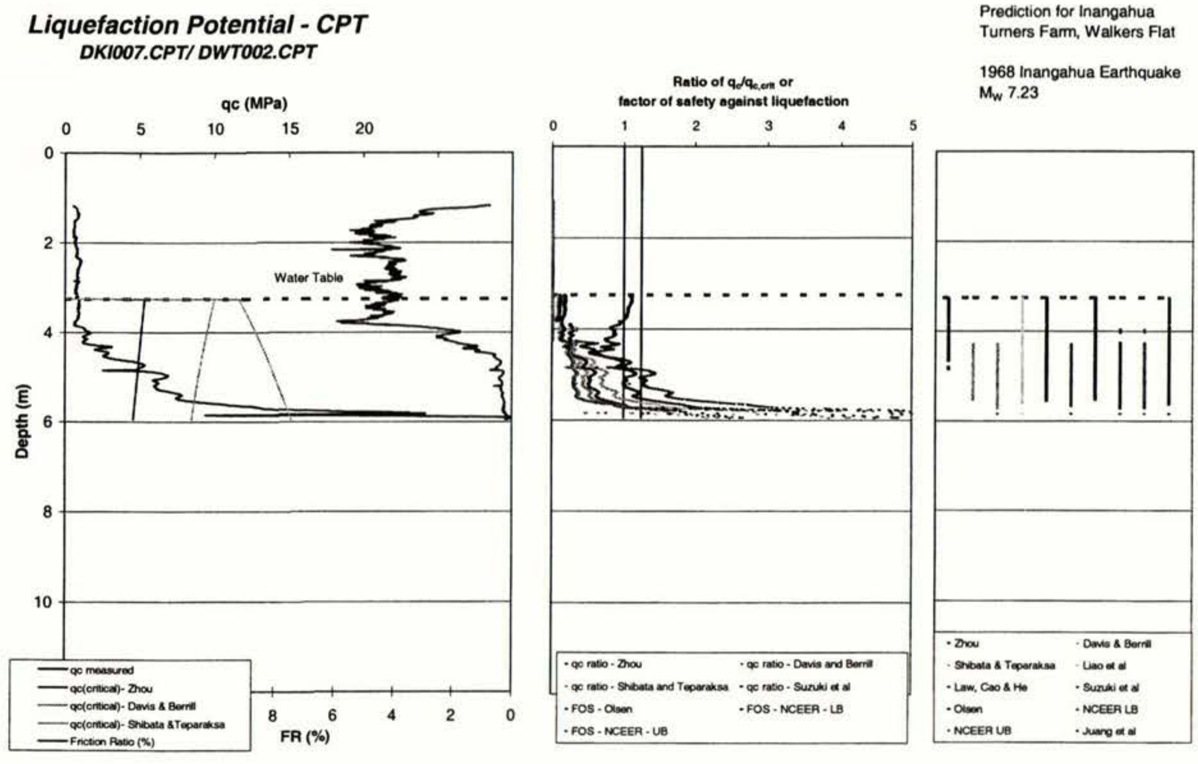

Liquefaction Potential - CPT
WT5012.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
Water Table
Depth (m)
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

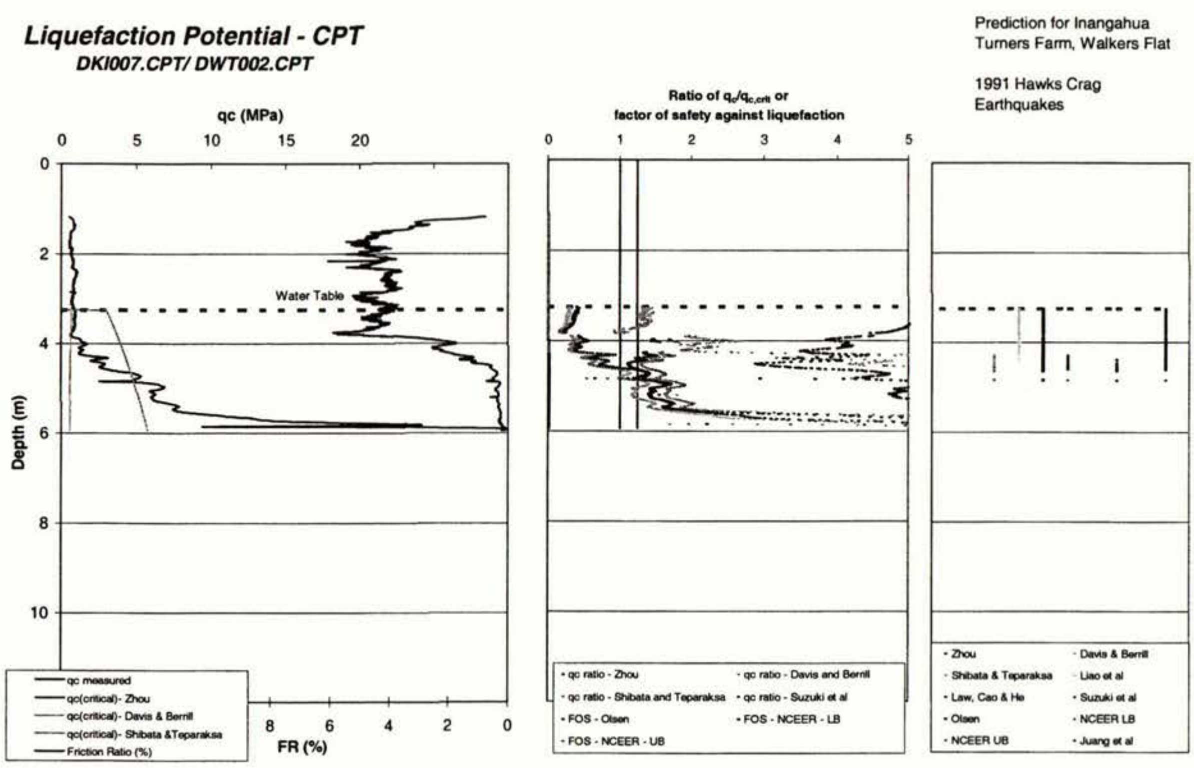

Liquefaction Potential - CPT
WT5012.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
Water Table
Depth (m)
FR (%)
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

TUR012.CPT

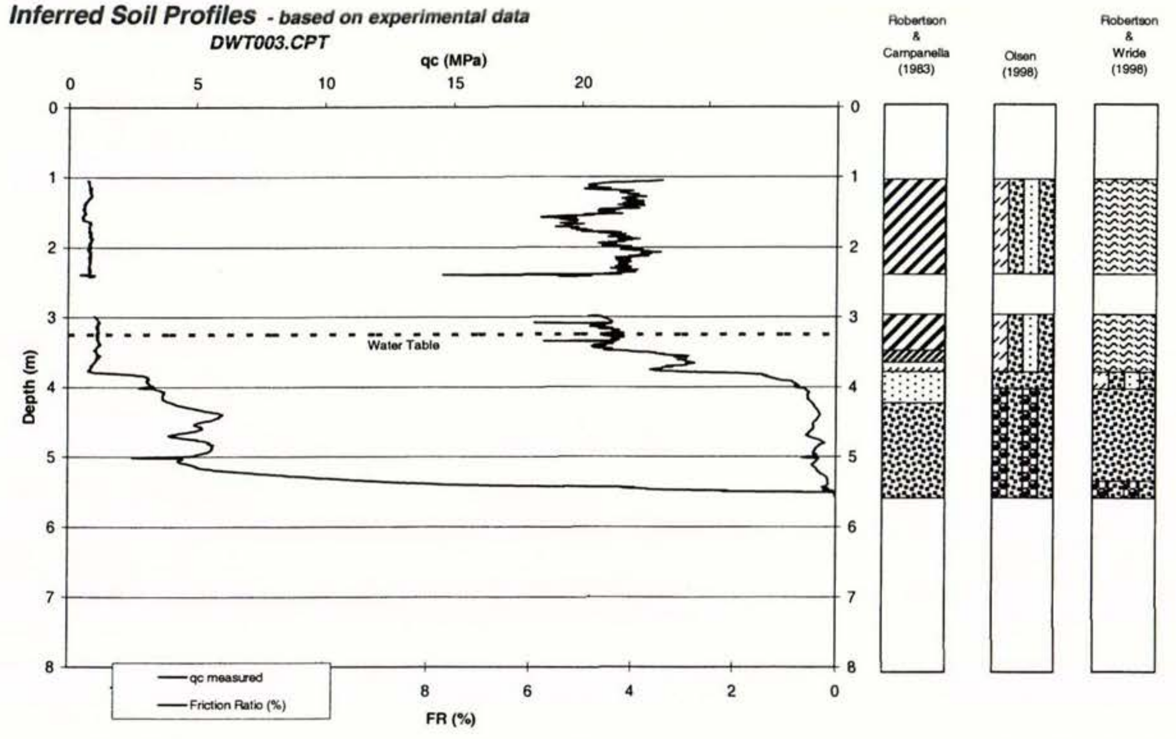

Inferred Soil Profiles - based on experimental data
DKI007.CPT/ DWT002.CPT
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
0 1 2 3 4 5 6 7 8 9 10 11 12
qc measured
Friction Ratio (%)
8 6 4 2 0
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)

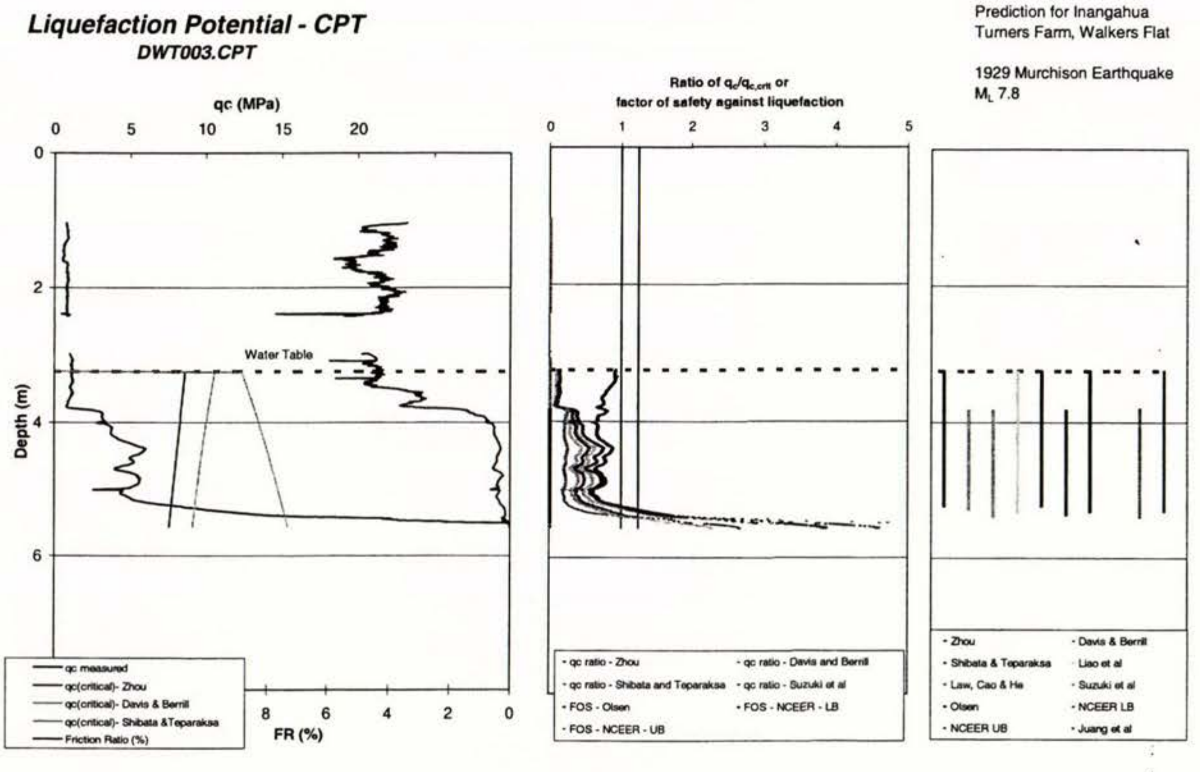

Liquefaction Potential - CPT
DKI007.CPT/ DWT002.CPT
qc (MPa)
0 5 10 15 20
Water Table
Depth (m)
0 2 4 6 8 10
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Prediction for Inangahua
Turners Farm, Walkers Flat
1929 Murchison Earthquake
M_L 7.8
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al


Liquefaction Potential - CPT
DKI007.CPT/ DWT002.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0 5 10 15 20
Depth (m)
0 2 4 6 8 10
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al


Liquefaction Potential - CPT
DKI007.CPT/ DWT002.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
0 5 10 15 20
Depth (m)
0 2 4 6 8 10
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
8 6 4 2 0
FR (%)
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0 1 2 3 4 5
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

TUR013.CPT

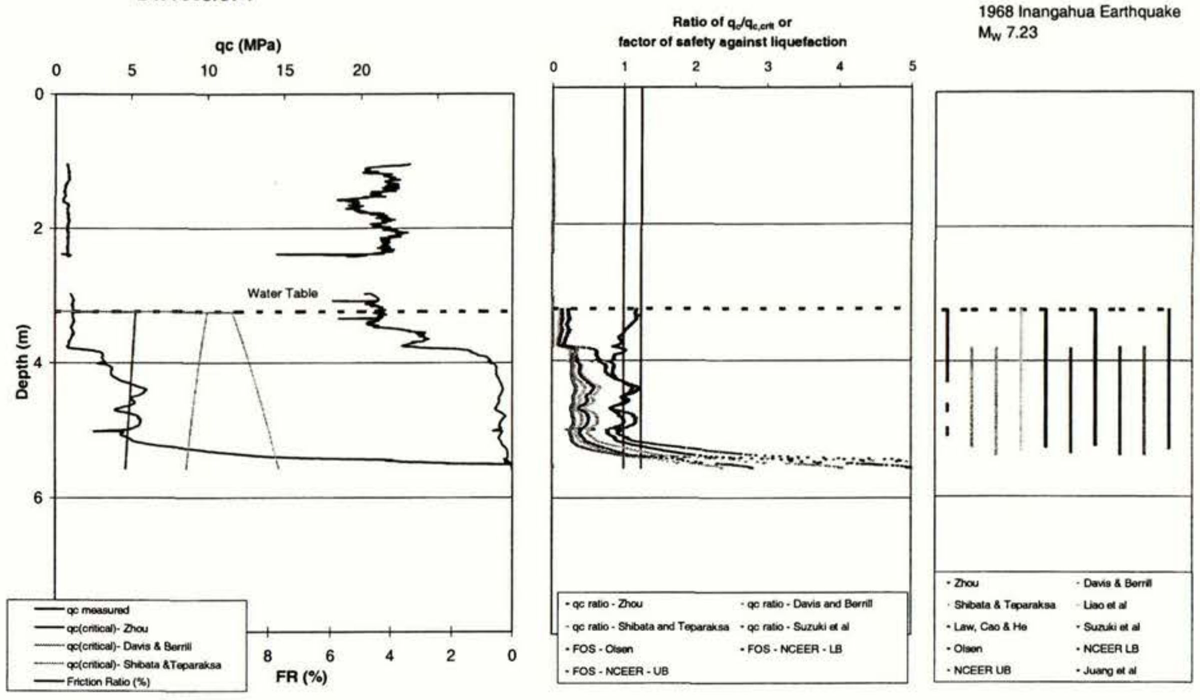

Inferred Soil Profiles - based on experimental data
DWT003.CPT
qc (MPa)
Depth (m)
Water Table
qc measured
Friction Ratio (%)
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)


Liquefaction Potential - CPT
DWT003.CPT
qc (MPa)
Depth (m)
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
- qc ratio - Zhou
- qc ratio - Davis and Berrill
- qc ratio - Shibata and Teparaksa
- qc ratio - Suzuki et al
- FOS - Olsen
- FOS - NCEER - LB
- FOS - NCEER - UB
Prediction for Inangahua
Turners Farm, Walkers Flat
1929 Murchison Earthquake
M_L 7.8
- Zhou
- Davis & Berrill
- Shibata & Teparaksa
- Liao et al
- Law, Cao & He
- Suzuki et al
- Olsen
- NCEER LB
- NCEER UB
- Juang et al

Liquefaction Potential - CPT
DWT003.CPT

Prediction for Inangahua
Turners Farm, Walkers Flat

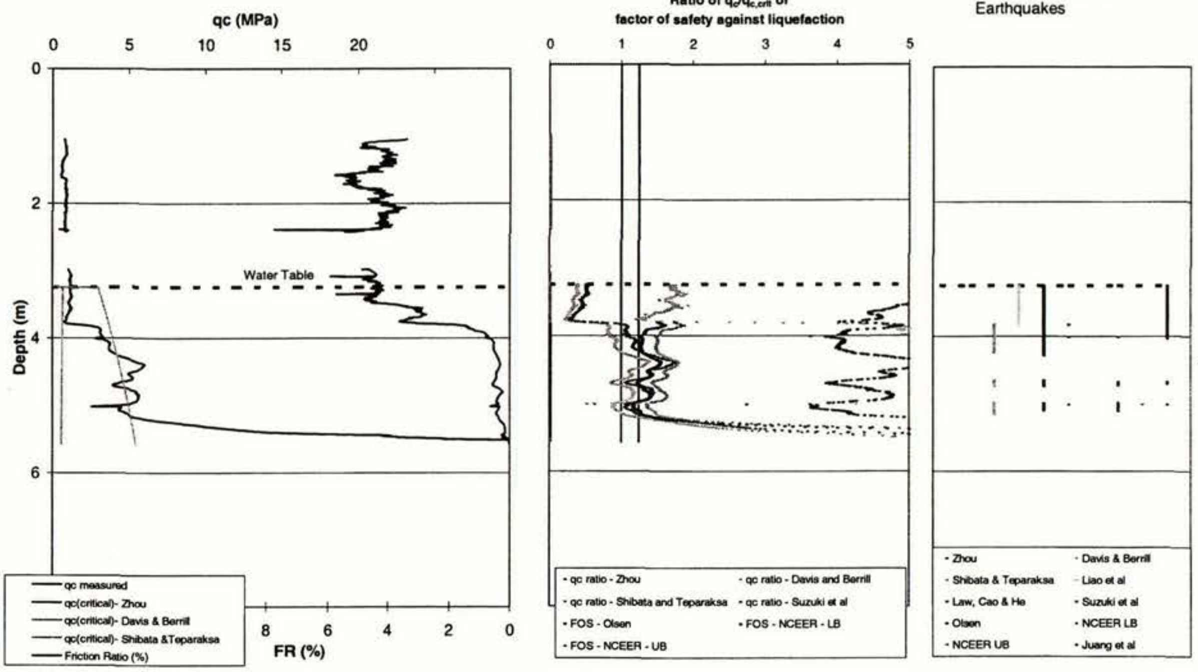
1968 Inangahua Earthquake
M_W 7.23



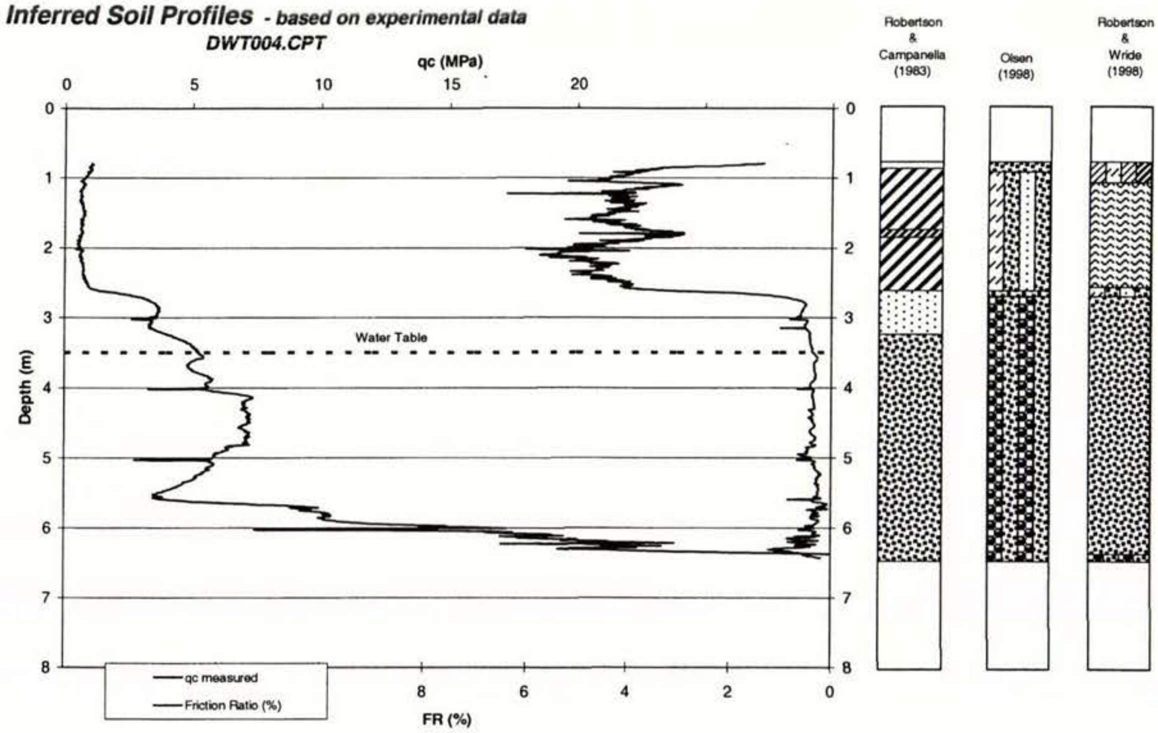
Liquefaction Potential - CPT
DWT003.CPT

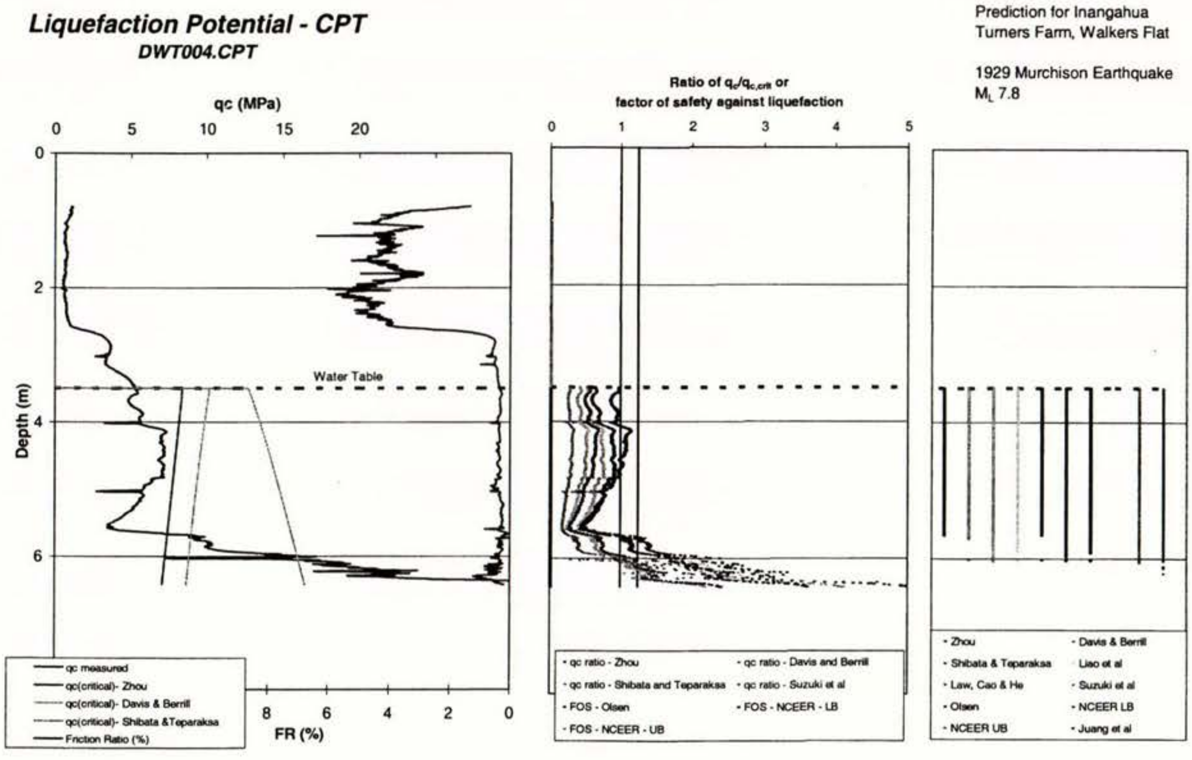
Prediction for Inangahua
Turners Farm, Walkers Flat

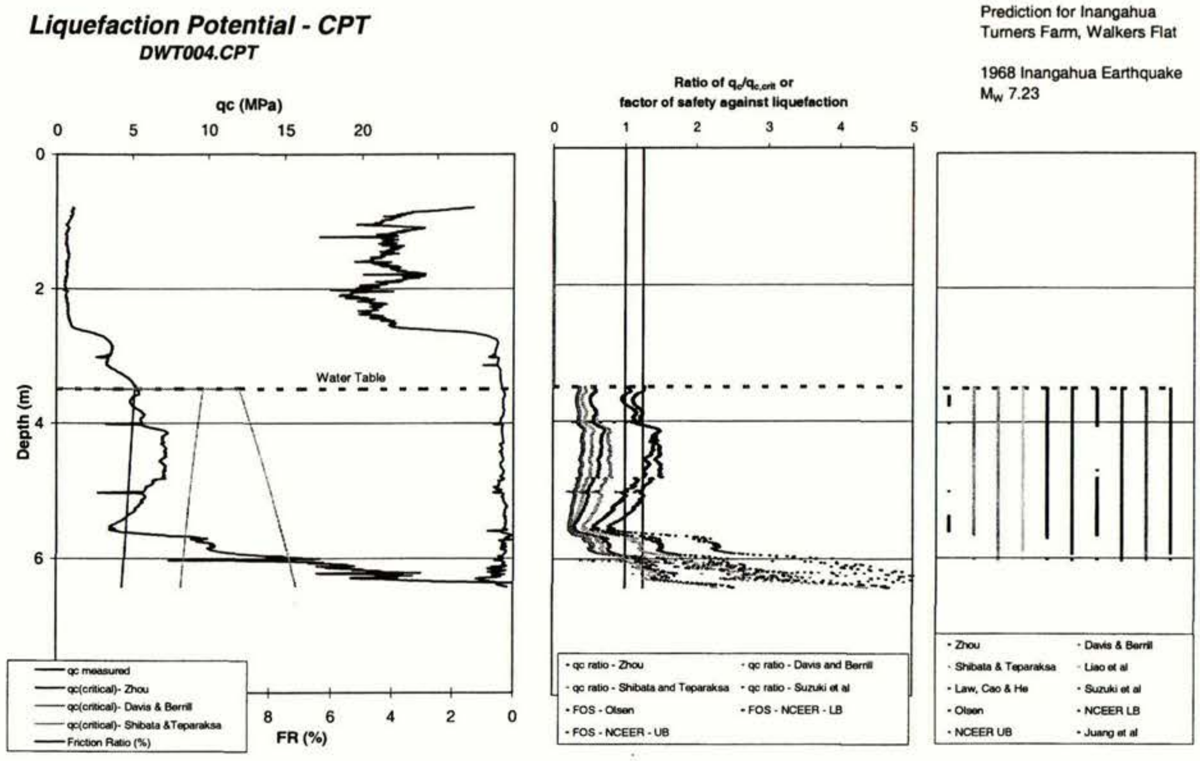
1991 Hawks Crag
Earthquakes



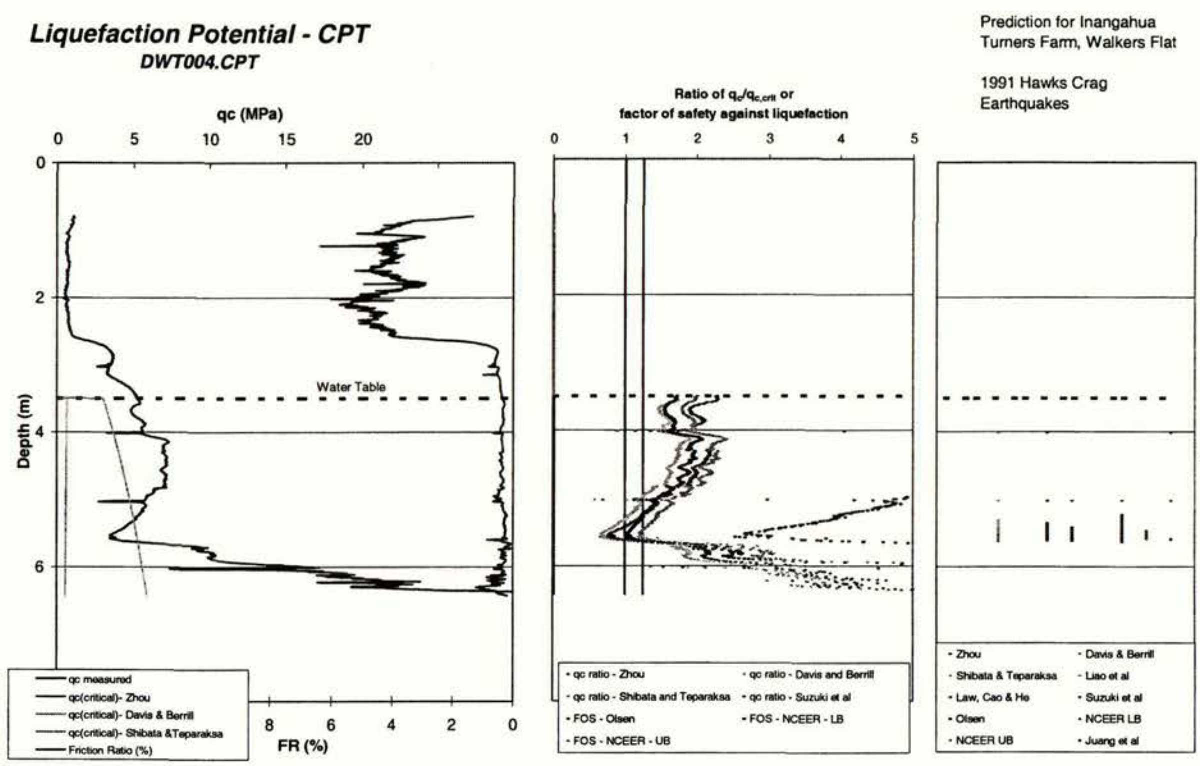
TUR014.CPT


Inferred Soil Profiles - based on experimental data
DWT004.CPT
qc (MPa)
0
5
10
15
20
Depth (m)
Water Table
qc measured
Friction Ratio (%)
8
6
4
2
0
FR (%)
Robertson & Campanella (1983)
Olsen (1998)
Robertson & Wride (1998)


Liquefaction Potential - CPT
DWT004.CPT
qc (MPa)
0
5
10
15
20
Depth (m)
Water Table
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
FR (%)
Ratio of $q_c/q_{c,crit}$ or factor of safety against liquefaction
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Prediction for Inangahua
Turners Farm, Walkers Flat
1929 Murchison Earthquake
M_L 7.8
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al



Liquefaction Potential - CPT
DWT004.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1968 Inangahua Earthquake
M_W 7.23
qc (MPa)
0
5
10
15
20
Depth (m)
0
2
4
6
Water Table
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0
1
2
3
4
5
FR (%)
8
6
4
2
0
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al



Liquefaction Potential - CPT
DWT004.CPT
Prediction for Inangahua
Turners Farm, Walkers Flat
1991 Hawks Crag
Earthquakes
qc (MPa)
0
5
10
15
20
Depth (m)
0
2
4
6
Water Table
Ratio of $q_c/q_{c,crit}$ or
factor of safety against liquefaction
0
1
2
3
4
5
FR (%)
8
6
4
2
0
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
qc ratio - Zhou
qc ratio - Davis and Berrill
qc ratio - Shibata and Teparaksa
qc ratio - Suzuki et al
FOS - Olsen
FOS - NCEER - LB
FOS - NCEER - UB
Zhou
Davis & Berrill
Shibata & Teparaksa
Liao et al
Law, Cao & He
Suzuki et al
Olsen
NCEER LB
NCEER UB
Juang et al

Appendix C: Sensitivity Study

Four sites were used in this study to see the variations produced in the results of the different models when the peak ground acceleration experienced at the site, a_{max}, and the distance from the epicentre of the earthquake to the site, R, were altered by +- 10%.

The following sites were used in the study:

Collins Street, Greymouth

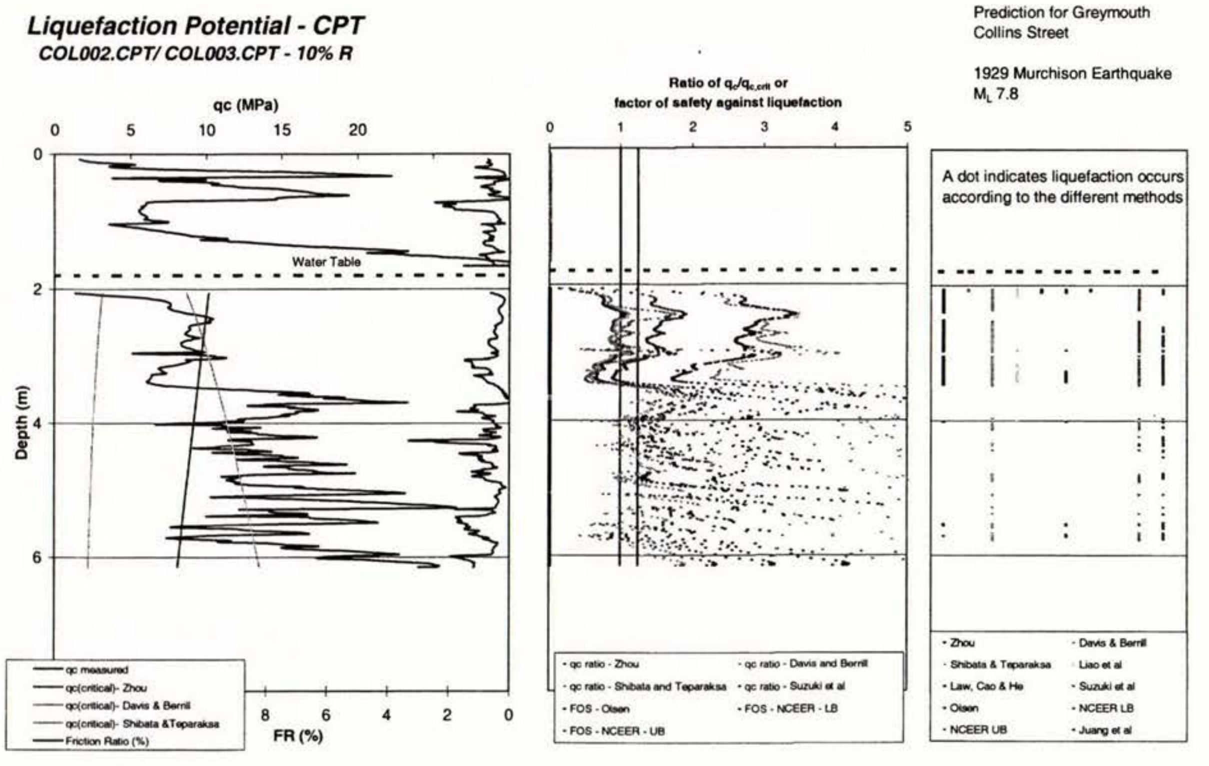
Fensom's Paddock, Karamea

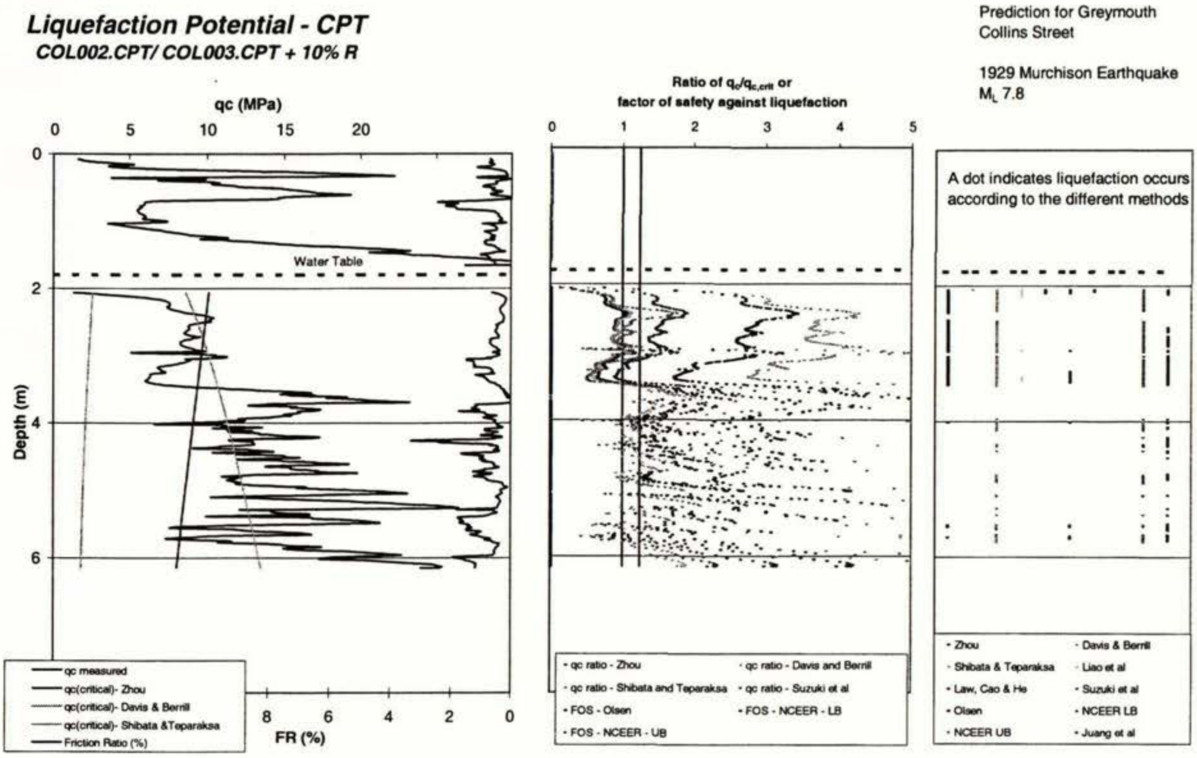
Simpson's Paddock, Karamea

Oparara School Site, Oparara

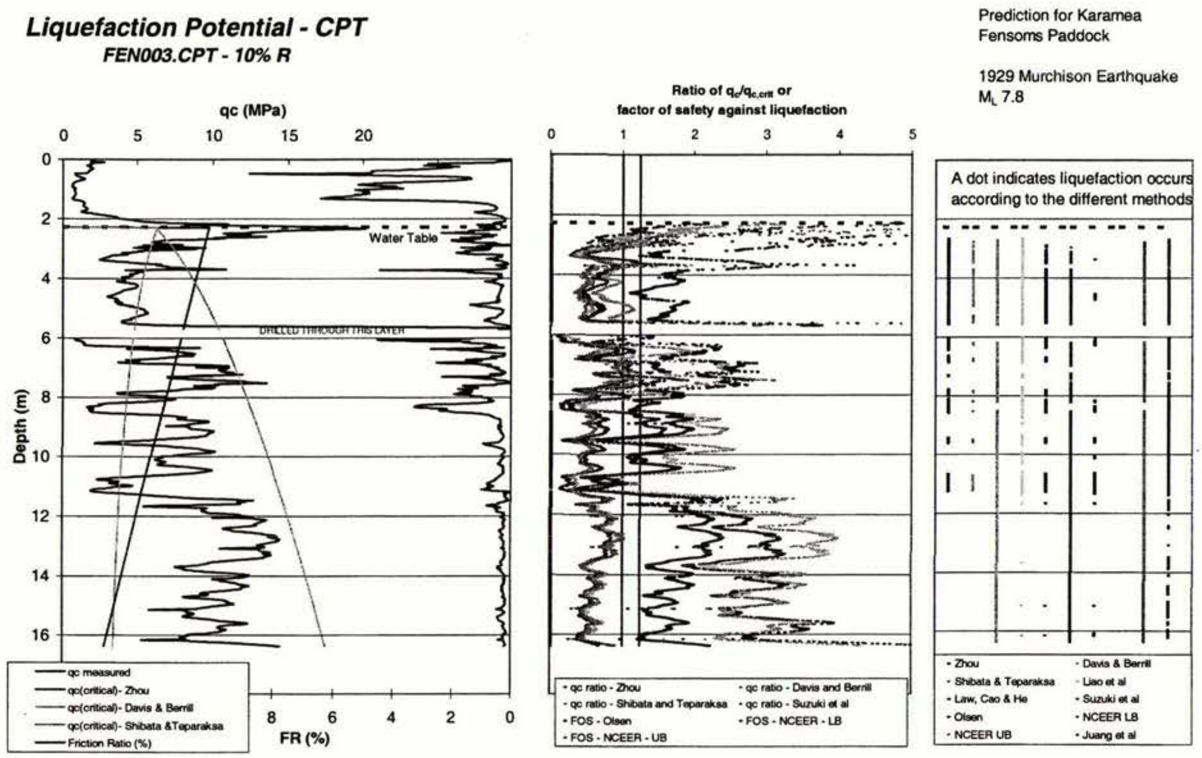
C1.1 Distance R Modifications

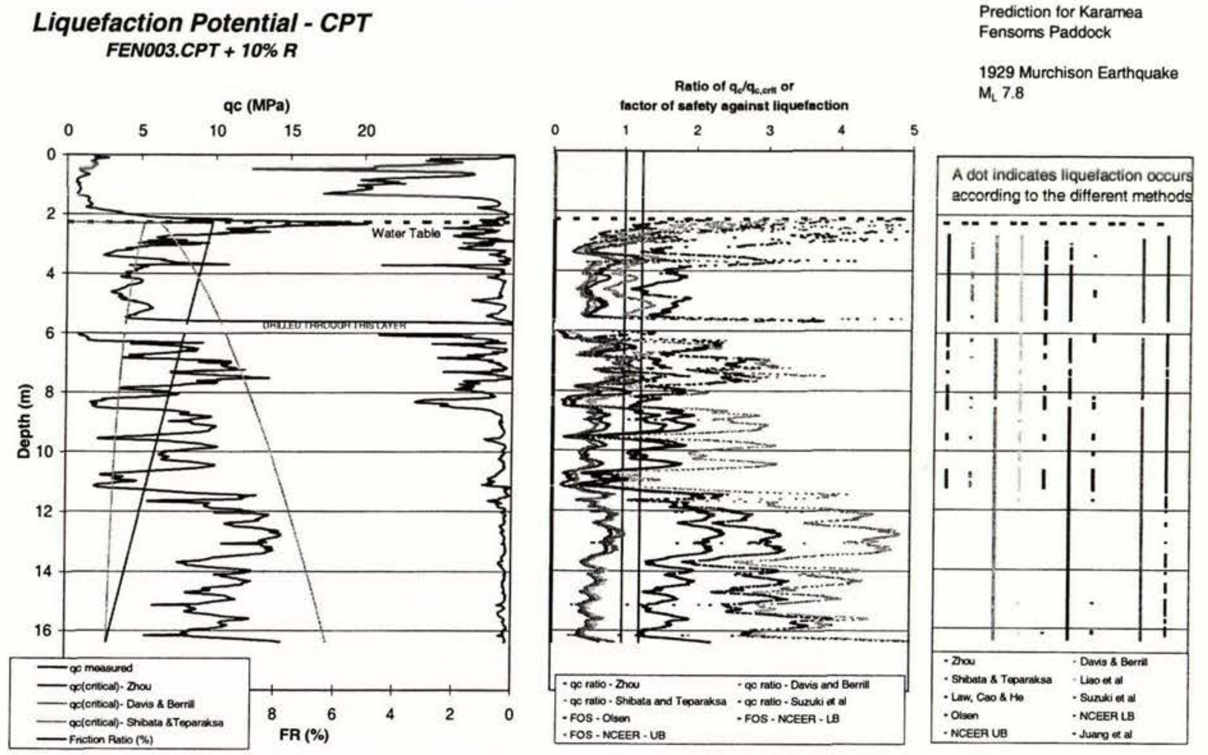
C1.1.1 Collins Street



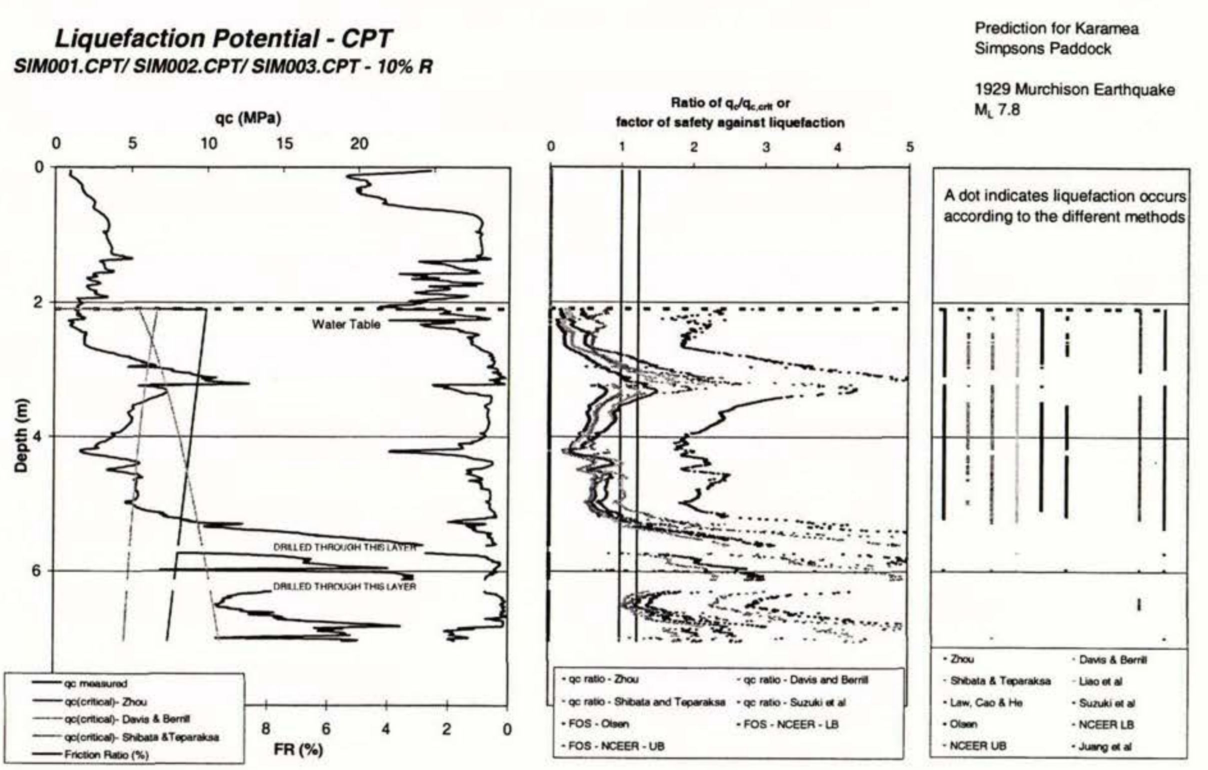


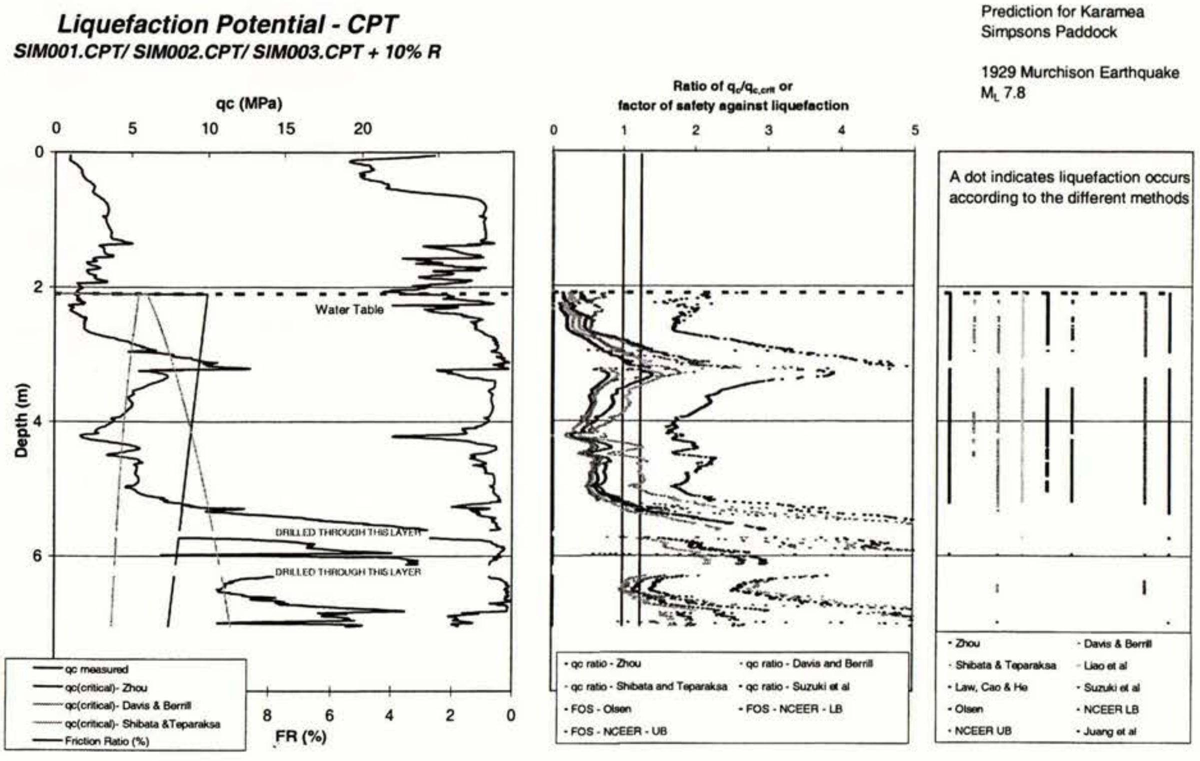
C1.1.2 Fensom's Paddock



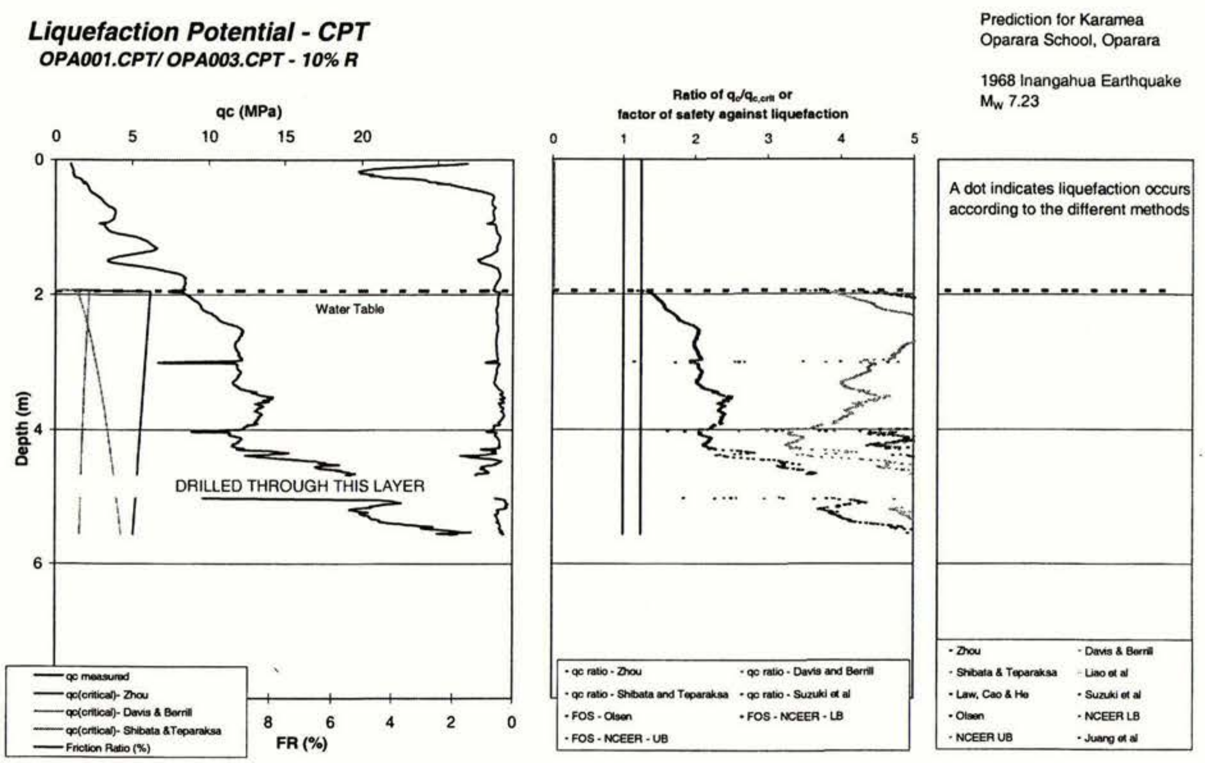


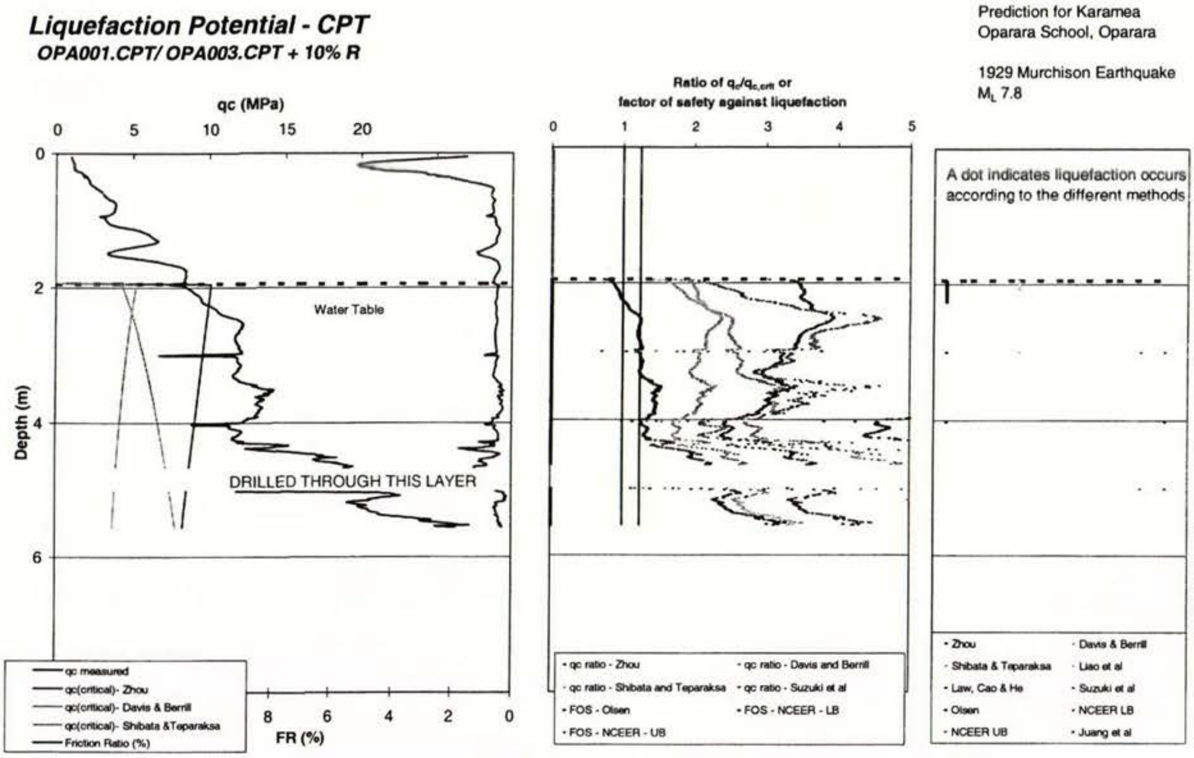
C1.1.3Simpson's Paddock





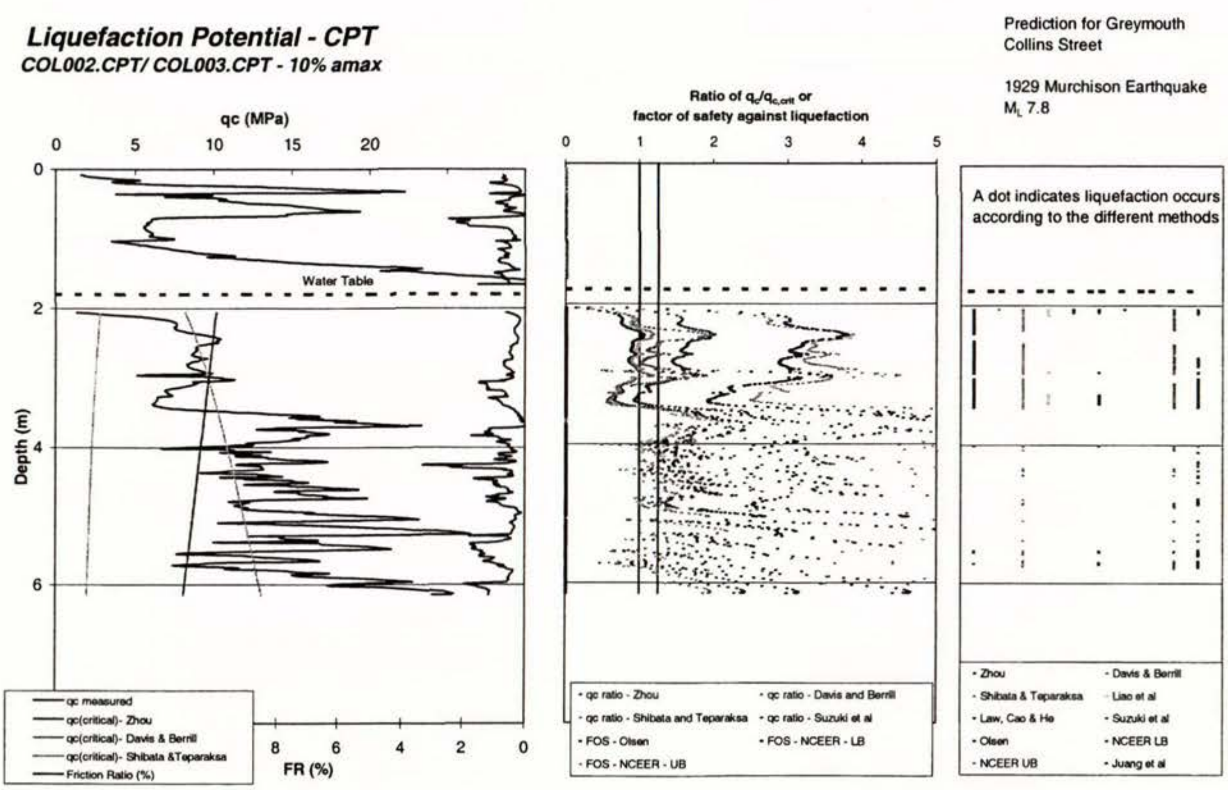
C1.1.4 Oparara School Site

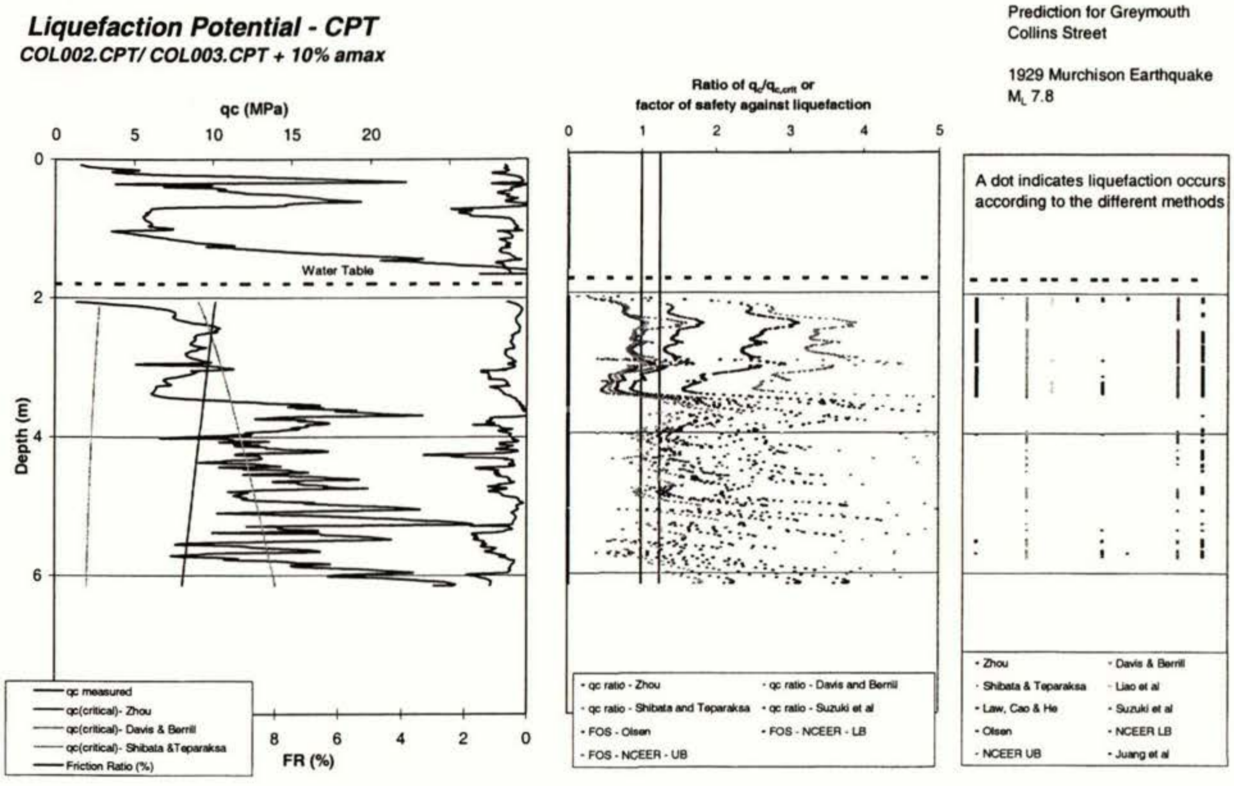




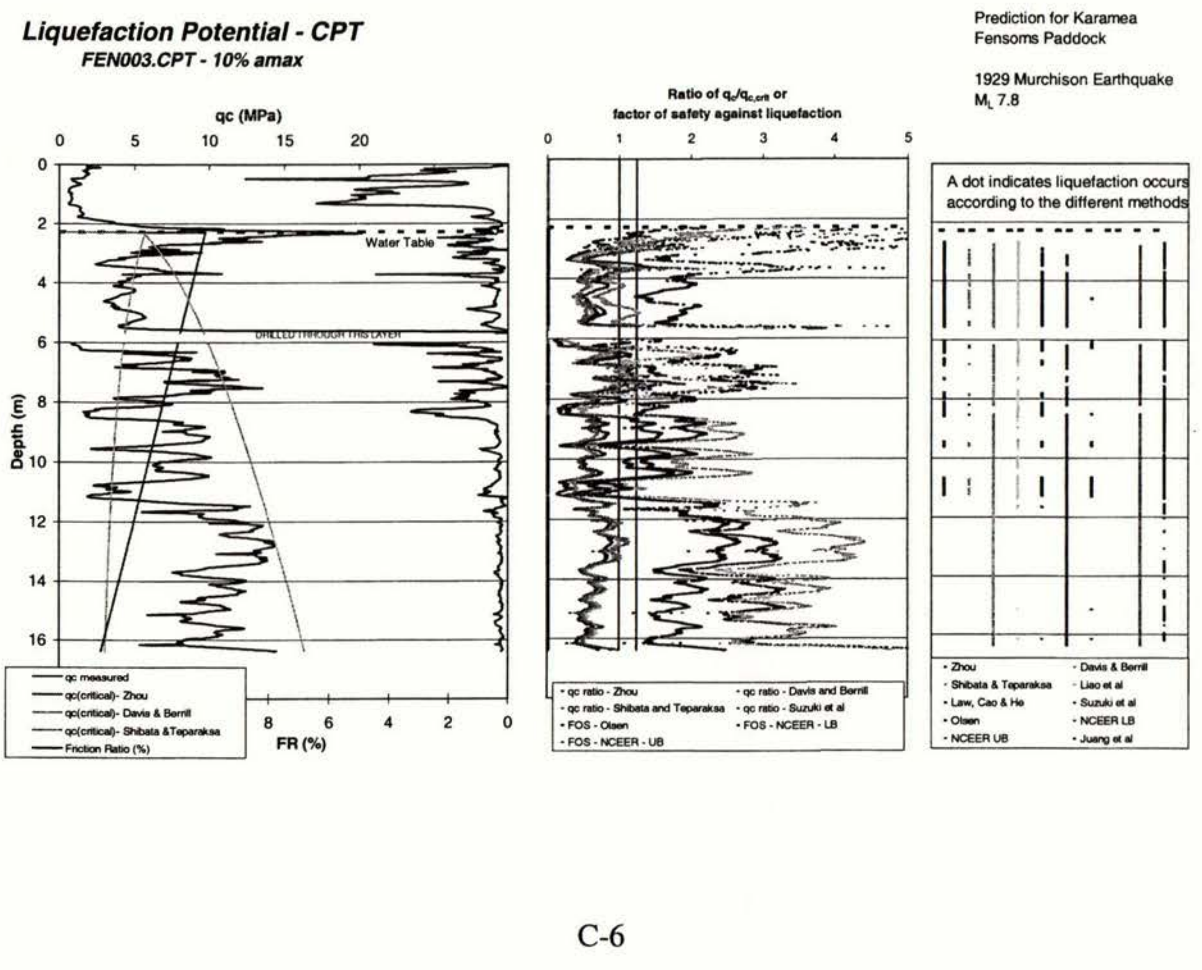
C1.2 Peak Ground Acceleration Modification

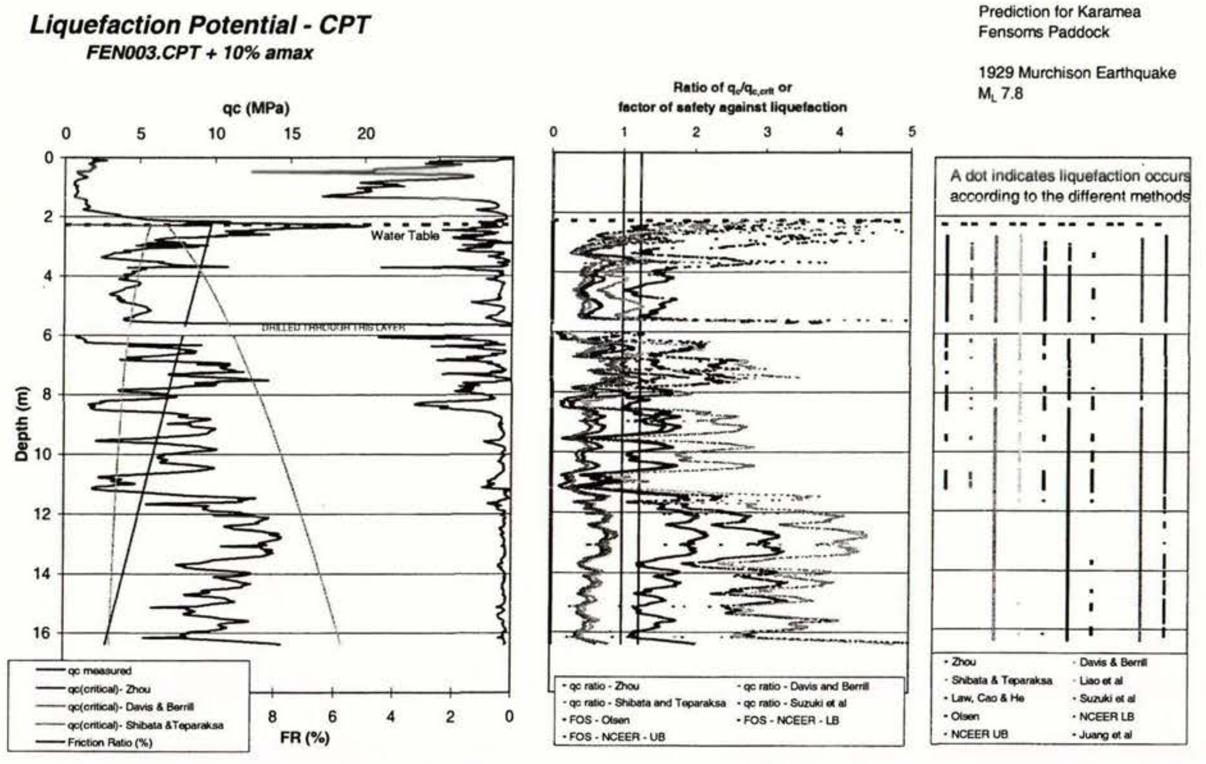
C1.2.1 Collins Street



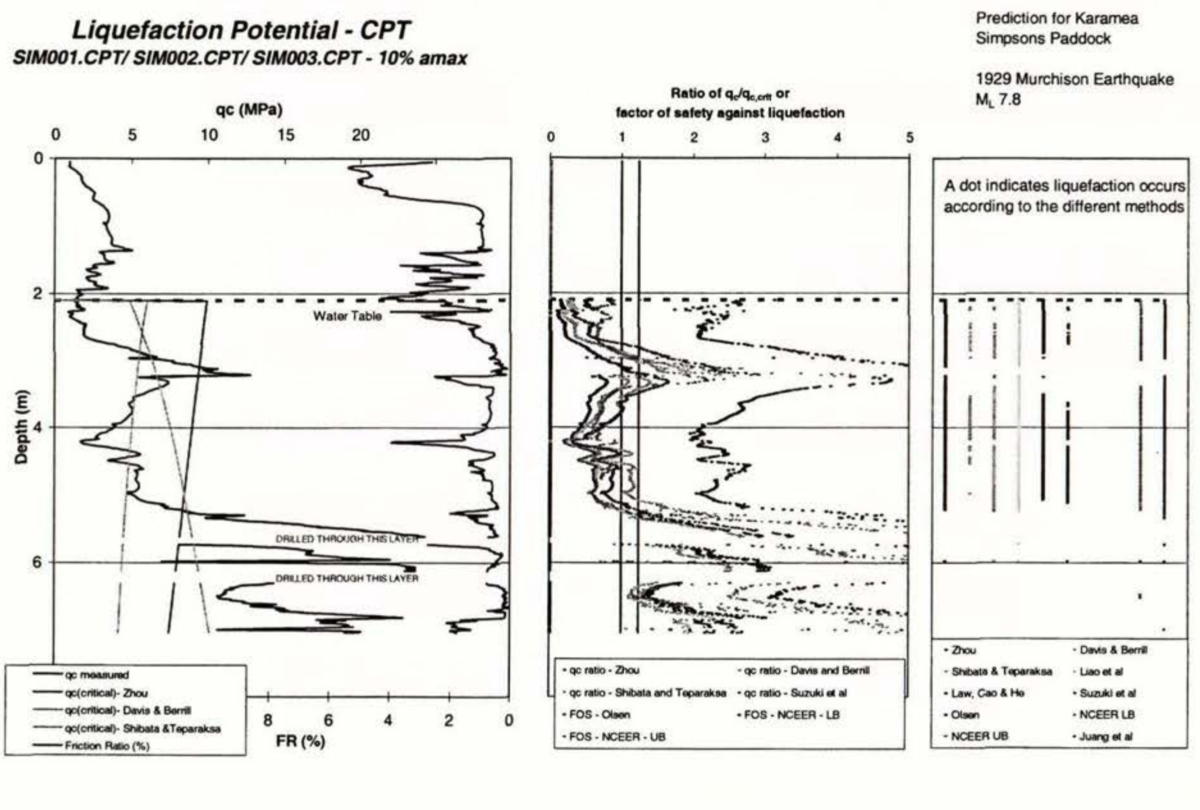


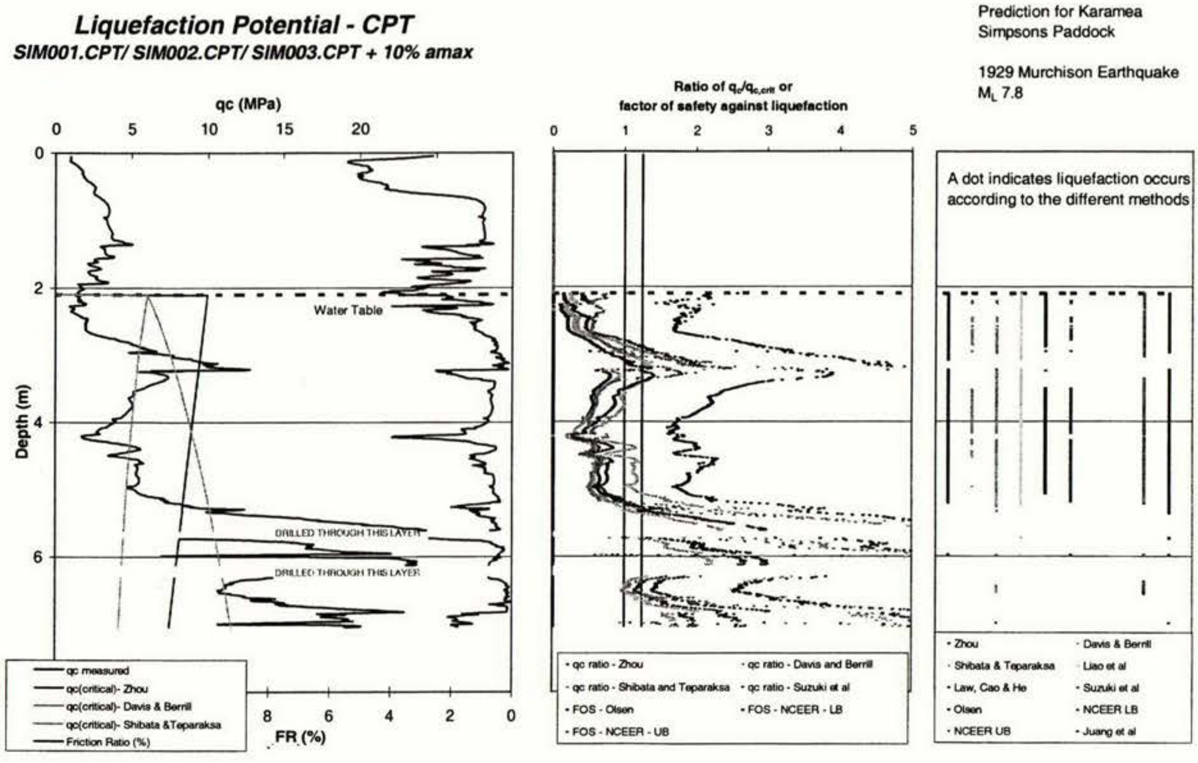
C1.2.2Fensom's Paddock



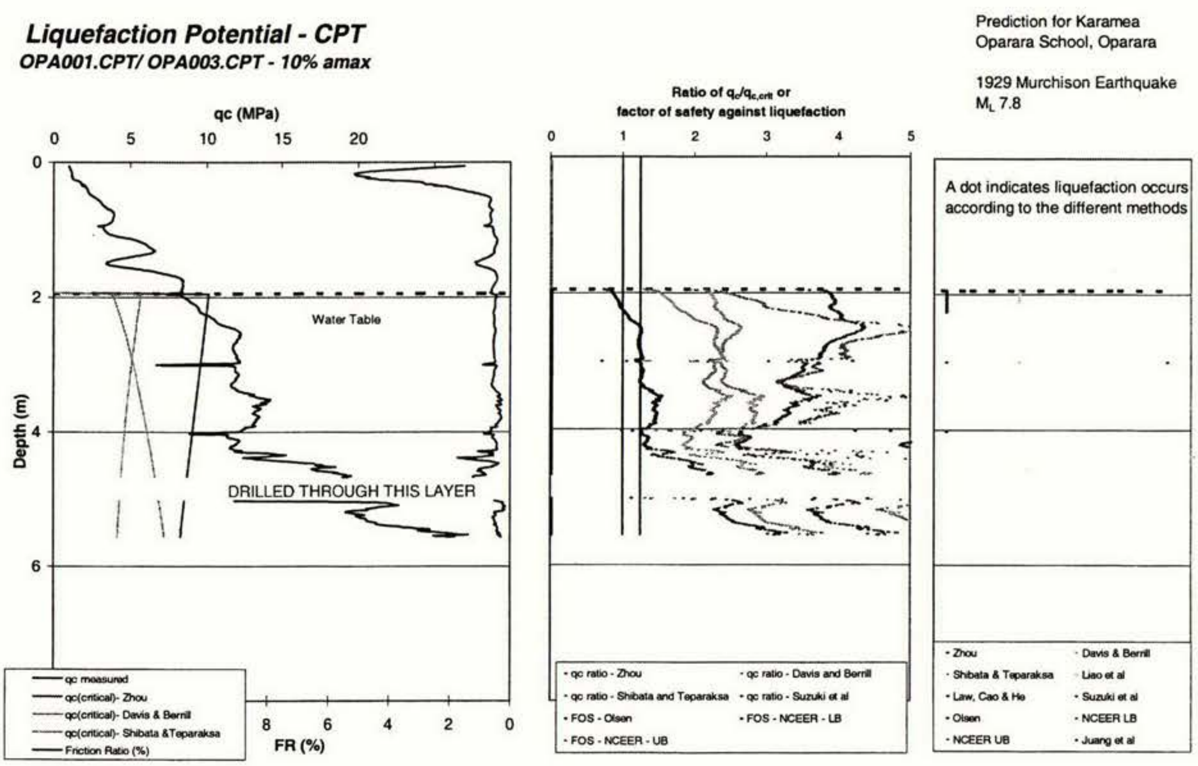


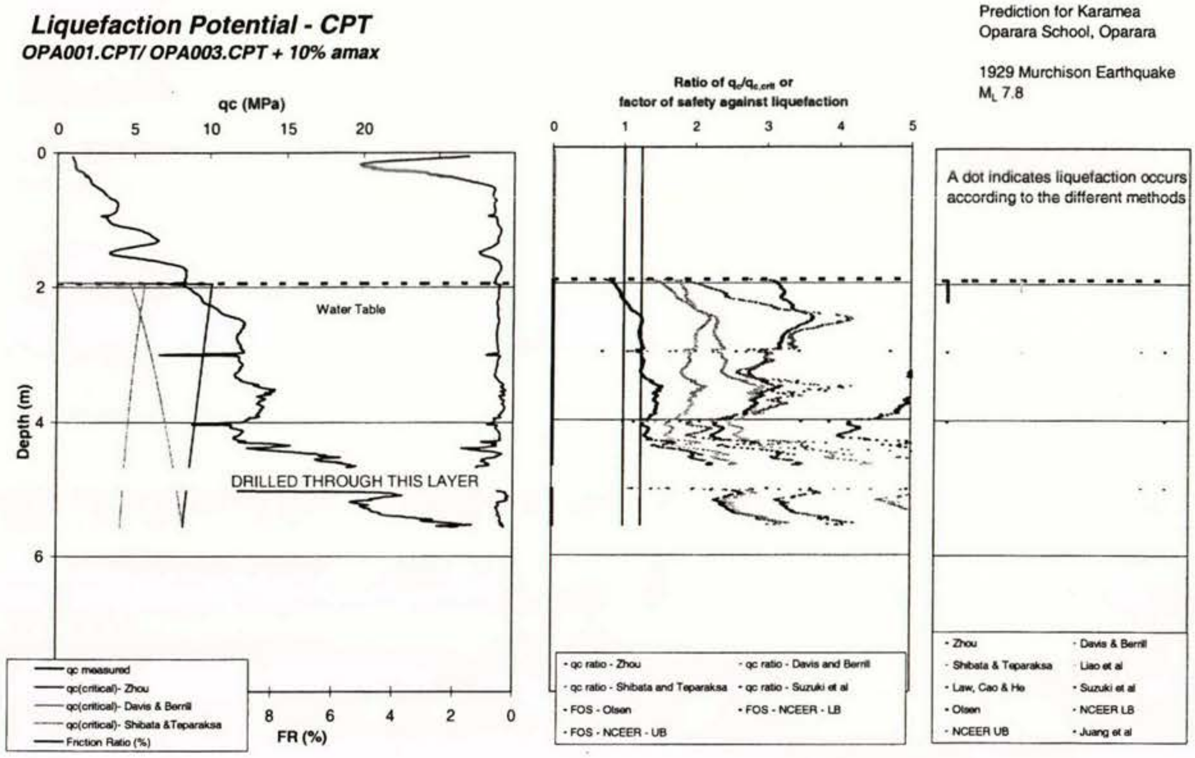
C1.2.3Simpson's Paddock





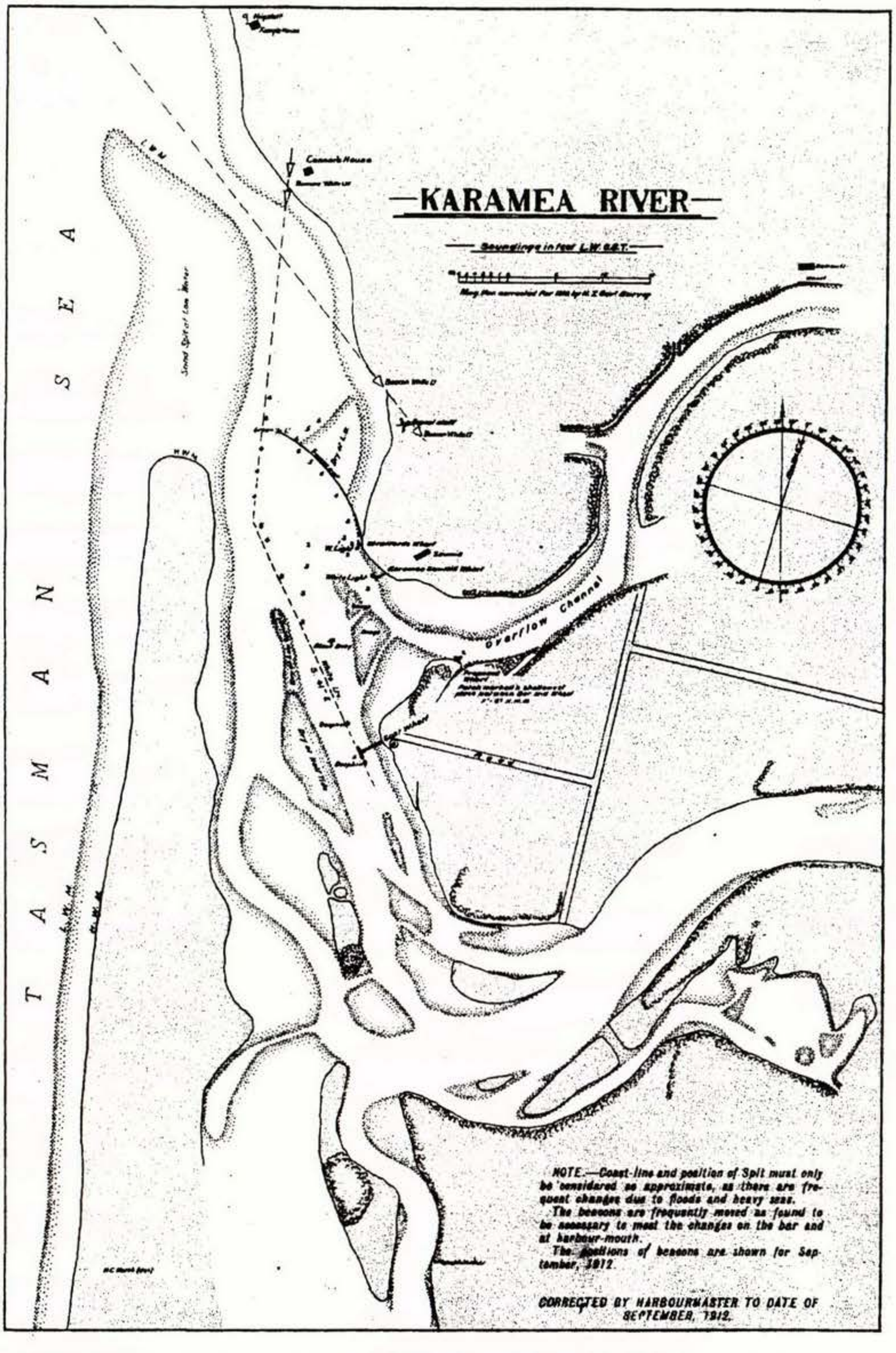
C1.2.4Oparara School Site




Liquefaction Potential - CPT
OPA001.CPT/ OPA003.CPT + 10% amax
Prediction for Karamea
Oparara School, Oparara
1929 Murchison Earthquake
M_L 7.8
qc (MPa)
0
5
10
15
20
Depth (m)
0
2
4
6
Water Table
DRILLED THROUGH THIS LAYER
FR (%)
8
6
4
2
0
qc measured
qc(critical)- Zhou
qc(critical)- Davis & Berrill
qc(critical)- Shibata &Teparaksa
Friction Ratio (%)
Ratio of qc/qc,crit or
factor of safety against liquefaction
0
1
2
3
4
5
- qc ratio - Zhou
- qc ratio - Davis and Berrill
- qc ratio - Shibata and Teparaksa
- qc ratio - Suzuki et al
- FOS - Olsen
- FOS - NCEER - LB
- FOS - NCEER - UB
A dot indicates liquefaction occurs according to the different methods
- Zhou
- Davis & Berrill
- Shibata & Teparaksa
Liao et al
- Law, Cao & He
- Suzuki et al
- Olsen
- NCEER LB
- NCEER UB
- Juang et al

Appendix D: Maps of Greymouth and Karamea

Churchouse (1982) showed maps of the entrances to the Greymouth and Karamea harbours which were dated 1912. These are shown in Figures D-1 and D-2 below.



THIS PLAN OF THE KARAMEA RIVER CLEARLY SHOWS THE EXPOSED BAR ENTRANCE AND THE TORTUOUS CHANNEL LEADING TO THE WHARVES.

Figure D- 1. 1912 Map of Karamea Harbour (Churchouse, 1982)

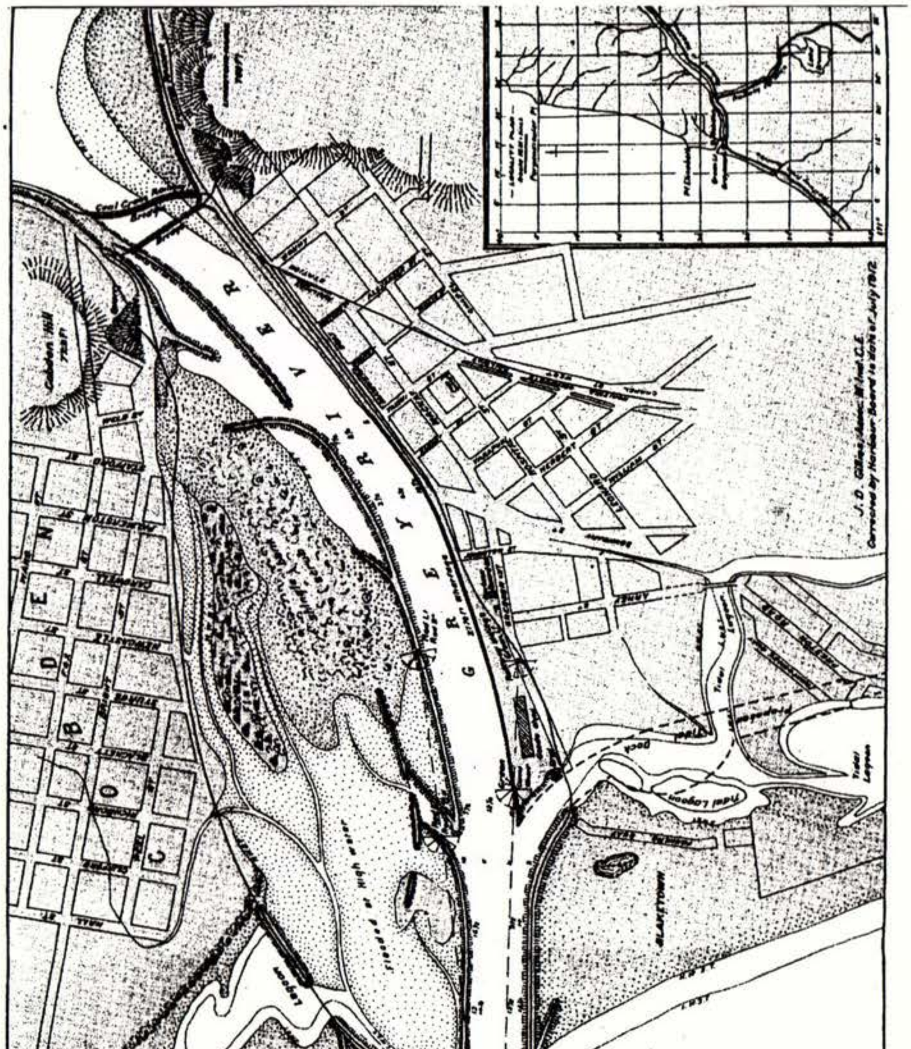


Figure D- 2. 1912 Map of Greymouth Harbour (Churchouse, 1982)

Appendix E: Sources of Information/ Thanks

The following organisations, voluntary groups and people all provided a wonderful source of knowledge and insight into the events of the past. Without the support of these groups the historical side of this project would have been impossible. The support of the council organisations and property owners in Greymouth and Karamea also meant that soil testing could be undertaken. Their assistance and enthusiasm is also gratefully acknowledged.

Alexander Turnbull Library	Cnr Molesworth and Aitken Streets P.O. Box 12349 Wellington www.natlib.govt.nz
Archives New Zealand	90 Peterborough Street P.O. Box 642 Christchurch
Blacks Point Museum	Franklin Street Blacks Point
Buller District Council	Brougham Street POBox 21 Westport
Christchurch City Library	Gloucester Street POBox 1466 Christchurch
Canterbury Museum	Rolleston Avenue Christchurch
Coaltown Museum	Upper Queen Street POBox 216 Westport
Collingwood Museum	Tasman Street Collingwood
Golden Bay Museum	69 Commercial Street Takaka 7172

Grey District Council	105 Tainui Street POBox 382 Greymouth
Greymouth Evening Star	5-9 Werita Street Greymouth
Grey District Library	POBox 382 Greymouth
History House	1 Tainui Street Greymouth
Institute of Geological and Nuclear Sciences (IGNS)	Gracefield Research Centre 69 Gracefield Road P.O. Box 30368 Lower Hutt
Karamea Information and Resource Centre	Market Cross Box 94 Karamea
Karamea Museum	Waverly Street Karamea
Motueka District Museum	140 High Street Motueka
Murchison District and Historical Society	60 Fairfax Street POBox 108 Murchison
Nelson Public Library	27 Halifax Street Nelson
Nelson Provincial Museum	Isel Park Hilliard Street Stoke Nelson
Museum of New Zealand Te Papa Tongarewa	Cable Street POBox 467 Wellington

University of Canterbury	Engineering Library MacMillan Brown Library Central Library University of Canterbury Private Bag 4800 Christchurch 1
Victoria University of Wellington	Earth Sciences Library Victoria University of Wellington POBox 600 Wellington 6001
West Coast Historical Museum	Corner of Tancred and Hamilton Streets P.O. Box 22 Hokitika
West Coast Regional Council	153 Tainui Street POBox 66 Greymouth
Westport North School	Cobden Street Westport

The Inangahua Museum was closed during course of study as the volunteer staff running the museum unfortunately did not have the time to open the museum.

The newspapers used in the study are listed below with the city or town of origin in parentheses.

The Auckland Weekly News (Auckland)

The Danniverke Evening News (Danniverke)

The Dominion (Wellington)

The Evening Post (Wellington)

The Greymouth Evening Star (Greymouth)

The Grey River Argus (Greymouth)

The Inangahua Herald (Inangahua)

The Nelson Evening Mail (Nelson)

The News (Westport)

The New Zealand Free Lance (Auckland)

The NZ Truth (Auckland)

The Press (Christchurch)

The Sun (Christchurch)