SEISMIC RISK AND GROUND IMPROVEMENT

by

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ABSTRACT

The west coast of British Columbia is located in one of the most seismically active areas in Canada. Historical earthquakes of magnitudes 7.0 to 8.0 have been recorded in this region and the Queen Charlotte Islands earthquake of 1940, magnitude 8.0, is the largest earthquake known to have occurred in Canada. The Vancouver Island earthquake of 1949, magnitude 7.3, caused extensive liquefaction soil failures along both coasts of Vancouver Island and along the west coast of mainland British Columbia.

This paper discusses the implications of the National Building Code of Canada for selecting design ground motion for liquefaction assessment, and the Cornell method of seismic risk assessment as used in the 1985 version of the Code. The Seed's simplified method of liquefaction assessment for level ground, based on field observations of the performance of sandy sites during actual earthquakes, is described, and a recently developed simple probabilistic method of liquefaction prediction, which incorporates the Seed's simplified method of assessing liquefaction potential into the Cornell seismic risk analysis framework, is outlined. Application of this simple method of calculating liquefaction probability to three case histories in the Lower Mainland of Vancouver, in which the foundation soils were or were not improved prior to development, is presented.

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INTRODUCTION

Following the Niigata earthquake (M 7.3) in Japan in 1964 and the Alaska earthquake (M 8.3) of 1964, the engineering profession began to pay serious attention to the phenomenon of ground failure by liquefaction. Compilations of observed occurrences of ground liquefaction by Youd (1977) and by Kuribayashi and Tatsuoka (1975) show a relationship between earthquake magnitude and maximum distance from the epicentre for liquefied sites. See Figure 1.

A comparison of this relationship with the observed seismicity of British Columbia shown on Figure 2 shows that there is potential for earthquake ground shaking to be strong enough to cause liquefaction of loose susceptible deposits in many of the more densely populated areas of British Columbia. For example, the Queen Charlotte fault is considered to be capable of a magnitude 8.5 earthquake, which would put Prince Rupert and Kitimat at risk from liquefaction. Similarly, the Lower Mainland and Victoria area are at risk from seismic activity in the Puget Sound area, and seismic activity in the northern Vancouver Island area could be the cause of liquefaction along the east coast of Vancouver Island and on the mainland in such communities as Powell River. In fact, liquefaction ground failures were observed in and around Comox, Powell River, and Campbell River due to the 1946 Vancouver Island earthquake (M 7.3) as reported by Hodgson (1946) and Rogers (1980).

One of the more practical and widely used procedures for evaluating subsoil liquefaction potential is the simplified method developed by Professor H.B. Seed and his co-workers (1975, 1976, 1979, 1981, 1984). Using Standard Penetration Test results from drilling investigations and laboratory particle size distribution tests on soil samples, together with a set of design charts, the experienced engineer can evaluate the liquefaction susceptibility of a site during the 'design earthquake'.

One difficulty that the engineer has in applying this method, however, is that it requires a definition of earthquake peak ground acceleration and magnitude. In areas of the world where seismicity is confined to well known active faults with a long period of recorded activity, a deterministic estimate of ground motion at the site can often be made. In British Columbia, the seismicity is not so well defined and, therefore, probabilistic methods are often used.

The recent 1985 version of the National Building Code of Canada (NBCC) has seismic design provisions and a zoning map that are based on a probabilistic assessment. This paper refers to this new earthquake zoning for Canada and describes a recently developed probabilistic approach to estimating liquefaction ground failure that incorporates Seed's simplified method. Some case histories are discussed where this and more conventional analyses were applied, and where ground improvement methods were used to substantially reduce the risk of liquefaction and related ground failure.

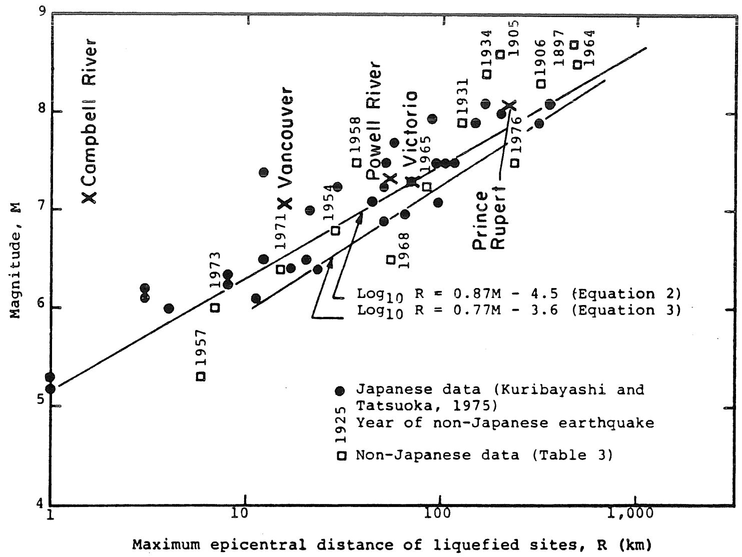
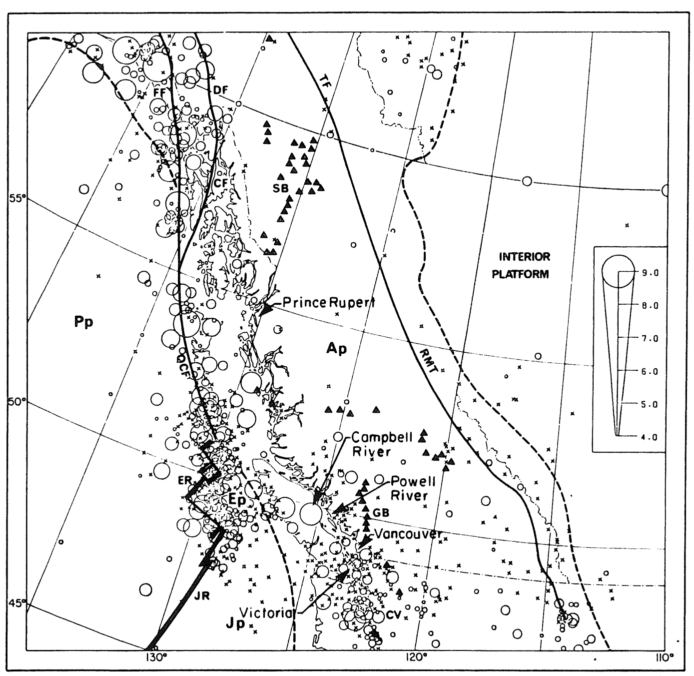


FIGURE 1. RELATIONSHIP BETWEEN MAXIMUM EPICENTRAL DISTANCE OF LIQUEFIED SITES AND EARTHQUAKE MAGNITUDE (After Youd, 1977).



Tectonic map of western Canada showing the locations of the main lithospheric boundaries superimposed on the seismicity map. Ap — American plate; CF — Chatham Strait fault; CV — Cascade volcanoes; DF — Denali fault; Ep — Explorer plate; ER — Explorer Ridge; FF — Fairweather fault; GB — Garibaldi Volcanic Belt; Jp — Juan de Fuca plate; JR — Juan de Fuca Ridge; Pp — Pacific plate; QCF — Queen Charlotte fault; RMT — Rocky Mountain Trench; SB — Stikine Volcanic Belt; TF — Tintina fault; solid lines — main faults and plate boundaries; dashed lines — continental slope and eastern margin of Rocky Mountains; triangles — recent volcanoes.

FIGURE 2. SEISMICITY AND TECTONIC MAP OF WESTERN CANADA. (After Milne et al, 1978).

DESIGN GROUND MOTION FOR LIQUEFACTION ASSESSMENT

The 1970 version of the NBCC gave a seismic zoning map for use in the earthquake design of normal structures which was used in subsequent versions of the code up to and including 1980. The map showed selected contours of peak ground acceleration (PGA) having a return period of 100 years (A_{100}) which were used to define the boundaries of seismic zones for building design. The contours were derived using a method of extreme value statistics applied to the historic catalogue of recorded earthquakes (Milne and Davenport, 1969).

The form of distance attenuation law used in this procedure produced a single value of PGA and incorporated none of the random scatter actually observed. The implicit assumption in this method was that future earthquakes would occur in the same places as past earthquakes. What is of more concern for the soils engineer was the use of A_{100} as the reference value for PGA contour map. It was stated, by Whitham et al (1970), that this probability level was chosen because "it bears a rational relationship to the expected lifetime of major structures and to the interval of time of fundamental available data". In the absence of any explicit guidance, this might be interpreted by the foundation engineer to imply that the A_{100} value represents an adequate design level for liquefaction. However, the A_{100} was simply a convenient reference level for ensuring a consistent variation in building base shear values across the country. The formula for calculating base shear, based on A_{100}, contained factors that rendered the effective PGA value significantly higher than A_{100}, so that the real level of probability of structural failure, although not stated, was much lower than 0.01 per annum. These code factors were set empirically based on experience from California. In fact, as shown by Anderson et al (1979), the effective PGA implied by the quasi-static design procedure in the 1977 NBCC had a return period varying from 80 to 1,700 years, depending on the natural period of the structure.

The old codes, therefore, did not provide engineers with any clear criteria for ground acceleration values to use in liquefaction assessment. Furthermore, there was no guidance regarding the duration of earthquake shaking (related to magnitude), which is a major factor causing liquefaction.

The 1985 NBCC includes a major change in the method of zoning and design for earthquake risk. The historical earthquakes are released from their presumed epicentres, but constrained within the boundaries of seismic source zones that are based on regional geology, faulting, tectonics, and observed seismicity. These zones are then assigned average activity rates, and the methods of Cornell (1968) and McGuire (1976) used to integrate the individual influences of potential earthquake sources, and derive contours of peak ground acceleration and peak ground velocity for chosen risk levels or return periods. The method is illustrated schematically, for a simple single source zone model, on Figure 3.

Figure 3 illustrates the steps in the Cornell method to calculate the probability, per annum, of exceeding a given PGA at a site of interest in the centre of a very large seismogenic source zone. The approach in the analysis is:

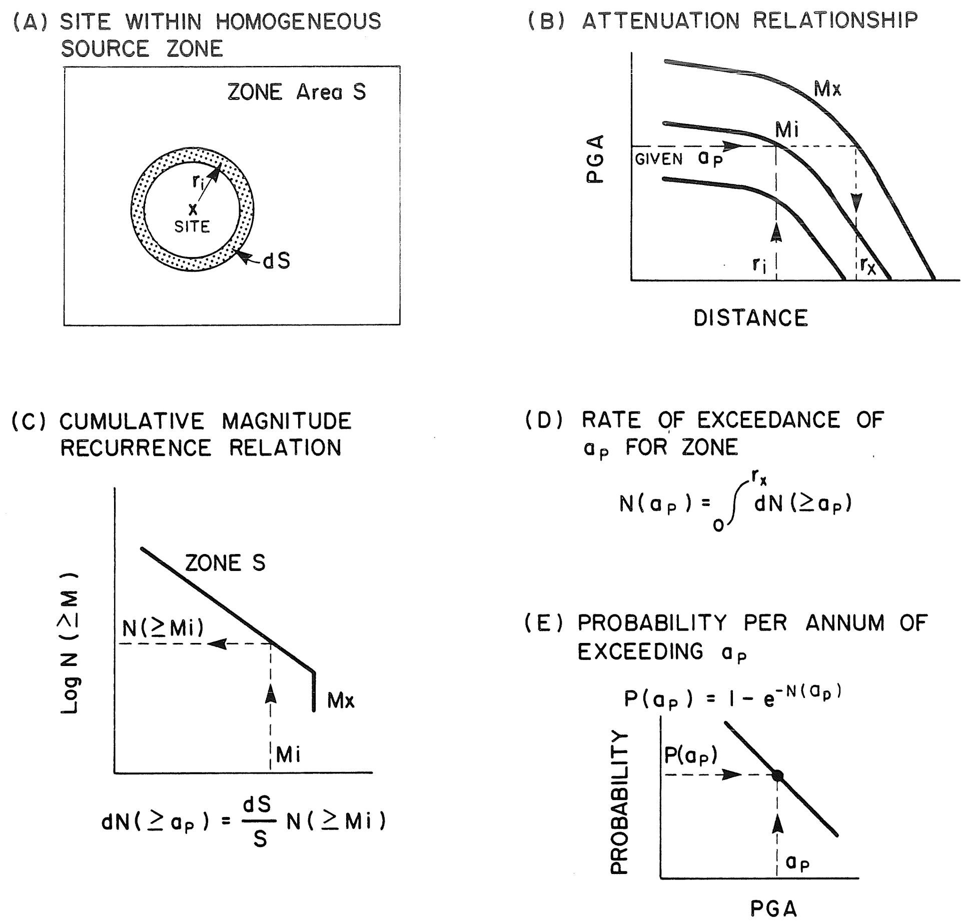


FIGURE 3. ILLUSTRATION OF THE CORNELL METHOD FOR SIMPLE CASE OF A SITE WITHIN A LARGE SEISMIC SOURCE ZONE.

1. Consider an annular element of area dS at distance r_i from the site.

2. Using an appropriate attenuation relationship, determine the magnitude M_i at distance r_i to exceed the given PGA a_p at the site.

3. For the M_i determined, use the cumulative magnitude recurrence relation for the zone to find the number of exceedances per year of PGA a_p at the site, dN ($>a_p$), due to earthquakes in the annular element.

4. The acceleration exceedances per year at the site from all annular elements which can affect the site are summed to obtain the rate of exceedance of PGA a_p for the zone, $N(a_p)$. The annular elements considered are those from the site out to a maximum distance R_x which is obtained by substituting the maximum magnitude M_x of the zone in the attenuation relation.

5. Assuming earthquake occurrence to be a Poisson process, the probability of exceeding a given PGA, a_p, at the site due to all earthquakes in the zone is $P(a_p) = 1-\exp(-N(a_p))$.

The simple case illustrated above can be extended to the general case of a number of irregular earthquake zones within the range of influence of the site. These cases are handled by numerical integration in the seismic risk analysis program of McGuire (1976).

The other change in the new code is a departure from the 100 year return period as a basis for the zoning map. There are now two maps, a peak horizontal acceleration map, and a peak horizontal velocity map. Both are based on the concept of a probability of exceedance of 10% during a 50 year period, which is equivalent to an annual probability of exceedance of 0.0021, or a return period of 475 years. This concept derives from the recommendations of the Applied Technology Council, formed in the United States in 1971, and is considered an appropriate risk level for the design of normal buildings to resist moderate earthquakes without significant damage, and major earthquakes without collapse. Ferritto and Forrest (1977) compare this risk level to the fatality rate from motor vehicle accidents and suggest that it is roughly equivalent to the probability that 10 people would have of being killed in 50 years of driving.

The A_{475} and V_{475} are used as reference levels for zoning in much the same way as the old A_{100}. The various factors included in the quasi-static base shear calculations in earlier, pre-1985 codes have been revised so that the minimum recommended lateral seismic forces have, on average across the country, not changed from the 1980 NBCC levels.

The implication (not explicitly stated) in the 1985 NBCC, however, is that to provide earthquake resistant foundations consistent with or exceeding the resistance of the superstructure, the geotechnical engineer should adopt ground motion parameters that have a probability of exceedance of about 10% in 50 years (Byrne and Anderson, 1987). This represents a much higher peak ground acceleration level than the old A_{100} or 0.01 annual probability of exceedance. In Vancouver, for example, the A_{100} value quoted in the 1970 NBCC is 8.9% of gravity, whereas the A_{475} value in the 1985 NBCC is 21% of gravity.

The Cornell-McGuire method as it is used in NBCC (1985) is intended for use in the design of 'normal' buildings and for 'normal' probability levels. The code gives zonal values, and also gives specific values for selected major cities, but it is often useful to run a site specific analysis. In this way, it is possible to separate out contributions to ground motion at the site from the various source zones, which can be useful when there is a need to estimate the dominant frequency range of ground motion, or when extrapolation to low probability levels is attempted.

Critical structures are not usually covered by national codes, and there is often, but not always, a requirement to consider ground motions more stringent than the 1/475 year values. Atkinson and Charlwood (1984) provide a useful overview of various code requirements for a range of critical structures as shown in Table 1. It is pertinent to a later case history to single out here the Standard governing LNG storage facilities, which requires design for 'safe shutdown' earthquake having a return period of 1 in 10,000 years.

TABLE 1

SUMMARY COMPARISON OF PRESENT CODE SEISMIC DESIGN CRITERIA IN CANADA (From Atkinson and Charlwood, 1984)

FACILITY (CODE)	"OPERATING OR "DESIGN" LEVEL"	"EXTREME" OR SAFETY LEVEL	CODE STATUS	BASED ON RISK LEVELS AS OPPOSED TO TRADITIONAL METHODS
Important Buildings (NBCC, 1980)	A A100 - design for allowable stresses in the elastic range using pseudo-static method or elastic dynamic analysis	Reserve strength implied through design formulae	Usually owner's decision to use NBCC. Use I=1.3 for key structures such as hospitals. Need consistent design for foundations. NBCC under development for 1985.	No
LNG Storage (CSA Z276)	OBE = A475 - allowable stresses in in elastic range - dynamic analysis using DRS	SSE = A10,000 - yield strength - buckling limit - dynamic analyses using DRS	CSA Standard seismic criteria under review. Possibly overall risk analysis related. Also use API 620.	Yes
LPG or Oil Storage	A = A100 Working stress design	Reserve strength implied through design formulae	Possible need for CSA standard with seismic criteria. Presently use API 620 or 650.	No
Hydro-Electric and Water Supply Dams	A = empirical seismic coefficient, -psuedo-static stability analyses and factors of safety	- Generally MCE or equivalent for ultimate strength check	Presently based on USBR & US Army Corps Methods	No
Thermal-Electric Power Plants	A = A100 - allowable stresses in elastic range - pseudo-static method or dynamic analysis	Not required	Presently based on NBCC. Could use cost/benefit analysis to set seismic design levels.	No
Nuclear Power Plants (CSA N289)	SDE > A100 - allowable stresses in elastic range - elastic dynamic analysis	DBE > A1000 - yield strength - elastic/plastic dynam. analysis	Subject to very comprehensive CSA Code and AECB review.	Yes
Electrical Transmission Systems CSA-C22.3	A = A100 for substations	Not required	No seismic design required in CSA Standard. Could be appropriate on long key lines.	No
Mines	Check pit slope stability and buildings using A A100	Not required	Mainly based on precedent. Subject to Prov. Reg. Agency review. NBCC used for buildings.	No
Mine Tailings Impoundments	A = empirical seismic coefficient - pseudo-static stability analysis and factors of safety	MCE used for ultimate strength check	Need to evaluate environmental & safety consequences for particular mineral.	No
Pipelines	A = A100 for compressor stations and slope stability checks	MCE used for major continental pipelines	Subject to review by NEB and Prov. regulatory agencies.	No
Offshore Petroleum Structures	OLE ≙ A25 for exploration islands or platforms OLE ≙ A475 for production facilities - Strength design	SLE = extreme event - Ductile design	Presently uses API-RP2A which recommends risk analysis based design levels and owner's decision.	Yes
Transportation (Marine & Railways)	A100 where applicable to buildings, bridges, and docks. Tunnel designs checked for fault displacements.	Not required	Usually owner's decision.	No
Hazardous Materials Storage & Disposal	May use A100 for buildings.	Needs specific risk assessment and design check	Subject to Fed. & Prov. regulatory review and EIS review.	Yes

The formulation of the Cornell-McGuire analysis used to develop the NBCC (1985) seismic design provisions is not generally appropriate for low probability ground motion estimates much less than 10^{-3} per annum. This fact is discussed in some detail by Milne and Weichert (1986) and is also referred to by Heidebrecht et al (1983). With decreasing annual probability of exceedance, the contributing earthquakes in the Cornell analysis increase in magnitude and decrease in epicentral distance, and the basic assumptions used in the model, i.e. attenuation relation, maximum magnitude, focal depth, recurrence rate, etc., can have a marked effect on the result. For example, the NBCC (1985) uses the distance attenuation relationship of Hasegawa et al (1981). This attenuation law for western Canada gives values of peak horizontal ground acceleration very similar to those obtained from other commonly used relations such as Schnabel and Seed (1973) and Joyner and Boore (1981), for magnitude 5.5 earthquakes, but far exceeds them for magnitude 7.5 events, as shown on Figure 4 from Basham et al (1985). The use of the Hasegawa attenuation can provide an upwards bias to PGA estimates for very low probability predictions by the Cornell method, if the contributing zones have maximum magnitudes greater than about M 6, because the effects of large earthquakes will dominate. Figure 5 shows the results of Cornell-McGuire analyses extended to low probabilities, for a site in the Fraser River delta area. One analysis used the basic NBCC (1985) model, and the other used the same source zone model, but with the attenuation relation of Joyner and Boore (1981). There is reasonable agreement between the two for probabilities down to the 0.002 level, but at lower probabilities, the two results diverge increasingly, until at the 0.0001 annual probability, there is almost a factor of two difference.

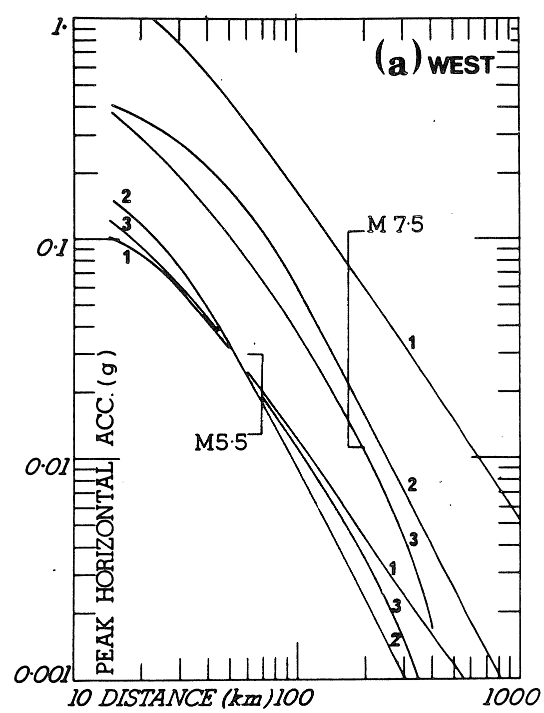


FIGURE 4.

COMPARISON OF ATTENUATION RELATIONS FOR MAGNITUDE 5.5 AND 7.5 IN WESTERN NORTH AMERICA:
1, HASEGAWA ET AL (1981);
2, SCHNABEL AND SEED (1973);
3, JOYNER AND BOORE (1981).
(From Basham et al, 1985).

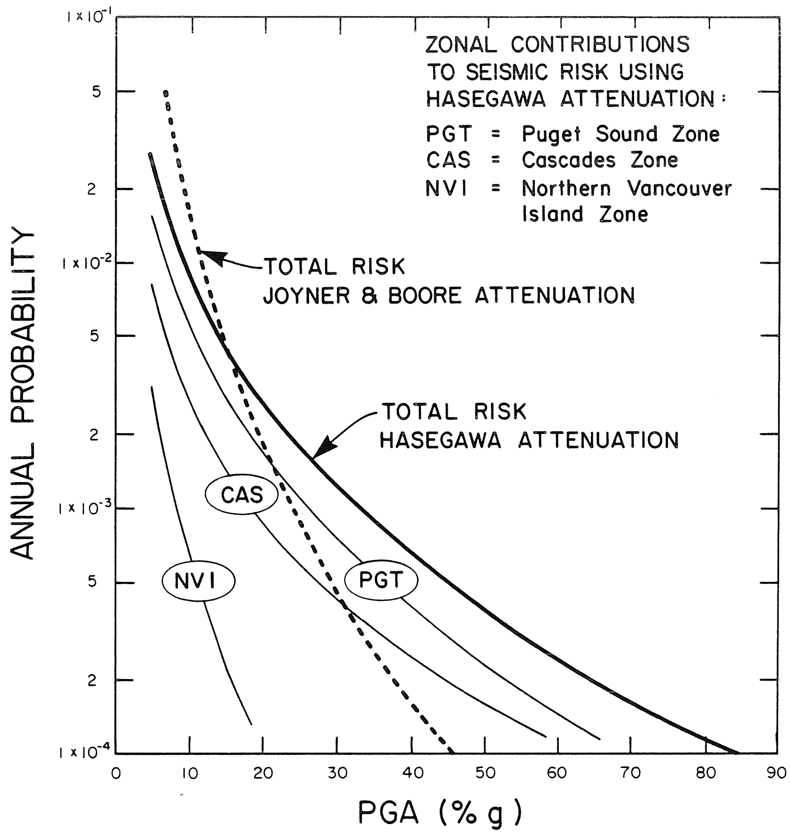


FIGURE 5. RESULTS OF CORNELL-MCGUIRE ANALYSIS USING NBCC (1985) EARTHQUAKE SOURCE ZONES FOR SITE IN TILBURY ISLAND, DELTA, BRITISH COLUMBIA.

Another major factor affecting the results of a Cornell-McGuire analysis is the treatment of the uncertainty in the attenuation relationships. Published attenuation equations represent the mean of a collection of scattered data. If the statistical scatter is incorporated into the analysis, by integrating over the range of mean plus one standard deviation using a stochastic factor σ_{lnk}, then another upwards bias is introduced. It is argued that this variability is real and should be included in the analysis, but the numerical value that should be adopted for the uncertainty term is debatable. The uncertainty factor, σ_{lnk}, varies from 0.3 to 0.7 for the most commonly used attenuation equations. The analysis used for the NBCC (1985) was done with a value for $\sigma_{lnk} = 0.7$.

A sensitivity analysis performed for Vancouver by Atkinson and Charlwood (1983) using a slightly different seismic source zone model than the NBCC, shows how the use of σ_{lnk} of 0, 0.2, 0.4 and 0.6 produces dramatically different PGA estimates, with the differences being most pronounced at low probability level. See Figure 6. At an annual probability of 0.0001, the results with $\sigma_{lnk} = 0.6$ are more than twice the mean value ($\sigma_{lnk} = 0$).

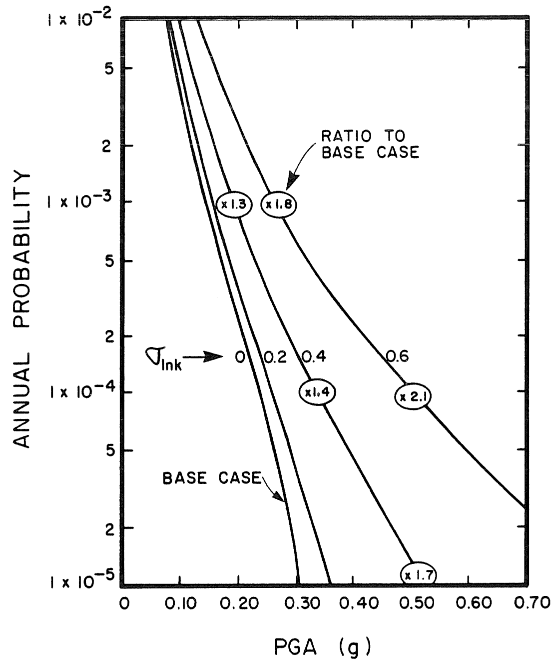


FIGURE 6. INFLUENCE OF UNCERTAINTY FACTOR, $\sigma_{\ln k}$, ON RESULTS OF SEISMIC RISK ANALYSIS. (From Atkinson and Charlwood, 1983).

Evidently, the NBCC (1985) Cornell-McGuire model gives reasonably stable answers down to about the 0.002 annual probability, but at lower probabilities, it may be somewhat unstable. When evaluating earthquake ground motion effects for critical structures, it behoves the geotechnical engineer to adopt a team approach and to bring together expertise in seismology, geology and soil dynamics, and carry out special studies which may include the Cornell-McGuire analyses tested for sensitivity to variation in several input parameters.

EMPIRICAL METHODS FOR LIQUEFACTION HAZARD ASSESSMENT BASED ON FIELD DATA

SEED'S SIMPLIFIED METHOD FOR LEVEL GROUND

Seed's simplified method of liquefaction assessment for level ground is based on field observations of the performance of sandy sites during actual earthquakes. The occurrence or non-occurrence of liquefaction at a site subject to earthquake shaking is correlated to the intensity of ground shaking and the Standard Penetration Test resistance of the soils underlying the site, as shown in a typical chart in Figure 7. The method was first proposed in 1975 (Seed et al, 1975) and was updated as new field data and interpretations became available (1976, 1979, 1981, 1983, 1984). The most recent revision was in 1984.

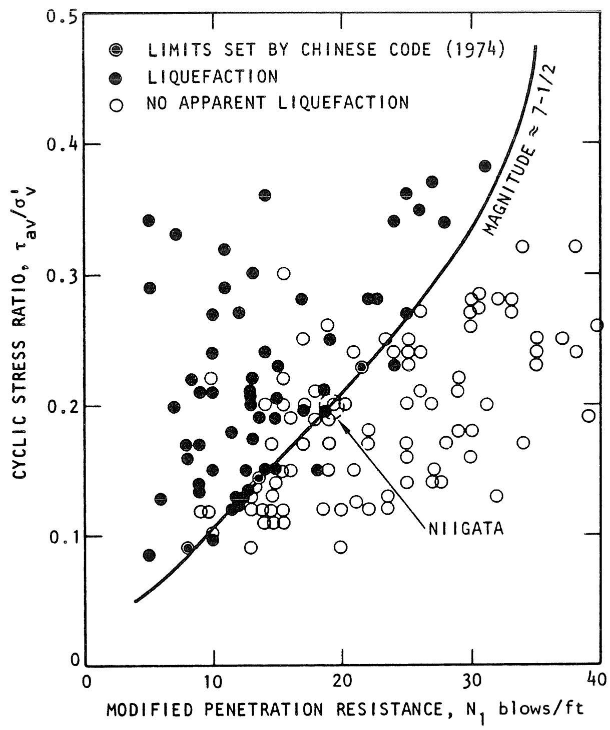


FIGURE 7. SEED'S LIQUEFACTION ASSESSMENT CHART FOR SANDS (D_{50} > 0.25 mm) FOR M 7.5 EARTHQUAKES. (From Seed and Idriss, 1981).

In Seed's approach, the earthquake loading in the soil at a site under consideration is represented by the cyclic stress ratio which can readily be computed by:

$$\frac{(\tau h)\ av}{\sigma o'} = 0.65 \frac{a_{max}}{g} \frac{\sigma o}{\sigma o'} r_d \qquad (1)$$

where $(\tau h)\ av$ = average horizontal shear stress induced by an earthquake

a_{max} = maximum acceleration at the ground surface

σo = total overburden pressure on sand layer under consideration

$\sigma o'$ = effective overburden pressure on sand layer under consideration

r_d = a stress reduction factor varying from a value of 1 at the ground surface to a value of 0.9 at a depth of about 9 m (30 ft).

The soil characteristics at the site are represented by the Standard Penetration Test (SPT) resistance values. The use of the SPT resistance is appropriate because it is affected in the same way by many of the factors that affect the liquefaction resistance of sand, i.e. changes in factors that tend to increase the cyclic loading or liquefaction resistance also tend to increase the penetration resistance (Seed, 1976). The penetration resistance used in Seed's correlations is the SPT resistance normalized to an effective confining pressure of 100 kPa:

$$N_1 = C_N \quad N \tag{2}$$

where N_1 = measured penetration resistance corrected to an effective overburden pressure of 100 kPa (1 tsf)

C_N = conversion factor which is a function of the effective overburden pressure

N = measured SPT resistance using standard procedures as described by Seed and Idriss (1981)

The boundary line shown on Figure 7 was derived largely based on earthquake data with magnitudes of about 7.5 and in sandy deposits with mean grain size, $D_{50} > 0.25$ mm. The possibility of liquefaction in a sand layer for such conditions can be assessed readily by plotting the point (N_1, $\tau av/\sigma'o$) defining the layer under consideration on the chart and noting whether it falls above or below the boundary line.

The liquefaction potential due to an earthquake is known to depend not only on the intensity of shaking, but also on duration which is related to the magnitude of the earthquake. The duration of the earthquake shaking is accounted for in Seed's analysis by the use of different boundary curves for different magnitude earthquakes. This feature was introduced in 1981 as shown in Figure 8. The extension of the chart to earthquakes with magnitudes other than M 7.5 was based on a set of scaling factors derived from a statistical analysis of earthquake records and the characteristic shape of a liquefaction curve determined by large scale laboratory cyclic simple shear tests (Seed and Idriss, 1981).

While it was suspected for some time that silty sands ($D_{50} < 0.15$ mm) are less vulnerable to liquefaction than clean sands with similar penetration resistance values, the first extensive set of field evidence was provided by Tokimatsu and Yoshimi (1981), based on data collected following the Miyagiken-Oki earthquakes of 1978 in Japan. Based on the Japanese data, Seed and Idriss (1981) introduced a correction for silty sand deposits. They suggested that the boundary established for sands with $D_{50} > 0.25$ mm can be used for silty sands ($D_{50} < 0.15$mm), provided the N_1 value for the silty sand site is increased by 7.5 before entering the liquefaction potential chart.

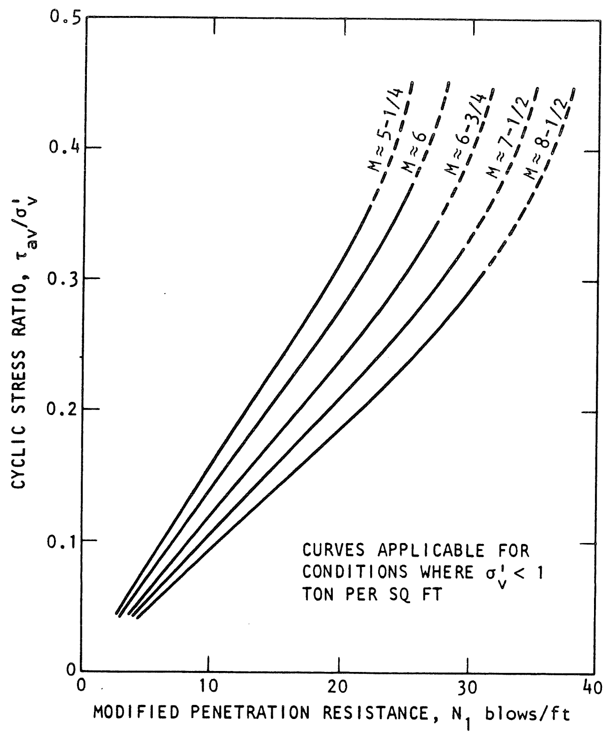


FIGURE 8. SEED'S LIQUEFACTION ASSESSMENT CHART FOR SANDS FOR DIFFERENT MAGNITUDE EARTHQUAKES. (From Seed and Idriss, 1981).

The data base of Figures 7 and 8 is limited to sites where liquefaction occurred under effective confining pressures of less than about 150 kPa, which is equivalent to a depth of about 15 m if the groundwater table is at the surface. For pressures and depths greater than this, the limiting cyclic stress ratios should be reduced as suggested by Seed (1983).

Seed et al (1984) reviewed available data on energy ratios associated with different SPT procedures used in practice worldwide and recommended that an energy ratio (i.e. percent of theoretical free-fall energy) of 60% be adopted as standard for liquefaction assessment. Measured penetration resistance values, Nm, for a hammer system which delivers an energy ratio other than 60%, ERm, can be corrected by the relationship:

$$N_{60} = Nm \frac{ERm}{60} \tag{3}$$

Based on a re-evaluation of the available SPT-liquefaction correlation data, Seed et al (1984) presented the relationships between stress ratio causing liquefaction and $(N_1)_{60}$ values for sands and silty sands for M 7.5 earthquakes as shown in Figure 9. For N_1 values up to about 25, however, the correlation line drawn for clean sands is very close to that proposed by Seed et al (1981), indicating that the N_1 values used in the earlier work, which were not corrected for variations in SPT procedures, correspond closely to $(N_1)_{60}$ values. At values of N_1 higher than 25, the curve in Figure 9 is less conservative than the earlier boundary line shown in Figure 7.

The new liquefaction resistance curves for M 7.5 earthquakes as shown in Figure 9 can be extended to other earthquake magnitudes following the procedure used by Seed et al (1981). The new silt "correction" is based on fines content (i.e. percent minus the No. 200 mesh sieve) and is believed to be more reliable than the previous correction based on mean grain size (i.e. D_{50}).

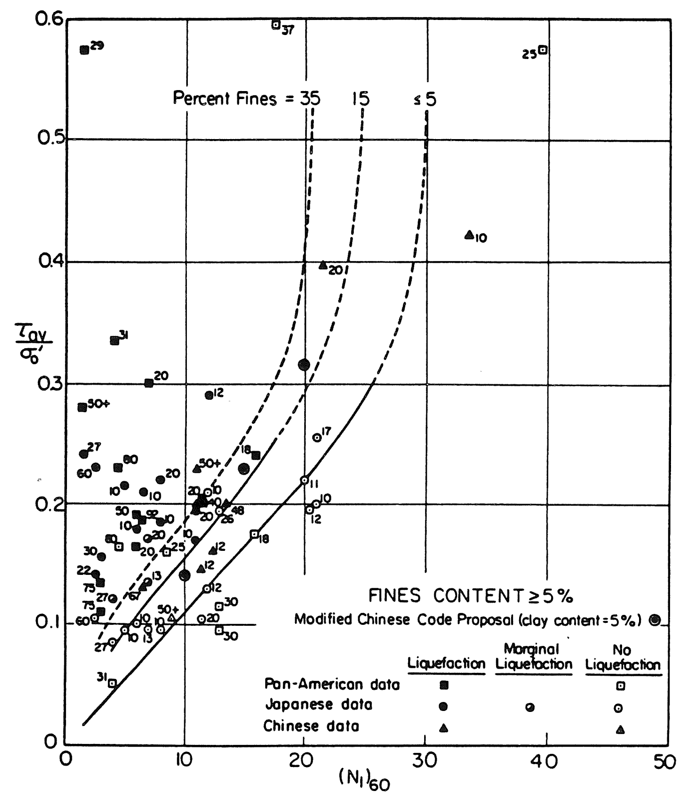


FIGURE 9. RELATIONSHIP BETWEEN STRESS RATIO CAUSING LIQUEFACTION AND $(N_1)_{60}$ VALUES FOR SANDS AND SILTY SANDS FOR M 7.5 EARTHQUAKES. (From Seed et al, 1984).

A PROBABILISTIC ESTIMATE OF LIQUEFACTION OCCURRENCE

Seed's simplifed method is deterministic in that the input earthquake characteristics are specified as single-valued numbers and the assessment provides a simple yes or no answer, or a computed factor of safety, to the possibility of liquefaction. The problem is often in the selection of a suitable "design earthquake" for the site. Increasingly, earthquake hazard is being treated probabilistically, the NBCC (1985) being but one example, so that it may be preferable to assess liquefaction potential probabilistically. A probabilistic assessment of liquefaction potential will also allow the comparison of liquefaction risk with other hazards, e.g. seismically induced structural damage, in order to provide an overall cost effective aseismic design.

When applying the results of a probabilistic seismic hazard study to a ground response or liquefaction problem, the question of earthquake duration must be addressed since the duration of shaking has a major influence on soil failure potential. The Cornell-McGuire probabilistic analysis quantifies the PGA at selected risk levels, but different magnitudes contribute differently at various levels of ground shaking. Stated another way, different magnitude earthquakes can produce the same level of shaking (PGA) at a site, depending on distance from the site. When using a probabilistically derived PGA, the question always arises as to which magnitude curve should be used in a simplified Seed procedure for liquefaction potential evaluation.

Klohn Leonoff Ltd. (1980a, 1980b) developed a simplified probability analysis which used a single seismic source zone and the Seed simplified liquefaction procedure, and considered the effects of both acceleration and magnitude. The calculations were originally performed on a TI-59 programmable calculator. Later, with the industrial sponsorship of Klohn Leonoff, the Soil Dynamics Group under Professor W.D.L. Finn at the University of British Columbia expanded the analysis into the Fortran computer program, PROLIQ2 (Atkinson et al, 1986). The program incorporates Seed's simplified method of assessing liquefaction potential into the Cornell framework, considering multiple source zones, in order to calculate liquefaction probability. To achieve this, the Cornell method is modified to consider the joint probability of occurrence of acceleration and magnitude (i.e. duration).

The theoretical formulation of PROLIQ2 is described in Atkinson et al (1986). To illustrate the methodology, the simple case of a site located inside a single, relatively large zone of homogenous earthquake occurrence is shown schematically on Figure 10. The steps involved in the analysis are as follows:

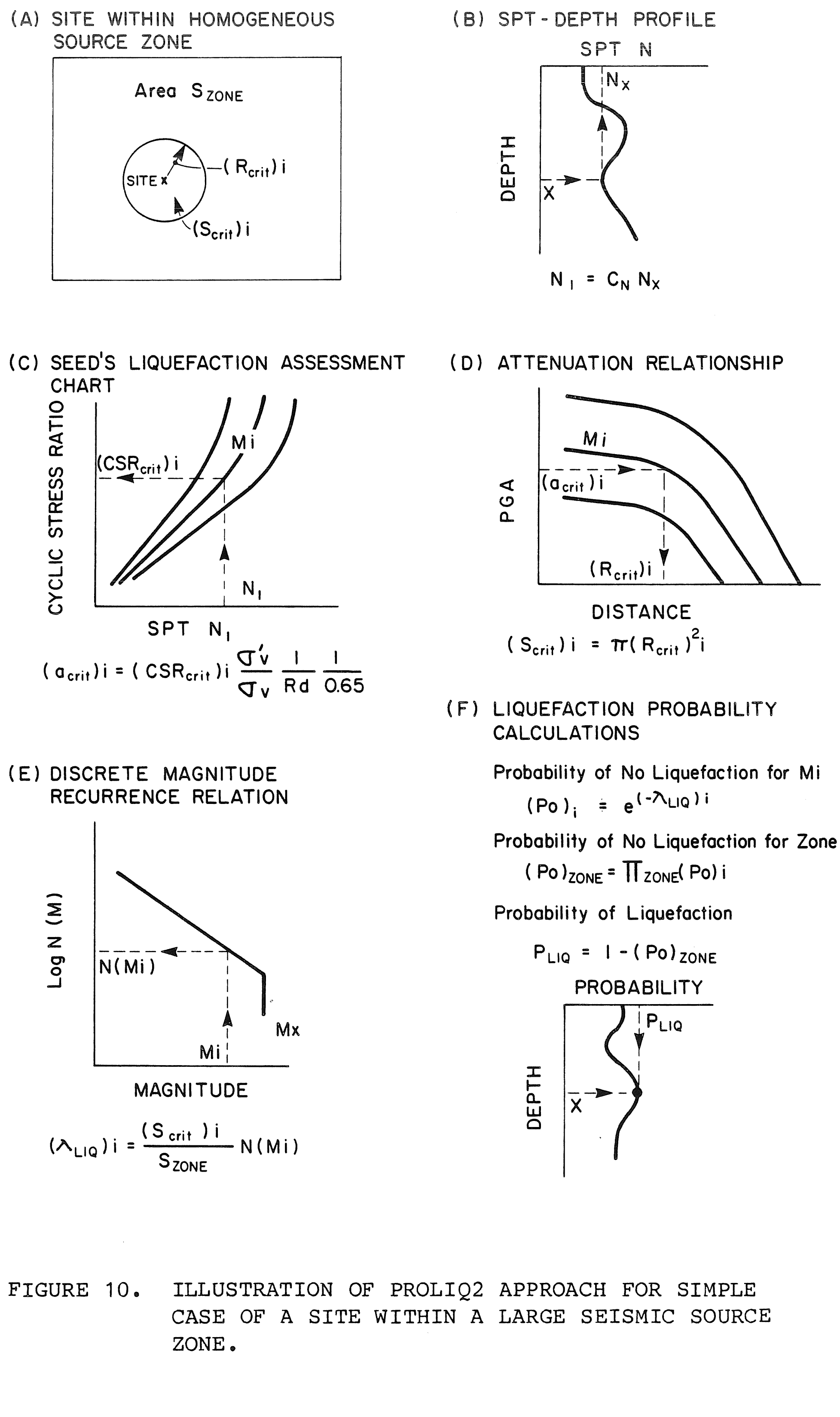


FIGURE 10. ILLUSTRATION OF PROLIQ2 APPROACH FOR SIMPLE CASE OF A SITE WITHIN A LARGE SEISMIC SOURCE ZONE.

1. Consider a site within a large homogenous seismic source zone with area S_{zone}.

2. At the sandy subsoil layer of concern, the SPT N value or values are selected and corrected to an effective overburden pressure of 100 kPa using Eqn (2). The SPT should be carried out using the "standard" procedures as outlined by Seed and Idriss (1981).

3. Each N_1 value is entered into Seed's liquefaction chart, or a similar chart defined by mean curves based on the method of least squares of misclassified points (Yegian and Whitman, 1978), to obtain the cyclic stress ratio (CSR) required to cause liquefaction for each different magnitude class, M_i. Magnitude classes from M 5.0 up to the maximum magnitude M_x considered possible for the zone are considered. For each critical CSR corresponding to a magnitude M_i, a critical acceleration (Acrit)i is calculated using Eqn (1).

4. The probability of liquefaction due to earthquakes of magnitude Mi is the probability of exceedance of (Acrit)i at the site due to all earthquakes of magnitude Mi. (Acrit)i will be exceeded at the site if an earthquake of magnitude Mi occurs within a critical distance or radius, (Rcrit)i. Thus for each (Acrit)i and Mi pair, an (Rcrit)i can be determined using an appropriate regional attenuation relationship.

5. The area around the site enclosed by (Rcrit)i is then used with the discrete magnitude-recurrence relation for the seismic zone, to calculate the annual rate of occurrence of earthquakes with magnitude Mi within the zone $(\lambda_{liq})i$.

6. Using the probability density function that derives from the assumption that earthquake occurrence is a random Poisson process, the probability of no liquefaction is calculated for each discrete magnitude Mi, and then for the entire zone $(P_o)_{zone}$. The probability of liquefaction for the zone (one or more occurrence) is, therefore, $P_{liq} = 1-(P_o)_{zone}$.

The above computation can be similarly extended to the general case of a site influenced by several seismic source zones to obtain the overall probability of liquefaction.

PROLIQ2 also allows static cone penetration test data and laboratory cyclic loading test data to be used to define the soil strength, instead of the more common SPT data. The seismicity input parameters are virtually the same as for a Cornell-McGuire analysis, being the site location, source zone geometry, magnitude-recurrence relation parameters for each zone, and the regional attenuation constants. The program outputs the probability of liquefaction for all the desired depths, and the contribution of each source zone to the overall probability.

CASE HISTORIES

LNG PEAK SHAVING PLANT, DELTA, BRITISH COLUMBIA

In 1980, Klohn Leonoff Ltd. was commissioned to carry out soil and structural response studies of a Liquefied Natural Gas peak shaving plant on Tilbury Island in the Fraser River delta, owned by British Columbia Hydro and Power Authority. At that time, the plant was already more than 10 years old and B.C. Hydro were concerned that it might not meet the new seismic design provisions of the governing code, CSA Z276 - Liquefied Natural Gas (LNG) Production, Storage, and Handling.

Extensive ground investigations and site response studies were undertaken in 1980, followed by structural analysis of the existing 39 m diameter LNG storage tank (with a storage capacity 27,000 m^3) and adjacent equipment and piping. The main conclusion arising from this work was that there was some risk of rupture of a full tank during a 'safe shutdown earthquake' characterised by a return period of 1 in 10,000 years. The ground directly beneath the tank had been treated with timber compaction piles, but the analysis indicated that the ground around the tank and supporting an earth containment berm could liquefy in a lesser, higher probability, earthquake. To improve the safety of the plant, a secondary concrete containment wall was built encircling the tank, founded on an annular zone of ground compacted by vibro-replacement as shown on Figure 11 (McGuire, 1984).

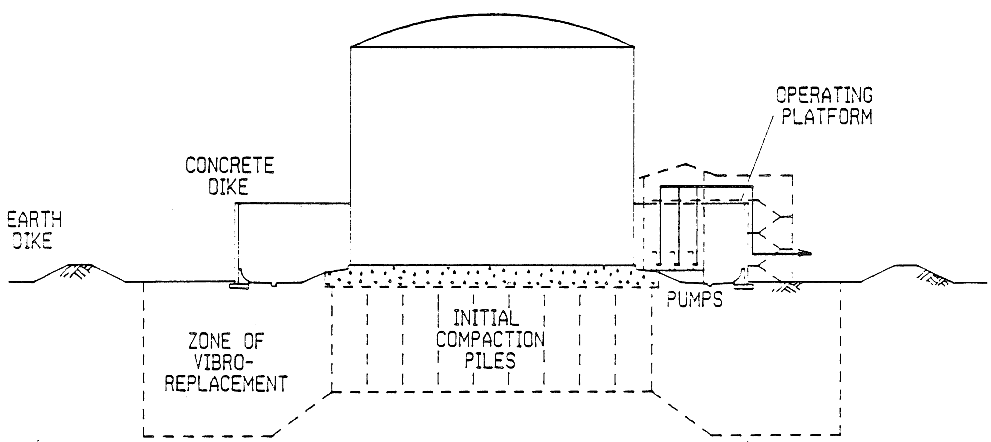
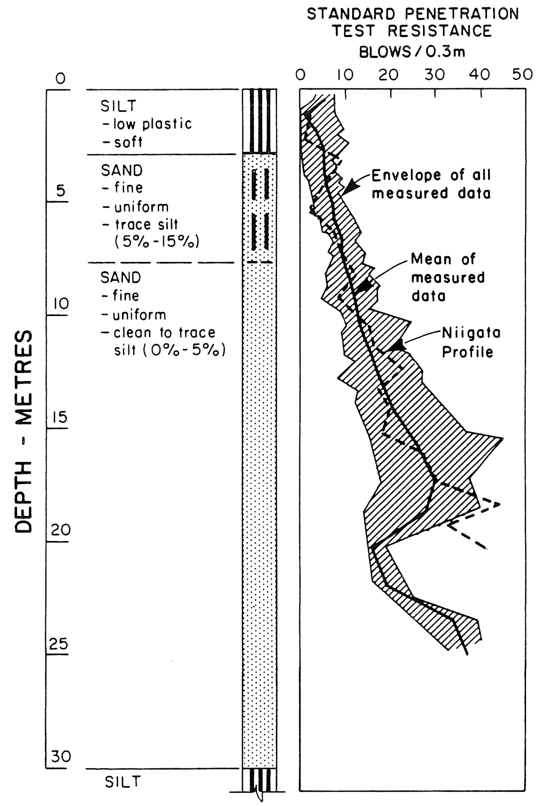


FIGURE 11. TANK, DIKES AND FOUNDATION AT LNG PEAK SHAVING PLANT, DELTA, BRITISH COLUMBIA. (From McGuire, 1984).

The field SPT data, from a total of 10 boreholes, are summarized on Figure 12. The SPT was done using a safety hammer hoisted with two wraps of rope around the cathead. Also shown on Figure 12 is a blowcount profile from Niigata, Japan (Ohsaki, 1966), which experienced extensive liquefaction in the 1964 earthquake. This SPT profile is from a site that subsequently liquefied in the 1964 earthquake, and corresponds closely to the mean of the measured values at the Tilbury Island site.

FIGURE 12.

SPT PROFILES, LNG
PEAK SHAVING PLANT,
DELTA, BRITISH COLUMBIA



In 1980, there was no clear guidance in the literature on how to evaluate silts and silty sands for liquefaction potential, or whether to be concerned about SPT energy efficiency effects. The liquefaction potential analysis was run using the Seed simplified procedure with the mean SPT profile, a mean r_d profile, and the lower bound curves from Seed (1979) for M 7.5 with a maximum ground surface acceleration of 33% g. This value was derived, using the empirical chart of Seed et al (1976), from a firm ground or bedrock PGA of 50% which was estimated from Cornell-McGuire analyses and deterministic assessment as being a reasonable value for an extreme event with a return period in the region of 1:10,000 years. This semi-deterministic analysis predicted liquefaction down to a depth of 22 m.

The solution adopted, that of ground densification by vibro-replacement, was contracted to GKN Keller Canada Ltd. in the fall of 1982, and construction was carried out in two phases in 1982 and 1983. Firstly, the surface mantle of low plastic river silt was removed from beneath the concrete containment dyke footing in an excavation and replaced by loose dumped clean sand and gravel. This sand and gravel fill and the sand beneath it to a depth of 23.5 m were densified in situ. Initially, trial patterns of compaction were made to select the probe spacing to be used, which led to an equilateral triangular grid of probes with a spacing of 2.5 m being chosen.

The specifications for compaction to be achieved were in the form of an SPT profile of blow counts with depth required at the centroid of each triangular probe pattern, as listed in Table 2. Compliance with the specifications was required to be checked by five SPT borings each, at the 25%, 50%, 75% and 100% completion milestones.

TABLE 2

SPECIFIED SPT BLOW-COUNT PROFILE

LNG PEAK SHAVING PLANT, DELTA, BRITISH COLUMBIA

DEPTH (m)	SPT N VALUE (blows/0.3 m)
3.4	24
4.9	27
6.4	29
7.9	32
9.5	34
11.0	36
21.6	36

The first set of control tests showed high blow counts, all above the specified values. Typical results are shown on Figure 13(a) for Drill Hole 8202. This was unexpected and the SPT equipment and procedures were reviewed to see if there was a problem there. At the same time, there was increasing awareness of the effects of hammer energy and hammer-anvil efficiency on the blow count value recorded in the SPT test (Kovacs and Salomone, 1982; Schmertmann and Palacios, 1979), and the UBC Civil Engineering department had acquired a Binary Instruments SPT force calibrator (Hall, 1982). The SPT calibrator is a microprocessor that measures the energy in the drill rods below the anvil, by means of a load cell, for each blow of the hammer. The drill-hammer-operator combinations being used for control testing at the site were then calibrated.

The Longyear 38 drill rig, rope and cathead hammer system and operator combination used for Drill Hole 8202 was found to deliver an average energy input of only 34% of the theoretical maximum energy of 475 J (4,200 in.lb). In fact, over 800 individual energy measurements were taken in that hole, and the average energy ratios per SPT varied from 17% to 45%. The measured blowcounts were then 'energy corrected' to a 55% efficiency level, considered to be close to the North American average at that time. These corrected values, shown plotted on Figure 13(b), failed to meet the specifications in the 3 m to 12 m depth range.

One possible reason for this was the higher silt content of the sand in this depth range inhibiting the compaction process. By this time, spring of 1982, published work by Seed and Idriss (1981) had shown how silt content in a loose sand effectively increased the liquefaction resistance, and how this fact could be simply incorporated into the empirical method of liquefaction assessment by adding 7.5 blows/0.3 m to the normalized N_1

value when $D_{50} < 0.15$ mm. This "silt correction" was applied and the corrected SPT values are plotted on Figure 13(c). For the range 0.15 mm < $D_{50} < 0.25$ mm, a correction of 4 blows/0.3 m was applied to N_1.

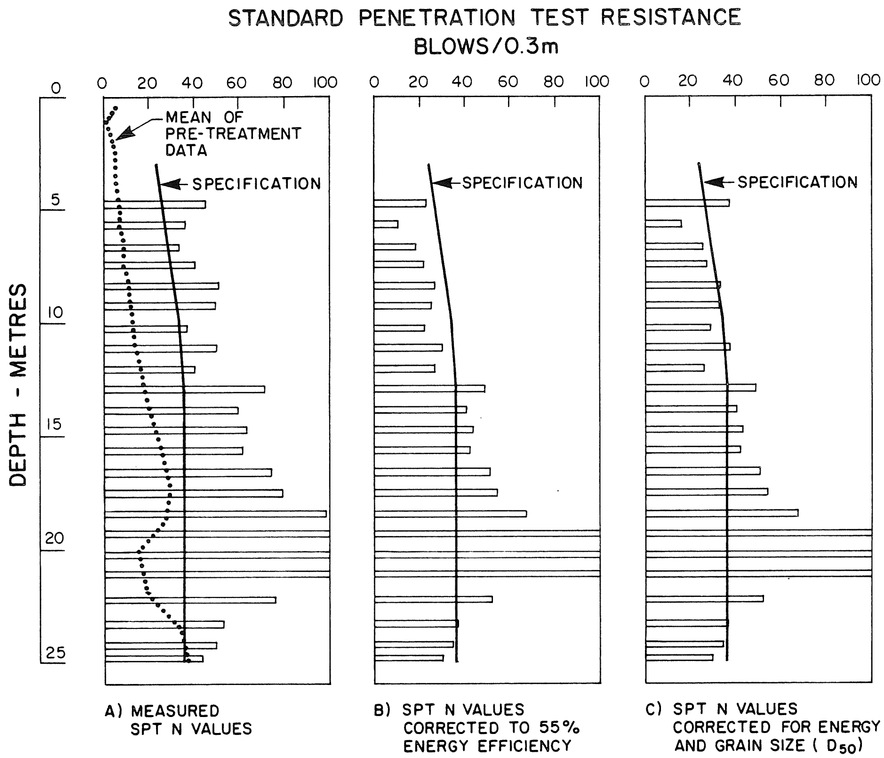


FIGURE 13. RESULTS OF SPT CONTROL TESTING - DRILL HOLE 8202, LNG PEAK SHAVING PLANT, DELTA, BRITISH COLUMBIA.

This method of interpreting the contract avoided the need for recompaction in many silty zones. Regardless of these adjustments to interpretation of the specifications, the Contractor worked hard to achieve the compaction required. Many improvements to procedures were developed, including monitoring of vibroflot probe amperage, changes to vibroflot probe extraction procedures and timing, and adjustment in the amount of crushed stone consumed. This latter was one of the main factors in achieving the required densification, and on average, the crushed stone consumption amounted to 1.4 tonnes per metre of stone column.

It is interesting, now, to re-analyze this site with the PROLIQ2 method. The results of an analysis using the mean values of SPT, and the NBCC (1985) source zone model and parameters are shown on Figure 14. Line A shows results obtained with measured SPT values, uncorrected for silt content, for comparison with the deterministic analysis of 1980. Line B results are for SPT values corrected for grain size using the D_{50} method of Seed and Idriss (1981). Either way, it is obvious that the probability of liquefaction is much higher than that of the specified design earthquake event. The PROLIQ2 results suggest that even for a conventional structure designed for an annual probability of 0.0021 (1 in 475 years), there is some risk of liquefaction ground failure.

FIGURE 14.

RESULTS OF PROLIQ2 ANALYSIS, LNG PEAK SHAVING PLANT, DELTA, BRITISH COLUMBIA

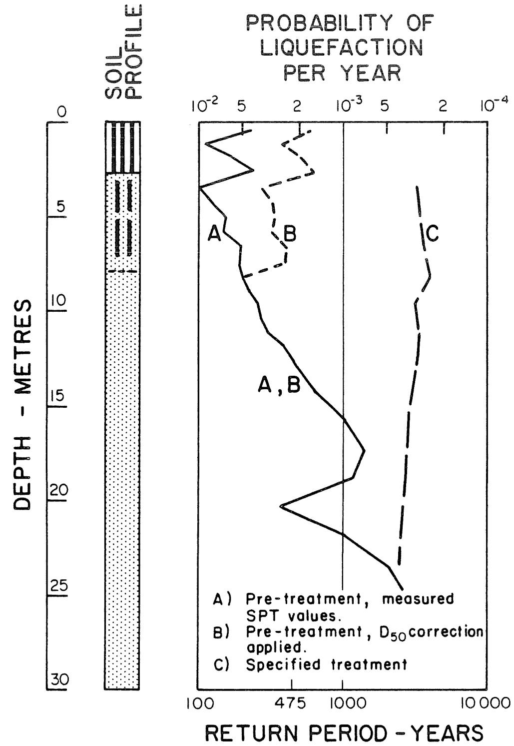


Figure 14 also shows the predicted liquefaction probabilities with the PROLIQ2 analysis using the specified blow count profile as input. The liquefaction probabilities predicted by the analysis are slightly higher than the governing code requirements for safe shutdown, but when considering other factors not included in the analysis, the real probability of damaging liquefaction is much lower. These subjective factors include:

a) the inherent problems, discussed previously, with high PGA values predicted by the NBCC (1985) Cornell-McGuire analysis at low probabilities,

b) the extra stabilizing effect of drainage due to stone columns, and

c) the 'set-up' or increase in SPT values that should be expected with time as documented by Mitchell (1986).

There is no explicit treatment of the site response in the PROLIQ2 method. It is considered that the use of the mean plus one standard deviation for attenuation incorporates all of the randomness observed, much of which is the result of differing site response due to varying geologic conditions and surface wave effects.

MANUFACTURING PLANT, TILBURY INDUSTRIAL PARK

Site investigations were carried out in 1982 and in 1983 for foundation design of a new manufacturing plant in Tilbury Industrial Park, Delta, British Columbia. The preliminary exploration in 1982 consisted of six electric cone penetration tests and one drill hole to 45 m depth with SPT sampling at 1.5 m to 3 m intervals and some shelby tube sampling in silt soils. The detailed field investigation in 1983 at the building site consisted of four drill holes to 25 m depth with sampling conducted at 1 m intervals using the SPT.

The generalized soil profile at the site is:

0	- 1.5 m	SAND FILL
1.5	- 4 m	SILT
4	- 9 m	SANDY SILT
9	- 12 m	SILTY SAND
12	- 33 m	SAND
33	- 45 m	CLAYEY SILT

The sandy deposits underlying the medium plastic native silt layer grade from fine sandy silt to fine to medium grained sand with depth, and can be conveniently classified into three substrata. The upper sandy silt zone has mean grain size, D_{50}, of less than 0.15 mm, the middle silty sand layer has D_{50} of between 0.15 mm and 0.25 mm, and the clean sand deposit has D_{50} of greater than 0.25 mm and grades from fine sand to fine to medium sand with increasing depth.

The 1982 mud rotary test hole was drilled using a Simco 2800 drill rig. A hydraulic-powered automatic trip hammer was used for the SPT. This type of hammer was known to deliver more energy to the drill rod string than the conventional rope and cathead system. Therefore, hammer blow efficiences were measured using an SPT calibrator supplied by the University of British Columbia, Civil Engineering Department. The energy measurements indicated an average energy efficiency of 85% of the theoretical maximum free-fall energy.

The 1983 mud rotary test holes were drilled using a Longyear 34 drill rig. The SPT's were performed using a donut hammer and two turns of rope around the cathead. The drill rig/operator combination was previously extensively calibrated at a nearby site in similar soil conditions and an average overall energy efficiency of 47% was measured (Robertson et al, 1983). This energy ratio was, therefore, assumed for this investigation.

The SPT N values from the 1982 and 1983 investigations, corrected to a common energy efficiency level of 55%, are shown plotted against depth in Figure 15.

An early development version of PROLIQ2 was used in this project to calculate the probability of liquefaction. Average SPT N values were used in the analysis and a surcharge, consisting of 1.5 m of sandfill placed above existing ground level, was incorporated in the analysis to simulate the proposed site grading fill to be placed during construction. The fill loading increased the overburden pressures used to calculate the cyclic stress ratio developed during an earthquake. However, corrections to the field blow count data were based on the overburden pressures before the planned fill placement. D_{50} corrections, generally as suggested by Seed and Idriss (1981), were applied to the N_1 values as follows:

$D_{50} < 0.15$ mm	Add 7.5 blows/0.3 m
$0.15 \text{ m} < D_{50} < 0.25$ mm	Add 4.0 blows/0.3 m
$D_{50} > 0.25$ mm	No correction

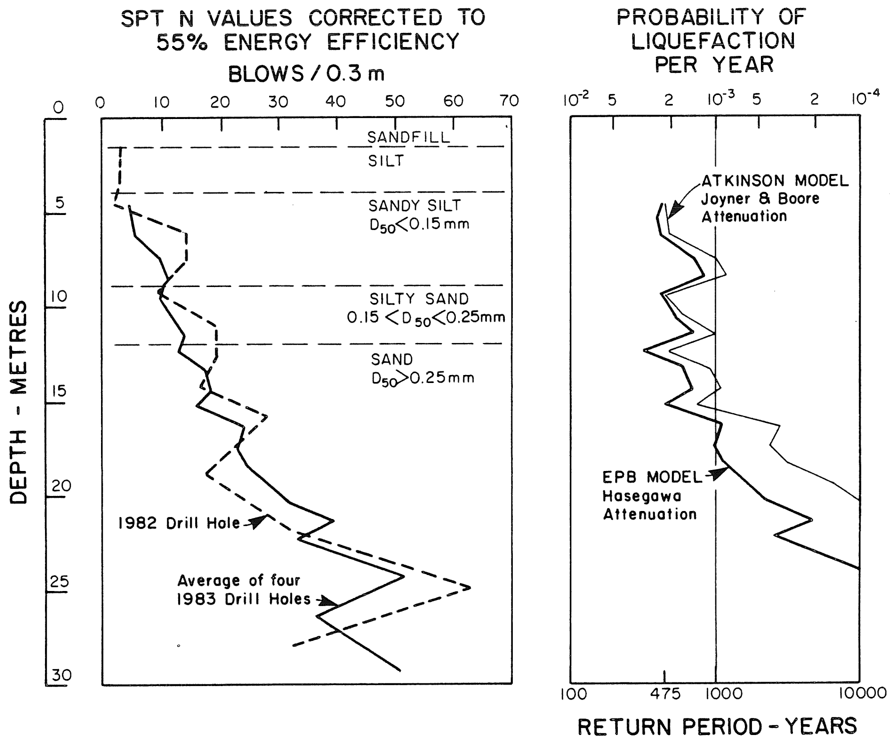


FIGURE 15. SPT PROFILES AND RESULTS OF PROBABILITSTIC LIQUEFACTION ANALYSIS, MANUFACTURING PLANT, TILBURY INDUSTRIAL PARK.

A seismogenic source zone model, as developed by Atkinson and Charlwood (1983), based on both historical seismicity and regional tectonics, and the Joyner and Boore (1981) regional attenuation relationship were used in the analysis. The computed profile of liquefaction probability versus depth is shown on Figure 15.

For comparison purposes, an analysis was performed using the PROLIQ2 program with the same soil input data but a different seismic model. The NBCC (1985) source zone model and the "western" version of Hasegawa et al (1981) attenuation were used (Basham et al, 1982). The computed probability of liquefaction curve is also shown on Figure 15. The results of the two analyses are comparable and indicate that the probability of significant liquefaction is approximately 0.002 per annum, which is equivalent to a 500 year return period.

For design of structures to NBCC (1985), an annual probability of 0.0021, or 1 in 475 year return period, is implied for structural damage. It was concluded that the probability of seismically induced liquefaction is consistent with the probability of structure damage, and that, therefore, no soil improvement was required.

SASKATCHEWAN WHEAT POOL EXPANSION, NORTH VANCOUVER, BRITISH COLUMBIA

Site investigations were carried out in 1976 at the location of the west grain storage annex at the Saskatchewan Wheat Pool site in North Vancouver, British Columbia. The annex structure is about 30 m wide by 150 m long and has an average base loading of about 330 kPa. The annex site had previously been occupied by sawmills and was partly covered with loose sandy fill. About two-thirds of the site was at Elevation 3.4 m (Geodetic datum), but one-third was below sea level. The toe of the additional fill required to expand the site was in 12 m depth of water and rested on a natural slope of loose granular soils up to 6 m thick, which extended to the bottom of an adjacent ship berth dredged to 15 m below sea level.

The field investigations consisted of several mud rotary drill holes in which SPT sampling was done, and numerous Becker drill holes including Becker penetration tests using the closed-ended 140 mm diameter casing. The Becker penetration test blow counts (blows/0.3 m) were assumed to be equal to the SPT N values. The penetration tests resistances are summarized on Figure 16.

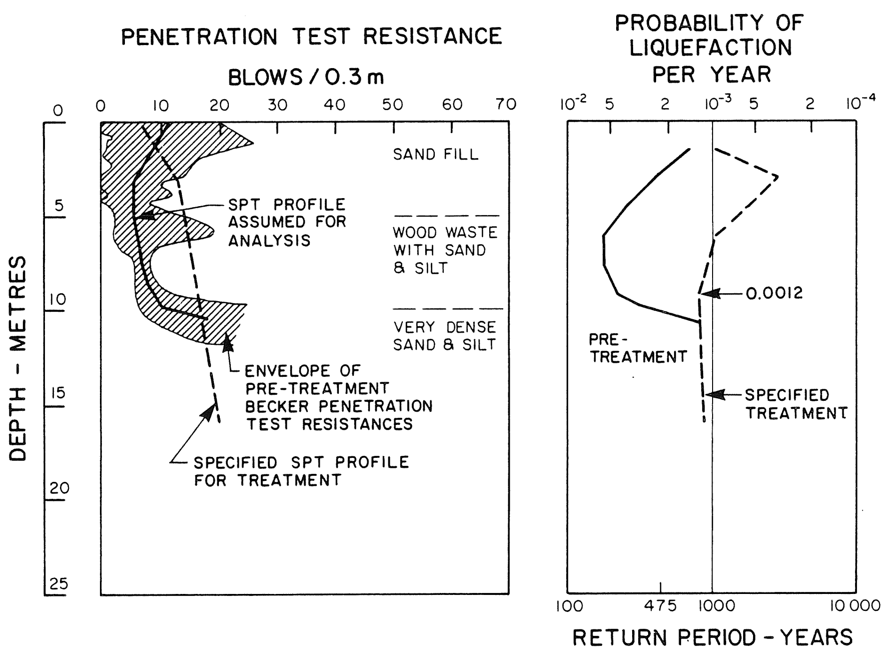


FIGURE 16. SUBSOIL DATA AND RESULTS OF PROLIQ2 ANALYSIS, SASKATCHEWAN WHAT POOL EXPANSION, NORTH VANCOUVER, BRITISH COLUMBIA

The subsoil stratigraphy varied with distance from the waterfront, but a representative profile, prior to placing additional site fill, would be 5 m thickness of loose sand fill overlying 5 m thickness of wood waste containing silt and silty fine sand. Below 10 m depth, the site is underlain by a very dense heavily over-consolidated interglacial sand and silt.

The sand and gravel fill placed to expand the building site and the existing sandy fill materials were loose, and susceptible to liquefaction. The structure was supported on cast-in-situ expanded base concrete piles bearing in the very dense subsoils. However, liquefaction of the loose soils above the bearing layer, as a result of earthquake activity, could result in a loss of 9 to 12 m depth of soil along the seaward side of the annex. The loss of ground would subject the heavily loaded foundation piles to lateral loading and loss of lateral support required to resist compressive buckling.

At the time of the investigation in 1976, the NBCC A_{100} acceleration for the site was 9% of gravity. For the assessment of liquefaction potential, design acceleration of 18% of gravity, produced by a M 7-7.5 earthquake was assumed. Deterministic analyses, based on Seed et al (1975), were first applied to define the zones of potential liquefaction, and secondly, to define the densification (in terms of SPT resistance) required to resist liquefaction under the assumed design earthquake.

The risk of liquefaction and ground loss adjacent to the seaward side of the structure foundations was reduced by vibro-replacement compaction of the loose sandy soils over a width of 21 m along the foreshore. The depth of compaction varied between 9 m and 15 m. Vibro probes were in rows at 1.7 m by 1.8 m spacing, staggered to give a triangular pattern. Minimum volume of stone placed was 20% to 30% of point area, or about 1 m diameter per probe point. After vibro-replacement, the compaction was checked by SPT tests at a number of drill holes put down at the centroids of the triangular probe pattern. The specified SPT values were achieved in the sandy soils; zones of wood waste, not considered liquefiable, were not required to meet the specified SPT resistance.

The 1976 soil test data have been re-analyzed using the PROLIQ2 program with the NBCC (1985) source zone model and the "western" version of Hasegawa et al (1981) attenuation. The computed probabilities of liquefaction, for both the pre-treatment and the specified treatment, are also shown on Figure 16. The computed probability of liquefaction for the specified treatment is 0.0012 per year, smaller than the implied current code requirement of 0.0021 per year.

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