EVALUATION OF LIQUEFACTION ASSESSMENT METHODS

Transfund New Zealand Research Report No. 109

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AN IMPORTANT NOTE FOR THE READER

The research detailed in this report was commissioned by Transit New Zealand when it had responsibility for funding roading in New Zealand. This funding is now the responsibility of Transfund New Zealand.

NOMENCLATURE

a_{max}	=	peak horizontal ground acceleration
A	=	material attenuation factor
c_2	=	constant dependant on the fines content (Sugawara method)
C_N	=	overburden correction factor
CSR	=	cyclic stress ratio
CSR_{crit}	=	critical cyclic stress ratio
CPT	=	cone penetration test
D_r	=	relative density
D_{50}	=	grain size corresponding to 50% by mass passing
g	=	gravitational acceleration
f_s	=	sleeve friction (from cone penetration test)
$F_{\%}$	=	percent fines
G	=	shear modulus
M_L	=	local (or Richter) magnitude
K_o	=	coefficient of lateral earth pressure at rest
MM	=	Modified Mercalli scale
M_o	=	seismic moment
M_s	=	surface wave magnitude
M_w	=	moment magnitude
N	=	field SPT blow count
N^{60}	=	SPT blow count corrected to an energy efficiency ratio of 60%
N_1^{60}	=	N^{60} corrected to SPT blow count at 100 kPa effective overburden stress
P_a	=	atmospheric pressure (101.3 kPa)
q_c	=	cone penetration resistance
$(q_c)_{crit}$	=	critical cone penetration resistance
$(q_{c1})_{crit}$	=	critical cone penetration resistance 100 kPa effective overburden stress
r	=	slope distance to source of rupture
r_d	=	stress reduction factor to take into flexibility of overlying soil
R	=	q_c / N^{60} (Mexican Method)
R_f	=	slope distance to fault
SPT	=	standard penetration test
σ_d	=	deviator stress
σ_o	=	total overburden pressure
σ'_o	=	effective overburden pressure
σ_{vo}	=	total vertical overburden pressure
σ'_{vo}	=	effective vertical overburden pressure
σ_1	=	major principal stress
σ_3	=	minor principal stress
σ'_{3c}	=	initial effective confining stress
τ_{av}	=	average peak shear stress
τ_m	=	average peak shear stress
U_c	=	uniformity coefficient
V_s	=	shear wave velocity
V_{s1}	=	normalised shear wave velocity
z	=	depth below ground surface
Δu	=	excess pore pressure
γ	=	shear strain

CONTENTS

EXECUTIVE SUMMARY

This document is intended as a guide for practitioners assessing sites for liquefaction potential in earthquake-prone areas in New Zealand.

Findings are based on a three year research project (1993-1996) that investigated 14 sites located at Buller, Foxton, Gisborne, Napier, Petone and Whakatane. Each site was associated with one of six earthquakes occurring between 1931 and 1993 (1931 Napier, 1934 Pahiatua, 1942 Masterton, 1968 Inangahua, 1987 Edgecumbe, or 1993 Ormond earthquakes). Details of the sites and earthquakes are provided in Appendix 1. Soil behaviour during each earthquake was established at each site based on historic information, and then various liquefaction assessment methods were applied to evaluate their effectiveness in predicting liquefaction.

An initial selection of 13 liquefaction assessment methods was reduced to 9 as a result of investigation problems associated with some of the assessment methods, and some methods being discarded in favour of similar but more accurate methods. Details of the evaluation process are contained in Appendix 2. The six methods finally recommended for use in liquefaction assessment are as follows:

Assessment method	Method type	Comments
Sugawara (1989)	CPT	Generally reliable.
Mexican (1991)	CPT	Generally reliable.
Ambraseys (1988)	SPT	Generally reliable.
Tokimatsu & Uchida (1990)	Shear Wave	Less reliable than CPT and SPT methods but field data lacking. Reliability may improve with more data.
Robertson (1990)	Shear Wave	Less reliable than CPT and SPT methods but field data lacking. Reliability may improve with more data.
Particle Size Distribution Criteria	Laboratory	Reliable indicator, however it is insensitive to earthquake magnitude.

The research results show that no single assessment method can accurately predict liquefaction potential for all site conditions and all earthquake characteristics. Consequently soil liquefaction evaluations should be carried out using at least two of the recommended assessment methods and final judgement should be based on the most conservative result. The application of the recommended assessment methods should also be limited to sites at short epicentral distances (up to 50 km from an epicentre), because insufficient information is available to determine whether the results apply equally to sites at greater epicentral distances. In fact the indications are that this is not likely to be the case.

ABSTRACT

This document is intended as a guide for practitioners in the assessment of site liquefaction potential. It is based on a three year research project (1993-1996) and recommends six liquefaction assessment methods which have given the best correlation with known soil behaviour under earthquake loading based on historic information from 14 sites in New Zealand. It also discusses the development of liquefaction prediction models and their advantages and limitations, as well as the application of the recommended liquefaction assessment methods, earthquake and site factors affecting liquefaction, and site investigation methods. Appendices detail the sites and provide information on which the assessments are based.

ACKNOWLEDGMENTS

Andrew Dodds (Opus International Consultants Limited, Central Laboratories, Lower Hutt) revised the report, and compiled Appendices 1 and 2 from the original full reports.

1. INTRODUCTION

In 1992,Transit New Zealand[1] commissioned Works Consultancy Services, Central Laboratories (WCL[2]), Lower Hutt, to carry out research to evaluate the usefulness of methods available to assess soil liquefaction probability.

For the evaluation, WCL carried out field and laboratory investigations at a total of twelve (12) sites located in Whakatane, Gisborne, Napier, Foxton and Petone, and the Department of Civil Engineering at the University of Canterbury provided site information for an additional two (2) sites in the Buller Gorge. The WCL investigations comprised geotechnical and geophysical testing including cone penetration test (CPT) and piezocone, hand-cored boreholes, shear wave velocity measurements, nuclear soil density measurements, sampling using a Van Staay sampler, and laboratory testing to determine gradings and relative densities. The Buller Gorge site investigation included testing with the Parez piezocone, electric and mechanical CPTs, shear wave velocity testing, and sampling using hand augering. Laboratory tests were also performed to determine the grading of recovered samples.

Seismicity and geology of the 12 sites were analysed by the Institute of Geological and Nuclear Sciences Ltd (IGNS), Lower Hutt, New Zealand.

Each of the locations was significantly affected by one of six earthquakes occurring between 1931 and 1993 (1931 Napier, 1934 Pahiatua, 1942 Masterton, 1968 Inangahua, 1987 Edgecumbe, 1993 Ormond earthquakes). Eight (8) sites were chosen where evidence of soil liquefaction has been documented, while the remaining four sites, where no liquefaction was observed, provided control data. The site data have been used to assess the performance of thirteen (13) liquefaction assessment methods based on a comparison of resultant predictions with the actual observed performance of the sites. The research results are fully reported in the WCL reports by Waugh et al. (1992), Smits et al. (1993) and Cheung et al. (1994).

The current report serves to document the main findings of the research undertaken, and provide guidance for practitioners undertaking liquefaction assessment of sites in New Zealand. The report comprises a main section and appendices. The main section contains an overall synopsis of liquefaction and the recommended assessment methods, while the appendices provide further background information as summarised from the WCL reports. Full sets of data are held on files at Opus International Consultants, Central Laboratories, Lower Hutt, and at Transfund New Zealand, Wellington.

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1 Transfund New Zealand has, from 1996, responsibility for funding road research in New Zealand.

2 Now Opus International Consultants, Central Laboratories (from 1997).

2. OVERVIEW OF LIQUEFACTION

2.1 Definition of Liquefaction

In 1985, the United States of America National Research Council (US NRC) published a review of the state of knowledge on the causes and effects of liquefaction of soils during earthquakes (NRC 1985). This report has subsequently been widely used and has become an important document in general geotechnical practice for earthquake resistant design.

In the 1985 NRC review, liquefaction is defined as follows:

> *During earthquakes the shaking of ground may cause a loss of strength or stiffness that results in the settlement of buildings, landslides, the failure of earth dams, or other hazards. The process leading to such loss of strength or stiffness is called soil liquefaction. It is a phenomenon associated primarily, but not exclusively, with saturated cohesionless soils.*

Small volcano-like features on the ground surface are commonly observed as a manifestation of liquefaction and are referred to as "sand boils". During an earthquake, shaking generates excess pore water pressure within the soil. Subsequent ground rupture may occur and allow water to carry liquefied soil to the surface where it is ejected. If the ground-water level is low or if a thick cohesive layer is present above the liquefied zone, then surface manifestations of liquefaction may not be observed.

2.2 Development

The 1964 Niigata earthquake in Japan caused extensive foundation failures. It is one of the best known cases where liquefaction has caused such damage. Since that earthquake, intensive research has been directed toward the prediction of liquefaction of soils. Between the mid 1960s and 1980s significant development of procedures for assessing liquefaction susceptibility was achieved in many earthquake-prone countries (Peck 1979).

In 1985, the Committee on Earthquake Engineering of the US NRC consolidated the state of knowledge on the causes and effects of liquefaction of soils during earthquakes (NRC 1985). The committee also documented state-of-the-art techniques for analysis of safety with respect to liquefaction. The information has been widely adopted by practitioners for liquefaction design.

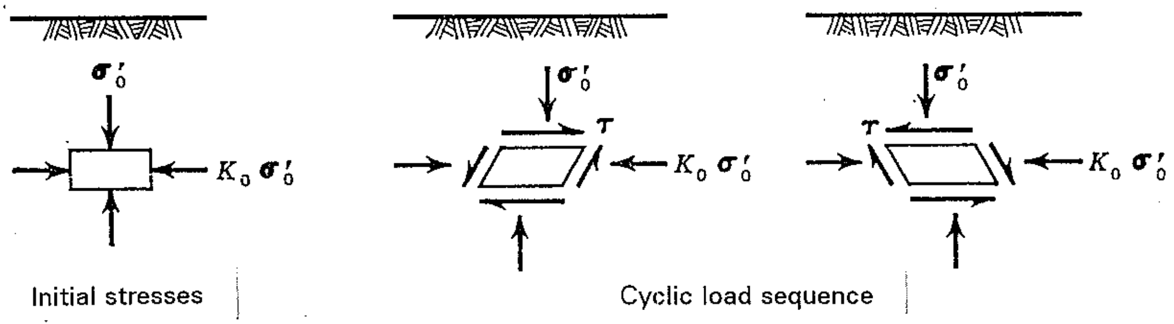
Since 1985, significant advances in the development of methods for assessing liquefaction probability have been made. For saturated sand deposits two basic approaches for assessing liquefaction susceptibility can be applied. The empirical approach is based on correlations between the results of field tests, such as standard penetration test (SPT), etc., and the observed occurrence or non-occurrence of ground failure at sites subjected to earthquakes in the past. The alternative is the analytical approach, usually based on the results of laboratory tests on high quality "undisturbed" samples.

2.3 Empirical Approach

The empirical approach is commonly employed in New Zealand to evaluate liquefaction probability at the site investigation stage. In this approach many of the correlations relate site parameters, such as SPT, CPT and shear-wave velocity profile, to earthquake-induced ground acceleration and/or shear stress (Seed et al. 1984, Robertson et al. 1992, Bierschwale & Stokoe 1984). A significant portion of the historical database being used for development of these empirical correlations is based on SPT information.

In the assessments the earthquake is generally assumed to consist of only vertically propagating shear waves. During the earthquake, soil elements are deformed by the induced cyclic horizontal shear stress (τ) which reverses direction a number of times and produces a cyclic loading effect as illustrated in Figure 2.1.

Figure 2.1 Idealised field loading conditions under earthquake.



The magnitude of the shear stress varies from one cycle to another. The loading on the (assumed) saturated soil is essentially occurring under undrained conditions and may lead to liquefaction of the soil. At any point in a soil deposit, the irregular stress sequence induced by the earthquake is assumed to be equivalent to a series of uniform cyclic shear stresses (τ_{av}). In terms of the cyclic stress ratio (CSR) this is given by:

$$CSR = \frac{\tau_{av}}{\sigma'_{vo}} = 0.65 \left(\frac{a_{max}}{g}\right) \left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right) r_d \qquad (1)$$

where: τ_{av} = average peak shear stress
a_{max} = peak horizontal ground acceleration
g = gravity
σ_{vo} = total overburden pressure
σ'_{vo} = effective overburden pressure
r_d = stress reduction factor to allow for flexibility derived of the overlying soil.

Figure 2.2 shows one of the early data plots for evaluation of liquefaction susceptibility of sites based on SPT data, that was initially developed at the University of California, Berkeley, by Seed et al. (1984). Modification to the measured SPT values has been made to account for differences in the effective overburden pressure of the soil at the testing depth, and variations in the SPT equipment used in different countries, so that direct comparisons of the data are possible.

In Figure 2.2, the shear stress within the soil subjected to earthquakes with local magnitude of 7.5 is estimated from Equation 1 and is plotted against the SPT penetration resistance of the soil. An artificial boundary line can be drawn (as shown on the figure) which approximately separates liquefiable and non-liquefiable deposits. Many other similar charts based on CPT or shear-wave velocity data have also been developed to assist identification of liquefaction-susceptible soils.

Based on analyses of statistical studies on earthquake records, Seed et al. (1975) recommended that the number of significant stress cycles representative of earthquakes of various magnitudes (local or Richter scales), at the average peak shear stress τ_{av}, can be satisfactorily represented by the values shown in Table 2.1.

Figure 2.2 Correlation between normalised SPT blow counts (N_1^{60}) in sands and Cyclic Stress Ratio (CSR) if M_L=7.5 (Seed et al. 1984).

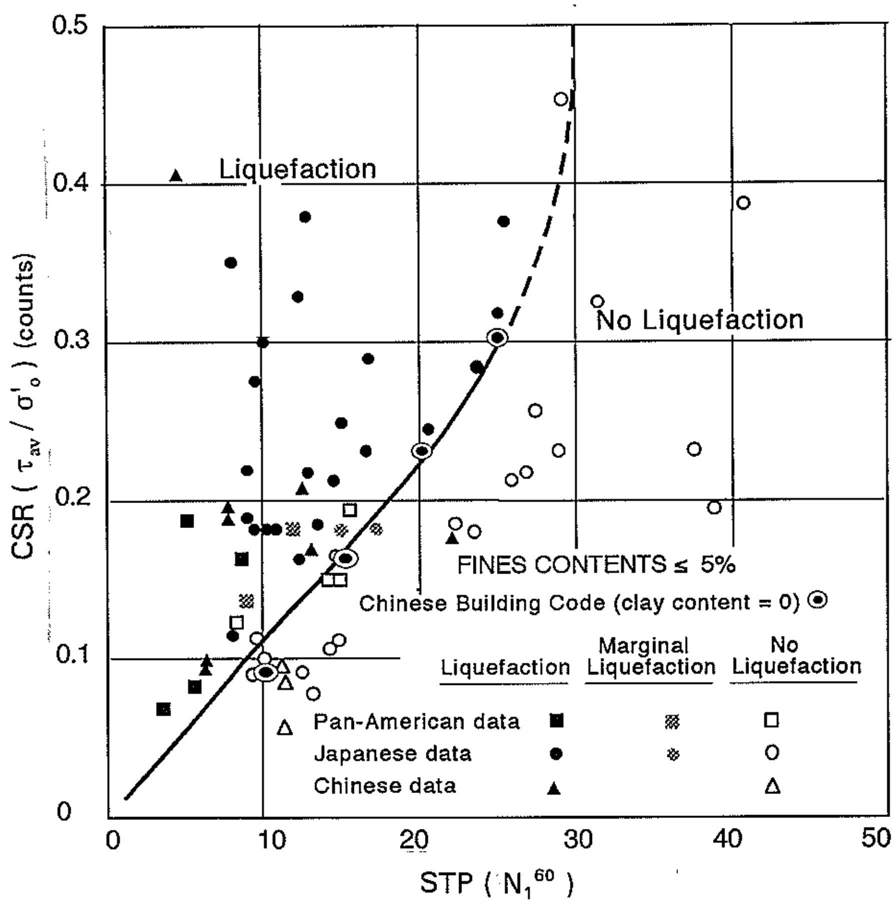


Table 2.1 Relationship between earthquake magnitude and number of representative stress cycles (extracted from NRC 1985).

Earthquake magnitude M_L (local magnitude)	Number of representative stress cycles at τ_{av}	Correction factor to Abscissa curve of M_L=7.5
8.5	26	0.89
7.5	15	1.0
6.75	10	1.13
6.0	5-6	1.32
5.25	2-3	1.5

2.4 Analytical Approach

In the analytical approach, "undisturbed" sand samples are required from the site for laboratory testing (Castro & Poulos 1977, Seed & Lee 1966, Lee & Seed 1967). Testing can be undertaken by means of cyclic triaxial tests, cyclic direct simple shear tests and cyclic torsional shear tests, but the cyclic triaxial test is the most common test. Seed & Idriss (1971) proposed using the undrained "cyclic triaxial compression testing" technique to determine the liquefaction susceptibility of sands. The test is performed by applying a cyclic deviator stress to a sample and observing the number of stress cycles required to cause initial liquefaction within the soil.

In this case the cyclic stress ratio (CSR) is defined by:

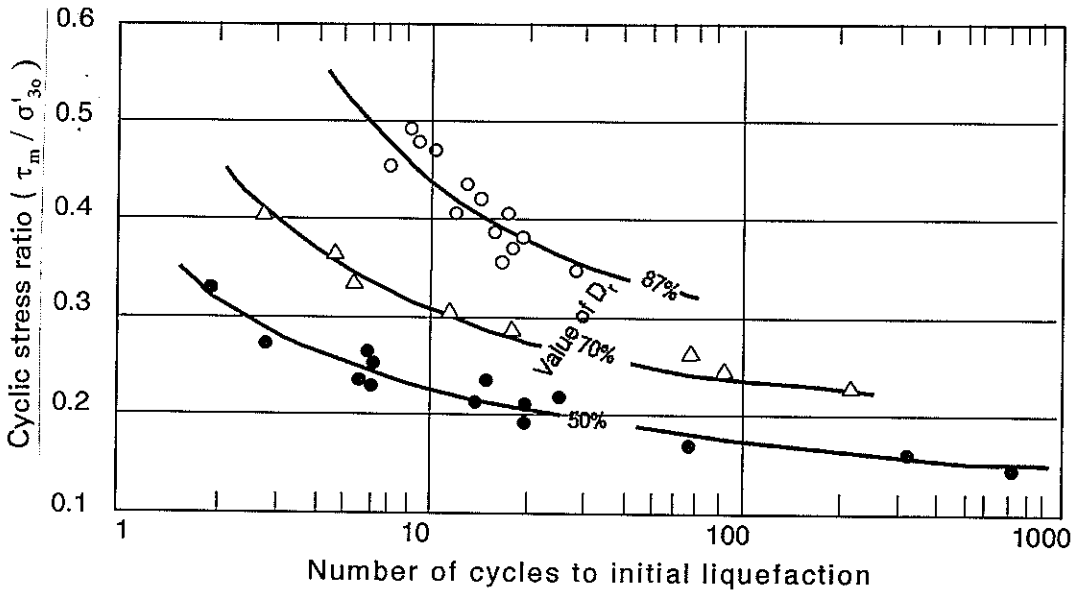
$$CSR = \frac{\sigma_d}{2\sigma'_{3c}} = \frac{(\sigma_1-\sigma_3)}{2\sigma'_{3c}} \qquad (2)$$

where: σ_d = deviator stress
σ_1 = vertical total stress
σ_3 = total confining stress
σ'_{3c} = initial effective confining stress

In most cyclic triaxial testing, the deviator stress is applied while the total confining stress is maintained at a constant level. This results in cycling of the vertical stress applied at the top of the soil sample. Depending on the appropriate design criteria, failure can be defined as either when the pore water pressure within the soil rises to the confining stress (initial liquefaction), or when the axial strain reaches a pre-determined value such as 5%, 10% or 20%.

Figure 2.3 shows a typical plot of data from a series of cyclic triaxial compression tests carried out on saturated sands. The graph illustrates that liquefaction resistance of sands, represented by the cyclic stress ratio, reduces when subject to an increasing number of stress cycles.

Figure 2.3 Typical cyclic triaxial compression test results.



2.5 Advantages and Limitations

The empirical approach is generally employed during the early stages of a site investigation, while the analytical approach is usually applied only when field evidence demonstrates that further evaluation is required. Users of either the empirical or the analytical approaches should be aware that both approaches have limitations, the most important of which are outlined below.

2.5.1 Empirical Approach

Many of the existing empirical correlations (such as that shown in Figure 2.2) which relate site parameters, earthquake-induced shear stresses, and the occurrence or non-occurrence of liquefaction (based on observed ground failure), have promise as useful tools in engineering practice. However, there are significant limitations with both the quantity and quality of the data on which the existing correlations are based. Quality of early SPT data is questionable because of inconsistent SPT practices carried out at the time and lack of test details to account for this. Unfortunately a large number of the empirical correlations are based on this SPT data.

Advantages of the empirical approach are:

- The required field investigation equipment, such as drilling and CPT rigs, are usually employed in routine geotechnical investigations anyway.
- The testing and analysis is relatively inexpensive.
- The empirical approach is widely accepted in the engineering community.

Limitations of the empirical approach are:

- A significant number of observations fall on the opposite side of the artificial boundary separating the liquefiable and non-liquefiable deposits. The inconsistency arises from statistical scatter caused by variations in local soil conditions and measuring techniques and errors in the observational data.
- The peak ground acceleration is commonly used as the key parameter in many correlations. However, this parameter is normally not measured from the actual investigation sites, but only estimated from earthquake records at nearby sites or from published magnitude-distance-acceleration correlations. This increases the scattering of the data.
- The effects of soil stratification, water level and soil type, which may affect the correlations, cannot always be adequately evaluated.
- Correlations are based on a database that has an unknown degree of consistency, because it has been compiled using different investigative tools and testing procedures as adopted in different countries.

2.5.2 Analytical Approach

The analytical approach is usually employed only when detailed analysis of soil behaviour under simulated earthquake-loading conditions is deemed necessary. The cyclic triaxial compression test is a relatively complicated and expensive test and can only be undertaken in a few laboratories in New Zealand.

Advantages of the analytical approach are:

- The analytical methods can be applied for materials such as slightly cemented sands or pumice sands, where empirical relationships have not been established.
- The dynamic material behaviour can be determined from the laboratory testing.
- With the aid of shear stress estimated from earthquake records or computational techniques, simulated shear stress conditions can be incorporated into the laboratory testing programme so that the factor of safety of the soil against liquefaction can be determined.

Limitations of the analytical approach are:

- The analytical procedures require a high level of specialist input.
- Undisturbed sampling of loose sands for triaxial testing is extremely difficult using conventional sampling techniques, and denser and stiffer materials, that are less likely to liquefy, are most successfully recovered.
- Even small changes in the sample volume have a significant effect on both the soil strength and behaviour. Since sand is a density-sensitive material, advancement of the sampler during the sampling process may cause slight compression of the material entering the tube and consequently affect the subsequent test results.

- Elaborate and expensive sampling techniques, such as ground freezing and chemical impregnation, have been employed to minimise sample disturbance. However, the resulting error from the disturbance still requires corrections to be applied to the results.
- Loose sand, which is highly susceptible to liquefaction, tends to be easily disturbed during sample preparation for the cyclic tests.
- Stress conditions in the laboratory tests are only an approximation of the actual earthquake-stress conditions occurring in the field.
- Interpretation of the test results requires specialist input to reasonably assess the cyclic stress strain behaviour of the material.

2.6 Cyclic Stress Ratio Variance

The cyclic stress ratio of the analytical approach (Equation 2), which applies for the cyclic triaxial test conditions, has been related to the case of the simple shear condition assumed in the empirical approach (Equation 1), which is an idealisation of the multi-directional, in-situ stress conditions that occur during an earthquake. Laboratory testing, utilising both triaxial and large scale simple shear testing, has demonstrated that the cyclic stress ratio assuming a simple shear condition is generally less than the cyclic stress ratio derived from triaxial testing by a factor of 0.6 to 0.9. The range of values is attributed to the dissimilar shearing modes in the simple shear test compared with the triaxial test (De Alba et al. 1975), the consolidation stress history (Seed 1976), and the selection of the failure criterion.

Multi-directional stresses occuring during an earthquake, as opposed to unidirectional stresses in the laboratory tests, cause further disparity between the cyclic stress ratio for field and laboratory conditions. Correction factors are therefore applied to laboratory test results in order to approximate more closely the conditions occurring in the field.

2.7 Report Approach

This report addresses the empirical approach only, using New Zealand sites as a means of testing the effectiveness of the various empirical methods proposed.

3. FACTORS AFFECTING LIQUEFACTION ASSESSMENT

In addition to the limitations specific to each of the approaches, as described in Section 2 of this report, other factors affect the assessment of liquefaction. They may be categorised as:

- seismological factors,
- geological factors,
- site characteristics, and
- geotechnical factors.

These factors are discussed in Sections 3.1 to 3.4 below.

3.1 Seismological Factors

Seismological factors that influence the probability of liquefaction potential include:

- earthquake magnitude,
- site to source distance,
- peak ground acceleration, and
- duration of ground motion.

3.1.1 Earthquake Magnitude

The earthquake magnitude is one of the primary factors influencing the acceleration and shear stress experienced during an earthquake.

Because several measures are used to describe the magnitude of earthquakes, it is important to use the earthquake magnitude appropriate to the particular liquefaction assessment model that is being applied. The three expressions of earthquake magnitude which are most commonly used are:

- moment magnitude, M_W
- local magnitude, M_L
- surface wave magnitude, M_S

Various forms of correlations between the three magnitudes have been proposed in the literature, though information most relevant to New Zealand conditions can be found in Dowrick (1991). The principles of these measures of earthquake magnitude are outlined in the following sections.

Moment Magnitude (M_W)

The moment magnitude is expressed in terms of the energy released from an earthquake. It is considered to be a good representation of earthquake size for many civil engineering applications, especially for extremely large events (Kanamori 1978, Ambraseys 1988).

For earthquakes with smaller fault ruptures, the moment magnitude M_W generally compares well with the local magnitude, M_L, and the surface wave magnitude, M_S.

The moment magnitude is defined in terms of the seismic moment (M_0) by:

$$M_w = \frac{2}{3} \log M_0 - 6.0 \tag{3}$$

The seismic moment (M_0) is expressed in dyne-cm and its calculation is based on the elastic dislocation theory. It relates to the rigidity of the ruptured material, the surface area and the displacement of the fault. This parameter may be derived from seismic records or field observations.

Local Magnitude (M_L)

Many liquefaction assessment models are developed based on earthquake data expressed in terms of the local magnitude M_L which, in civil engineering, is often referred to as the Richter magnitude.

Local magnitude is determined from seismic records which are extrapolated or interpolated to distances of about 1000 km. The validity of the method relies on determination of high frequency radiation of the earthquake waves. Using M_L has disadvantages because recording instruments may be overloaded in large earthquake events, and interpretation of the records is difficult when the source size becomes comparable with station distance (Ambraseys 1988). Also, for magnitudes less than about 6, M_L tends to be a less reliable measure of energy release than the surface wave magnitude M_S.

Surface Wave Magnitude (M_S)

For engineering applications in New Zealand the surface wave magnitude M_S has gradually become popular in describing the size of earthquakes. It is one of the most easily and reliably determined expressions of magnitude, especially for historic events where the local magnitude cannot be determined because the seismic records are lacking (Dowrick 1991).

Estimation of surface wave magnitude is based on observations at large distances compared to the size of the source, and on the measurement of energy from the entire earthquake source but at low frequencies.

For very large earthquakes (M_S>8), interpretation of surface wave magnitude from seismic records is difficult because of saturation, though this does not occur as early as it does for measures of other magnitudes such as the local magnitude. For shallow New Zealand earthquakes with M_S<6, the surface wave magnitude, M_S, has been proposed (for example by Dowrick 1991),as a better measure of the size in terms of the seismic moment than local magnitude, M_L.

3.1.2 Site to Source Distance

The distance from a site to the earthquake source is an important factor affecting site performance under earthquake loading. Throughout this report (except for the Ambraseys method), the distance from the liquefaction sites to the seismic energy source or nearest fault rupture is defined as the horizontal distance from the site to the nearest boundary of the zone of seismic energy release, projected to the ground surface. The Ambraseys method utilises the slope distance.

3.1.3 Peak Ground Acceleration

In many simplified liquefaction assessment models, the peak horizontal ground acceleration (a_{max}) is assumed to be the dominant ground motion factor influencing liquefaction, and the effects of vertical ground motion are ignored.

At locations where ground motion accelerometers are installed, the peak horizontal ground accelerations can be determined directly from the seismic records. Unfortunately, seismic records are rarely available for the actual site undergoing assessment. For sites without any instrumentation, determination of a_{max} relies on extrapolation or interpolation of data recorded at nearby sites or sites with similar conditions. This type of extrapolation should only be carried out by competent engineering seismologists and geologists with consideration of the following factors:

- characteristics and size of the earthquake source,
- site geology at the accelerometer station,
- seismic wave propagation and attenuation characteristics in the region,
- geology of the site undergoing the assessment.

3.1.4 Duration of Ground Motion

Although the duration of an earthquake may have an important influence on the liquefaction of sands, this factor has generally been ignored in the simplified assessment models investigated in this research project.

3.2 Geological Factors

Geological factors include:

- depositional process
- geological history (age and cementation)

3.2.1 Depositional Process

In most cases, liquefiable sand layers were laid down at the site as fluvial deposits, and tend to be under-consolidated to slightly over-consolidated. Knowledge of the stress history of a deposit will therefore provide useful information when assessing the results of any of the liquefaction assessment methods. Laboratory oedometer tests on samples recovered from the field may be useful in determining the stress history of the site. Unfortunately sample disturbance is practically unavoidable for loose sands during the sampling process, and so useful and reliable test results may not always be obtained. As an alternative, oedometer tests may be performed on samples from clay-silt seams, adjacent to the sand layers, to obtain an estimation of the stress history.

3.2.2 Geological History

Liquefaction susceptibility generally decreases with the increase in age of soil deposits. Hence soils of Holocene age are more susceptible than those of Pleistocene age. Cementation, which reduces liquefaction potential, can also occur with age and is an important consideration when using assessment methods. The assessment methods used in this study were evaluated using data from uncemented sand deposits only. Therefore, before applying recommended methods, an assessment of material cementation must be carried out. If the materials undergoing investigation are cemented then the assessment methods should not be applied for liquefaction evaluation.

3.3 Site Characteristics

Site characteristics include:

- ground-water level,
- thickness and extent of liquefiable layers, and
- ground slope.

3.3.1 Ground-water Level

Generally only sands which are saturated and below ground-water level are liquefiable. Therefore the ground-water level should always be recorded as part of the site investigation. The observation period should be sufficiently long to determine any seasonal variation in the ground-water level.

3.3.2 Thickness of Sediment Layers

At sites where the ground-water level is low or where the liquefiable layer is overlain by significantly strong layers, surface expression of liquefaction-induced ground failure, e.g. sand boils, may not occur. However, even in these cases, the assessment methods presented in this report can still be used to identify liquefiable and non-liquefiable layers.

3.3.3 Extent of Liquefiable Sediment

Estimation of both the horizontal and vertical extent of a potentially liquefiable sediment is important during the site investigation. Field observations at sites where liquefiable sediments are thin or are of small extent indicate that soil liquefaction may not necessarily induce noticeable ground damage or significant settlement of structures. However, when liquefiable sediments are of large vertical and horizontal extent, liquefaction may cause significant foundation settlement and structural damage, and cause failure of underground facilities.

3.3.4 Ground Slope

Liquefaction of sands denotes a loss of shear resistance and may cause sliding of adjacent non-liquefiable layers, particularly in the case of liquefiable sand deposits under sloping ground. Earthquake-induced forces assisted by gravity-induced forces may result in significant lateral ground movement. Reports of massive flow-sliding of slopes and earth dams caused by liquefaction have been widely published.

Prediction of this type of large scale earthquake-induced ground displacement is an area of major international research and is outside the scope of this report.

3.4 Geotechnical Factors

The accumulation of experience through observation and testing has enabled the identification of various geotechnical factors to assist and guide in the course of liquefaction assessment. Geotechnical factors include:

- particle size distribution limits,
- fines content, and
- soil strength assessment.

3.4.1 Particle Size Distribution

Based on results of particle size distribution analyses of sands from liquefaction sites, it has been widely accepted that most liquefiable soil lies within the range of fine to medium sand. The permeability of these materials is not usually high enough to dissipate the pore water pressure within the soil during the earthquake, and thus the generation of excess pore pressure leads to liquefaction.

The limits in the particle size distribution curves which have been established to differentiate liquefiable soil from non-liquefiable soil are shown in Figure 3.1. These curves also illustrate that even material with very high fines content may be potentially liquefiable.

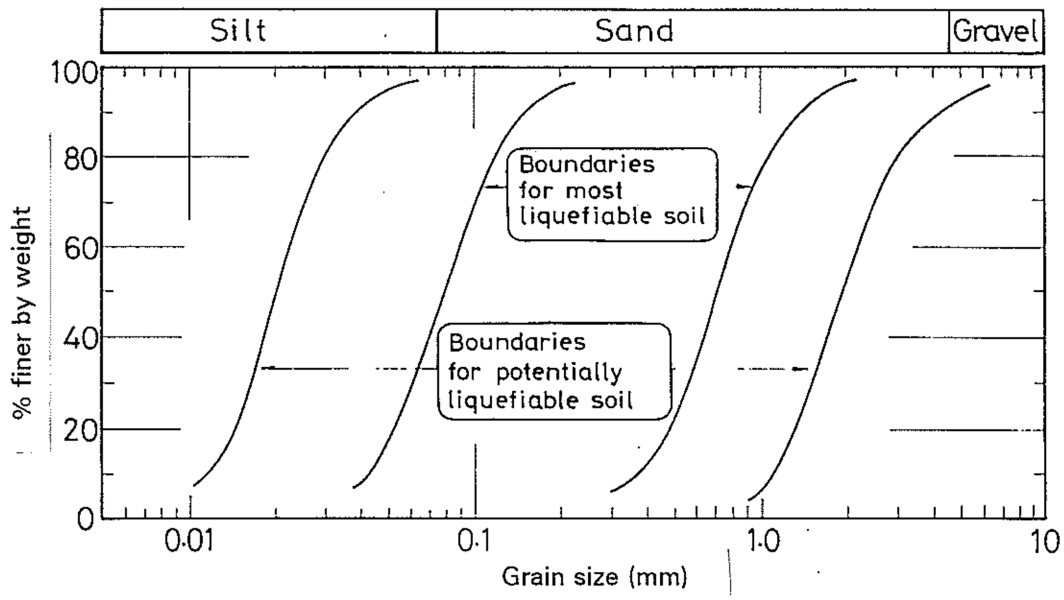
3.4.2 Fines Content

Increase in the fines content generally increases the material cohesiveness and decreases the liquefaction probability of the materials. The fines content ($F_{\%}$), used in the research for this report, is defined as the fraction, by weight, of dry soil passing through a (NZS) sieve of 75 μm screen size.

3.4.3 Soil Strength

Soil strength is a function of effective overburden pressure, cohesiveness and angle of friction, and these are also factors affecting liquefaction resistance. Several of the measures used to characterise soil strength – SPT, CPT penetration resistance and shear wave velocity – have subsequently found extensive use as input parameters for the assessment of liquefaction potential.

Figure 3.1 Particle size distributions of liquefiable and non-liquefiable soils (NRC 1985).



The SPT value, expressed in terms of SPT blow count for 300 mm penetration into the soil within a drillhole, was used early in the 1960s for estimation of liquefaction resistance of materials. However, limitations in the SPT technique, both in terms of equipment and methodology, have influenced the reliability of the results, and the CPT has subsequently become more widely adopted in site investigations for liquefaction assessment. The CPT provides a continuous record of penetration resistance, improving the ability to detect thinner soil layers susceptible to liquefaction. However, it does not provide a sample for inspection as is the case with the SPT.

Field measurement of the shear wave velocity of soils may be carried out using a variety of techniques, and these are discussed in Section 4.5. It is to be noted that measurement of shear wave velocity alone may not be sufficient for liquefaction assessment purposes. Jamiolkowski & Lo Presti (1992) reported the lack of sensitivity of shear wave velocity to stress and strain history but a strong influence of stress and strain history on liquefaction resistance. Age of soil and soil type also appear to affect shear wave velocity to a lesser extent than liquefaction resistance. However, while it is accepted that shear wave velocity measurement is not adequate for the evaluation of all deposits, the exceptions are mainly related to older sediments (>10,000 years), often with complex stress histories. These concerns may not extend to the liquefiable sands in New Zealand, which are commonly alluvial and young deposits.

4. INVESTIGATION METHODS FOR LIQUEFACTION ASSESSMENT

4.1 Augering and Drilling

Hand-augered boreholes are usually sufficient for assessing shallow depth material, but drilling may be required for investigation of liquefaction probability at sites with thick and/or deep soft sediments. Rotary core drilling is usually preferred to obtain continuous soil cores for visual inspection. At depths of particular interest, high quality push-tube samples (such as those obtained using a suction core piston sampler) should also be retrieved for laboratory testing.

Drilling also provides access for in-situ testing, tube sampling, and for installation of piezometers and casings for in situ shearwave velocity and bulk density measurements. Drilling may also be used to create access through shallow material (e.g. gravels) and allow cone testing to be carried out within deeper layers.

4.2 Standard Penetration Test

The Standard Penetration Test (SPT) is one of the most commonly used methods in New Zealand site investigations. It is preferred by many engineering practitioners because the additional cost for performing the test during a routine drilling programme is not significant. The test should be carried out to New Zealand Standard NZS 4402 : 1986, Test 6.5.1 *Determination of the Penetration Resistance of a Soil - Standard Penetration Test (SPT)* (SANZ 1986).

The SPT procedures adopted in different countries may be very different. The assessment methods discussed in this report are mainly based on the SPT database reported by Seed et al. (1984). To establish this database, SPT data from liquefaction sites in many parts of the world were corrected to 60% energy efficiency (compared with the theoretical free-fall energy), based on the safety-hammer SPT device most commonly used in the US. This enables direct comparisons of site performance to be made where the energy efficiency of SPT equipment is known (definitive work on the subject of SPT corrections is that of Skempton, 1986). The database has subsequently been widely used for development of assessment methods.

4.3 Cone Penetration Test

The Cone Penetration Test (CPT) is becoming more favoured as a tool that provides reliable and consistent soil information, and has become more widely adopted in site investigations for liquefaction assessment. The CPT technique comprises pushing a cone with a 60° apex and a projectional area of 1000 mm^2 into the ground at a constant rate of 20 mm/s while measuring the cone penetration resistance and the friction along the cone sleeve.

The test procedure is described in New Zealand Standard NZS 4402 : 1986, Test 6.5.3 *Cone Penetration Resistance Using a Cone or a Friction Cone* (SANZ 1986).

4.4 CPT with Pore Pressure Measurement (Piezocone Testing)

Laboratory experiments have demonstrated that low cone resistance combined with excess pore pressure (measured using a piezocone) is indicative of liquefiable soils (Canou 1988). The use of the piezocone for liquefaction assessment has also been evaluated in the study recorded in this report. Unfortunately, the measured pore water response generally lead to inconclusive results in the assessment, and therefore the method was not further pursued in this investigation. A simple liquefaction assessment using the piezocone test appears, at this stage, to be unreliable because of, for example, complexities in the pore water pressure response and the interaction between the pore water and the soil skeleton.

4.5 Shear Wave Velocity Measurement

Shear wave velocity measurement provides an alternative means of characterising soils for liquefaction purposes, since the shear wave velocity is influenced by many of the variables that influence liquefaction. Several techniques for measuring in-situ shear wave velocity exist, though none are specified in a New Zealand Standard. Both the downhole and crosshole techniques are well established, while the spectral analysis of surface waves (SASW) technique is a relatively recent (past decade) development, becoming established as a routine investigation tool. The downhole technique has been most widely adopted because it can be applied in conjunction with either drilling or cone testing and is less costly than crosshole testing. Details of each technique are outlined in Sections 4.5.1 to 4.5.3 below.

4.5.1 Downhole Technique

Downhole shear wave measurements are made by lowering one or more geophones inside a vertical tube installed in the ground. In the case of a drillhole, a PVC liner may be grouted in place after drilling is completed. Any space between the liner and the ground must be grouted to ensure that adequate coupling between the soil and the geophones is achieved.

Horizontal shear waves are generated at the ground surface and the travel time of the wave to the downhole geophones recorded. The shear wave velocity of each layer can then be calculated either from a plot of travel time versus depth or from the incremental change in travel time between two or more geophones. The time delay method using multiple geophones is favoured since any trigger delay errors in the recording system do not affect the analysis.

The shear wave velocity can also be calculated using digital signal processing to determine the change in phase angle between two geophones.

Alternatively, downhole measurements may be carried out during cone penetration testing using either a geophone located at the tip of the cone rod (seismic cone) or by using the cone rig to push in a special casing designed to house a multiple geophone array. The seismic cone method usually incorporates a single geophone, and consequently the results, which may be affected by triggering delay errors, are likely to be less accurate than those using multiple geophones.

The standard practice is to generate shear waves of reverse polarity and compare the recorded wave forms, which should also show a reversal of polarity if the shear wave has been correctly identified.

4.5.2 Crosshole Technique

Crosshole shear wave measurements are carried out by measuring the transit time of a vertical shear wave travelling in a horizontal plane between a seismic source in one borehole and geophones located at the same level in adjacent holes, typically 3-5 m apart. A total of three holes is desirable to allow the velocity to be based on interval times between two geophones, since the total travel times are small and triggering delay errors may affect data quality. To calculate velocities, a deviation survey is required in each hole to determine the correct travel distance at each depth that the shear wave transit time is measured.

The crosshole technique can offer better layer resolution than the downhole method, especially for deep layers, and may be employed on projects where the additional costs for drilling and for deviation surveys are justified. The crosshole method may also be used if ground conditions prevent the generation of shear waves at the surface (e.g. swamps or offshore investigations), or where high energy loss in the shallow layers hinders investigation of deeper soil.

A standard test method for crosshole testing has been proposed by Ballard et al. (1983).

4.5.3 Spectral Analysis of Surface Waves (SASW) Technique

The spectral analysis of surface waves technique is based on the analysis of surface waves generated by a hammer blow and recorded at two geophones. The velocity of a surface wave with a particular wavelength is a function of the shear wave velocity of the soil to a depth of approximately one third the wavelength. By determining the surface wave velocity over a wide range of wavelengths, a profile of shear wave velocity versus depth is derived. The shear wave profile is determined using a complex inversion procedure, based on an assumed theoretical profile.

Surface wave testing may also be performed using a controlled source usually comprising a mechanical or electromagnetic shaker. These generate surface waves in a narrow frequency band, but may be automatically adjusted to step through a range of frequencies to acquire data equivalent to that obtained using hammers or dropped weights.

The SASW technique offers a relatively economical method of exploration without the need of boreholes. Depth of penetration can be considerable (up to 30-40 m), but is more typically 8-10 m, unless large masses are used to generate very long wavelength surface waves. Layers that are thin relative to their depth have little influence on the surface wave velocity and so may not be detected.

4.6 Soil Testing

4.6.1 Density Measurement

In-situ or laboratory measurement of density is required in order to determine total and effective vertical stresses in the field, which in turn are input parameters for most of the liquefaction assessment methods. Relative density (R_D), comparing in-situ density with laboratory determined minimum and maximum densities, is also a well recognised indicator of liquefaction potential. Unfortunately, for cohesionless materials (which tend to be liquefiable), estimation of soil density using the laboratory technique tends to be inaccurate because of water loss from the soil during sampling. Laboratory determination of soil density is more accurate for cohesive materials which, however, are less likely to liquefy.

An alternative approach is to determine the soil density profile using both laboratory and geophysical methods. The laboratory method is used to measure the soil density from the stiffer soil layers while geophysical (gamma-gamma) logging is carried out over the full depth of the soil from within a drillhole or special casing installed using a cone rig. The geophysical results may be correlated with the laboratory measurements to establish a calibration factor from which to calculate the bulk density profile from the gamma-gamma log.

Laboratory testing may be performed according to NZS 4402 : 1986, Test 5.1.5 *Soil Density Tests - Water Displacement Method* (SANZ 1986).

4.6.2 Particle Size Distribution

Particle size distribution is in itself a useful indicator of potentially liquefiable soils, and is also used to determine fines content for use in other assessment methods. Determination of particle size distribution is recommended to be carried out to NZS 4402 : 1986, Test 2.8.1 *Determination of the Particle Size Distribution - Standard Method by Wet Sieving* (SANZ 1986).

Fines content is usually defined in the assessment methods as the fraction of the particles by weight passing an NZS sieve with a screen aperture of 75 μm.

5. RECOMMENDED ASSESSMENT METHODS

The liquefaction assessment guidelines contained in this report are the product of a three year research project which evaluated a total of 13 liquefaction assessment methods including 4 CPT and 2 SPT methods, 5 shear wave velocity methods, and 2 laboratory methods. Geotechnical and geophysical investigations were carried out at 10 liquefaction and 4 non-liquefaction sites to obtain soil data for input to the assessment methods. The evaluation of the assessment methods was made by comparing the predictions from each method with the observed liquefaction or non-liquefaction of soil at sites known to have been affected by earthquakes in New Zealand. Site and evaluation details are contained in Appendices 1 and 2 respectively.

The recommended methods are:

- Sugawara (1989) - CPT method
- Mexican (Diaz-Rodriguez & Armijo-Palacio 1991) - CPT method
- Ambraseys (1988) - SPT method
- Tokimatsu & Uchida (1990) - Shear wave velocity method
- Robertson (1990) - Shear wave velocity method
- Particle Size Distribution - Laboratory method

A description of each method follows.

5.1 Sugawara CPT Method

The Sugawara (1989) method is based on CPT results and is a variation of the method proposed by Shibata & Teparaksa (1988). The technique is similar to the approach recommended by Seed & Idriss (1981), except that CPT results are used to calibrate the model rather than using SPT data obtained in boreholes.

The basis of the method is that the soil is potentially liquefiable if the measured cone resistance, q_c, is less than the critical cone resistance, $(q_c)_{crit}$, where $(q_c)_{crit}$ is obtained from the following relationship (Meyerhoff 1957):

$$(q_c)_{crit} = (q_{c1})_{crit} \left(\frac{\sigma'_{vo} + 0.07}{0.17} \right) \tag{4}$$

and

$$(q_{c1})_{crit} = c_2 \left(5 + \frac{20\,(CSR_{crit} - 0.1)}{(CSR_{crit} + 0.1)} \right) \tag{5}$$

$$CSR_{crit} = \left(0.1\ (M_L - 1)\ \frac{a_{max}}{g}\ \frac{\sigma_{vo}}{\sigma'_{vo}}\ (1 - 0.015\,z) \right) \tag{6}$$

where: $(q_c)_{crit}$	=	critical cone penetration resistance
$(q_{c1})_{crit}$	=	critical cone penetration resistance at 100 kPa effective overburden stress
CSR_{crit}	=	critical cyclic stress ratio
a_{max}	=	peak horizontal ground acceleration
σ'_{vo}	=	effective overburden pressure
M_L	=	earthquake magnitude in terms of local magnitude scale (Richter magnitude)
c_2	=	constant dependent on the fines content
g	=	gravity
z	=	depth below ground level

The method presents direct correlations between CPT cone resistance and the cyclic stress ratio. The factor c_2 in the model allows for the relationship between the fines content (% silt and clay) and critical cone resistance for equivalent liquefaction probability. This relationship is:

For $F_{\%} \leq 5\%$:

$$c_2 \equiv \frac{(q_{c1})_{crit}}{(q_{c1})_{crit} \text{ for clean sand}} = 1.0 \tag{7}$$

or, for $F_{\%} > 5\%$:

$$c_2 \equiv \frac{(q_{c1})_{crit}}{(q_{c1})_{crit} \text{ for clean sand}} = 1.58 - 0.87\ (\log_{10}F\%) \tag{8}$$

where $F_{\%}$ = % fines.

In many circumstances only CPT testing of the soil profile is undertaken with no direct sampling of the soils to obtain grading curves for the determination of the percentage fines. In these cases the percentage fines can be inferred from the CPT results using one of several established correlations. For instance, Robertson & Campanella (1983) proposed a simple soil classification method deduced from cone resistance and friction ratio (Figure 5.1).

5.2 Mexican CPT Method

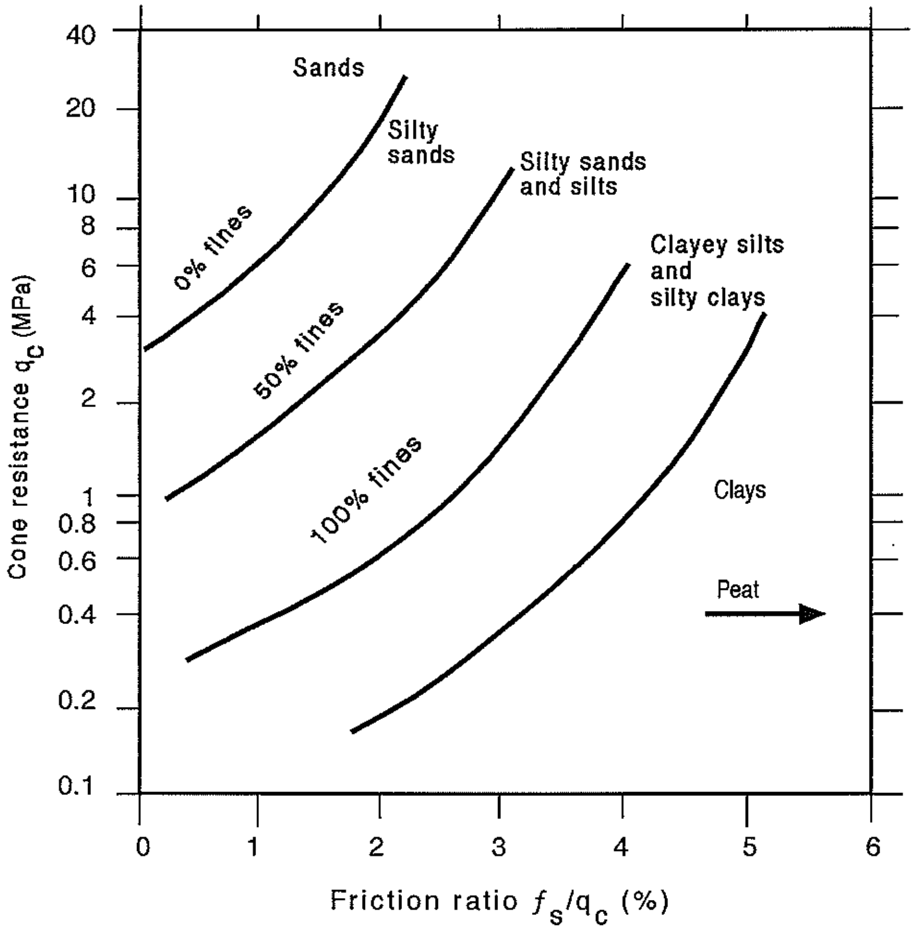
Diaz-Rodriguez & Armijo-Palacio (1991) report on a method of liquefaction assessment using the CPT. This approach makes use of the sleeve friction, f_s, together with the cone resistance, q_c, to identify those soil layers which are likely to liquefy. The liquefaction probability of that soil layer is then determined by comparing the measured q_c values with a critical value $(q_c)_{crit}$ which is based on an empirical correlation.

The criteria for liquefaction are taken to be:

- CPT friction ratio $(f_s / q_c) \leq 2.5\%$
- and q_c / σ'_{vo} to be $\leq (q_c)_{crit} / \sigma'_{vo}$

where $(q_c)_{crit}$ is derived from the Seed et al. method.

Figure 5.1 Soil classification and presumptive percentage fines from electric CPT data (modified from Robertson & Campanella 1983).



Calculation of critical cone penetration resistance $(q_c)_{crit}$:

In the current report and the previous research based on historic data (Waugh et al. 1992, Smits et al. 1993, Cheung et al. 1994), the derivation of $(q_c)_{crit}$ values is based on the approach of Liam Finn et al. (1987). This approach demonstrated that the critical cyclic stress ratio (CSR_{crit}) from Seed's SPT method could be approximated for values of CSR less than 0.4 by the relationship:

$$CSR_{crit} = \frac{N_1^{60}}{12.9M_L - 15.7} \tag{9}$$

where: N_1^{60} = N^{60} corrected to the SPT blow count at 100 kPa effective overburden stress

= $C_N N^{60}$ (10)

and

C_N = overburden correction factor

= $1/(10\sigma'_{vo})^{0.5}$ where σ'_{vo} is in MPa (Laio & Whitman, 1986) (11)

N^{60} = SPT blow count corrected to an energy efficiency ratio of 60%

There is a relationship between N^{60} and q_c which has been found from testing to be dependent on mean grain size (D_{50}), i.e.:

$$q_c = f_n(N^{60}, D_{50}) \tag{12}$$

In Seed & De Alba (1986) it is reported that q_c can be related to N^{60} by the following equation (Figure 5.2):

$$R = \frac{q_c}{N^{60}}, \quad 0.2 \geq R \geq 0.8 \qquad (q_c \text{ in MPa}) \tag{13}$$

Given the expression for the cyclic shear ratio in Equation 1, by combining with Equations 9, 10, and 13, and dividing by the effective overburden pressure, the following expression is obtained:

$$\frac{(q_c)_{crit}}{\sigma'_{vo}} = \left(0.65 \frac{\sigma_{vo}}{\sigma'_{vo}} \frac{a_{max}}{g} r_d \right) \frac{(12.9M - 15.7)\,R}{C_N\,\sigma'_{vo}} \tag{14}$$

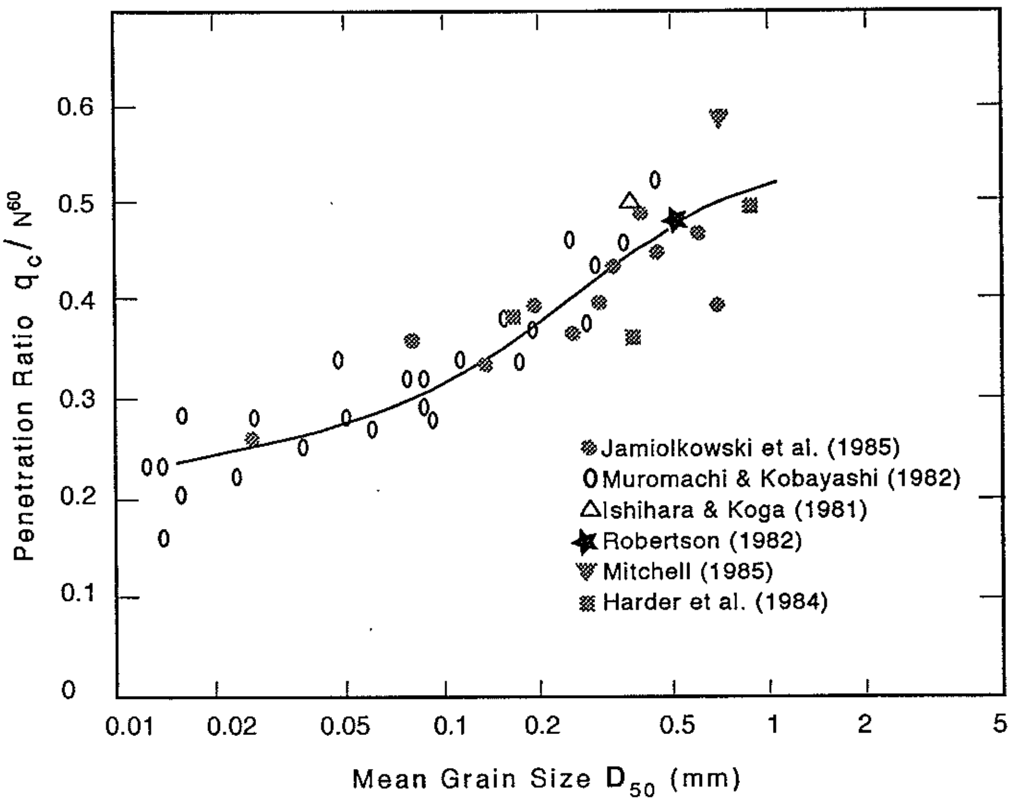
In this approach the stress reduction factor r_d is given by:

$$r_d = 1 - (z^2 / 1486) \tag{15}$$

where: z = depth in m (for $z \leq 30$ m)

For particular earthquake design parameters, a_{max} and M_L, the value of $(q_c)_{crit}$ can then be calculated from Equation 14.

Figure 5.2 Variation of q_c/N^{60} with mean grain size, q_c in MPa, N^{60} = SPT value at 60% energy efficiency (from Seed & De Alba 1986).



5.3 Ambraseys SPT Method

Ambraseys (1988) conducted a detailed review of the base data used for the Seed approach and incorporated further case histories that had come to hand. A total of 137 case histories of earthquakes which were known to have caused ground failures through liquefaction were reviewed. The study also included a number of corrections to previously published data that were found to contain errors.

Ambraseys concluded that, while the curve proposed by Seed et al. (1984) and shown in Figure 2.2 generally agrees with the case history data, the scaling factor for earthquakes other than those with magnitude M = 7.5 was significantly at variance with the case histories.

Ambraseys recommended using the following equations for all earthquake magnitudes and clean sands (fines content ≤5%).

$$CSR_{crit} = 0.4 \exp(0.06N_1^{60})\,(N_1^{60})^{0.755} \exp(-0.525\,M_w) \qquad (16)$$

for ($6.0 \leq M_w \leq 7.5$), and

$$CSR_{crit} = 3.29 \exp(0.06N_1^{60})\,(N_1^{60})^{0.755} \exp(-0.81\,M_w) \qquad (17)$$

for ($M_w \geq 7.5$)

where: M_w = moment magnitude

N_1^{60} = $C_N N^{60}$

in which N^{60} is the SPT blow count corrected to an energy efficiency ratio of 60% and C_N is defined by Equation 11.

A simple chart was prepared by Ambraseys which summarises the work (Figure 5.3).

Input alternatives to utilise the chart are:

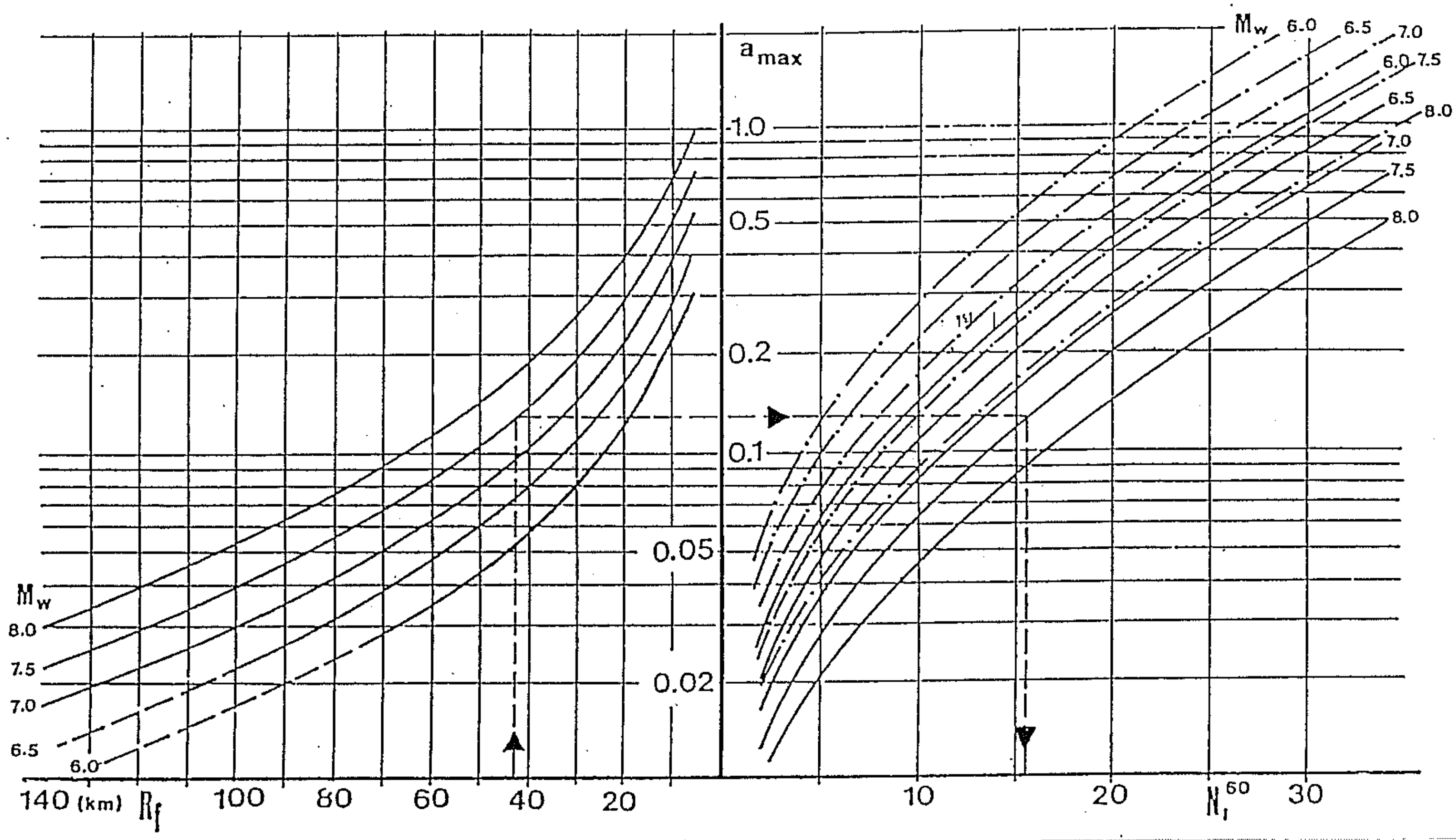
Input M_w and R_f to obtain a_{max}, $(N_1^{60})_{crit}$

or M_w and a_{max} to obtain $(N_1^{60})_{crit}$

where R_f is defined as the slope distance of the liquefaction site from the seismic source.

Figure 5.3 Ambraseys liquefaction assessment model (Ambraseys 1988).

M_w	= moment magnitude	---- - ----	water table at critical depth
R_f	= slope distance to fault	————	water table at ground surface



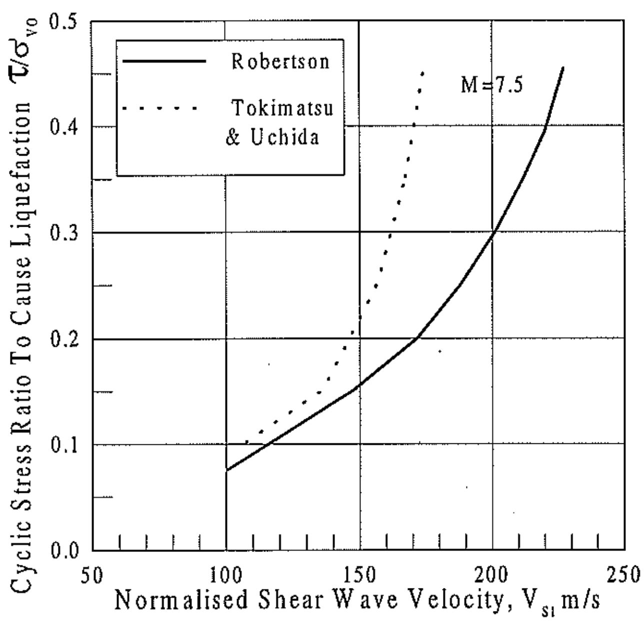
5.4 Tokimatsu & Uchida Shear Wave Velocity Method

Both Tokimatsu & Uchida (1990) and Robertson (1990) independently developed similar liquefaction resistance and shear wave velocity relationships. Both methods adopted Seed's approach to determine the cyclic stress ratio using Equation 1. Tokimatsu & Uchida (1990) derived their results based on laboratory dynamic triaxial testing on isotropically consolidated sand samples. Liquefaction of the tested sample is defined when 5% peak to peak axial strain is reached rather than when the generated excess pore water pressure is equal to the confining pressure. The results of their study are presented in Figure 5.4. The shear wave velocity is related to its normalised value by:

$$V_{s1} = V_s \left(\frac{P_a}{\sigma'_{v0}} \right)^{0.25} \tag{18}$$

where: P_a = atmospheric pressure (101.3 kPa)
V_{s1} = normalised shear wave velocity

Figure 5.4 Relationship between liquefaction resistance and shear wave velocity of sands (Tokimatsu & Uchida 1990, Robertson 1990).



5.5 Robertson Shear Wave Velocity Method

Robertson (1990) established a cyclic stress ratio versus normalised shear wave velocity relationship based on observed liquefaction in the Imperial Valley, and some limited data from other sites (Figure 5.4).

The Robertson and the Tokimatsu & Uchida methods predict very similar results at cyclic stress ratios of less than 0.2, while the methods differ for stress ratios larger than 0.2. At this stage it is uncertain which of these two methods is more reliable because field data are lacking. Further evaluation of the two methods will be required when new field data become available in the future.

5.6 Particle Size Distribution Method

Particle size distribution of soil is one of the most commonly employed methods to determine liquefaction probability for cohesionless soils. For liquefaction, the criteria (NRC 1985) typically require that:

- the percentage of silt and clay size particles is less than 10%,
- the uniformity coefficient $Uc = D_{60} / D_{10}$ is less than 6,
- the 20% passing size D_{20} satisfies $0.04 < D_{20} < 0.5$ mm.

5.7 Use of the Recommended Methods

The above methods have been recommended based on comparison of predictions with observed behaviour for selected sites in New Zealand. The recommendations are therefore empirically based and are subject to limitations. The following discussion provides some comments on this aspect for use in applying each method. As no single assessment method is reliable in every case, a combination of at least two or more of the methods should be used when undertaking a site evaluation. The site evaluation should then be based on the more conservative results of the selected assessment methods.

The recommended CPT methods and SPT method are relatively reliable for prediction of liquefaction probability of New Zealand sites.

The recommended shear wave velocity methods are generally less reliable than the CPT and SPT methods, possibly because the evaluated models are founded upon relatively small amount of field data. Tokimatsu & Uchida's method is based on correlation between the results of laboratory cyclic triaxial compression tests and the shear wave velocity of sands, whereas Robertson's method is founded on field measurement of shear wave velocity and estimated earthquake-induced shear stress. The methods give similar results but accuracy of the two methods should be re-evaluated in the future as more field data become available.

The recommended particle size distribution method is well founded and can identify whether the soil is liquefiable or not. However, it cannot provide an estimate of the probable earthquake magnitude at which liquefaction may occur.

All the assessment methods have shown a good degree of consistency in their ability to predict liquefaction performance for earthquakes at short (<50 km) distances. However, the applicability of the assessment methods for greater distances is uncertain due to inconsistencies noted with the predictions of the methods at these distances. Smits et al. (1993) have cited uncertainties in calculated accelerations or limitations of the assessment methods as possible reasons for the inconsistencies. However, insufficient information is available to confirm either.

Table 5.1 provides a summary of the recommended methods.

Table 5.1 Summary of recommended methods.

Assessment method	Method type	Comments
Sugawara (1989)	CPT	Generally reliable.
Mexican (1991)	CPT	Generally reliable.
Ambraseys (1988)	SPT	Generally reliable.
Tokimatsu & Uchida (1990)	Shear Wave	Less reliable than CPT and SPT methods but field data lacking. Reliability may improve with more data.
Robertson (1990)	Shear Wave	Less reliable than CPT and SPT methods but field data lacking. Reliability may improve with more data.
Particle Size Distribution Criteria	Laboratory	Reliable indicator, however it is insensitive to earthquake magnitude.

APPENDIX 1
SITE AND SEISMOLOGICAL INFORMATION

A1.1 INTRODUCTION

The liquefaction assessment methods recommended in Chapter 5 of this report are the result of a three-year research project that concentrated on 6 areas in New Zealand where sites were identified as having experienced liquefaction during past earthquake events. Each site was associated with one of six earthquakes occurring between 1931 and 1993. These sites were used to evaluate the effectiveness of liquefaction assessment methods that are available to practitioners. This appendix provides: (a) summarised details of the site locations and investigation details, and (b) seismological information pertaining to each earthquake and the relevant site affected. The reader is referred to WCL reports 92-3220 (Waugh et al. 1992), 93-3220 (Smits et al. 1993) and 94-3220 (Cheung et al. 1994) for full site details and investigation results.

A1.2 DESCRIPTION OF SITES

A1.2.1 Site Selection

The correct identification of sites where liquefaction has occurred is difficult. Field observations may classify an area as non-liquefied because surface manifestations, such as sand boils, lateral spreading or dynamic consolidation, may not be obvious, although at depth soils may have liquefied. Furthermore, the phenomenon of liquefaction was not recognised until the mid-1960s. Consequently, liquefaction was not identified separately in the early descriptions of historic surface damage recorded during earthquakes.

To identify the test sites used in this research project, a comprehensive study was made of available information in libraries, museums, council records, as well as interviews with people who had experienced the earthquake or witnessed the results. The reader is referred to the WCL Reports for these details. Table A1.1 lists the selected earthquakes and sites, and Figure A1.1 indicates site locations. The following abbreviations are used to identify each site:

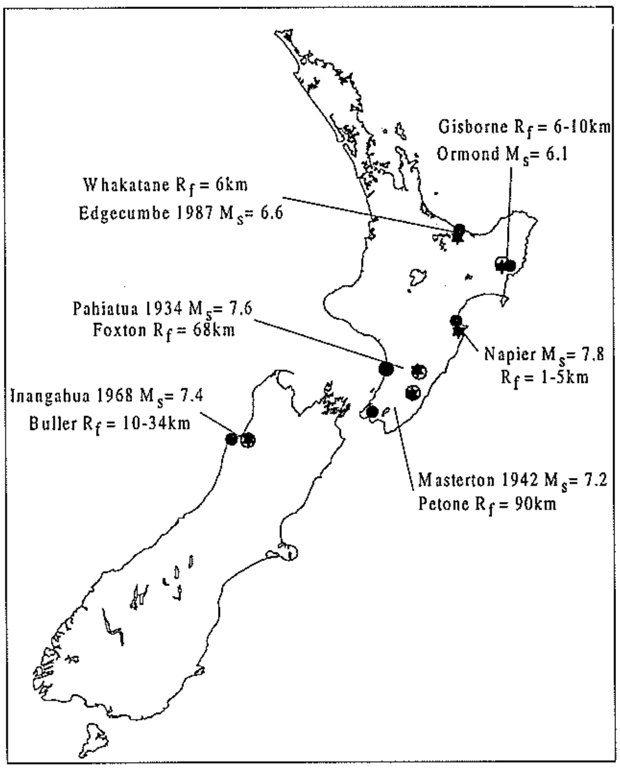
W = Edgecumbe (Whakatane) sites	G = Gisborne sites
N = Napier sites	B = Buller sites
F = Foxton sites	L = Liquefied site
P = Petone sites	C = Control site, i.e. not liquefied

Example: A site identified as NL1 means that it is liquefied site number one (L1) at Napier (N).

Table A1.1 Earthquakes (in chronological order) and locations identified as liquefaction and control sites for the evaluation study.

Date	Earthquake	Location	Liquefied Site	Control Site
1931	Napier	Napier	Nelson Park (NL1) Embankment Road (NL2)	Todd Street (NC1)
1934	Pahiatua	Foxton	Foxton Beach (FL1)	–
1942	Masterton	Petone	Petone Pipe Bridge (PL1)	Heretaunga Street (PC1)
1968	Inangahua	Buller Gorge/ Westport	Kilkenny Park (BL1) Three Channel Flat (BL2)	–
1987	Edgecumbe	Edgecumbe/ Whakatane	Tarawera R. bridge (WL1) Whakatane R. bridge (WL2)	Edgecumbe Substation (WC1)
1993	Ormond	Gisborne	Te Karaka (GL1) Humphrey Road (GL2)	Caesar Road (GC1)

Figure A1.1 Test locations and epicentres of the associated earthquakes that caused liquefaction in New Zealand over the period 1931 to 1993.



A1.3 SITE INVESTIGATIONS

A1.3.1 Testing Chronology

The WCL site investigations commenced in 1989 with an initial investigation and assessment of the Edgecumbe earthquake sites undertaken by Jennings & Smith (1991). Testing continued between 1992 and 1993 with investigation of the Napier, Pahiatua, Masterton, Ormond and Inangahua earthquake sites, in that order. The Inangahua earthquake sites required WCL to carry out only SASW testing, as the University of Canterbury had already carried out other site investigations that had commenced in 1986. A re-visit to the Edgecumbe sites in 1993 was required to carry out SASW testing also, as the Jennings & Smith (1991) investigation had not included this test.

A1.3.2 Test Methods

The WCL testing regime for each site consisted of CPT probes using a 20 tonne rig and Delft piezo-electric cone unit, then casing of the hole for downhole in-situ shear wave and nuclear density measurements. A combination of hand-augered (above water table) and Van Staay Sampler (below water table) samples were obtained for relative density and particle size distribution determination in the laboratory. SASW testing was also carried out at every site. The University of Canterbury investigations differed only in that SPTs were also carried out, and the CPT method included electric cone sounding without pore pressure measurement, CPTU (Parez piezometer cone sounding), and mechanical cone testing (Dutch cone sounding).

A1.3.3 Test Results

The volume of test results precludes their inclusion here, but the interested reader is referred to WCL Reports 92-3220 (Waugh et al. 1992), 93-3220 (Smits et al. 1993) and 94-3220 (Cheung et al. 1994). The latter report includes only selected test results for the Inangahua earthquake sites investigated by the University of Canterbury.

A1.4 SEISMOLOGICAL INFORMATION

The following provides a summary of the earthquake and site response parameters used in the evaluation study, as determined by the Institute of Geological and Nuclear Sciences Ltd (IGNS), Lower Hutt.

A1.4.1 1987 Edgecumbe Earthquake

The main shock of the 1987 Edgecumbe earthquake occurred on 2 March 1987 at 13:42h, with an epicentre near the Tarawera River mouth, a focal depth of eight kilometres, and an average fault slip of 0.4-1.6 metres. Rupture occurred at a normal fault with the down-thrown side to the north. The rupture plane was estimated to be twenty square kilometres. The main shock was followed by after shocks, the largest being about one tenth of that of the main shock (Pender & Robertson 1987). The magnitude of the main shock has been estimated as $M_s = 6.6$, $M_L = 6.3$ (Dowrick 1991), and high intensities of shaking were experienced near the epicentral region.

The "effective depth" (h_e) of the main shock, as opposed to the focal depth, was four kilometres (Dowrick 1991). This is extremely shallow and helps explain why high intensity shaking (MM IX) was experienced for a comparatively modest earthquake magnitude. Strong motion seismograph records from the base of Matahina Dam (in eastern Bay of Plenty) gave a peak horizontal acceleration (a_{max}) of 0.33g in a NE direction (Pender & Robertson 1987).

Table A1.4.1 lists the relevant seismological information for the test sites associated with the 1987 Edgecumbe earthquake.

Table A1.4.1 Seismological information for the test sites associated with the 1987 Edgecumbe earthquake.

Location	**Site**	**Epicentral Distance (km)**	**Distance to Focus (km)**	**Modified Mercalli scale (MM)**	**Peak Ground Acceleration a_{max} (g)**	
					Rock Outcrops	**Soil Deposits**
Tarawera River Bridge	WL1	6	7.2	IX	0.37	0.30
Whakatane River Bridge	WL2	18	18.4	VIII	0.32	0.25
Edgecumbe Sub-Station	WC1	6	7.2	IX	0.37	0.30
Matahina Dam		14	14.6	VIII	0.33 (measured)	-

A1.4.2 1931 Napier Earthquake

The Napier earthquake occurred at 10:46h on 3 February 1931. The fault rupture is believed to have been close to the surface and, with an epicentre only a few kilometres away from the city centre, a great deal of damage to buildings occurred. Exensive damage to rail and road systems were reported, with landslides and slips, raised seabeds and damaged wharfs. Table A1.4.2 gives seismological information for the Napier earthquake test sites.

A1.4.3 1934 Pahiatua Earthquake

The Pahiatua earthquake occurred at 23:46h on 5 March 1934. Extensive damage occurred over a large area of the North Island, including serious structural failures at Napier, Wellington and Wanganui. Ground damage included cracks, lateral spreading and subsidence. Roads and railways were damaged and many bridge abutments failed. Only one site could be unambiguously identified as having liquefied, at Foxton Beach, Manawatu. The reports of serious liquefaction at this site are sufficiently detailed to allow exact positions to be identified at Foxton Beach, the Manawatu Heads, and the river flat of the Manawatu River at Foxton Beach.

Table A1.4.2 gives seismological information for the Pahiatua earthquake test sites.

A1.4.4 1942 Masterton Earthquake

The Masterton earthquake, which occurred at 23:16h on 24 June 1942, resulted in extensive damage to structures in the Wairarapa. Liquefaction, dynamic consolidation and lateral spreading of soft soils are inferred to have occurred as a result of this earthquake. In the Wellington region, reports of ground damage were identified including subsidence of reclaimed land between Lambton Quay and the water front, a sand boil at Thorndon Quay, as well as settlement of the approach of the Petone Pipe bridge and the abutment of the Waiwhetu bridge in Seaview Road.

Only the damage to the Petone Pipe Bridge had been documented well enough to warrant testing in the evaluation study. A site at Heretaunga Street in Petone was selected as the control site where liquefaction did not occur. Ground surface levels are believed to be similar in 1993 to those in 1942, as are the ground-water levels. The water level ranged from 1.7 m to 1.5 m below ground level at the sites in 1993.

Seismological information for the sites is given in Table A1.4.2.

Table A1.4.2 Seismological information for test sites associated with the Napier, Pahiatua and Masterton earthquakes.

Test Site Identification			Ground Water Level (m)	Seismic Intensity Parameters					
Earthquake	**Location**	**Site**		**Moment Magnitude (M_W)**	**Surface Wave Magnitude (M_S)**	**Modified Mercalli scale (MM)**	**Epicentral Distance (km)**	**Focus Distance (km)**	**Peak Ground Acceleration a_{max} (g)**
Napier	Nelson Park	NL1	0.5	7.8	7.8	X	3	6	0.56
	Embankment Road	NL2	1.4	7.8	7.8	X	5	7	0.54
	Todd Street	NC1	1.5	7.8	7.8	X	1.5	5	0.56
Pahiatua	Foxton Beach	FL1	0.3	7.6	7.6	VII	68	68	0.16
Masterton	Petone Pipe Bridge	PL1	1.7	7.2	7.2	VII	90	90	0.10
	Heretaunga Street	PC1	1.5	7.2	7.2	VII	90	90	0.10

Note: M_W is calculated from $M_W = 2/3 \{ 1.5 M_S + (16.1 \pm 0.1)\} - 10.7$ (Hanks & Kanamori 1979).

A1.4.5 1993 Ormond Earthquake

The Ormond earthquake occurred at 21:46h on 10 August 1993, with Richter local magnitude of $M_L = 6.3$ and the epicentre of the main shock located between Ormond and Te Karaka, about 25 km north of Gisborne City. Evidence of liquefaction was found on terraces on both sides of the Waipaoa River, and up to 20 km away from the epicentre, near the mouth of the Waipaoa River. All three test sites were within 10km distance from the epicentre. Seismological information is given in Table A1.4.3.

A1.4.6 1968 Inangahua Earthquake

On 23 May 1968, at 04:00h, an earthquake with a Richter local magnitude of $M_L = 7$ struck the Buller region, South Island, resulting in three deaths and injuries to fourteen people. The earthquake also caused serious damage to houses, bridges, roads, railway lines and underground facilities. The epicentre was near Inangahua, but significant damage was also caused at Reefton, Westport and Greymouth (Ooi 1987).

The main shock occurred in the Southern Alps, and the Modified Mercalli felt intensity in the epicentral area was X. The depth, location and orientation of the fault rupture and faulting mechanism have been well defined (Dowrick & Sritharan 1993). On terraces along both sides of the Buller River, Ooi (1987) reported evidence at many locations indicating liquefaction of underlying sand layers. The test sites were located on terraces adjacent to the Buller River, at Kilkenny Park in Westport and at Three Channel Flat near Inangahua (close to the epicentre). No control site was used for the Buller Gorge analysis. Table A1.4.3 lists the seismological information.

Table A1.4.3 Seismological information for test sites associated with the Ormond and Inangahua earthquakes.

Test Site Identification			Ground Water Level (m)	Seismic Intensity Parameters					
Earthquake	**Location**	**Site**		**Moment Magnitude (M_W)**	**Surface Wave Magnitude (M_S)**	**Modified Mercalli scale (MM)**	**Epicentral Distance (km)**	**Focus Distance (km)**	**Peak Ground Acceleration a_{max} (g)**
Ormond	Manuel's Farm, Te Karaka	GL1	5.7	6.5	6.1 (uncertain)	VII	9	24	0.23
	Humphrey Road, Ormond	GL2	3.3			VII	6	23	0.23
	Caesar Road, Ormond	GC1	7.0			VII	10	24	0.22
Inangahua	Kilkenny Park, Westport	BL1	1.0	7.3	7.4	VII-VIII	34	34	0.34
	Three Channel Flat, Inangahua	BL2	3.4			X	10	11	0.66

APPENDIX 2
EVALUATION OF LIQUEFACTION ASSESSMENT METHODS

A2.1 INTRODUCTION

The initial stage of the three-year research project involved an extensive literature review that identified various liquefaction assessment methods. Based on practice-orientated criteria, a selection of these assessment methods were first applied to the test sites associated with the 1987 Edgecumbe earthquake, and then to further sites affected by other earthquakes in the later stages of the research project. Evaluation of the selected assessment methods was based on the findings obtained, and Chapter 5 of the main report provides details of the six methods finally recommended and an overall summary of the evaluation results, as collated from WCL reports 92-3220 (Waugh et al. 1992), 93-3220 (Smits et al. 1993) and 94-3220 (Cheung et al. 1994). The list of these recommended methods is given in Section A2.4.3.

This appendix provides details of the evaluation procedures undertaken. Each of the methods originally selected for assessment are discussed and descriptions given of those methods that were not recommended in the final evaluation. Discussion is made in the context of both the New Zealand environment and general observation, to provide the reader with the background information and reasoning behind the recommendations made. Full details are given in the WCL reports referenced.

A2.2 LIQUEFACTION ASSESSMENT METHODS

The literature search undertaken in the first stage of the research project generated a total of 377 publications concerning liquefaction. Based on a review of this literature, various assessment methods were initially identified. The methods in addition to those recommended are discussed below.

A2.2.1 Shibata & Teparaksa Method

The Shibata & Teparaksa (1988) CPT method is similar to the recommended method by Sugawara (1989), in that the latter is a variation which introduced a new expression for c_2 to take into account fines content. Thus both methods use the same equation for the critical cone resistance ($(q_c)_{crit}$), except the Shibata & Teparaksa expression for c_2 is as follows:

$$c_2 = 1.0 \text{ for } D_{50} \geq 0.25\text{mm}$$

$$c_2 = D_{50} / 0.25 \text{ for } 0.06\text{mm} < D_{50} < 0.25\text{mm}$$

A2.2.2 Piezocone Evaluation Method

Laboratory experiments (Canou 1988) have shown that excess pore pressure (Δu), combined with low cone resistance (q_c), are indicative of potentially liquefiable soil. This method was utilised during the evaluation study but erratic data and operational difficulties prevented any firm correlation between liquefaction and pore pressure development to be made. The test was subsequently discarded from the evaluation study.

A2.2.3 Berrill & Davis Method

The Berrill & Davis method is based on a model for the seismic liquefaction of sands. Their improved version (Berrill & Davis 1985) relates the cumulative pore pressure increase (Δu) to earthquake magnitude, distance to centre of energy release, and corrected SPT values to represent ground conditions and material attenuation. As in the approach originally put forward by Seed et al. (1985), case histories of sites that have or have not liquefied were used to calibrate the model. Liquefaction is presumed to occur if the seismic-induced excess pore pressure is greater than or equal to the effective overburden pressure (i.e. $\Delta u \geq \sigma'_{vo}$). The model is particularly useful if probabilistic calculations of liquefaction hazard are required for a particular site.

The model predicts that the cumulative seismic-induced excess pore water pressure (Δu) is given by:

$$\frac{\Delta u}{\sigma'_{vo}} = \frac{120A^{0.5}\ 10^{0.75M}}{r\ N_1^{1.5}\left(1000\sigma'_{vo}\right)^{0.75}} \qquad \text{(A2-1)}$$

where: A = material attenuation factor ≈ 0.9

σ'_{vo} = effective vertical overburden pressure for stratum of interest (MPa)

M = magnitude of earthquake

r = slope distance to the source of rupture (metres)

N_1 = corrected SPT value = $C_N N$

where:

$$C_N = 0.77\ \log_{10}\left(\frac{20}{\sigma'_{vo}}\right) \quad (\text{tons/ft}^2\ \text{units}) \qquad \text{(A2-2)}$$

A2.2.4 Shear Wave Velocity Methods

The recommended shear wave methods by Tokimatsu & Uchida (1990) and Robertson (1990) were part of what was initally a total of five shear wave methods considered. The other methods are discussed below.

A2.2.4.1 Threshold Shear Strain Method

The concept of threshold strain (NRC 1985) can be used to evaluate the seismic stability for soils during seismic events. If the earthquake-induced shear strain in soil is less than the threshold strain, soil particle movement will be minimal and therefore no excess pore pressure will be generated within the soil. Consequently dynamic consolidation for dry soils or liquefaction for saturated materials will not occur.

Using a typical threshold strain value of 0.01% for most sands, the following relationship can be obtained (NRC 1985):

$$V_s = 100\sqrt{1.2\ a_{max}\ D} \tag{A2-3}$$

where: V_s = threshold shear wave velocity (m/s)
a_{max} = peak ground acceleration (m/s^2)
D = depth below ground level (m)

The threshold shear wave velocity for a particular soil layer subject to a site-specific ground acceleration can therefore be calculated. If the measured shear wave velocity is higher than the threshold value, liquefaction is not to be expected. However, if the shear wave velocity is lower than the threshold value, further liquefaction assessment of the site is required using methods as proposed by Bierschwale & Stokoe (NRC 1985), by Tokimatsu & Uchida (1990), or by Robertson (1990).

A2.2.4.2 Liquefaction Shear Wave Method

The threshold shear strain method is often considered to be too conservative for the liquefaction assessment because particle motion can induce considerable pore pressure increases before actual liquefaction occurs. An improvement on this method was suggested during the evaluation study, and is described below.

The peak shear strain caused by an earthquake can be estimated by the following equation:

$$\gamma = \frac{\tau}{G} = \frac{(\frac{a}{g})\ \sigma_o\ r_d}{G} \tag{A2-4}$$

By assuming that the mass density of the soil is constant with depth, the above equation can be rewritten as:

$$\gamma = \frac{a\ z\ r_d}{(\frac{G}{G_{max}})V_s^2} \tag{A2-5}$$

where: γ = shear strain
τ = shear stress
G = shear modulus
G_{max} = maximum shear modulus
σ_o = overburden pressure
r_d = stress reduction factor to allow for flexibility of the soil
a = acceleration
g = gravitational acceleration
z = depth
V_s = shear wave velocity

In the threshold shear strain method, the peak shear strain has been chosen typically as 0.01%, and the corresponding modulus reduction factor (G/G_{max}) selected as 0.8.

As liquefaction of soil occurs at a higher shear strain, the above equations can equally be applied for prediction of liquefaction probability.

Using the information from the Edgecumbe test sites, good agreement with other methods was obtained by assuming that soil liquefaction occurred at 1% peak shear strain and that the modulus reduction factor was equal to 0.1 at that strain level. Based on this and also assuming that r_d was approximately equal to unity, the liquefaction probability of a soil could be evaluated from the following equation:

$$V_s = 10\sqrt{a\ z} \tag{A2-5}$$

where liquefaction will occur if the shear wave velocity (V_s) of the soil at a depth z is lower than the calculated value which is required for the soil to resist liquefaction at peak ground acceleration a_{max}.

A2.2.4.3 Bierschwale & Stokoe Method

A method reported by Bierschwale & Stokoe (NRC 1985) correlates field observations of liquefied soils in the Imperial Valley, California, with their measured shear wave velocities. Areas of no liquefaction, likely liquefaction and liquefaction are identified on a graph of the surface acceleration versus the measured shear wave velocity.

A2.2.5 Method using Electrical Properties of Soils

The electrical properties of the soil (electrical conductivity of pore fluid, horizontal and vertical conductivity of the soil), which are dependent on porosity, particle orientation and cementation from aging, have been combined with the stress conditions to form a structure index (SI) for the soil. This index can then be uniquely correlated to the stress ratio required to cause liquefaction, settlement characteristics, and rate of pore pressure generation.
The electrical method is insensitive to the details of the apparatus and the test procedures, and thus provides a means for indexing the grain and aggregate compositions of particulate materials with good reproducibility. However, the method is very much in the experimental stage and was considered insufficiently proven to justify further assessment in the evaluation study.

A2.2.6 Flat Plate Dilatometer Method

The flat plate dilatometer was originally developed for the determination of in-situ soil stresses and moduli. The horizontal stress index (k_d) measured by the dilatometer is dependent on relative density, in-situ stresses, stress history, aging and cementation, as is liquefaction resistance. A correlation has subsequently been reported by Robertson & Campanella (1986), between k_d and the critical cyclic stress ratio for sands without fines.

However, considering there is no verification of the method with case histories, and the equipment is not available in New Zealand, the method was not used in the evaluation study.

A2.2.7 General Comments on Methods Identified

The methods initially identified for assessment were for the most part an enhancement on the well known "Seed's Method", which employs the cyclic stress ratio concept and correlation with field observations to assess liquefaction potential. Seed's Method itself was therefore not used in the evaluation study, as it is inherent in many of the assessment methods chosen.

Also, as the evaluation study was restricted to the empirical approach to liquefaction assessment, assessment methods using only field derived data and/or simple laboratory tests (i.e. particle size distribution and relative density tests) were chosen. The long established alternative approach based on the Casagrande school of thought (Poulos 1988), relying more on the evaluation of advanced laboratory tests, was therefore excluded from the evaluation study.

A2.3 SELECTION OF LIQUEFACTION ASSESSMENT METHODS

Identification of various assessment methods was followed by a selection process whereby selection criteria were established and each method was tested for compliance against these criteria. The selection criteria established were as follows:

- The assessment method should be based on common field testing and simple laboratory testing methods.
- The results of the test methods utilised for the assessment method should have recorded correlation with observed liquefaction performance of different sites exposed to a range of seismic loading conditions.
- The required field and laboratory results for the assessment method should be readily available from the test sites used in the evaluation study.
- Where a method required test results other than those obtained from the sites, the assessment method should be selected if generally accepted correlations exist for data conversion, as for example the well established SPT versus CPT correlation.

Based on these criteria a total of twelve (12) assessment methods were selected for initial assessment, with the Piezocone Evaluation Method also being chosen but subsequently discarded as noted in Section A2.2.2. The selected methods were as follows:

- Laboratory Test Assessment Methods
  - Particle Size Distribution
  - Relative Density

- SPT and CPT Assessment Methods
  - Shibata & Teparaksa Method
  - Sugawara Method
  - Ambraseys Method
  - Mexican Method
  - Berrill & Davis Method

- Shear Wave Velocity Assessment Methods
  - Threshold Shear Strain Method
  - Liquefaction Shear Wave Method
  - Bierschwale & Stokoe Method
  - Tokimatsu & Uchida Method
  - Robertson Method

Evaluation of the selected methods was then undertaken in order to make final recommendations.

A2.4 EVALUATION RESULTS

A2.4.1 Discussion

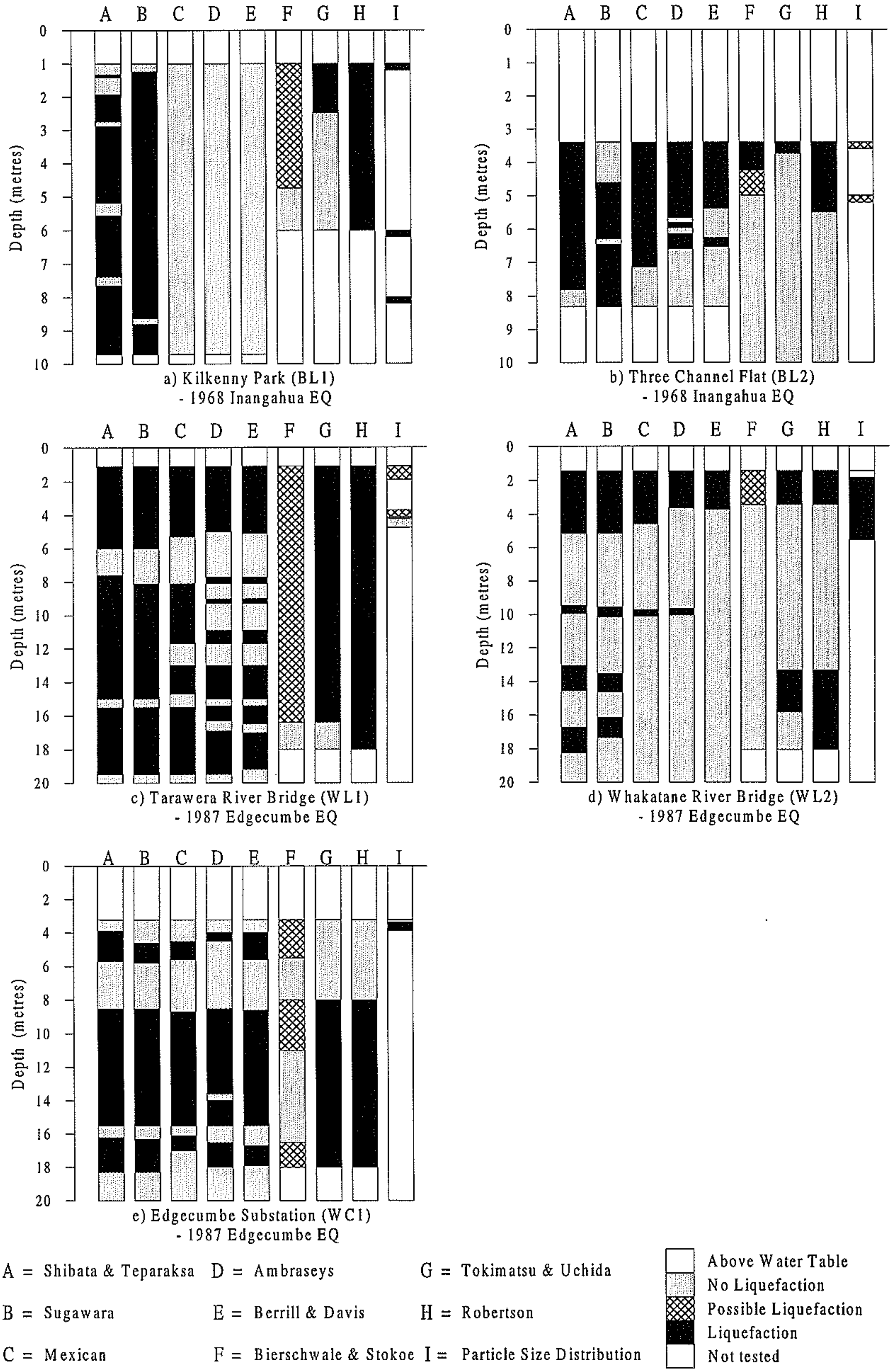
In the course of the evaluation study the more crude shear wave methods (i.e. the Threshold Shear Strain and Liquefaction Shear Wave methods) were discarded in favour of the higher ranked Bierschwale & Stokoe, Tokimatsu & Uchida, and Robertson shear wave methods, as these latter methods provided more consistent predictions in line with the other methods assessed. The Threshold Shear Strain method in particular was noted as being over-conservative, and was not recommended for use in liquefaction assessment. The Relative Density laboratory method was also discarded because not enough samples were available on which to base conclusive results.

The remaining methods formed the final group on which general findings were based and recommendations made. The general findings and recommended methods are given below, and prediction results are shown graphically in Figures A2-1a, A2-1b and A2-1c.

A2.4.2 General Findings

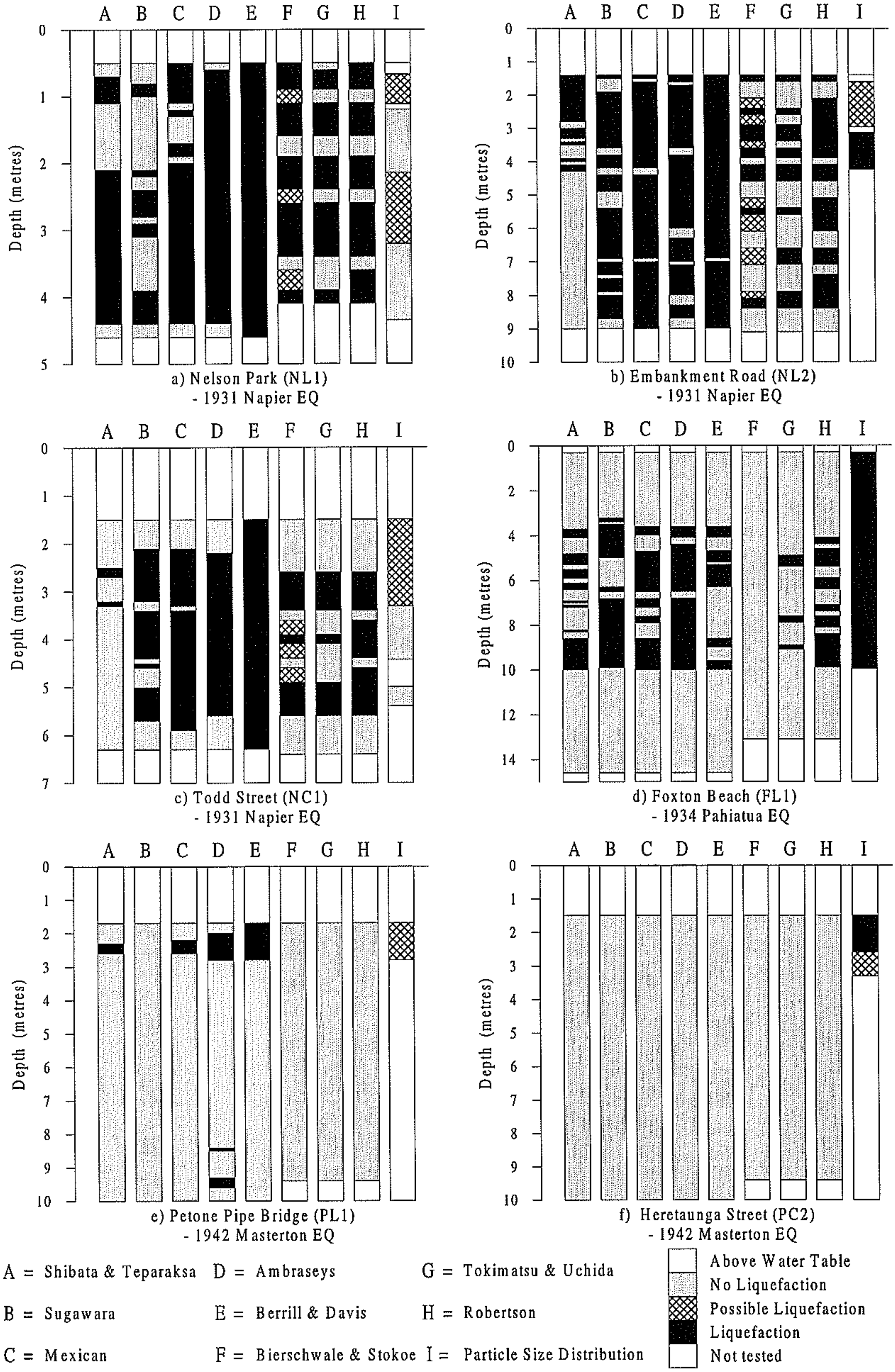
- For the three CPT methods, predictions varied significantly in their accuracy, from very consistent to totally conflicting. It appeared that the Sugawara method is more consistent in identifying the liquefaction layers, with the least reliable being the Shibata & Teparaksa method.

- The predictions from the SPT methods, Ambraseys and Berrill & Davis, are generally similar, while Ambraseys method generally provides better agreement with the CPT methods.



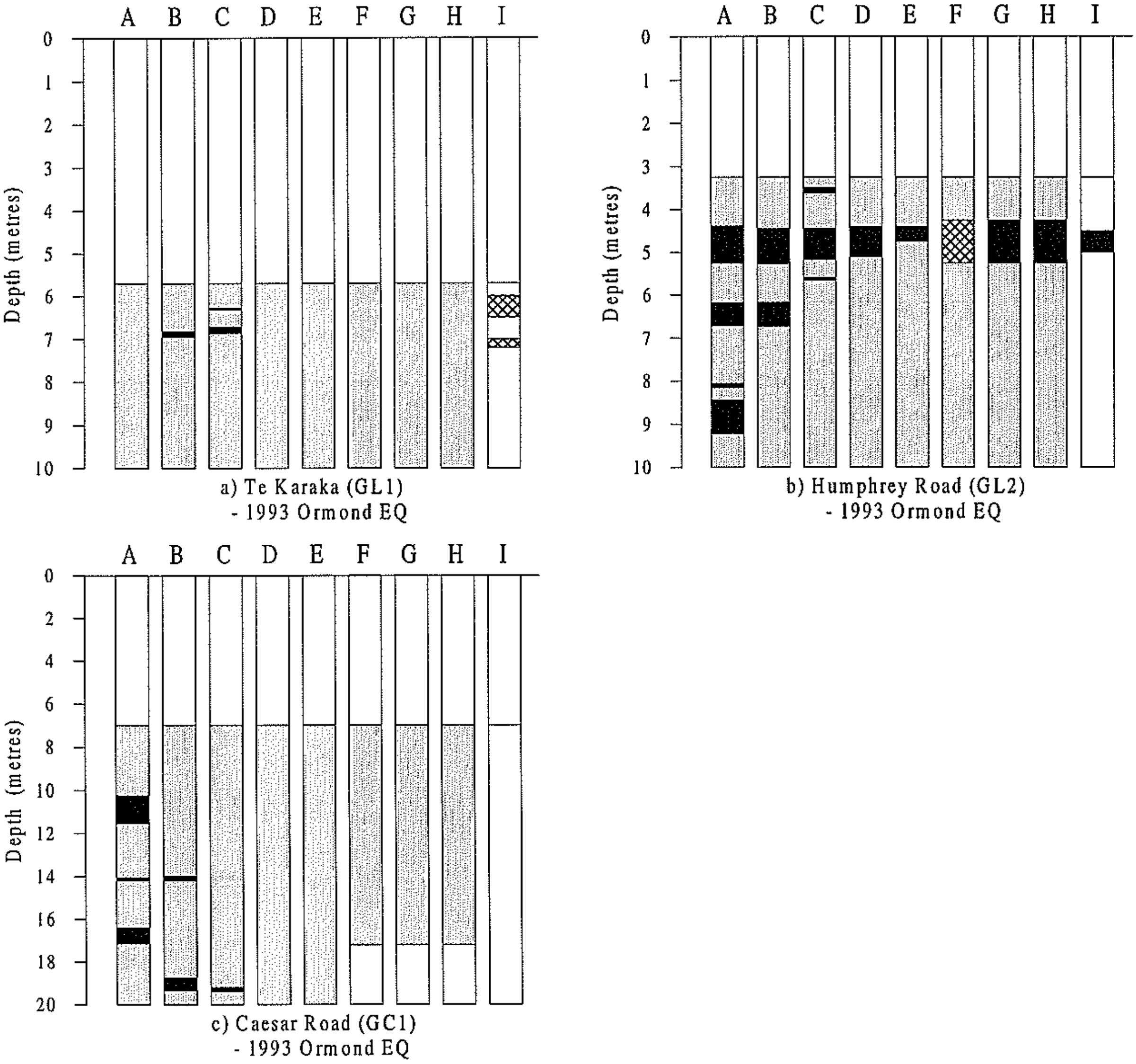
Note: For site nomenclature, refer to section A1.2.1, Appendix 1

Figure A2-1a: Liquefaction prediction results obtained from the 9 assessment methods selected for evaluation



Note: For site nomenclature, refer to section A1.2.1, Appendix 1

Figure A2-1b: Liquefaction prediction results obtained from the 9 assessment methods selected for evaluation



A = Shibata & Teparaksa D = Ambraseys G = Tokimatsu & Uchida

B = Sugawara E = Berrill & Davis H = Robertson

C = Mexican F = Bierschwale & Stokoe I = Particle Size Distribution

Above Water Table
No Liquefaction
Possible Liquefaction
Liquefaction
Not tested

Note: For site nomenclature, refer to section A1.2.1, Appendix 1

Figure A2-1c: Liquefaction prediction results obtained from the 9 assessment methods selected for evaluation

- For the three shear wave velocity methods, the Tokimatsu & Uchida and Robertson methods give similar predictions, whereas the Bierschwale & Stokoe method tends to predict less liquefaction, especially for a cyclic stress ratio larger than 0.2. However, the results obtained in the study could not identify which of the methods is more accurate.
- Of all the methods evaluated, the Sugawara method appears to be the most capable method of identifying liquefiable layers, where liquefaction was traced from particle size distribution tests on the sand boils and cored materials.
- All the methods appear to be unreliable in predicting liquefaction of soils at sites subjected to peak ground accelerations approximately less than 0.3g. Whether this result is a shortcoming of the seismological estimation methods or the liquefaction assessment could not be established from the evaluation study.

A2.4.3 Recommendations

Based on these findings the following six assessment methods were recommended for use as liquefaction assessment methods in New Zealand:

- Laboratory Test Assessment Methods
  - Particle Size Distribution

- SPT and CPT Assessment Methods
  - Sugawara Method
  - Ambraseys Method
  - Mexican Method

- Shear Wave Velocity Assessment Methods
  - Tokimatsu & Uchida Method
  - Robertson Method

These methods are described in Chapter 5 of the main report. Important points that came out of the evaluation study were as follows:

- The evaluation study revealed inconsistency of predictions using the methods assessed. Refinement of the methods based on local geology and seismology would address this problem.
- Liquefaction assessment should not rely on the prediction from only one method. Methods such as the Sugawara, Mexican and Ambraseys methods should be used simultaneously in the assessment.
- More data are required to be included in the database to evaluate the reliability of the Tokimatsu & Uchida and Robertson methods.
- Further investigation on the assessment methods for prediction of liquefaction at low ground accelerations is needed.

BIBLIOGRAPHY

BIBLIOGRAPHY

Ambraseys, N.N. 1988. Engineering seismology. *Journal of Earthquake Engineering and Structural Dynamics 17:* 1-105.

Arulanandan, K. et al. 1986. Comparison of the SPT, CPT, SV and electrical methods of evaluating earthquake induced liquefaction susceptibility in Ying Kou City during the Haicheng earthquake. In *Use of In-Situ Tests in Geotechnical Engineering*: 389-415.

Arulanandan, K. et al. 1988a. Level ground soil-liquefaction analysis using in-situ properties: I. *ASCE Journal of Geotechnical Engineering 114(7):* 753-770.

Arulanandan, K. et al. 1988b. Level ground soil-liquefaction analysis using in-situ properties: II. *ASCE Journal of Geotechnical Engineering 114(7):* 771-790.

Ballard, R.F., Stokoe, K.H., McLemore, R. 1983. Proposed standard test method for cross-hole seismic testing. *Geotechnical Testing Journal, ASTM, 6(4):* 210-219.

Berrill, J.B., Davis, R.O. 1985. Energy dissipation and seismic liquefaction of sands : revised model. *Soils & Foundations 25(2):* 106-118.

Berrill, J.B. et al. 1987. Seismic liquefaction in the Buller : Preliminary results. *Pacific Conference on Earthquake Engineering:* 13-23.

Bierschwale, J.G., Stokoe, K.H. 1984. Analytical evaluation of liquefaction potential of sands subjected to the 1981 Westmorland earthquake. *Geotechnical Engineering Report GR-84-15,* Civil Engineering Department, University of Texas, Austin.

Boon, K. 1990. *The Napier Earthquake Story, events in New Zealand history.* Nelson Price Milburn Ltd, Petone, New Zealand. ISBN 0-7055-1480-3.

Callaghan, F.R. 1933. Hawke's Bay earthquake of 3rd February 1931. *NZ Journal of Science and Technology 15(1):* 1-38. DSIR, Wellington, New Zealand.

Canou, J. 1988. Mini-piezocone (M-CPT) investigation related to sand liquefaction analysis. *International Conference on Penetration Testing, ISOPT-1:* 699-706. De Ruiter (Ed.), Balkema, Rotterdam, Netherlands.

Castro, G., Poulos, S.J. 1977. Factors affecting liquefaction and cyclic mobility. *Journal of Geotechnical Engineering Division, ASCE, 103 (GT6):* 501-516.

Castro, G. et al. 1992. Steady state strength analysis of Lower San Fernando Dam slide. *ASCE Journal of Geotechnical Engineering 118(3):* 406-427.

Cheung, K.C., Sutherland A.J., Logan T.C. 1994. Analysis of areas of previous liquefaction, stage 3. Part 1: Factual report. *Central Laboratories Report 94-22320,* Works Consultancy Services Ltd, Lower Hutt.

Chung-Tien Chin et al. 1988. SPT-CPT correlation for granular soils. *International Seminar on Penetration Testing 1988, ISOPT-1:* 335-339.

Conly, G. 1980. *The Shock of '31 - the Hawke's Bay Earthquake.* A.H. & A.W. Reed Ltd, Wellington. 234pp.

De Alba, P., Chan, C.K., Seed, H.B. 1975. Determination of soil liquefaction characteristics by large scale laboratory tests. *Report No. EERC 75-14,* Earthquake Engineering Research Centre, University of California, Berkeley, California.

De Alba, P. et al. 1984. Elastic wave velocities and liquefaction potential. *Geotechnical Testing Journal 2:* 77-88.

Dellow, G.D. 1988. Earthquake generated landslides in the Wellington-Lower Hutt area. MSc Thesis, University of Canterbury, Christchurch, New Zealand.

Diaz-Rodriguez, J.A., Armijo-Palacio, G. 1991. Liquefaction potential of fine cohesionless soils using the CPT. *Soils and Foundations 31(3):* 111-119.

Dowrick, D.J. 1991a. Magnitude reassessment of New Zealand earthquakes. *Journal of Earthquake Engineering and Structural Dynamics 20:* 577-596.

Dowrick, D.J. 1991b. A revision of attenuation relationships for Modified Mercalli intensity in New Zealand earthquakes. *Bulletin of NZ National Society of Earthquake Engineering 24(3):* 210-224.

Dowrick, D.J. 1992. Attenuation of Modified Mercalli intensity in New Zealand earthquakes. *Journal of Earthquake Engineering & Structural Dynamics 21(3):* 181-196.

Dowrick, D.J., Sritharan, S. 1993. Peak ground acceleration recorded in the 1968 Inangahua earthquake and some attentuation implications. *Bulletin of NZ National Society for Earthquake Engineering 26:* 349-355.

Eiby, G. 1989. *Earthquakes.* Heinemann Reed, Auckland, New Zealand. 166pp. Revised edition.

Evans, M.D., Seed, H.B., Seed, R.B. 1992. Membrane compliance and the liquefaction of sluiced gravel specimens. *ASCE Journal of Geotechnical Engineering 118(6):* 856-872.

Fairless, G.J., Berrill, J.B. 1984. Liquefaction during historic earthquakes in New Zealand. *Bulletin of NZ National Society of Earthquake Engineering 17(4):* 280-291.

Hanks, T.C., Kanamori, H. 1979. A moment magnitude scale. *Journal of Geophysical Research 84:* 2348-2350.

Hayes, R.C. 1937. The Pahiatua Earthquake of 5 March 1934. *NZ Journal of Science and Technology 19:* 382-388.

Henderson, J. 1933. The geological aspects of the Hawke's Bay earthquakes. *NZ Journal of Science & Technology 15(1):* 38-78.

Hull, A.G. 1986. Pre-AD 1931 tectonic subsidence of Ahuriri Lagoon, Napier, Hawke's Bay, New Zealand. *NZ Journal of Geology & Geophysics 29:* 75-82.

Hull, A.G. 1990. Tectonics of the 1931 Hawke's Bay Earthquake. *NZ Journal of Geology & Geophysics 33:* 309-320.

Hunt, A.N. 1988. *Foxton 1888-1988. The first 100 years.* Foxton Borough Council.

Ishihara, K. 1985. Stability of natural deposits during earthquakes. *11th International Conference of Society of Mechanical & Foundation Engineering 1:* 321-376.

Jamiolkowski, M., Lo Presti, D.C.F. 1992. Correlation between liquefaction resistance and shear wave velocity - Discussions. *Soils & Foundations, June 1992, 32(2):* 145-148.

Jennings, D.N., Edward, M.R., Franks, C.A.M. 1988. Some observation of sand liquefaction in the 2 March 1987 Edgecumbe (New Zealand) earthquake. *Proceedings 5th ANZ Conference on Geomechanics:* 602-607. Sydney, August 1988.

Jennings, D.N., Smith, P.L. 1991. In-situ ground conditions in areas of previous liquefaction (Edgecumbe, Bay of Plenty). *Works Consultancy Services Ltd, Hamilton, Report No.1730.*

Kanamori, H. 1978. Quantification of earthquakes. *Nature 271:* 411-414.

Kingma, J.T. 1971. Geology of Te Aute Subdivision. *NZ Geological Survey Bulletin ns 70.* NZ DSIR, Wellington, New Zealand.

Laio, S.S.C., Whitman, R.V. 1986. Overburden correction factors for SPT in sand. *Journal of Geotechnical Engineering, ASCE, 112(3):* 373-377.

Lee, K.L., Seed, H.B. 1967. Cyclic stress causing liquefaction of sand. *Journal of Soil Mechanics and Foundations Division, ASCE, 93 (SM1):* 47-70.

Liam Finn, W.D., et al. 1987. Estimating the probability of liquefaction during earthquakes. *Reliability and Risk Analysis in Civil Engineering.* Proceedings of Fifth International Conference on Implications of Statistic and Probability in Soil and Structural Engineering (ICASP5), UBC, Vancouver, Canada.

Marshall, P. 1953. Effects of earthquake on coastline near Napier. *NZ Journal of Science & Technology 15(1):* 79-92.

Martin, G.R. 1991. A practical assessment of site liquefaction effects and remedial measures. *NZ Geomechanics News No. 42.*

McMinn, J., Brabhaharan, P. 1993. *Liquefaction hazard study - Wellington region. A review of historical records of liquefaction.* Works Consultancy Services Ltd and Wellington Regional Council, Wellington, New Zealand.

Meyerhoff, G.G. 1957. Discussion. *Proceedings of Fourth International Conference of Soil Mechanics and Foundation Engineering 3:* 110.

NRC. 1985. *Liquefaction of soils during earthquakes.* Committee on Earthquake Engineering, National Research Council, National Academy Press, WashingtonDC.

Ongley, M. 1942. Wairarapa Earthquake of 24th June 1942, together with map showing surface traces of faults recently active. *NZ Journal of Science & Technology 25:* 67-78.

Ooi, C.T. 1987. *Investigation of liquefaction in the Buller region.* University of Canterbury, Department of Civil Engineering, Christchurch, New Zealand.

Peck, R.B. 1979. Liquefaction potential : science versus practice. *Journal of Geotechnical Engineering Division, ASCE, 105 (GT3):* 393-398.

Pender, M.J., Robertson, T.W. 1987. Edgecumbe Earthquake : reconnaissance report. *Bulletin of NZ National Society of Earthquake Engineering 20(3):* 201-249.

Poulos, S.J. 1988. Liquefaction and related phenomena. Pp. 292-320 in *Advanced dam engineering for design, construction and rehabilitation.* Jansen, R.B. (Ed.), Van Nostrand Reinhold, New York.

Read, S.A.L., Sritharan, S. 1993. Reconniassance report on the Ormond Earthquake - 10 August 1993. *Bulletin of NZ National Society for Earthquake Engineering 26(3):* 292-308.

Robertson, P.K. 1986. In situ testing and its application to foundation engineering. *Canadian Geotechnical Journal 23(4):* 573-594.

Robertson, P.K. 1990. Seismic cone penetration testing for evaluating liquefaction potential. *Proceedings of a Seminar on Recent Advances in Earthquake Design using Laboratory and In-situ Tests*, sponsored by ConeTec Investigations Ltd., Vancouver, Canada.

Robertson, P.K., Campanella, R.G. 1983. Interpretation of cone penetration tests - Parts I and II. *Canadian Geotechnical Journal 20(4):* 718-745.

Robertson, P.K., Campanella, R.G. 1985. Liquefaction potential of sands using CPT. *Proceedings ASCE Journal of Geotechnical Engineering 111(3):* 384-403.

Robertson, P.K., Campanella, R.G. 1986. Estimating liquefaction potential of sands using the Flat Plate dilatometer. *Geotechnical Testing Journal ASTM 9(1):* 38-40.

Robertson, P.K., Woeller, D.J., Finn, W.D.L. 1992. Seismic cone penetration test for evaluating liquefaction potential under cyclic loading. *Canadian Geotechnical Journal 29:* 686-695.

SANZ. 1986. Methods of testing soils for civil engineering purposes. *NZS 4402:1986.* Standards Association of NZ, Wellington, New Zealand.

Seed, H.B. 1976. Evaluation of soil liquefaction effects on level ground during earthquakes. *Specialty Session on Liquefaction Problems in Geotechnical Engineering,* ASCE National Convention, Philadelphia.

Seed, H.B., De Alba, P.1986. Use of SPT and CPT tests for evaluating the liquefaction resistance of sands. *Proceedings of the In-situ Testing in Geotechnical Engineering Conference, ASCE:* 281-302.

Seed, H.B., Idriss, I.M. 1971. A simplified procedure for evaluating soil liquefaction potential. *Journal of Soil Mechanics and Foundations Division, ASCE, 97 (SM9):* 1249-1273.

Seed, H.B., Idriss, I.M. 1981. Evaluation of liquefaction potential of sand deposits based on observations of performance in previous earthquakes. *In Situ Testing to Evaluate Liquefaction Susceptibility.* Proceedings of Session No. 24, ASCE National Convention, St Louis, Missouri.

Seed, H.B., Idriss, I.M., Makdisi, F., Banerjee, N. 1975. Representation of irregular stress time histories by equivalent uniform stress series in liquefaction analyses. *Report No. EERC 75-29,* Earthquake Engineering Research Centre, University of California, Berkeley, California.

Seed, H.B., Lee, K.L. 1966. Liquefaction of saturated sands during cyclic loading. *Journal of Soil Mechanics and Foundations Division, ASCE, 92 (SM6):* 105-134.

Seed, H.B., Romo, M.P., Sun, J., Jaime, A., Lysmer, J. 1987. Relationships between soil conditions and earthquake ground motions in Mexico city in the earthquake of Sept. 19, 1985. *Report No.UCB/EERC-87/15,* Earthquake Engineering Research Centre, University of California, Berkeley, California.

Seed, H.B., Tokimatsu, K., Harder, L.F., Chung, R.M. 1984. The influence of SPT procedures in soil liquefaction resistance evaluations. *Report No. UBC/EERC-84/15,* Earthquake Engineering Research Centre, University of California, Berkeley, California.

Seed, H.B., Tokimatsu, K., Harder, L.F., Chung, R.M. 1985. Influence of SPT procedures in soil liquefaction resistance evaluations. *Journal of Geotechnical Engineering, ASCE, 111(12):* 1425-1445.

Shibata, T., Teparaksa, W. 1988. Evaluation of liquefaction potential of soils using cone penetration tests. *Soils & Foundations 28(2):* 49-60.

Skempton, A.W. 1986. Standard penetration test procedures and the effects in sands of overburden pressure, relative density, particle size, ageing and overconsolidation. *Geotechnique 36(3):* 425-447.

Smith, W.D. 1990. Earthquake hazard in New Zealand : Some implications of the Edgecumbe earthquake, March 1987. *Bulletin of NZ National Society of Earthquake Engineering 23(3):* 211-219.

Smits A.P., Cheung K.C., Sutherland A.J. 1993. Analysis of areas of previous liquefaction, report on stage 2 research. Part 1: Factual report. *Central Laboratories Report 93-22320,* Works Consultancy Services Ltd, Lower Hutt.

Sugawara. N. 1989. Empirical correlation of liquefaction potential using CPT. *International Conference of Soil Mechanics & Foundation Engineering 12(1):* 335-338.

Tokimatsu, K., Uchida, A. 1990. Correlation between liquefaction resistance and shear wave velocity. *Soils and Foundations 30(2):* 33-42.

Tokimatsu, K. et al. 1991. V_s determination from steady state Rayleigh wave method. *Soils & Foundations 31(2):* 153-163.

Tsuchida, H. 1970. Prediction and countermeasure against the liquefaction in sand deposits. *Soils & Foundations 23(1):* 47-60.

Waugh, P.J., Jennings D.N., Cheung, K.C., Smits A.P. 1992. Analysis of areas of previous liquefaction, report on stage 1 research. *Central Laboratories Report 92-22320,* Works Consultancy Services Ltd, Lower Hutt.