Proceedings of Ninth World Conference on Earthquake Engineering
August 2-9, 1988, Tokyo-Kyoto, JAPAN (Vol.III)

4-2-3

# SEISMIC AND GEOTECHNICAL ASSESSMENTS OF A PROPOSED RADIOACTIVE WASTE DISPOSAL SITE

K. Tim Law[1] and Jas S. Devgun[2]

1 Institute for Research in Construction
National Research Council of Canada
Ottawa, Ontario, Canada K1A 0R6

2. Chalk River Nuclear Laboratories
Atomic Energy of Canada Ltd.
Chalk River, Ontario, Canada

## SUMMARY

An Intrusion-Resistant Underground Structure for a low- and intermediate-level waste disposal facility is proposed for a sand site in Chalk River. The seismic conditions and liquefaction potential at the site have been studied. The study shows that, for a return period of 500 years, the design earthquake may be characterized by a peak horizontal ground acceleration of 0.167g and magnitude of 6.5. The sand layer is normally unsaturated and as such is stable. Should the sand layer at the footing level become saturated, the safety factor against liquefaction failure is 1.06. Densification of this layer is recommended.

## INTRODUCTION

The Chalk River Nuclear Laboratories (CRNL) have initiated a program to convert the site's radioactive waste storage operation to disposal. One concept that is being pursued is an Intrusion-Resistant Underground Structure (IRUS) for low- and intermediate-level wastes with hazardous lifetime $\leq$ 500 years. A schematic diagram of this facility is shown in Figure 1. It consists of a concrete-walled trench 100 m long, 20 m wide and 8 m deep (Ref. 1). The top of the facility will be covered with a reinforced concrete cap. Additional water-shedding barriers may be added on top of the cap and a soil and vegetation cover will be placed on the top. The facility will be based above the normal water table in unsaturated sand deposits. The bottom of the facility will be kept permeable to allow rapid draining of any water that may get into the facility.

Many technical and socio-economic factors are being considered in the site characterization program. One factor is the liquefaction potential of the sand under earthquake conditions that prevail at the site. This paper describes a study on the seismic and geotechnical considerations with regard to liquefaction potential.

## SEISMIC ASSESSMENT

Historical Seismicity Canadian seismic records are maintained by the Geophysics Division of Geological Survey of Canada (GDGSC). Canada is divided into five seismological regions: Eastern, Central, Western, Northern and St. Elias. For the Eastern region, further subdivision into zones were made by Basham et al. (Ref. 2) based on historical seismic data. Chalk River lies in the southern part of the Western Quebec zone (WQU) in this region.

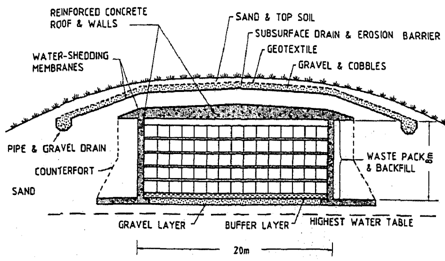



FIGURE 1

SCHEMATIC CROSS-SECTION OF IRUS

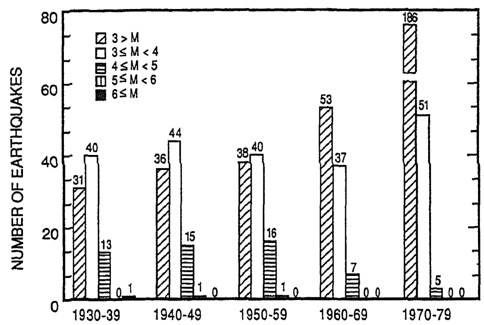
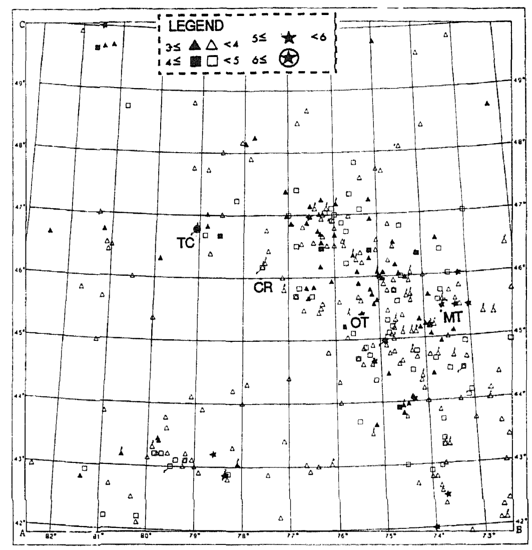



FIGURE 3

EARTHQUAKE DATA 1930 -1979 IN TERMS OF TEN YEAR TIME INTERVALS. (800 km BY 800 km SQUARE REGION WITH CHALK RIVER AT CENTRE)




(Scale : Distance AB = 800 km, AC = 800 km; the number attached to a mapped symbol refers to the number of events at that place; Reference Points CR = Chalk River, TC = Témiscaming, OT = Ottawa, MT = Montreal

FIGURE 2

PLOT OF ALL KNOWN EARTHQUAKE SINCE 1661 WITH M ≥ 3.0 FOR CHALK RIVER REGION (PROVIDED BY GD GSC)

Figure 2 shows a composite plot of all the earthquakes from 1661 to 1985 with a magnitude (M) > 3 for an area of 800 km x 800 km with Chalk River at the centre. This plot is based on the digital listing of all catalogued events supplied by GDGSC. The earliest historical earthquakes in this list are based on the assessments of available records. The felt reports of earthquakes during the first two and half centuries were dependent on the existence of earliest settlements in particular areas. Only after about 1927 were a significant number of earthquakes being located instrumentally. During the past 30 years the lower magnitude instrumental coverage has been available in parts of eastern Canada (Ref. 2).

Examination of the data for the area in Figure 2 shows a relatively high level of seismic activity near Chalk River. Most of the earthquakes have M < 4. Some large earthquakes, however, have occurred in this area. These include: the 1732 earthquake near Montreal (M ≃ 6), the 1935 Temiscaming earthquake (M = 6.2), the 1944 Cornwall earthquake (M = 5.6), and the 1983 earthquake in the Adirondacks area of northern New York State (M = 5.6). Since 1661, there have been 20 documented events with M > 5, 104 with M > 4 and 459 with M > 3.

Data for the period 1930 to 1979 have been plotted in a different way (Fig. 3) to show the frequency of events over ten year time intervals. It appears that the pattern of seismic activity for earthquakes with M < 5 is relatively constant. The last interval (1970-1979) shows a large number of microearthquakes because complete coverage of such events was available only since 1968.

Probabilistic Seismic risk  Seismic risk is normally defined as the per annum probability of occurrence or exceedence of any given value of parameter. In the study of liquefaction of granular soil, two parameters are required: the peak horizontal acceleration, $a_{max}$, and the earthquake magnitude, M.

The evaluation of seismic risk in terms of $a_{max}$ for this region is described in Ref. 3. Briefly, the methodology involves several steps. 1) Define the earthquake source zones based on the historic and recent seismic data and any geologic or tectonic evidence that can be used to constrain the probable extent of future seismic activity. 2) Determine the magnitude recurrence relation that gives the cumulative number of earthquake exceeding a certain magnitude. 3) Choose an appropriate attenuation relation to calculate $a_{max}$ as a function of magnitude and distance. 4) For any location, a distribution function for the probability of exceedance is computed by numerical integration of contributions from all relevant source zones. This methodology was applied to Chalk River and the results are shown in Table I.

Table I. Seismic Risk of Peak Horizontal Acceleration, $a_{max}$, for Chalk River

| | | | | |
|---|---|---|---|---|
| Probability of exceedence per annum | .01 | .005 | .002 | .001 |
| $a_{max}$ (g) | .072 | .105 | .167 | .241 |

(Source: Geophysics Division, Geological Survey of Canada)

Since the hazardous life of the waste is 500 years, the design $a_{max}$ for the IRUS is taken equal to .167g, which corresponds to a return period of 500 years.

As the maximum magnitude recorded in this zone is 6.2 from the Temiscaming earthquake, it is prudent to consider the return period of a 6.5 M earthquake that will give rise to $a_{max}$= .167g in Chalk River. This can be determined using the recurrence relation for this zone by Basham et al. (Ref. 3) and the attenuation equation for eastern Canada by Hasegawa et al. (Ref. 4). The result shows that such an earthquake has a return period of about 1850 years. This is less than the return period of $a_{max}$= .167g and M = 6.5 is used for the ensuing study on liquefaction potential of the sand deposit.

## GEOTECHNICAL ASSESSMENT

Test Program  The test program included piezocone penetrometer tests and sampling of the sand from the site. The piezocone penetrometer was fitted with a 60° tapered, 10 $cm^2$ cone. The pore pressure was measured through a 4 mm thick cylinder porous filter located immediately above the cylindrical tip. As most of the tests were conducted in unsaturated sand, no pore pressure was measured until reaching the water table. A total of 3 piezocone soundings were performed at the proposed site for the IRUS.

Soil Profiles  The soil profiles at the site were deduced from the piezocone data using the interpretation of Douglas and Olsen (Ref. 5). The results are summarized in Table II. Briefly, the subsoil consists of a loose to compact fine sand overlying a compact to dense silty sand. The thickness of the fine sand layer varies with the surface elevation: 8.5 m thick at the highest location C1 and 5.2 m thick at the lowest location C3.

Relative Density  Based on a comparison of mineralogy and grain shape, the Chalk River sand is closest to the Monterey sand studied by Villet and Mitchell (Ref. 6). Therefore, their correlation curves for relative density have been used and are shown in Figure 4 for C1. It appears that the top loose to dense fine sand has a relative density in the neighbourhood of 40%.

Table II. Soil Profiles Determined from Piezocone Tests

| Location | Surface Elevation m | Depth m | Soil Type |
|---|---|---|---|
| C1 | 168.66 | 0 - 5.5 | Loose to compact sand |
| | | 5.5 - 8.5 | Loose sand |
| | | 8.5 - 10.5 | Compact to dense, fine to silty sand |
| C2 | 167.51 | 0 - 3 | Compact sand |
| | | 3 - 6.5 | Loose to compact sand |
| | | 6.5 - 13.0 | Compact to dense, fine to silty sand |
| C3 | 165.20 | 0 - 3 | Loose to compact sand |
| | | 3 - 5.2 | Loose sand |
| | | 5.2- 10.5 | Compact to dense, fine to silty sand |

Liquefaction Potential The piezocone tip resistance $q_c$ can be used to estimate the liquefaction potential of saturated granular material as proposed by Seed et al. (Ref. 7). This method is based on the study of many earthquakes with $M \simeq 7.5$. Extension to other earthquake magnitudes is possible by multiplying by a correction factor, $\mu$, which is determined from laboratory cyclic triaxial tests and by considering the effective durations associated with different M. The method has two main steps. First, a measurement of the resistance at liquefaction failure, or liquefaction resistance ($\tau_\ell$) is made. Secondly, the dynamic stress ($\tau_h$) corresponding to a design peak horizontal acceleration ($a_{max}$) is computed. The liquefaction resistance is then compared with the dynamic stress and when the latter exceeds the former, liquefaction will occur.

The liquefaction resistance is commonly expressed in terms of a shear stress ratio, $\tau_\ell/\sigma'_{vo}$, where $\sigma'_{vo}$ is the effective overburden pressure. This ratio can be expressed as:

$$\tau_\ell/\sigma'_{vo} = \mu\frac{N_1}{90} \text{ (for } N_1 < 25) \qquad (1)$$

where $N_1$ = modified standard penetration resistance
= $C_N \cdot N$
$N$ = standard penetration resistance in blows per foot
$C_N$ = a function of $\sigma'_{vo}$ at the depth at which N is measured (Ref. 7)

Based on cyclic triaxial tests on this sand, the value $\mu$ is practically identical to that given by Seed et al. (Ref. 7) as listed in Table III.

Table III. Correction Factors for Different Earthquake Magnitudes

| | | | | |
|---|---|---|---|---|
| Earthquake magnitude, M | 7.5 | 7.0 | 6.25 | 5.25 |
| Correction factor, $\mu$ | 1.0 | 1.13 | 1.32 | 1.5 |

N can be converted from $q_c$ using:

$$N = q_c / C \qquad (2)$$

where C is a coefficient dependent on the mean grain size, $D_{50}$. Based on the work of Robertson et al. (Ref. 8), C is equal to 0.45 MPa for the type of sand studied here.

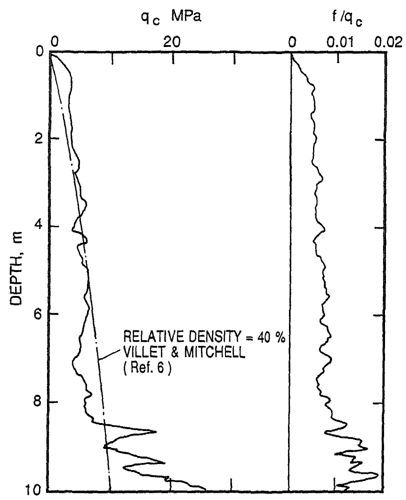



FIGURE 4

CONE TIP RESISTANCE ($q_c$) AND FRICTION RATIO ($f/q_c$) AT LOCATION C1, CRNL, CHALK RIVER

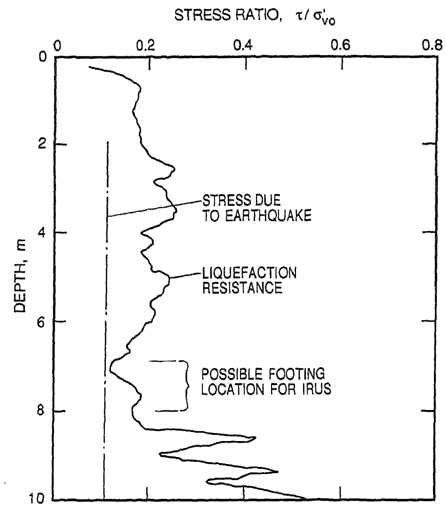



FIGURE 5

LIQUEFACTION POTENTIAL AT LOCATION C1, CRNL, CHALK RIVER FOR M = 6.5, $a_{max}$ = 0.167 g

From Equations (1) and (2), one obtains

$$\tau_\ell/\sigma'_{vo} = \frac{C_N q_c}{40.5} \qquad (3)$$

where $q_c$ is in MPa.

The dynamic stress can also be expressed in terms of a ratio, $\tau_h/\sigma'_{vo}$, given by Seed and Idriss (Ref. 9) as:

$$\tau_h/\sigma'_{vo} = 0.65 \frac{a_{max}}{g} \frac{\sigma_v}{\sigma'_{vo}} r_d \qquad (4)$$

where $\sigma_v$ = total overburden stress
$r_d$ = a stress reduction factor = 1-0.015 z
and z = depth in m.

The dynamic stress and liquefaction resistance at location C1 are compared in Figure 5 for the design condition of $a_{max}$ = 0.167g and M = 6.5. The comparison suggests that the sand at or below the IRUS footing level will not liquefy. However, the safety factor against liquefaction failure is only 1.06. In order to increase the safety factor under such conditions, densification of the loose sand at and below the footing level is required.

It should be emphasized that the sand will only liquefy when it is saturated by a water table rising to the footing level of the facility. As the designed footing level is normally above the water table as documented in Ref. 10, the joint probability of a rising water table and an earthquake occurring at the same time is less than the probability of the earthquake occurring alone.

## CONCLUSIONS

The seismic and geotechnical study on a sand site for a proposed low- and intermediate-level radioactive waste disposal facility shows:

(1) The site is located in an area of moderate seismic risk. Based on historical seismic data, peak horizontal ground acceleration is computed to be .167g for a probability of exceedence of .002 p.a., i.e., approximately a return period of 500 years.
(2) Cone penetration tests indicate that the upper part of the soil deposit is composed of a unsaturated, loose sand of relative density of about 40%. A liquefaction study reveals that, should this layer become saturated, it would possess a safety factor of 1.06 under the above seismic condition.
(3) Since it is desired to locate the foundation of the facility at the level of the loose sand, it is recommended that the safety factor be increased by compacting the loose sand layer.

## ACKNOWLEDGEMENT

The authors are indebted to the valuable information received from P.W. Basham and J.E. Adams, both of the Geophysical Division, Geological Survey of Canada in establishing the seismic conditions in the Chalk River area.

## REFERENCES

1. Hardy, D.G., Dixon, D.F., Devgun, J.S. and Jarvis, R.G., "Concept Safety Assessment of a Prototype, Intrusion-Resistant, Shallow Land Burial Facility for the Permanent Disposal of Low- and Intermediate-Level Radioactive Waste", Atomic Energy of Canada Report, AECL-MISC-277, (1986).
2. Basham, P.W., Weichart, D.H. and Berry, M.J., "Regional Assessment of Seismic Risk in Eastern Canada", Bull. Seism. Soc. Am. Vol. 69, 1567-1602 (1979).
3. Basham, P.W., Weicheret, D.H., Anglin, F.M. and Berry, M.J., "New Probabilistic Strong Seismic Ground Motion Maps of Canada: A Compilation of Earthquake Source Zones, Methods and Results", Earth Physics Branch (EMR), Open File No. 82-33, Ottawa, Canada (1982).
4. Hasegawa, H.S., Basham, P.W. and Berry, J.M., "Attenuation Relations for Strong Seismic Ground Motion in Canada", Bull. Seism. Soc. Am., Vol. 71, No. 6, 1943-1962, (1981).
5. Douglas, B.J. and Olsen, R.S., "Soil Classification Using Electric Cone Penetrometer", Sym. Cone Penetration Testing & Experience, Geot. Engg. Division, ASCE, St. Louis, 209-227, (1981).
6. Villet, W.C. and Mitchell, J.K., "Cone Resistance, Relative Density and Friction Angle", Cone Penetration Testing and Experience, Geot. Engg. Division, ASCE, St. Louis, 178-208, (1981).
7. Seed, H.B., Idriss, I.M. and Arango, I., "Evaluation of Liquefaction Potential Using Field Performance Data", J. Geot. Eng. Div., ASCE, Vol. 109, No. 3, 458-482, (1983).
8. Robertson, P.K. and Campanella, R., "Liquefaction Potential of Sands Using the CPT", J. Geo. Engg. Div., ASCE, Vol. 111, No. 3, 384-403, (1985).
9. Seed, H.B. and Idriss, I.M., "Ground Motions and Soil Liquefaction during Earthquakes", Earthquake Engienering Research Institute, Berkeley, California, (1982).
10. Buckley, L.P., "Activities Related to Low Level Radioactive Waste Management at AECL's Nuclear Research Sites", CRNL, Chalk River, Ontario TR-360-3, (1987).