A.R. MacNeill School – site development and foundation design

Ernest Naesgaard
Terra Macleod Engineering Ltd, Vancouver, B.C.

Gunther Yip
Terra Macleod Engineering Ltd, Vancouver, B.C.

Abstract: The A.R. MacNeill Secondary School is a one to two story structure in Richmond B.C. which is currently under construction. The school site was a bog with approximately 1.5m of peat overlying 3.5m very soft to soft organic silt over approximately 20 m of loose to dense sand over clayey silts to approximately 240 m depth. Foundation design considerations include the high compressibility of the peat, organic silts, and underlying clayey silts, the low bearing capacity of the peats and organic silts, and the consequences of liquefaction in the underlying sands. Alternative foundation designs considered included: (i) pile foundations with and without ground densification, and (ii) removal of the peat soil and spread foundations with and without ground densification. The chosen foundation design / site preparation scheme includes excavating the peat, filling and preloading the building footprint with river sand, and founding the building on an inter-connected grid of spread footing foundations. Ground densification to mitigate liquefaction in the underlying sands was not conducted. Bearing pressures were kept low in order that the building would not punch into underlying liquefied sands and the building was designed to tolerate large post-earthquake differential settlements without collapse. A two dimensional dynamic numerical analyses using the program FLAC was conducted to check performance during design earthquake conditions. The building structure and liquefaction triggering was included in the model. The dynamic analyses demonstrated the importance of structurally tying the building together laterally and gave insight into deformation patterns within and around the building. In the surrounding playing fields, parking areas and driveways the peat is being preloaded but not removed except in areas where storm and sanitary sewers are to be located. The site preparation methodology, foundation design rational, and preload settlement data are presented.

Introduction

The A.R. MacNeill Secondary School is currently under construction and is scheduled for opening in the fall of 2001. The school site was a peat bog creating a myriad of geotechnical challenges, including low bearing capacity, large and potentially ongoing settlements, lateral ground displacements and liquefaction of underlying loose sand soils. This paper describes the ground conditions, foundation alternatives, design methodology, construction procedures, and preload settlement data. All elevations given in this paper are relative to geodetic datum.

The work was done in conjunction with the structural engineers Bush Bohlman and Partners and architects Killick Metz Bowen Rose.

Site description – pre-development

The site is located west of No. 4 Road between Alberta and Granville Streets in Richmond, B.C. (Fig. 1). The site is L shaped and has an area of approximately 36,000 m^2. The site was a bog covered with light brush and cottonwood trees to approximately 200mm diameter. Site elevation varied from 1.7 to 2.2 m, except for local areas which were higher due to placement of fill. Adjacent residential lots to the north, east, and south were at similar elevation. Adjacent roads have been filled and are approximately 1 to 2 meters above site grade. To the west is located the existing Henry Anderson Elementary School with a surrounding grade of approximately elevation 3 m.

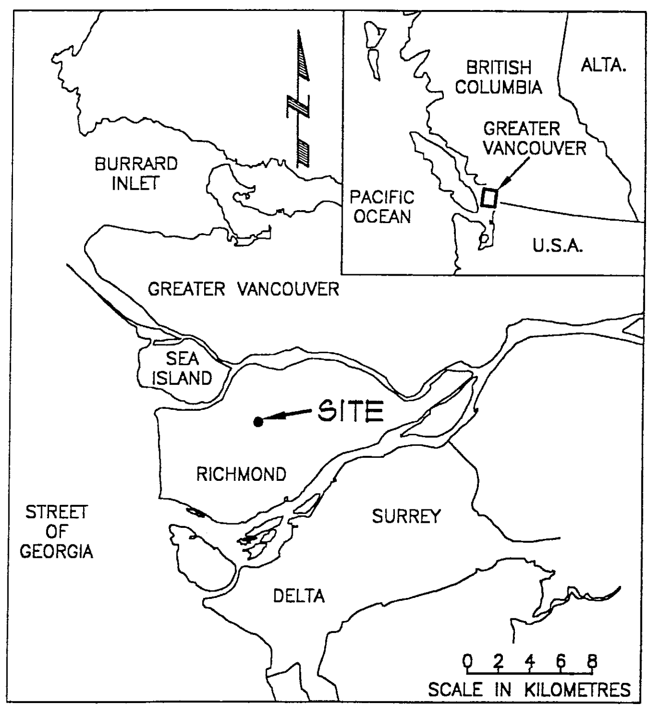


Figure 1. Key Plan

The proposed school

The school is currently under construction. It is a one to two story building with a footprint of approximately 9000m^2 (Fig. 2). Around the school are driveways, parking areas and playing fields. The school is of reinforced concrete tilt-up, light metal stud, and steel construction with composite steel/concrete floors. Service wall loads are less than 100 kN/m. The school has a building code importance factor of 1.3. During the design earthquake, the building should not collapse and endanger the lives of persons. However it need not be functional following the earthquake.

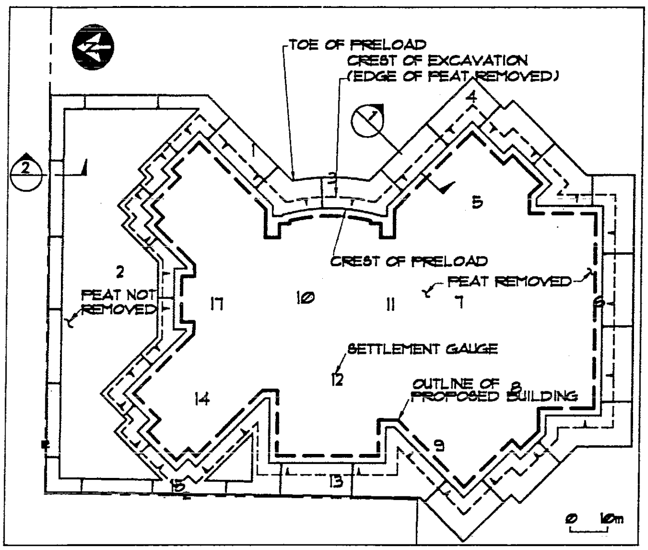


Figure 2. Building and preload layout with location of settlement points.

The sub-surface profile

The subsurface soil profile is summarized in Table 1 and as a typical Cone Penetration and auger hole log in Fig. 3. Groundwater elevation prior to development varied from the ground surface to approximately 1 m below depending on time of year and precipitation levels.

Seismic design parameters

Richmond is an area with the potential for large earthquakes. The site soils are soft and loose, which could lead to potential ground motion amplification, shift in predominant period of motion, and liquefaction of loose sand layers in the unit B1 and B2 sands. The Building was designed according to the 1998 British Columbia Building Code with seismic zone $Z_a = Z_v = 4$ and a foundation factor of 2. The design earthquake was assumed to have a magnitude of 7.0, a peak firm ground acceleration of 0.21 g, a peak firm ground velocity of 0.21 m/s and an amplified surface ground acceleration of 0.3g (Byrne and Anderson - Task Force Report, 1991).

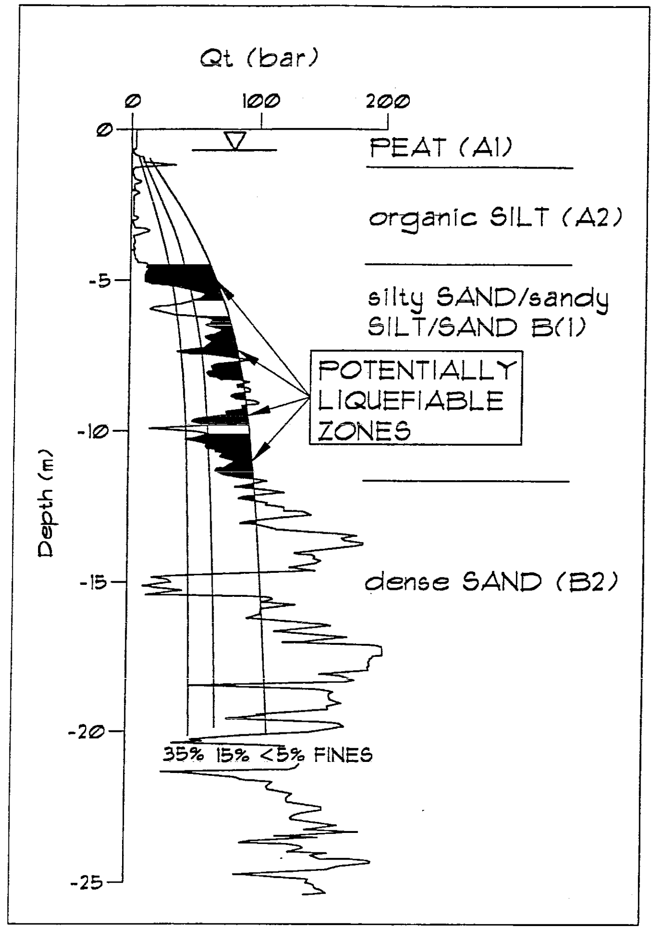


Figure 3. Typical soil profile with zones of potential liquefaction.

Foundation options

Key foundation design challenges to be addressed in the building foundation design include:

- the high compressibility of the peat, organic silts, and underlying clayey silts,
- the low shear strength and bearing capacity of the peats and organic silts, and
- liquefaction in the underlying loose silty sands and sands and related deformations and settlements.

Table 1 – Generalized soil profile

Unit	Thickness (m)	Description
A1	0.9 to 1.8	**PEAT** – dark brown and fibrous except for the lower 0.1 to 0.2 m, which may be amorphous – water content 100 to 1000%.
A2	2.4 to 4.6	**ORGANIC SILT** – very soft to soft grey clayey to sandy organic silt with some layers of fine sand. Generally coarser and less organic content with depth. – water content 35 to 130% - undrained shear strengths (prior to preloading) of 8 to 25 kPa.
B1	4 to 7	**SILTY SAND** – very loose to medium dense silty sand with layers of sandy silt and sand
B2	Approx. 17	**SAND** – loose to dense fine to medium sand with silty sand and occasional sandy silt layers
C1	Approx. 240 m[1]	**CLAY/SILT** – firm to stiff silty clay and clayey silt with fine sand layers in places
D1	N/A	**PLEISTOCENE** – very dense sands, silts and till soils

[1] Inferred from GSC hole 94-4 approximately 1.5 km north of site (Dallimore et al., 1995)

Five foundation schemes, as summarized on Table 2, were reviewed by the design team during the preliminary design phase. Preliminary foundation capacities and costs were derived and comparisons made. As the peat soil was only 1.5m thick it was removed from under the building footprint in all options to mitigate constructability and methane gas generation concerns.

Option 3 was deemed to be the least costly and was chosen for the final design. This option addresses the soil compressibility by preloading, the low soil shear strength by having a light building with low bearing pressures, and the soil liquefaction by having a light weight building with low bearing pressures which is well tied together laterally and can tolerate large differential settlements with out collapse. In final design the raft foundation option was revised to a grid of continuous spread footings interconnected to the building's slab-on-grade.

The construction sequence for the building included the following steps:

- excavation of peat soils (bottom of peat approx. el. 0),
- placement of geofabric (Nilex 2016),
- backfill with compacted river sand to elevation 3.4m,
- placement of riversand preload to elevation 6.0 m,
- preloading for 18 months,
- removal of preload to elevation 3.4 m, and
- construction of interconnected spread footing and reinforced slab-on-grade foundation.

A typical section near the building edge which shows the extent of peat removal and replacement is given in Fig. 4. The foundation design parameters for the building are summarized in Table 3.

Foundation design methodology

Geotechnical analyses for the building included calculation of foundation bearing capacity and ground deformations for both static and seismic conditions.

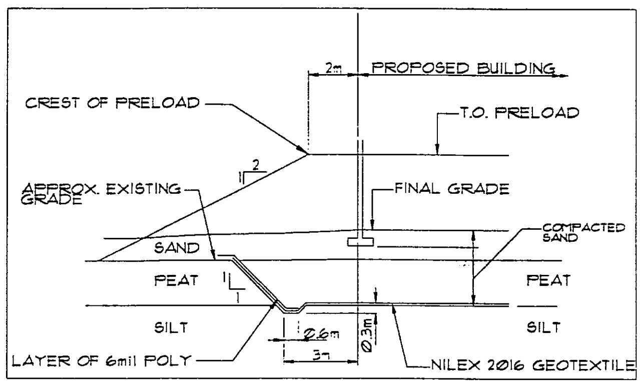


Figure 4. (Section 1, on fig. 2) Showing relative location of building preload and excavation.

Foundation design for static loading

Bearing capacity - For static (non-seismic) conditions bearing capacity was largely controlled by the presence of the underlying very soft Zone A2 organic silts. Allowable bearing capacity was calculated by assuming an equivalent footing on top of the organic silt. To get the size of the equivalent footing the foundations stresses were assumed to spread in a two vertical to one horizontal distribution. The bearing capacity (q_{ult}) of the silt was taken as 5.3 times the undrained shear strength (c_u). A

Table 2 – Summary of foundation alternatives

Option No.	Peat removal	Preloading	Ground densification	Foundation type	Remarks
1	yes	no	yes	expanded base piles	- structural slab
2	yes	for slab-on-grade only	yes	spread foundations	- Zone A2 silt excavated and replaced in vicinity of spread foundations - slab-on-grade
3	yes	yes	no	Raft / spread foundations	- light ductile building with low bearing pressures - slab-on-grade integral with spread foundations
4	yes	no	partially	expanded base piles	- same as option 1 but with partial densification
5	yes	no	no	deep pipe piles	- structural slab - piles designed to tolerate ground deformations without collapse

factor of safety of 3 was applied to get the allowable bearing pressure.

Elastic settlement - For static conditions the building settlement was broken into two parts. There would be elastic distortion of the underlying Zone A2 silt soils, which would occur mainly upon the initial application of the building loads, and there would be long term consolidation settlements of the Zone A2 and deeper Zone C soils. An estimate of the elastic settlements was obtained using the equivalent footing on top of the organic silt and closed form equations for an elastic layer of limited thickness from Bowles, 1977 and were less than 5 mm.

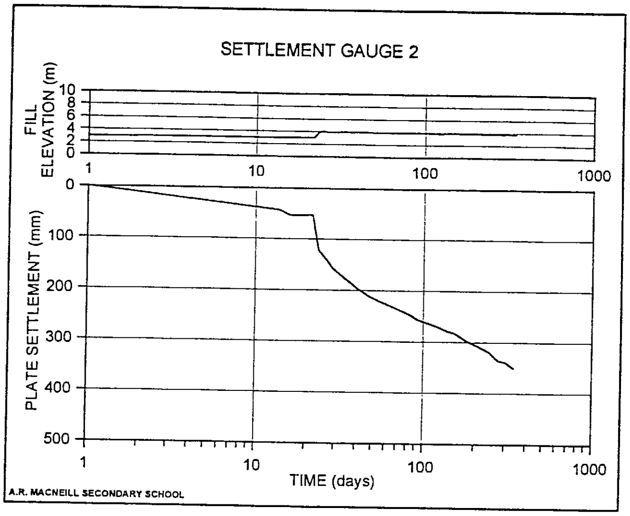
Consolidation Settlement - Consolidation settlements are generally considered to consist of two components, primary and secondary. These long term settlements were reduced to thresholds tolerable by the structure by preloading with a surcharge. The height of the preload was selected such that the stresses in the compressible Zone A2 silts at the time of preloading were 50% higher than the stresses that would be with the building in place. From experience and case histories of other sites in the Fraser delta (Ripley, 1995; Crawford and Morrison, 1996), it has been found that preloading reduces the long term settlement but does not necessarily eliminate it. Predicting the magnitude of these settlements is difficult. Typically the long term settlements decrease approximately proportional to the logarithm of time and will continue for many years. At the time of preload and surcharge removal there will be an elastic rebound and a period of little settlement, then with time the long term settlements will re-initiate. Generally the net effect of the surcharge unload is to reduce the amount of long term settlements. An upper-bound estimate of the ongoing settlement rates can be obtained from preload monitoring.

Recorded preload settlement magnitude and rate per log cycle of time are shown in Table 4. The location of the settlement points is in Fig. 2 and a typical settlement time plot is in Fig. 5. From Table 4 it can be seen that the rate of settlement within the building area varies from 80 to 160 mm/log cycle of time. With the preload being in place for 18 months prior to unload settlement in 50 years in the future is approximately 1.5 log cycles of time. If it is assumed that settlements will be ongoing at the same rate as the surcharged preload then total settlements of the building over the next 50 years will be 120 (1.5x80) to 240 mm (1.5x160) and differential settlements across the

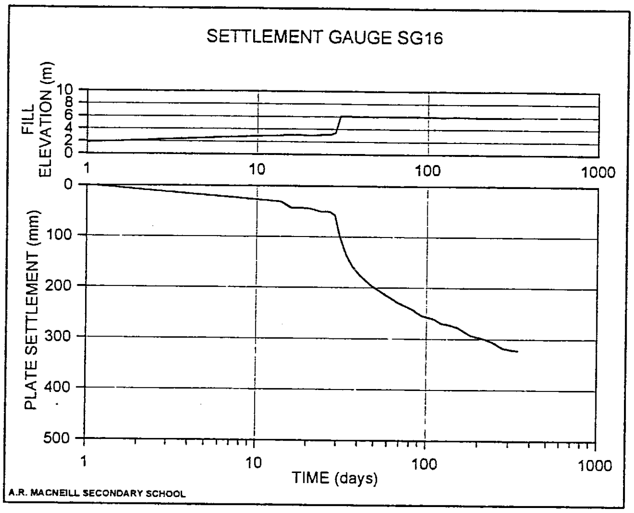
Table 3 - Summary of building foundation design parameters

Allowable foundation bearing pressure	40 kPa
Factored ultimate bearing pressure for seismic conditions	54 kPa
Factored ultimate punching resistance of foundation perimeter	20 kN/m
Total settlement within building over 50 years	200 mm
Differential settlement within building over 50 years	1:400 75mm/30m
Total settlement due to design earthquake	300 mm
Differential settlement within building due to design earthquake	150 mm over 6 m

site would be up to approximately 120mm (240-120). Actual long term settlements should be less then this amount due to the 50% surcharge unload.



(a) Parking area with peat left in place.



(b) Building area where peat has been removed.

Figure 5. Typical settlement vs. time graphs.

Foundation design for seismic loading

Seismic considerations are a key parameter for foundation design in the Fraser River delta. As indicated above, the underlying loose sands are not to be densified and are expected to liquefy during the design earthquake. The consequences of liquefaction that must be addressed in the design include punching failure and settlements of spread footing foundations, lateral spreading displacements, and post-earthquake consolidation settlements.

The original building design was initially conducted with force-equilibrium, empirical and non-dynamic numerical methods. A more detailed dynamic numerical analysis was conducted recently as part of preparing for this paper. The procedures used for the initial design are first discussed. Following this is a description of the dynamic analyses and discussion on the results.

Liquefaction triggering - Liquefaction triggering assessment was initially conducted by converting the cone penetration test tip bearing to equivalent $(N_1)_{60}$ and using the simplified Seed method (Seed and Idriss, 1982; Youd and Idriss, 1997). Typical potentially liquefiable zones are shown on Fig. 3.

Lateral spreading - A static shear stress bias was created when the peat was excavated and replaced with sand fill to a grade higher then the surround. When the underlying soil liquefies the shear stress bias will cause lateral spreading. The amount of spreading was estimated using the empirical procedure by Bartlett and Youd (1995) to be in the range of 0.3 to 0.8 m. The intent of the design was that the building should be structurally tied together so that lateral spreading of the ground would not pull the building apart. The required lateral design force was not easily derived. It was felt that the upper bound would be the force that would slide half the building over the ground. This was deemed to be too severe and a lesser force based more on engineering judgment than calculation was selected. The building has several features to help it resist lateral spreading displacements. These include:

- a slab-on-grade which is reinforced with 10M reinforcing bars in each direction,
- footings which are tied to or integral with the reinforced slab-on-grade,
- a grid of longitudinally reinforced continuous spread footings in lieu of individual pad footings, and
- significant redundancy in the structural lateral resisting elements.

Foundation bearing and punching shear capacity - When portions of the underlying Zone B1 and B2 sands liquefy there is a tendency for the spread footing foundations to punch into the weak soil. During the initial building design the punching resistance was assessed by three methods. One was a limit equilibrium analysis where the punching resistance of the foundation was calculated as the shear strength of the soil between the bottom of the footing and the top of the liquefied layer divided by a factor of safety of 1.5. A factored ultimate punching shear resistance of 20 kPa per meter of footing perimeter was calculated. The second procedure was based on an equation correlating foundation displacement with the factor of safety against bearing failure within the

Table 4 Summary of preload settlement results

STATION (1)	Approximate Net Increase in σ_{vo}' due to preload (kPa)	Total settlement over 10 months (mm)	Settlement per log cycle of time at end of 18 months (mm/log cycle)	Peat excavated and Replaced	
1	40	320	130	no	Parking area
2	40	340	160	no	"
3	40	250	140	no	"
4	40	310	120	no	"
5	90	280	110	yes	Building area
6	90	330	80	yes	"
7	90	310	110	yes	"
8	90	330	100	yes	"
9	90	-		yes	Gauge lost
10	90	430	160	yes	Building area
11	90	400	150	yes	"
12	90	410	130	yes	"
13	90	-	150	yes	"
14	90	360	130	yes	"
15	40	300	90	no?	Parking area
16	90	330	130	yes	Building area
17	90	310	140	yes	"

(1) see Fig. 2 for settlement

liquefied ground (Naesgaard et al., 1998). This Naesgaard et al. (1998) procedure is for continuous spread foundations and is based on the results of a series of pseudo-static and dynamic numerical analyses. With this method the factor of safety against punching or bearing failure is calculated as follows:

$$FS = ((2 \times z_c \times c_u) + (5.14 \times \tau_{res} \times B)) / q \times B$$

Where

B = footing width,
q = bearing pressure,
τ_{res} = residual strength of the liquefied ground
z_c = thickness of non-liquefiable cohesive crust,and
c_u = undrained shear strength of non-liquefiable cohesive crust.

Based on these correlations, with a factor of safety greater than 3, footing settlements during the earthquake were estimated to be less than 200 mm.

The third method used was a site specific static numerical analysis conducted using the program FLAC (ITASCA, 1998). In this analysis method the footing was brought to static equilibrium and then the properties of the loose sand layers were changed from pre-liquefaction to post-liquefaction properties. This procedure indicated that a 2.5 wide strip footing loaded with a bearing pressure of 80 kPa gave liquefaction induced settlements of 250 mm. With the factored design bearing pressure of 55 kPa settlements would be less.

From the above procedures a factored ultimate punching resistance of 20 kPa and a punching induced settlement of less than 200 mm was selected.

Post-liquefaction Consolidation Settlements - Post-liquefaction consolidation settlements which are in addition to the shear strain induced settlements discussed above will also occur. These are due to consolidation of the liquefied ground. For the conditions at the site these were estimated using the procedure by Tokimatsu and Seed, (1987) and were found to be in the order of 2% of the thickness of the liquefied soil layers. Based on CPT testholes on the site the thickness of liquefied ground is expected to varying from 1 to 4 m. This results in post-liquefaction consolidation settlements in the range of 20 to 80mm.

Based on the above analyses combined shear induced and post-liquefaction consolidation settlements were assumed to be up to 300 mm with a differential settlement between adjacent foundations (assumed to be at least 6m apart) of up to 150 mm.

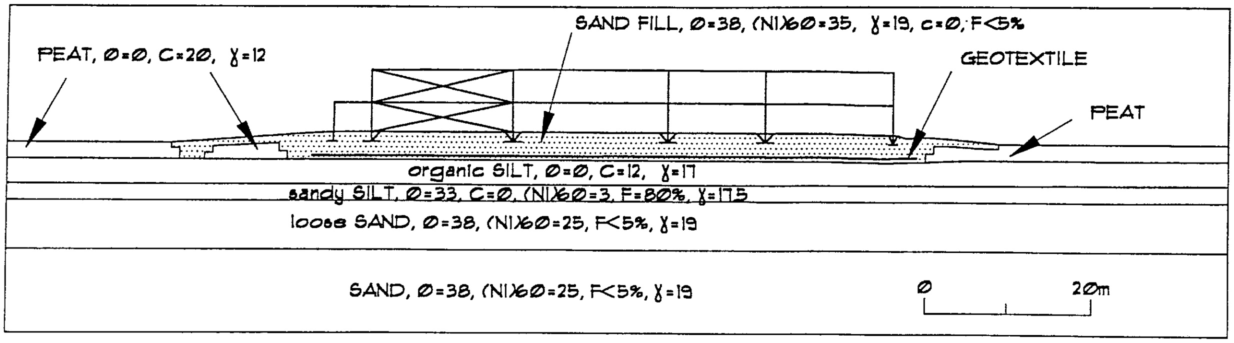


Figure 6. Soil profile and properties use in dynamic numerical model.

Dynamic Numerical Analyses

Dynamic analyses of typical sections through the building were conducted after completion of the design as part of this paper. The analyses were conducted using the program FLAC with the liquefaction triggering model by Byrne and Beaty (Beaty & Byrne, 1999; Byrne & Beaty, 1999). The method by Byrne and Beaty combines the ground response analysis, the liquefaction triggering analysis and the consequences of liquefaction and ground shaking into one model. During the dynamic analysis the cycles of shear stress within each element are cumulated and upon reaching a predetermined threshold of cycles the soil properties are changed from pre-liquefaction to post-liquefaction values. The key steps in the analysis are as follows:

- develop a two dimensional grid with soil as shown in Fig. 6,
- allow the grid and structure to come to static equilibrium under gravity loads,
- conduct a ground response analysis to get representative earthquake input motion for the base of the grid,
- conduct the dynamic analyses by changing to undrained soil properties, by applying free-field boundaries to the sides of the grid, and by applying the earthquake input motion to the base of the grid, and
- monitor the response of the soil and structure.

Three dynamic analyses were conducted with varying building configurations. Fig. 7 is a typical plot showing zones where liquefaction was triggered and Fig. 8 shows a distorted grid where displacements have been magnified by 10. During the dynamic analyses the building moved to the left about 200 mm as a unit and the soil adjacent to the building on either side moved away 200 to 400 mm. Building settlements were up to 200 mm and were greatest toward the edges of the building. Post-

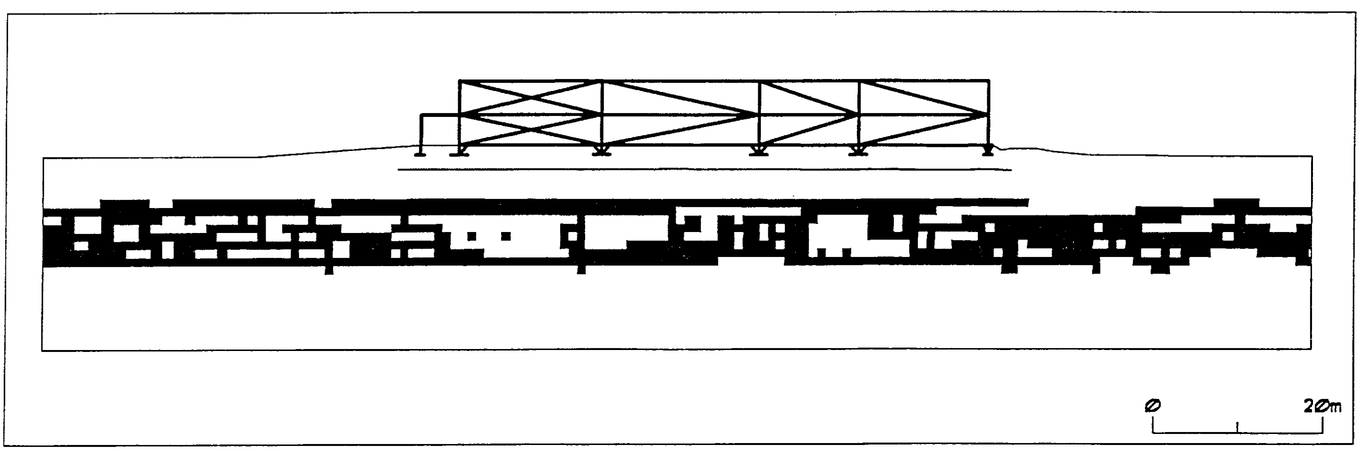


Figure 7. Zones and soil liquefaction (in black) in typical dynamic analysis.

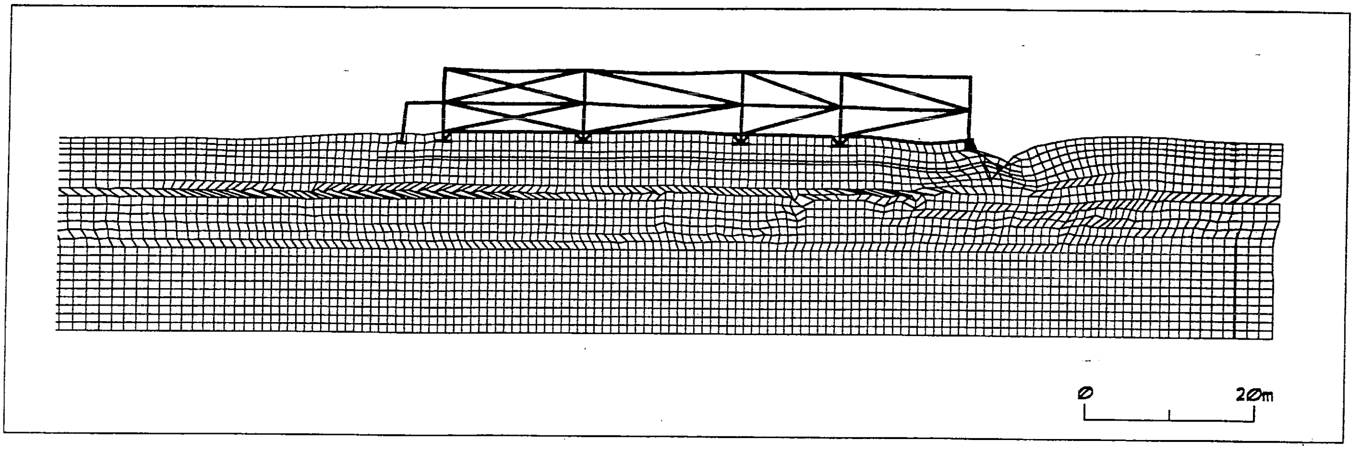


Figure 8. Deformed grid at the end of analysis with deformations magnified by a factor of 10.

liquefaction consolidation settlements would be additional to the above vertical settlements. Based on the extent of liquefaction the consolidation vertical settlements would vary from 20 to 120 mm. Lateral tensile forces in the structure during the earthquake shaking were 100 to 300 kN/m width. Fig. 9 shows a typical time history of axial force in a meter width of the building slab.

Key conclusions from the dynamic analyses are:

(i) The building foundations did not punch into the liquefied soil layers and vertical ground deformations were within the range estimated in the design phase.

(ii) Topography changes and/or changes in soil density surrounding a building can create static bias that would have significant effect on liquefaction related ground deformations at the perimeter of the structure.

(iii) Structurally tying the building together (laterally) is critical, especially when there is a static bias within the underlying soil. Determining the required tie capacity is difficult and the use of a dynamic analysis with liquefaction triggering capabilities is suggested for this purpose. Alternatively the tie capacity should be sufficient to slide one-half the building across the ground surface. If separation is allowed within the building then the location of separation should be along pre-selected weak links that are designed to allow separation without collapse of the structure. This may require redundancy in the lateral resisting elements within the building (shear walls. bracing, etc.) and special consideration for the support of floor and roof beams, etc.

Roadways, playing fields and parking areas

The development procedure for the parking and playing field areas was similar to that used for the building except that the peat soil was not removed and the extent of preloading was less. The construction procedure for the playing field and parking areas is as follows:

- clearing & grubbing including removal of trees greater than 150 mm diameter, bucking of trees less than 150 mm diameter into 1 m lengths and flattening of brush,
- preloading to elevation 4.0 m,
- removal of preload sand,
- placement of geofabric (Nilex 2006), and
- filling with compacted sand to final sub-grade elevation.

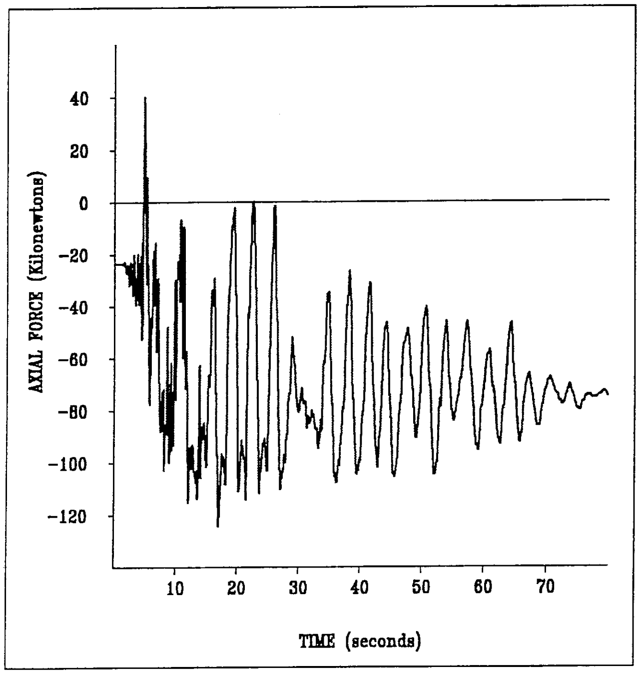


Figure 9. Typical time history of axial force in slab-on-grade.

Roadways and parking areas founded over the peat soils were designed to have a total structure (asphalt, crushed gravel base course, and pit-run sub-base, and sand sub-grade) thickness of at least 700 mm. Preloading for the building, playing fields, parking areas and roadways was designed to have a surcharge unload of at least 150% (ie. Effective stress induced by preload divided by effective stress with building in place > 1.5). On a 3m wide strip around the perimeter of the site the peat was excavated and replaced with sand fill (Fig. 10) in order to reduce the impact of the preloading and filling on the neighboring property.

Service corridors

Typically all service corridors were preloaded in a similar manner to the roadways and playing fields. Generally the invert of the main sewers was below the peat soil. If not, the peat was excavated and replaced with granular backfill. Where the sewers excavation was within a 1 vertical to 4 horizontal line from the foundations of neighboring existing structures it was specified that tightly sheeted shoring or directional drilling methods which would minimize ground movement should be used.

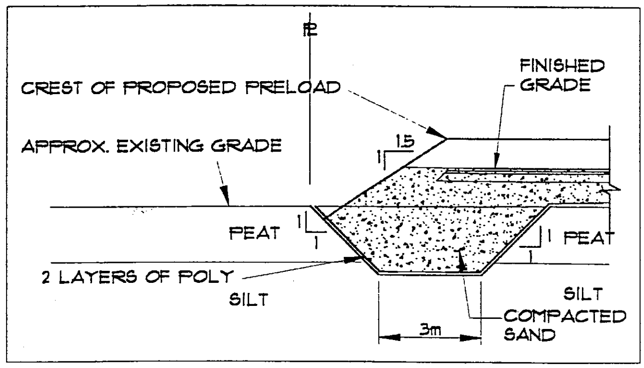


Figure 10. (Section 2, on fig. 2) Typical edge detail adjacent to neighbouring property.

Conclusions

The proposed school site contains compressible peat and weak clay/silt soil, and loose sands that may liquefy during the design earthquake. Removing the peat and founding the building on a pad of compacted and preloaded sand fill, without densifying the underlying soil, provides an economical site preparation and foundation solution. Bearing pressures were kept low in order that the building would not punch into underlying liquefied sands during the design earthquake. The building was designed to tolerate large post-earthquake differential settlements without collapse. 2D dynamic numerical analyses using the program FLAC were conducted to check the performance during design earthquake conditions. These analyses confirmed that light flexible structures could be founded over potentially liquefiable soil without collapse. The dynamic analyses demonstrated the importance of laterally tying the building together and demonstrated the usefulness of this type of analyses in designing structures over weak and liquefied ground.

Acknowledgements

The authors wish to thank the Richmond School Board for the opportunity to work on the project and permission to publish this paper. We also wish thank Greg Ven Huizen and Glen Merkley for assistance on the project. Graeme Macleod and Uthaya Uthayakumur for reviewing the manuscript on short notice, and to Peter Byrne and Michael Beaty for providing the subroutine for the dynamic analyses.

References

Bartlett, S.F. and Youd, T.L., 1995. "Empirical prediction of liquefaction-induced lateral spread," *J. of* Geotech Engrg., ASCE, Vol 121, No. 4, April, 316-329.

Beaty M. and Byrme, P.M. 1999. "Predicting liquefaction displacements with application to field experience," 8th Canadian *Conf. on* Earthquake Engineering, Vancouver, June 1999.

Bowles, J.E., 1977. Foundation Analysis and Design – second edition, MacGraw-Hill, New York.

Byrne, P.M. and Anderson, D.L., 1991. "Task Force Report – Earthquake Design in the Fraser Delta, City of Richmond, June.

Byrme, P.M. and Beaty M., 1999. "Assessment of residual strength of embankments." *Proceedings of* the Second International Conference on Earthquake Geotechnical Engineering, June 21-25, 1999, Lisboa, Portuagal. pp. 1069-1075.

Crawford C.B. and Morrison K.I., 1996. "Case histories illustrate the importance of secondary-type consolidation settlements in the Fraser River delta," Can. Geotech. J. 33: pp: 866-878.

Dallimore, S.R., Edwardson, K.A., Hunter, J.A., Clague, J.J. and Luternauer, J.L., 1995. "Composite geotechnical logs for two deep boreholes in the Fraser River delta, British Columbia", Geological Survey of Canada Open File # 3018

ITASCA, 1998. FLAC – Fast lagrangian analysis of continua, Version 3.40. ITASCA Consulting Group Inc., Thrasher Square East, 708 South Third Street, Ste. 310, Minneapolis, Minnesota.

Naesgaard, E., Byrne, P.B., and Ven Huizen, G., 1998. "Behaviour of light structures founded on soil 'crust' over liquefied ground", *Proceedings of* Geotechnical Earthquake Engineering and Soil Dynamics III. ASCE Specialty conference, Seattle, Washington, pp. 422-433.

Ripley, C.F., 1995. "Preloading thick compressible subsoils: a case history," Can. Geotech. J. 32, pp: 465-480.

Seed, H.B. and Idriss, I.M. 1982. "Ground motions and soil liquefaction during earthquakes". Earthquake Engineering Research Institute Monograph.

Tokimatsu, K. and Seed, H.B., 1987. "Evaluation of settlement in sands due to earthquake shaking," *J. of* Geotech. Engr., ASCE, Vol. 113, No. 8, pp: 861-878

Youd, T.L. and Idriss, I.M., 1997. Proceeding of the NCEER workshop on evaluation of liquefaction resistance of soils. *Technical report* NCEER-97-0022, December, 31, 1997.