EARTHQUAKE DESIGN IN RICHMOND

by

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ABSTRACT

The liquefaction potential of the saturated loose to medium dense sands which underlie much of the Municipality of Richmond, has been assessed for the design earthquake having a 10% chance of occuring once every 50 years. The results show that the sites with the looser sand deposits would be susceptible to liquefaction.

The effect of the deep foundation soils on the response spectra of the surface motion shows amplification of the longer period content of the motion. A design spectrum for the Richmond area is proposed.

INTRODUCTION

The Municipality of Richmond lies within the Fraser Delta area, and is located in the highest risk earthquake zone in Canada. It is a dyked low lying area sectioned by the arms of the Fraser river, and underlain by deep deposits of loose to medium dense sands and silts.

In other high seismic risk areas where the foundation soils were comprised of water saturated loose to medium dense sands and silts, very severe earthquake damage has resulted. Tall buildings located on such soils may sink into the ground and tip over if the soil liquefies. This occurred at Niigata, Japan in 1964 during an earthquake of M7.3 (Magnitude 7.3 on the Richter Scale). The foundation soils at Niigata were comprised of deltaic deposits similar to those of the Fraser Delta. Such liquefaction is not restricted to Japan but has occurred during many earthquakes including the 1906 San Francisco earthquake, the 1946 British Columbia earthquake, the 1964 Alaska earthquake and the 1971 San Fernando, California earthquake.

Earthquake motions on rock or firm ground have a predominantly low period. If, however, the earthquake motion travels up through hundreds of metres of soft sediment as it would in the delta, its predominant period may increase dramatically. For buildings on rock sites, short stiff buildings of 1 or 2 storeys could be in quasi-resonance with the earthquake motion. However, in the delta it would be the tall buildings of 10-20 storeys that could be in quasi-resonance with the earthquake motion. Such resonance led to very severe damage to hi-rise buildings in Caracas, Venezuala during the earthquake of 1967.

Buildings in Canada must be designed under the National Building Code of Canada. However, because the underlying soil conditions in Richmond are poor from the earthquake point of view, special attention is required in earthquake design that is not adequately covered by the code. The two main concerns are: (1) the liquefaction potential of the underlying soils, and (2) possible quasi-resonance effects on hi-rise buildings due to a shift in the predominant period of the earthquake motion. These two items and their implication for design are addressed in some detail in this paper.

This paper is a summary of a report prepared for the Township of Richmond, B.C. The full report is available from the Richmond Building Department, and as a Soil Mechanics Series Report from the Department of Civil Engineering, U.B.C.

MECHANISM OF LIQUEFACTION

Loose to medium dense sandy soils are quite stable under static or normal loading conditions, and structures founded on such soils can be expected to behave in a satisfactory manner. However, during a severe earthquake, the shaking causes such materials to compact or densify. If the material is not water saturated, such compaction leads to vertical movements or settlements. The soil suffers no strength loss and the movements are generally relatively small. However, if the material is saturated, compaction cannot occur because the water is essentially incompressible and cannot escape from the soil pores during the short period of shaking, and so the load on the soil is transferred from the sand grains to the water with a resulting increase in porewater pressure and drop in strength. If all the load is transferred to the water, the soil loses all of its strength and behaves like a liquid and is said to have liquefied. The high water pressures can lead to expulsion of water and sand at the ground surface in the form of miniature volcanoes and the loss of strength can result in large movements of structures and services founded in or above the liquefied zone.

The magnitude of the movements resulting from liquefaction depend on the static driving forces, and the density of the soil. If the ground surface is level, then although the sand may liquefy, the movements will be small because the driving forces are essentially zero. On sloping ground or where building loads occur, the magnitude of the movements depend primarily on the density of the sand. Loose sands which liquefy regain little strength as shearing due to the static driving forces takes place, and hence can essentially flow as a viscous fluid. Denser sands, while they may initially liquefy under the earthquake shaking tend to expand or dilate under the shearing action of the static load, and this causes a drop in porewater pressure and a gain in strength. If the shearing causes a strength regain that is equal to the static shear stress, then a limited movement rather than a flow slide occurs. Such movement is referred to as lateral spreading and would normally only occur during the period of strong shaking.

The density of the sands and silts is thus a key factor governing their earthquake behaviour. If the sands are loose, then they are readily liquefied and resulting movements may be large, whereas if they are dense they are difficult if not impossible to liquefy and resulting movements will be small. A measure of density or liquefaction resistance can be obtained from the standard penetration test, (SPT) in which a standard sampling tube is driven into the soil with a standard hammer. The number of blows to drive the tube 0.3 metres is termed the Standard Penetration value, N. There are other more sophisticated measures of the liquefation resistance of soil, however, because the standard penetration test has been carried out at many sites in the Fraser Delta and because it is presently the most widely accepted measure of liquefaction resistance, it will be used throughout this report.

SOIL CONDITIONS IN THE FRASER DELTA

The geology of the Fraser Delta has been described in some detail by Blunden 1973 and Wallis, 1979. Basically the area is underlain by:

1. A surficial deposit comprised of a thin discontinuous veneer of clays, silts and peats up to 8 metres in thickness, underlain by
2. sand deposits to a maximum thickness of 45 metres, underlain by
3. silt-clay deposits up to 200 metres in thickness, underlain by
4. glacial deposits up to 100 metres in thickness, underlain by
5. bedrock at depths in the range 200 to 700 metres.

The deposits of concern from a liquefaction point of view are the sands of medium grain size that underly the surficial layer in the typical depth range of 3 to 30 metres. A detailed study of N values of these sands in the region was undertaken by Wallis (1979). Recent additional data, which includes cone penetration data, has been reviewed and confirms the study by Wallis (1979). The density of the sand deposits vary significantly with location and depth, and no well defined areas or zones of similar sand density could be identified with the available data. It is important, therefore, that each site or area be treated on an individual basis.

The range in N values from various sites in Richmond are shown in Figure 1. At each site the mean value of N at various depths was calculated. Figure 1 then shows the average value of the mean values, as well as the high and low mean values. A detailed study of the N values at any one site at any depth indicates that they range between one half and twice the mean value. A lower bound for N values in Richmond is therefore one half the lower mean value and this lower bound is also shown. The liquefaction resistance of the sand depends on the N value corrected or normalized to a confining stress of 1 Atmosphere and termed N_1.

DESIGN EARTHQUAKE

The lower mainland area of British Columbia, which includes Richmond, is in an active earthquake or seismic region. Since the year 1872, which is the beginning of the historial record, 11 earthquakes with Richter magnitudes in the range 5-1/2 to 7-1/2 have occured in the region.

Buildings in Canada are generally designed to satisfy the current National Building Code of Canada [NBCC (1980)]. This code requires that structures be able to survive a level of seismic activity, expressed as a peak horizontal ground acceleration. This is achieved by requiring that the structures have a lateral strength sufficient to resist the forces induced by a specific horizontal acceleration or shaking level, together with an ability to deform plastically. It is envisioned that if the structure is subjected to such seismic forces, it would be heavily damaged, and may have to be torn down, but that it should not collapse, and loss of life would be minimal. NBCC (1980) uses the peak ground acceleration that would be exceeded with a probability of 1/100 per annum, termed the 100 year acceleration and denoted by A_{100}, as a means of dividing the country into seismic zones. It also uses A_{100} in the formulas used to calculated the design lateral forces. However, in these formulas there are other factors and it has been shown (3) that the NBCC (1980) formula implicitly results in buildings designed to resist seismic activity with a probability of exceedence of 1/200 to 1/500 per annum (the A_{200} to A_{500} accelerations). This has led to confusion amongst the engineering community. Structural engineers following the code, knowingly or unknowingly design for A_{200} to A_{500} accelerations, while soils and foundation engineers have no such guidelines in the code and commonly design for the A_{100} acceleration.

Recently it has been reported (4) that CANCEE (Canadian National Committee on Earthquake Engineering) has adopted the A_{475} acceleration level as one measure to be used for future Canadian seismic zoning. The use of A_{475} corresponds to a 10% probability of exceedence in 50 years, which is deemed a realistic risk level for structures. The A_{475} acceleration will also be used in formulas to calculate the design lateral forces, with other factors in the formulas calibrated so that the proposed new code is essentially equivalent to the present code. The implication is that structures should be designed so that they will not collapse when subjected to the A_{475} acceleration, and consequently this acceleration has been used as the design acceleration in this study.

The seismicity of the area has been examined by Klohn Leonoff Consultants Ltd. The A_{475} acceleration, which is the expected peak acceleration on rock or firm soil, is reported to be 0.19g. This agrees well with the 0.20g that has been proposed for Vancouver (4). The Richter magnitude of earthquakes that might produce such an acceleration range from M5.5 to 7.5.

SURFACE MOTION OF DESIGN EARTHQUAKE

Earthquake induced shaking is caused by stress waves propagating in the rock crust. Where deep soil deposits overlay the rock as in the Fraser Delta, the motion is then carried up to the surface by shear waves propagating upwards through the soil. Because the soil is softer than the rock, it tends to amplify the long period components of the rock motion.

In determining the expected surface motion at deep soil sites there are at least three methods that can be used:

1) scaling of surface motions recorded at the site, if available;
2) calculation of response using rock motions input at the base of the soil;
3) scaling of surface motions recorded at other but similar sites.

In this study methods 2 and 3 were used to determine the design surface motions . The expected peak acceleration of the surface motion is 0.17g, a slight reduction from the base rock acceleration of 0.19g. These motions were used to compute the surface design spectra and this is discussed in a later section.

In addition to surface response, method 2 also yields the dynamic stresses within the soil required for liquefaction evaluation, and these are now discussed.

DYNAMIC STRESSES IN FOUNDATION SOIL

The dynamic analysis of the foundation soil yields the time history of earthquake induced shear stresses for the design earthquake. At any depth, these stresses vary in a random fashion with time, and it is common for liquefaction analysis purposes to replace this random pattern with an equivalent number of uniform stress cycles. An equivalent shear stress τ_{eq}, equal to 0.65 times the maximum computed shear stress is generally chosen and the number of cycles or pulses of such a stress level depend on the magnitude of the earthquake considered. For an M6 earthquake, 5 cycles are appropriate whereas for an M7.5 earthquake, 15 cycles are appropriate (5).

From the point of view of both deformation and possible liquefaction, it is the dynamic stress ratio τ_{eq}/σ'_o, in which σ'_o equals the effective overburden pressure, that is of interest. The variation of this ratio with depth for the design earthquake is shown in Figure 2. It is these stress pulses that may cause liquefaction of the sand and this will be considered in the next section.

ASSESSMENT OF LIQUEFACTION POTENTIAL

The liquefaction potential of the delta is assessed herein by the method proposed by Seed (6), and Seed and Idriss (5). Seed's method is widely used in North America and incorporates a chart, shown in Figure 3, that was obtained by examining a number of sites at which liquefaction had and had not occurred and where both the shaking levels and the normalized standard penetration test N_1 values were known. Points that liquefied are shown as solid circles while points that did not liquefy are shown as open circles. The solid line is a lower bound below which liquefaction was not observed to occur. The chart was obtained for earthquakes of about M7.5 and for sands of medium grain size and so is appropriate for the design earthquake and sand deposits under consideration. The chart therefore allows an assessment of liquefation to be made if the N_1 values and shaking levels are known.

At any depth the dynamic stress ratio τ_{eq}/σ'_o can be determined from Figure 2, and the required N_1 to prevent liquefaction determined from Figure 3. These N_1 values are then plotted in Figure 4 as the solid line. Also plotted in Figure 4 are the mean values of N_1 found in the Fraser Delta. The results indicate that for the average mean and lower mean soil conditions, liquefaction to depths of 9m and 16m respectively are predicted to occur for the design earthquake.

The liquefaction potential was also assessed using a method developed by Japanese researchers and yielded results similar to Seed's method above.

The Fraser Delta sands, and those of Niigata Japan where severe liquefaction damage occurred in 1964, are compared in Figure 5. It may be seen that based upon average mean N_1 values the Fraser Delta sand profile has N_1 values that are higher than those of the heavily damaged zone of Niigata and lower than those of the lightly damaged zone. However, based upon the lower mean values, the Fraser Delta sands have lower N_1 values than the heavily damaged zone at Niigata. This indicates that significant areas of the delta could suffer severe damage in the event of an earthquake of severity comparable to that which occurred at Niigata. (A_{gmax} = 0.16g).

SURFACE RESPONSE SPECTRA

For structural design purposes the surface acceleration response spectrum is the most widely used measure of seismic shaking. As mentioned previously there are at least three methods that can be used to determine the surface motion. In Figure 6 acceleration spectrum of the calculated surface motion and the acceleration spectrum of scaled motions recorded at other sites (7) are presented. Also plotted is the spectrum of the NBCC multiplied by the suggested soil factor of 1.5, for a peak ground acceleration of 0.19g. The calculated and scaled response are the mean plus one standard deviation values, as is the NBCC spectrum.

The data indicates that the NBCC spectrum is overly conservative for periods less than 0.8 seconds, but non-conservative for periods greater than 0.8 seconds. This is consistent in that the deep soft soil layers damp out the low period motions but amplify the longer period motions. The solid line in Figure 6 is the recommended design response spectrum for the Richmond region.

STRUCTURAL DESIGN CONSIDERATIONS

The recommended design response spectrum shown in Figure 6 is based upon the 475 year earthquake and reflects the increased longer period respose associated with this level of shaking in the deep soil deposits present in the Fraser Delta. Since the present code uses the A_{100} acceleration in conjunction with various multipliers to calculate the seismic forces, the proposed spectrum cannot be used directly with the present code. It is suggested that if a dynamic analysis using the response spectrum approach is used to calculate the seismic forces, then for buildings with a fundamental period (T) greater than 0.8 seconds the response spectrum given in Figure 6, divided by a factor of two, should be used in place of the response spectrum presented in the NBCC. Note that Figure 6 includes the soil amplification factors, and should be used in conjunction with $F = 1.0$. For buildings with $T < 0.8$ seconds, it is suggested that the present code spectrum be used, along with the foundation factor of $F = 1.5$.

If the quasi-static seismic analysis method is used, it is suggested that for structures with $T < 0.8$ seconds, the present code should be used. For structures with $T > 0.8$ seconds, the forces calculated using the presnt code should be increased by 50% for $T > 1.2$ seconds. For $0.8 < T < 1.2$ seconds a linear interpolation from zero increase at $T = 0.8$ seconds to a 50% increase at $T = 1.2$ seconds should be made. A foundation factor of $F = 1.5$ should be used in all the quasi-static analysis.

CONCLUSIONS

Based on a probability of exceedence of 10% in 50 years it is shown that there is a possibility of liquefaction in the loose saturated sands prevalent in the Richmond area of the Fraser Delta. However since the sands are so variable it is important that each site or area be treated on an individual basis.

The deep layers of loose to medium dense sands and silts amplify the longer period motions of earthquakes, resulting in larger forces on buildings with fundamental periods exceeding 0.8 seconds than would be predicted using the present National Building Code of Canada. Design guidelines for structures are given.

ACKNOWLEDGEMENTS

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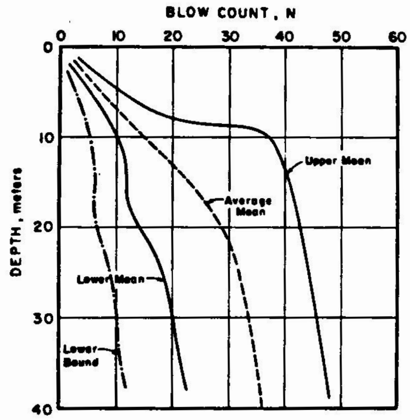

FIG. 1
PENETRATION RESISTANCE PROFILES FOR
FRASER DELTA SANDS

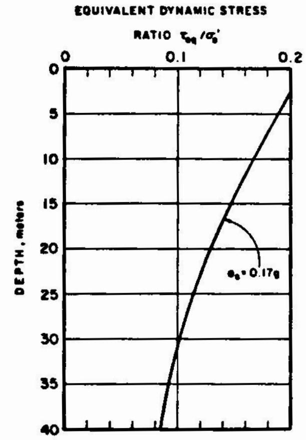

FIG. 2
EQUIVALENT DYNAMIC STRESS RATIO
PROFILE FOR FRASER DELTA

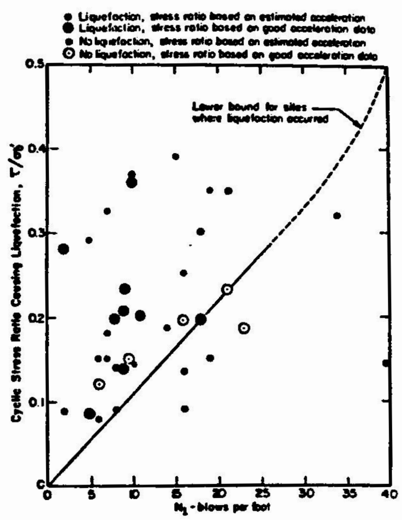

FIG.3 CORRELATION BETWEEN STRESS RATIO CAUSING
LIQUEFACTION IN THE FIELD AND PENETRATION
RESISTANCE OF SAND (After Seed et al. 1975).

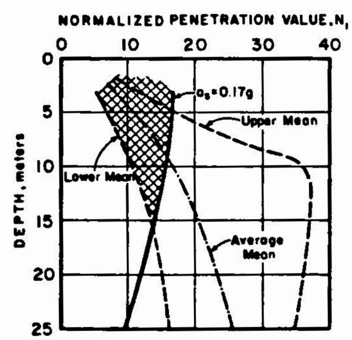

FIG. 4
LIQUEFACTION POTENTIAL OF FRASER DELTA SANDS,
SEEDS METHOD

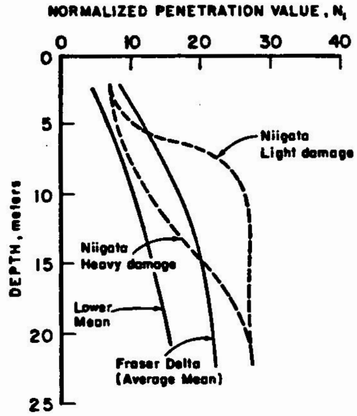

FIG. 5

PENETRATION RESISTANCE COMPARISON OF FRASER DELTA AND NIIGATA SANDS.

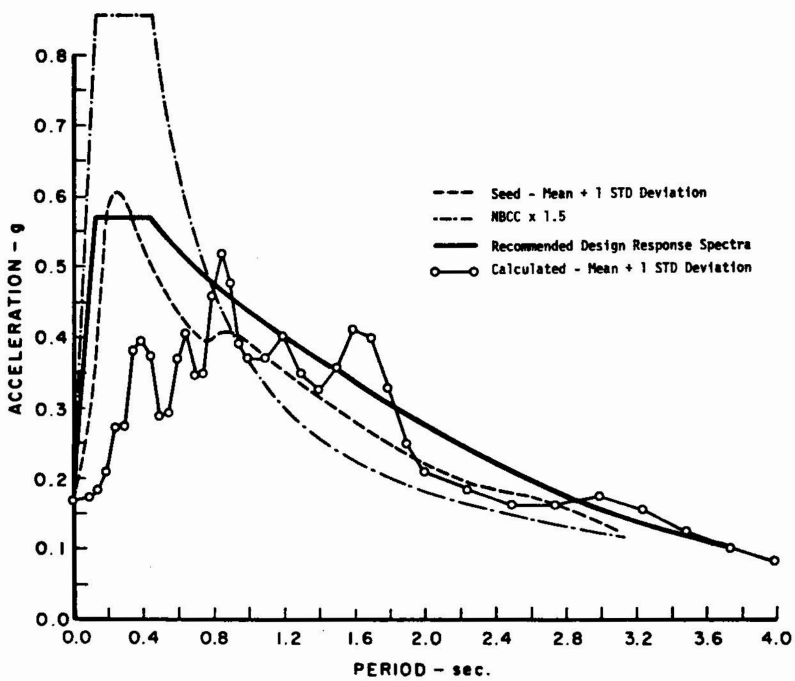

FIG. 6 RECOMMENDED DESIGN ACCELERATION RESPONSE SPECTRA, 5% DAMPING.