Report

DISTRICT OF CAMPBELL RIVER

CAMPBELL RIVER NO. 1 FIREHALL PRELIMINARY SEISMIC ASSESSMENT

May 1995

ASSOCIATED
ENGINEERING

ASSOCIATED
ENGINEERING

May 30, 1995
File: 952448

Robert Owens
Fire Chief
District of Campbell River
675 - 13th Avenue
Campbell River, B.C.
V9W 6C1

**Associated
Engineering
(B.C.) Ltd.**

Suite 300
4940 Canada Way
Burnaby, B.C.
V5G 4M5

TEL (604) 293-1411
FAX (604) 291-6163

**Re: CAMPBELL RIVER NO. 1 FIREHALL
PRELIMINARY SEISMIC ASSESSMENT**

Dear Mr. Owens:

We are pleased to enclose two (2) copies of the Preliminary Seismic Assessment for the Campbell River #1 Firehall.

Thank you for selecting Associated Engineering (B.C.) Ltd. for this assignment. Please do not hesitate to contact us if you any question regarding the report or require further assistance with a detailed seismic assessment.

Yours truly,

D. B. Harrison, P.eng.
Structural Engineer

DALE HARRISON
May 31/95

DBH/arp/report/campbell.595/letter

Enclosure

TABLE OF CONTENTS

1.0 INTRODUCTION

1.1 OBJECTIVES

Associated Engineering (B.C.) Ltd. has been retained by the District of Campbell River to provide a preliminary assessment of the #1 Fire Hall located at 675 - 13th Avenue.

The objective of the review is to provide a preliminary assessment of the damage potential for the fire hall based on preliminary structural and geotechnical reviews.

1.2 PRELIMINARY STRUCTURAL ASSESSMENT

The procedure for the structural assessment included a preliminary visual inspection to obtain information on the structural system and connection details. A review of the architectural and structural drawings on file with the engineering department and photographs taken during construction.

The preliminary analysis was based on the NRC "Guidelines for Seismic Evaluation of Existing Buildings". The scope of the assignment limited the analysis to a preliminary review of the Evaluation Statements contained in the guideline.

1.3 AVAILABLE DATA

The District of Campbell River engineering department provided copies of the following drawings of the existing #1 fire hall.

.1 Allan Murnaghan Architect
Dwg No 7529 / S01
Dwg No 7529 / A02 to 7529 / A08 inclusive

.2 Willis Cunliffe Tait
Dwg No W5294 - 1 - 1 to W5294 - 1- 3 inclusive

Associated Engineering (B.C.) Ltd. obtained a copy of the original soils report, referenced on the structural drawings, from Cook Pickering & Doyle dated January, 1974 and June, 1976. A copy of the original report is enclosed in Appendix D.

1.4 SITE INSPECTION

A visual site inspection of the fire hall was conducted by Mr. D.B. Harrison, P.Eng., assisted by Mr. S. Haigh, Assistant Fire Chief, on May 19, 1995.

The inspection identified the following information:

.1 Basement floor slab was not cast monolithic with the walls as shown on the structural drawings but cast as a slab-on-grade with the walls cast on footings.

.2 Site was reported to have been preloaded as noted on Architectural Dwg 7529 / S01.

.3 Basement height was reduced to 7 feet clear from 8 feet specified apparently due to problems with a high water table.

.4 Reinforced masonry wall between offices section and apparatus floor was previously exploratory drilled to determine location of reinforcing at new door opening. Drilling showed that grouted reinforced lintel above existing office window did not extend past opening.

.5 Metal roof and floor deck is nominally puddle welded to supports.

In addition the photos on Plate 2 to 4 in the Appendix show some typical structural details.

1.5 PRELIMINARY GEOTECHNICAL ASSESSMENT

The preliminary geotechnical report conducted by AGRA Earth and Environmental is included in Appendix C.

2.0 DISCUSSION

2.1 SEISMIC RISK

Seismic risk in Canada is established by a statistical analysis of geological data and past seismological history of the area. Campbell River is located in National Building Code Zone 6 which is the highest seismic risk zone in Canada and is equal to or exceeds many areas of California.

Pacific Geoscience Centre of the Geological Survey of Canada provided a site specific seismic hazard calculation for the Campbell River #1 Fire Hall. Their seismic hazard calculation is included in Appendix B. The National Building Code has designated the 1:475 year event as the design level for new buildings. For this preliminary assessment we have used the 1:475 year event with force levels reduced by a 0.6 factor as recommended by the NRC "Guidelines for Seismic Evaluation of Existing Buildings".

Fire halls are designated by the National Building Code as post disaster structures because they contain equipment and facilities which are essential for the protection of the community in the event of a major earthquake. Therefore the facility was evaluated based on an importance factor of 1.5 as recommended by the NRC guidelines.

2.2 DESCRIPTION OF THE EXISTING STRUCTURE

The fire hall is a 500 m^2 two story office facility with an attached 600 m^2 single story apparatus floor. Both sections of the building are constructed of reinforced masonry with metal decking on open web steel joists. The second floor in the office area consists of a concrete topping on metal decking on open web steel joists. The office area has a partial basement of reinforced concrete construction.

2.3 LATERAL LOAD RESISTING SYSTEM

A building is subjected to lateral forces during an earthquake which it does not experience in normal service. Buildings which are constructed to older standards often have details which are inadequate to resist seismic loads. To resist seismic loads it is essential that a structure have a complete load resisting system which will transfer seismic lateral loads from the roof and floors into the shear walls and foundations. It is also important that the structure have connections with adequate strength and ductility to tie the structure together.

A lack of seismically resistant connections is a major weakness in buildings of this type which were designed for vertical loads with only nominal lateral resistance.

2.4 PRELIMINARY STRUCTURAL ASSESSMENT SUMMARY

The preliminary seismic assessment has indicated deficiencies in the following areas:

Element	Damage Potential	Recommendation
Roof Diaphragm	Low to High	Detailed assessment required
2nd Floor Diaphragm	Low to Moderate	Detailed assessment required at opening
Masonry Shear Walls	Preliminary shear check indicates significant deficiency in shear capacity in both north- south and east-west directions. Reinforcing at bond beams and opening may be deficient.	Detailed analysis required Pachometer Survey required
Connections	Connection anchoring walls to roof and 2nd floor to walls are seriously deficient.	Site investigation to verify anchor type and capacity. Detailed analysis required.
Foundations	Moderate to High	Detailed liquefaction analysis
Non-Structural Items	Not assessed.	

The most significant deficiency identified by the preliminary structural assessment is the anchorage of the roof and 2nd floor to the walls for out-of-plane forces and in-plane shear. The second most serious deficiency is the inadequate shear capacity of the masonry shear walls. Either of these deficiencies could cause serious damage or collapse to occur in a significantly smaller earthquake than the design level. A detailed analysis and more comprehensive site investigation will better define the scope and magnitude of these deficiencies and the cost of retrofitting.

2.5 PRELIMINARY GEOTECHNICAL ASSESSMENT

See AGRA Earth & Environmental report in Appendix C for the preliminary geotechnical assessment.

3.0 CONCLUSION AND RECOMMENDATIONS

Based on a preliminary seismic assessment of the Campbell River #1 Fire Hall, it is concluded that there is a significant potential for significant damage and collapse in the event of an earthquake which could occur during the life of the building. Such an event could result in partial or complete loss of function and/or loss of life for building occupants.

We would recommend that a detailed seismic assessment be undertaken to provide an analytical assessment of the seismic deficiencies and risk of collapse for various levels of earthquake. We would also recommend that preliminary cost estimates be developed for seismic upgrading of the facility or key portion of the building to provide life safety protection. The cost estimates would include an evaluation of upgrading cost versus seismic risk reduction and the report will examine the opportunity to retrofit or replace the existing building. We would also recommend that detailed seismic assessment include a pachometer survey to identify the reinforcing and roof connection details in the masonry.

As noted in AGRA Earth and Environmental report we would recommend further geotechnical work be conducted to better define the risk of liquefaction.

APPENDIX A

PHOTO SHEETS - PL1 TO PL4

North Elevation

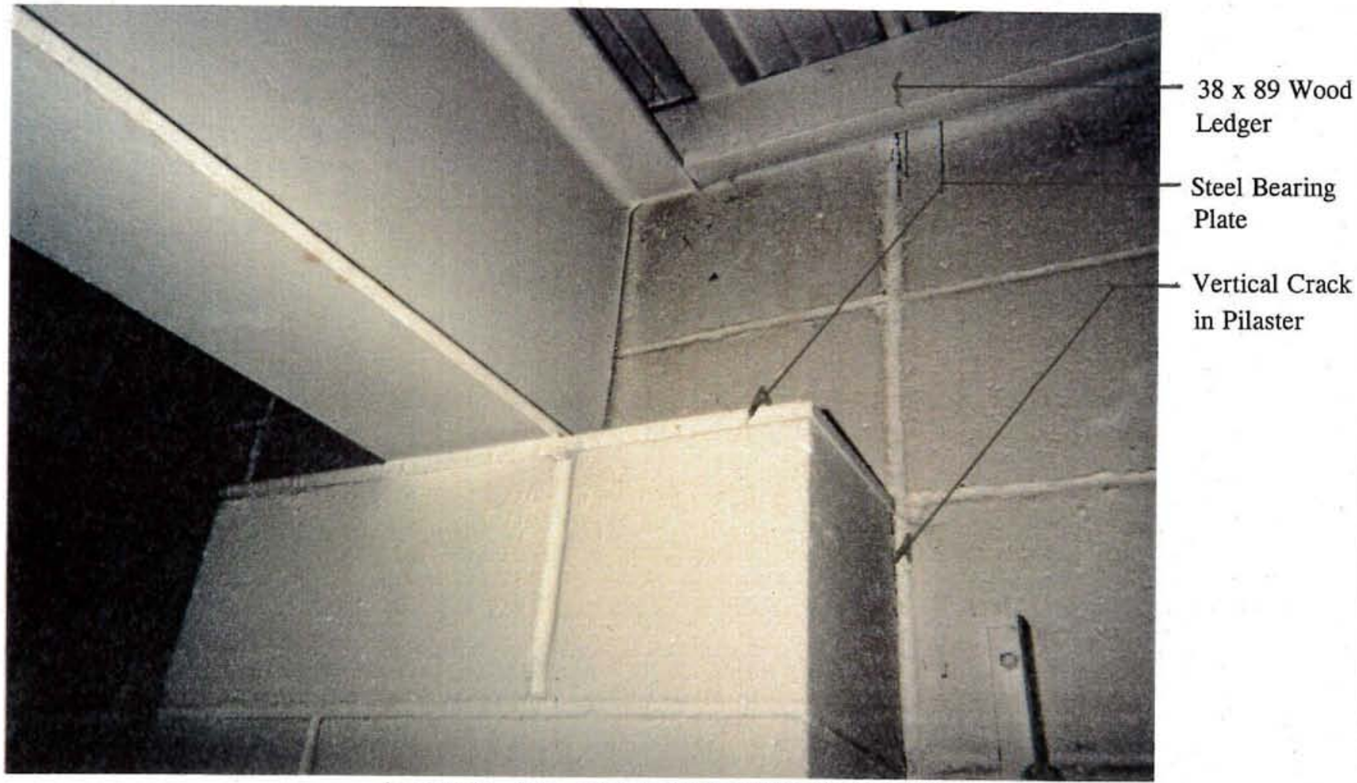
Southeast Elevation

PROJECT NO. 952448
DATE May 24, 1995
APPROVED DBH
SCALE N.T.S.
DWG. NO. PL-1

District of Campbell River
No. 1 Firehall
Firehall Elevations

ASSOCIATED
ENGINEERING

952448/reports/campbell.595/photo.1



Apparatus Floor Roof Beam/Pilaster Detail

Roof Joist at South Wall of Maintenance Shop

PROJECT NO. 952448
DATE May 24, 1995
APPROVED DBH
SCALE N.T.S.
DWG. NO. PL-2

District of Campbell River
No. 1 Firehall

Structural Details

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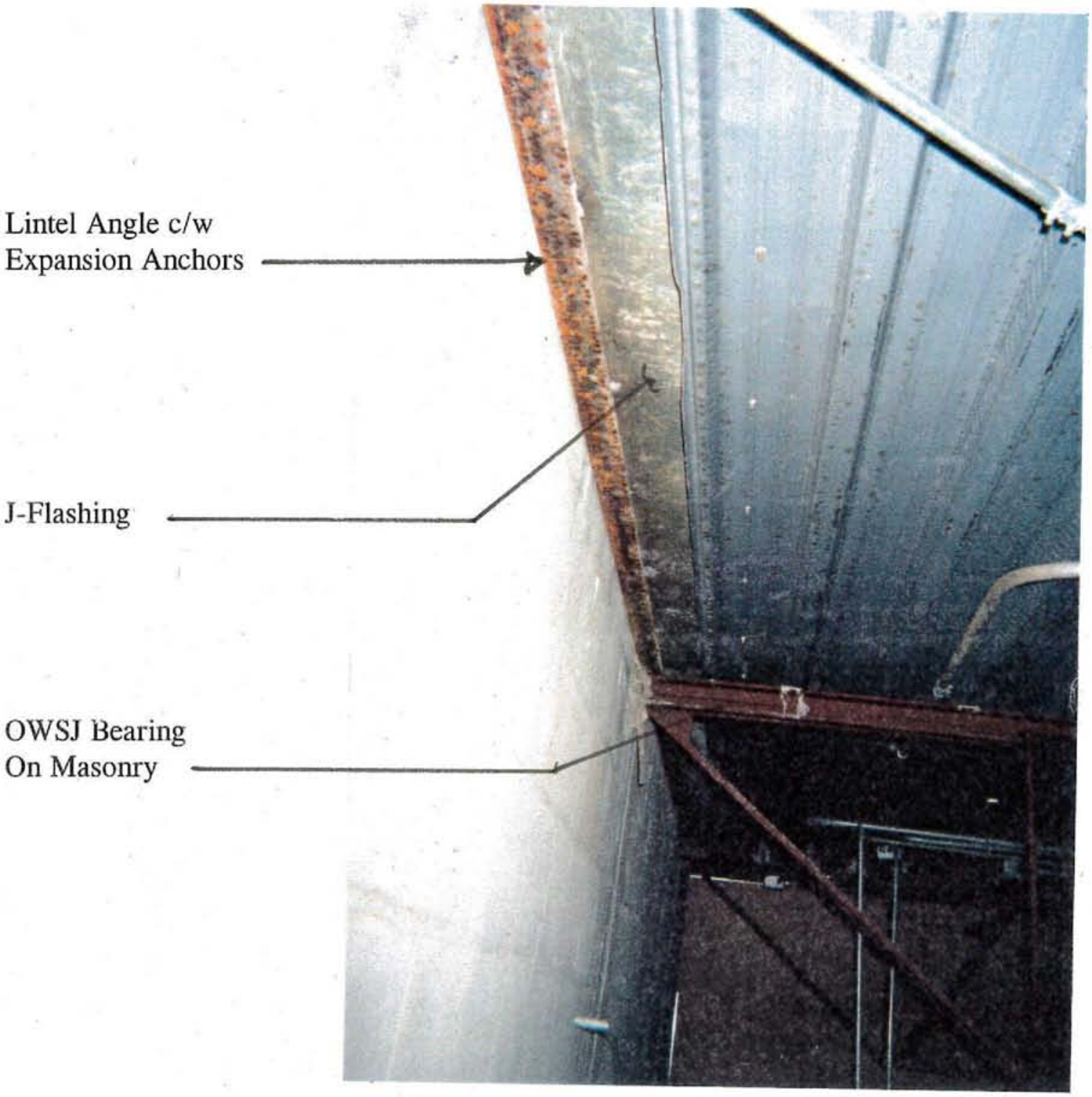
952448/reports/campbell.595/photo.2



2nd Floor/Masonry Wall Connection Above Locker Room

PROJECT NO. 952448
DATE May 24, 1995
APPROVED DBH
SCALE N.T.S.
DWG. NO. PL-3

District of Campbell River
No. 1 Firehall
Structural Details

ASSOCIATED
ENGINEERING

Roof Joist at North Office Wall

PROJECT NO. 952448
DATE May 24, 1995
APPROVED DBH
SCALE N.T.S.
DWG. NO. PL-4

952448/reports/campbell.595/photo.4

District of Campbell River
No. 1 Firehall

Structural Details

ASSOCIATED
ENGINEERING

APPENDIX B

PACIFIC GEOSCIENCE - SEISMIC RISK CALCULATION

NATURAL RESOURCES CANADA GEOLOGICAL SURVEY OF CANADA	RESSOURCES NATURELLES CANADA COMMISSION GEOLOGIQUE DU CANADA
SEISMIC RISK CALCULATION *	CALCUL DE RISQUE SEISMIQUE *
REQUESTED BY/ DEMANDE PAR	D.B. Harrison / Associated Eng. Ltd.
SITE	Campbell River Firehall #1
LOCATED AT/ SITUE AU	50.03 NORTH/NORD 125.25 WEST/OUEST

PROBABILITY OF EXCEEDENCE PER ANNUM/ PROBABILITE DE DEPASSEMENT PAR ANNEE	0.010	0.005	0.0021	0.001
PROBABILITY OF EXCEEDENCE IN 50 YEARS/ PROBABILITE DE DEPASSEMENT EN 50 ANS	40 %	22 %	10 %	5 %
PEAK HORIZONTAL GROUND ACCELERATION (G) ACCELERATION HORIZONTALE MAXIMALE DU SOL (G)	0.150	0.249	0.440	0.673
PEAK HORIZONTAL GROUND VELOCITY (M/SEC) VITESSE HORIZONTALE MAXIMALE DU SOL (M/SEC)	0.115	0.200	0.383	0.637

* REFERENCES

1. NEW PROBABILISTIC STRONG SEISMIC GROUND MOTION MAPS OF CANADA: A COMPILATION OF EARTHQUAKE SOURCE ZONES, METHODS AND RESULTS.
P.W. BASHAM, D.H. WEICHERT, F.M. ANGLIN, AND M.J. BERRY
EARTH PHYSICS BRANCH OPEN FILE NUMBER 82-33, OTTAWA, CANADA 1982.

2. ENGINEERING APPLICATIONS OF NEW PROBABILISTIC SEISMIC GROUND-MOTION MAPS OF CANADA.
A.C. HEIDEBRECHT, P.W. BASHAM, J.H. RAINER, AND M.J. BERRY
CANADIAN JOURNAL OF CIVIL ENGINEERING, VOL. 10, NO. 4, P. 670-680, 1983.

3. NEW PROBABILISTIC STRONG GROUND MOTION MAPS OF CANADA.
P.W. BASHAM, D.H. WEICHERT, F.M. ANGLIN, AND M.J. BERRY, BULLETIN OF THE SEISMOLOGICAL SOCIETY OF AMERICA, VOL. 75, NO. 2, P. 563-595, 1985.

4A. SUPPLEMENT TO THE NATIONAL BUILDING CODE OF CANADA 1985, NRCC NO. 23178.
CHAPTER 1: CLIMATIC INFORMATION FOR BUILDING DESIGN IN CANADA.
CHAPTER 4: COMMENTARY J: EFFECTS OF EARTHQUAKES.

4B. SUPPLEMENT DU CODE NATIONAL DU BATIMENT DU CANADA 1985, CNRC NO 23178F.
CHAPITRE 1: DONNEES CLIMATIQUES POUR LE CALCUL DES BATIMENTS AU CANADA.
CHAPITRE 4: COMMENTAIRE J; EFFETS DES SEISMES.

AGRA
Earth & Environmental

3070 Barons Road
Nanaimo, B.C. V9T 4B5
Tel (604) 758-1887
Fax (604) 758-1899

May 29, 1995
File No: NX30215

Associated Engineering (BC) Ltd.
8927 McLarey Avenue
Black Creek, B.C.
V9J 1A3

Attention: Mr. Dale Harrison, P.Eng.

PROJECT: FIRE HALL No. 1 - 675 13th AVENUE
CAMPBELL RIVER

SUBJECT: PRELIMINARY SEISMIC OVERVIEW

Dear Sir:

1.0 INTRODUCTION

AGRA Earth & Environmental Limited (AEE) has carried out a preliminary geotechnical assessment relating to the seismic stability of the site of the existing No. 1 Fire Hall in Campbell River. The purpose of the assessment was to estimate what the potential ground response would be at the site during a major earthquake. A "major" earthquake would be defined as the 1 in 475 year seismic event.

The purpose and scope of the assessment were presented in a letter - dated May 23, 1995 - from Mr. Dale Harrison, P.Eng. of Associated Engineering (B.C.) Ltd (Associated) to AEE.

The following report has been prepared solely from available information, as described below. It should be emphasized that no field investigation was carried out to confirm the subsurface conditions at the site.

2.0 SITE DESCRIPTION

The existing No. 1 Fire Hall in Campbell River is located in the southeast corner of Elm Street and 13th Avenue. The fire hall building consists of a single-storey garage area that contains five bays, and a two-storey office/assembly area with a basement. It is understood that the building is supported on conventional spread footings.

The garage area was primarily of concrete masonry block construction. The two-storey section was masonry and steel construction.

The site is located approximately 200 m north of the crest of the valley slope and 500 m west of the current shoreline of Georgia Strait. The Campbell River is located at least 1 km to the north and west of the site. In general, the site was flat and level, and had been finished with paved parking/access areas and landscaping.

3.0 GEOLOGICAL SETTING

A review of aerial photographs (air photos) of the Campbell River area indicated that the site is situated on the flood plain of the Campbell River. A Terrain Map - produced by the BC Department of Lands and Forests - indicates that the site is situated on an "undulating/gently undulating, gravelly, fluvial fan".

Based on previous investigations carried out by AEE in the local area, it is anticipated that the ground conditions at the site would consist - in general - of granular fills overlying loose/compact, clean, saturated, sandy gravel. It should be noted that these previous investigations were carried out with test pits of limited (less than 3 m) depth.

A copy of the original geotechnical report for the fire hall building was provided to AEE by Associated (report reference: Cook Pickering & Doyle Ltd. report dated January 31, 1974 - Project #3867-C). The soils information provided in the report - from test pits excavated to less than 2 m depth - indicated that the site was underlain, in general, by sand and silty sand. The report also indicated that the groundwater level was within about 0.6 m (2 ft) of ground surface.

4.0 SEISMIC SETTING

A seismic risk calculation, carried out by the Geological Survey of Canada in Sydney, B.C., indicated that there is a 10% probability that the Campbell River area will be subjected to seismic accelerations of about 44% of gravity (0.44 g) within a 50 year period. This corresponds to the 1 in 475 year seismic event under consideration in this report.

It should be noted that, in 1946, a major earthquake took place in the Campbell River area. The event resulted in liquefaction of the local soils and significant ground motions were observed and recorded.

5.0 DISCUSSIONS AND RECOMMENDATIONS

5.1 Ground Deformation Potential

It is understood that the fire hall is considered to be a "post-disaster" building, and, as such, would be expected to resist the expected 1 in 475 year event without collapse. Collapse is defined - by the National Building Code of Canada - as a state where occupants can no longer exit the building due to structural failure.

The amount of damage to any structure subjected to seismic accelerations will be partly dependent on the type (consistency and composition) of soil on which it is founded. Based on the available geotechnical information and on previous work - by AEE and others - in the area, it is expected that the fire hall would be founded upon a relatively thick deposit of unconsolidated granular deposits of fluvial origin. It is anticipated that these deposits would consist of generally variable sands and gravels, and silty sands. Recent (Holocene) deposits such as these are typically susceptible to liquefaction in the event of an earthquake.

Therefore, based on the soil and groundwater conditions assumed to be underlying the existing fire hall, and on the anticipated magnitude of the earthquake under consideration, it is judged that the soils underlying the site would liquefy in response to a 1 in 475 year seismic event. Note that determination of the zone of potentially liquefiable soil, and estimation of potential ground displacements, are outside the scope of this preliminary assessment.

5.2 Motion Potential Investigation

It is recommended that, due to the assigned importance of the fire hall building, and the anticipated potential for liquefaction at the site, a subsurface investigation and quantitative liquefaction/ground motion assessment. Such a study would assist in design of a retro-fit of the structure and/or cost benefit analysis for replacement.

It should be noted that the probabilistic approach, used by the GSC, to calculate the seismic risk of a site tends to over-predict the Peak Ground Acceleration (PGA) for near-field (close to anticipated epicentre) sites. Calculations of PGA in the Campbell River area by others have indicated relatively lower values of 0.25 to 0.3 g. It is recommended that further assessment be carried out to determine a more representative value of the PGA for the fire hall site. This value would then be used in the calculation of the ground displacement potential.

A subsurface investigation would confirm the assumed soil and groundwater conditions described above, and would provide information regarding:

- soil density;
- location of groundwater table;
- soil composition - especially grain size distribution.

Due to the expected gravelly soils, it is considered that a Becker Hammer drill rig would be the appropriate method of investigation at this site.

The results of the subsurface investigation would allow for evaluation of the Foundation Factor (F) for structural design, as defined in the National Building Code of Canada.

Based on the subsurface information obtained and on the site specific PGA value, a Seed Simplified liquefaction assessment would be carried out. This assessment would allow for:

- definition of the potentially liquefiable zone;
- an estimate of potential ground displacements (although a detailed analysis of potential displacements would still be beyond of the scope of the work).

In addition, appropriate site remediation methods - such as ground densification - could be presented. It should be noted at this point, that these techniques are generally very expensive and should only be considered after a detailed cost-benefit analysis.

6.0 CLOSURE

The discussions and recommendations presented above were based on currently available information, and should be considered as preliminary only. As stated, no site specific investigation has been carried out as part of this assessment, and the results of the liquefaction potential are strictly qualitative.

Campbell River Firehall #1

ZONING FOR ABOVE SITE/ ZONAGE DU SITE CI-DESSUS

1985 NBCC/CNBC: ZA = 6; ZV = 6; V = 0.40 M/S

ACCELERATION ZONE/ ZONE D'ACCELERATION ZA=6
ZONAL ACCELERATION/ ACCELERATION ZONALE 0.40 G

VELOCITY ZONE/ ZONE DE VITESSE ZV=6
ZONAL VELOCITY/ VITESSE ZONALE 0.40 M/S

1985 NBCC/CNBC **
SEISMIC ZONING MAPS/ CARTES DU ZONAGE SEISMIQUE

PROBABILITY LEVEL: 10% IN 50 YEARS
NIVEAU DE PROBABILITE: 10% EN 50 ANNEES

G OR M/S	ZONE	ZONAL VALUE/ VALEUR ZONALE
0.00		
	0	0.00
0.04		
	1	0.05
0.08		
	2	0.10
0.11		
	3	0.15
0.16		
	4	0.20
0.23		
	5	0.30
0.32		
	6*	0.40

* ZONE 6: NOMINAL VALUE/ VALEUR NOMINALE 0.40;
SITE-SPECIFIC STUDIES SUGGESTED FOR IMPORTANT PROJECTS/
ETUDES COMPLEMENTAIRES SUGGEREES POUR DES PROJETS D'IMPORTANCE.

** FOR NBCC APPLICATIONS, CALCULATED ZONE VALUES AT A SITE SHOULD BE REPLACED BY EFFECTIVE ZONE VALUES [ZA(EFF) OR ZV(EFF)] AS SHOWN BELOW/ POUR APPLICATIONS SELON LE CNBC, ON DOIT REMPLACER LES VALEURS ZONALES CALCULEES POUR UN SITE PAR LES VALEURS EFFECTIVES [ZA(EFF) OU ZV(EFF)] COMME MONTRE CI-DESSOUS:

1. IF/SI (ZA - ZV) > 1, ===> ZA(EFF) = ZV + 1.

OR/OU

2. IF/SI (ZA - ZV) < 1, ===> ZA(EFF) = ZV - 1.

OR/OU

3. IF/SI ZV=0 AND/ET ZA > 0, ===> ZV(EFF) = 1.

(SEE REFERENCE 2 CITED ABOVE, PAGE 677)
(VOIR PAGE 677 DE LA REFERENCE 2 CI-DESSUS)

May 24, 1995.

APPENDIX C

AGRA EARTH AND ENVIRONMENTAL GEOTECHNICAL REPORT

We trust that the information presented here meets your current requirements. If you have any questions, or require further information, please do not hesitate to contact the undersigned.

Yours truly,
AGRA Earth & Environmental Limited

Tom W. Oxland

Professional Engineer, Province of British Columbia
29 May 95
T. W. OXLAND

Tom W. Oxland, P.Eng.
Geotechnical Project Engineer

Carl Miller, M.Sc., P.Eng.
Manager, Nanaimo Office

cc: Associated Engineering (Burnaby) Ltd.
Attention: Ms. Louise Parwell

APPENDIX D

COOK PICKERING AND DOYLE LTD. SOILS REPORT

Cook Pickering & Doyle Ltd.
CONSULTING ENGINEERS
PHONE 879-0494
835 WEST 7TH AVENUE
VANCOUVER 9, B.C.

January 31, 1974.
Project #3867 - C.

Campbell River Fire Protection District,
1000 Island Highway,
Campbell River, B. C.

Attention: Mr. D. E. Lanyon, Chairman.

Dear Sirs:

Re: Campbell River Firehall,
13th & Elm

This letter with the attached test pit logs and location plan comprises our report on soil conditions and foundation recommendations at the above site.

Most of this site has had prior use and there is 1 to 2 1/2 ft. of sand and gravel fill over the majority of the area. The test pits exposed up to 5 ft. of organic silt under the fill. Silty sand exists beneath the entire site at depths from 4 ft. to 8 ft. below ground surface. The water table in December was about 2 ft. below ground surface.

We understand that the Firehall will consist of a single storey section enclosing the truck bays and a two storey section with offices, dormitory and recreation facilities. We anticipate 30 to 40 kip column loads in the single storey portion and up to 100 kip column loads in the two storey section.

We recommend that the building area on this site be preloaded with 3 ft. of fill to eliminate floor settlements. The fill (likely sand and gravel) should be placed to an elevation 3 ft. above final floor elevation and left in place for three to four weeks before removing. The preload fill should cover the total Firehall plan area. If preloading is not undertaken, we estimate that floor settlements could be as high as 1 1/2 to 2 in.; particularly in the area of TP-1.

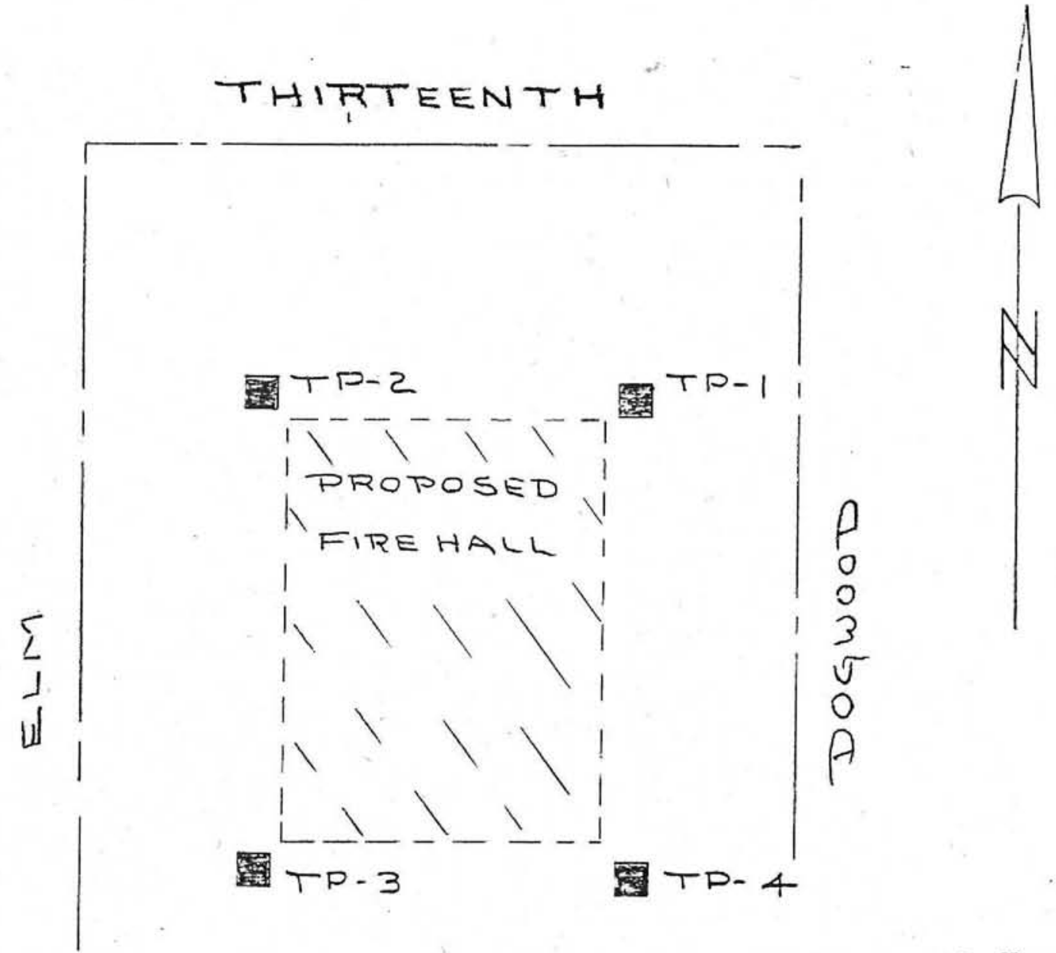
We recommend that the Firehall be placed on spread footings designed for a maximum allowable bearing pressure of 2000 psf. However; to carry these pressures without settlements exceeding 1 in., we recommend that there must be a minimum of 3 ft. of compacted clean granular material beneath footings loaded to 50 kips and 4 ft. of compacted granular beneath footings loaded to 100 kips. The size of the excavation and replacement in plan should be equal to the width of the footings plus 4 ft. We recommend that this excavation and replacement be done in the "wet" to ensure that the underlying layered silty sand and sandy silt does not heave and thereby lose strength. Compaction of clean sand and gravel below the water table can be accomplished by carefully lowering a concrete vibrator into the granular material. Above the water table, all sand and gravel fill should be compacted to at least 95% standard Proctor density.

The only alternative to the above recommended excavation and replacement procedure would be to pile-support the building. We consider this to be more expensive and recommend that spread footings be used.

Yours very truly,

D. E. SNEAD, P. ENG.
COOK, PICKERING & DOYLE LTD.

DES:jet
enc. - Invoice.
c.c. Frank Anderson Architect,
#201 - 1533 Pender St.,
Vancouver, B. C.



SCALE :- SKETCH

▨ TEST PIT

TEST HOLE LOCATION SKETCH
CAMPBELL RIVER FIREHALL

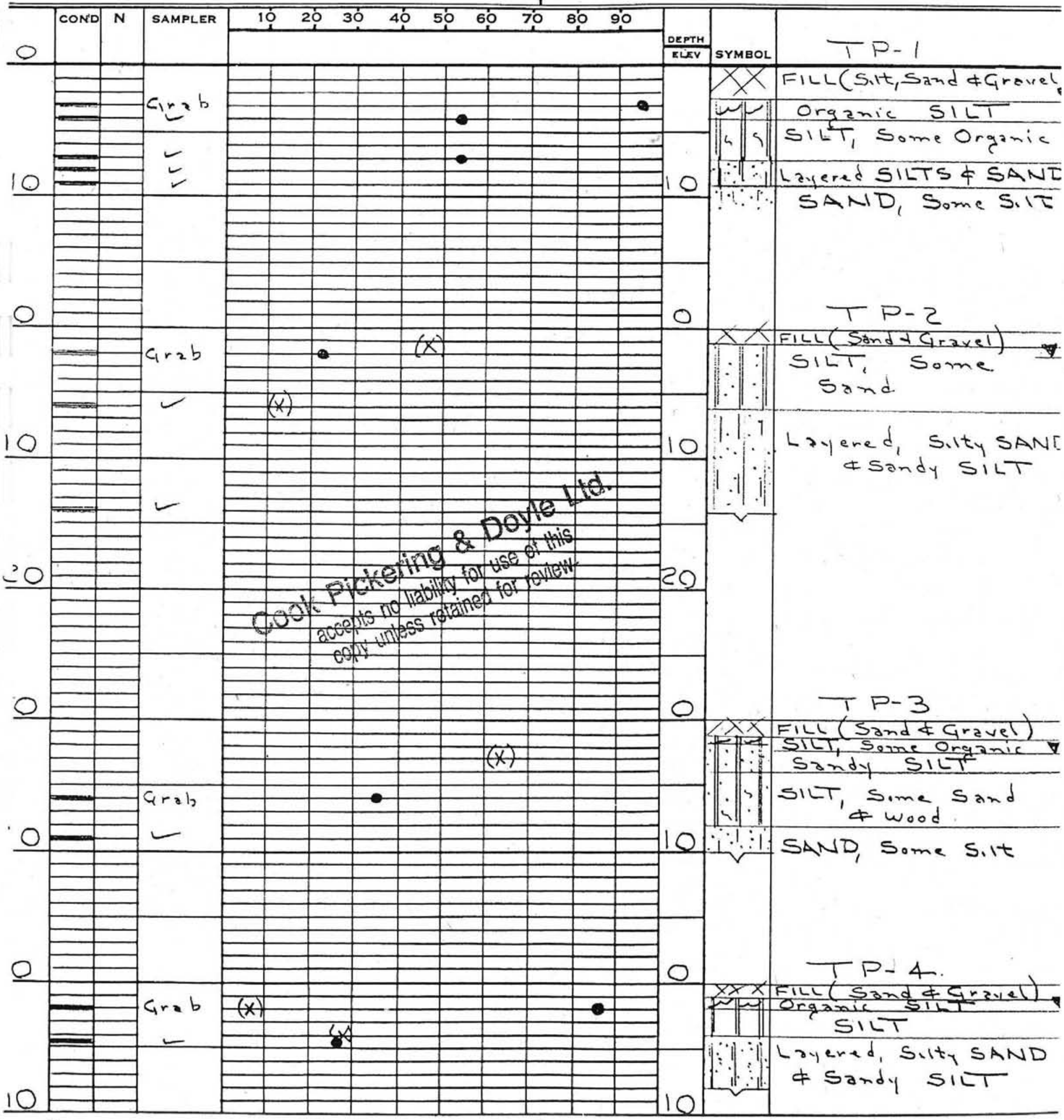
Cook, Pickering & Doyle Ltd.
CONSULTING ENGINEERS
DEC 1973 LBJ

Dwg. No. 1
3867 C

Cook Pickering & Doyle Ltd.
835 WEST 7TH AVENUE, VANCOUVER 9, B.C.

PROJECT FIRE HALL CAMPBELL RIVER

JOB NO. 3867C



TP-1

FILL (Silt, Sand & Gravel

Organic SILT

SILT, Some Organic

Layered SILTS & SAND

SAND, Some Silt

TP-2

FILL (Sand & Gravel)

SILT, Some Sand

Layered, Silty SAND & Sandy SILT

TP-3

FILL (Sand & Gravel)

SILT, Some Organic

Sandy SILT

SILT, Some Sand & Wood

SAND, Some Silt

TP-4.

FILL (Sand & Gravel)

Organic SILT

SILT

Layered, Silty SAND & Sandy SILT

N — STANDARD PENETRATION TEST — NUMBER OF BLOWS OF 140 LB. HAMMER DROPPED 30" TO PRODUCE 12" OF PENETRATION OR PENETRATION AS NOTED — INCHES

● — MOISTURE CONTENT IN % BY DRY WT. OF SOIL ▶—PLASTIC LIMIT ◀—LIQUID LIMIT ▼-GROUND WATER LEVEL

X — UNCONFINED COMPRESSION STRENGTH — PSI. (X) — TORVANE OR PENETROMETER IN EQUIV. UNCONFINED STRENGTH — PSI.

TYPE OF SAMPLER — 2" STA. — 2" STANDARD S—SHELBY-TUBE FP—FIXED PISTON STL—SPLIT TUBE WITH LINER

COND — SAMPLE CONDITION ■ GOOD ▥ DIST'D ▤ NIL

Cook Pickering & Doyle Ltd.
CONSULTING ENGINEERS

PHONE 879-0494

835 WEST 7TH AVENUE
VANCOUVER, B.C.
V5Z 1C2

June 16, 1976

Project #3867 - C.

Willis Cunliffe Tait & Co. Ltd.
827 Fort Street
Victoria, B. C.
V8W 2Y5

Attention: Mr. A. Jordan, P. Eng.

Cook Pickering & Doyle Ltd.
accepts no liability for use of this
copy unless retained for review.

Dear Sirs:

Re: Proposed Campbell River
#1 Fire Hall

As requested in your letter of June 3, 1976, we have reviewed the information in our report of January 31, 1974 with respect to the proposed #1 Firehall as shown on the drawings which you forwarded.

The most significant change from the original concept is the excavation for a basement in the office area. The proposed excavation is about 6½ ft. below ground surface with a water table noted in Jan./74 about 2 ft. below ground surface. Therefore, the structure will have to be "tanked", or drains installed with fail safe drainage to ensure the water table is permanently drawn down. If the structure is tanked, the weight of the building must be checked to ensure that it does not float (assume water table at ground surface when checking this). Drains to lower the water table would involve perimeter drains plus underfloor drains on about 10 ft. centers. This drainage would have to be "fail safe" (i.e., positive drainage) otherwise the basement level of the building may flood.

During construction, the water table will have to be lowered to avoid heaving of the bottom sands and silts. We recommend that a dewatering system be installed for this purpose rather than attempting the construction with a series of sumps and pumps.

Providing the short and long term water problem is handled, the following recommendations regarding allowable bearing pressures apply. Bearing wall supporting the office section and loaded up to 5 klf can be founded directly on the excavated grade designed for an allowable bearing pressure of 1500 psf.

The fire truck area constructed at about existing grade should be preloaded for three to four weeks as stated in our original report. Due to the elimination of heavy column loads (100 kips) at or near present ground surface, the preload can be reduced to 2 ft. from the originally specified 3 ft. In addition, footings loaded up to 40 kips can be designed using a bearing pressure of 1500 psf with a sub-excavation and replacement of only $1\frac{1}{2}$ ft. The width of the sub-excavated area should be equal to the footing width plus $1\frac{1}{2}$ ft. Recommendations for suitability of material and compaction are presented in our original report.

Yours very truly,

D. E. SNEAD, P. ENG.
COOK, PICKERING & DOYLE LTD.

DES:nk
Enclosure

Geotechnical Seismic Assessment
Fire Hall No. 1
675 13th Avenue
Campbell River, BC

Submitted to:

District of Campbell River
Maintenance Services Manager
c/o Engineering Departments
301 St. Ann's Road
Campbell River, BC V9W 4C7

Submitted by:

AMEC Earth & Environmental Limited
Nanaimo, British Columbia

18 March 2003

AMEC File No. NX01790-100

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LIST OF APPENDICES

1.0 INTRODUCTION

This report presents a geotechnical seismic assessment prepared by AMEC Earth & Environmental Limited (AMEC) for the Fire Hall No. 1 in Campbell River, BC.

The scope of work for this assessment was described in AMEC's proposal dated 15 October 2002 (AMEC File NXP2277). Authorization to proceed with the scope of work outlined was obtained from Mr. David Morris, Maintenance Services Manager for the District of Campbell River via facsimile on 13 November 2002.

This report outlines the exploration program, laboratory soil testing program, and provides an assessment of the seismic liquefaction potential of the foundation soils underlying Fire Hall No. 1.

2.0 SITE & PROJECT DESCRIPTION

AMEC Earth & Environmental Limited (AMEC) completed a preliminary seismic overview relating to the anticipated seismic performance of the site and the Fire Hall No. 1 in May 1995 (AMEC File: NX30215). The overview identified the potential for poor ground response, including seismic liquefaction of the foundation soils, and presented a series of recommendations to assess the seismic performance of the ground and structure. The recommendations included field exploration to collect site-specific information, geotechnical analysis, and the provision of a report to present conclusions and recommendations. The initial assessment was initiated by the District of Campbell River. Fire Halls are understood to be "post-disaster" structures that are required to remain functional after a major earthquake. A major earthquake as defined by the National Building Code of Canada is an earthquake which results in accelerations and velocities with a 10% chance of being exceeded in 50 years (1 in 475 year earthquake).

Fire Hall No. 1 is located on 13 Avenue as shown on Figure 1. The Fire Hall is situated within the Campbell River flood plain. Based on the results of the preliminary geotechnical study completed by AMEC (as AGRA Earth & Environmental) in 1995, the area is believed to be underlain by loose deltaic deposits that are susceptible to liquefaction when subjected to strong seismic shaking. Liquefaction is a dramatic reduction in soil strength that can occur as a result of strong cyclic loading as applied by an earthquake. Structures founded on subsoils that liquefy can undergo a range of unacceptable response ranging from severe structural damage to outright collapse.

Subsurface exploration and quantitative seismic assessment was recommended by AMEC in the 1995 report at this site to address the liquefaction potential.

3.0 FIELD ASSESSMENT

The subsurface exploration program included advancing three boreholes, as shown on Figure 2. Boreholes varied in depth from 9.1 to 15.9 m below existing grade.

The boreholes were advanced under the supervision of AMEC personnel on 03 December 2002 using equipment supplied by Dynamic Drilling Inc. including electronic piezocone penetration testing (CPT) equipment, support truck, and an operator for CPT testing as well as a truck mounted rig with solid stem augers for collecting soil samples. CPT was used at the three locations with two of the locations also being advanced by solid stem auger drilling to obtain soil samples for laboratory analysis. The soil stratigraphy encountered in the boreholes is described on the borehole logs presented in Appendix A. The soil was described using the Modified Unified Soil Classification System that is described in Appendix A.

The CPT consists of hydraulically pushing an electronically equipped cone with a base area 10 cm^2 tip into the ground. Continuous data is recorded with depth of the pressure on the cone tip (Q_c), the stress on the friction sleeve located immediately behind the cone tip (f_s) and the water pressure in a pressure transducer located behind the friction sleeve (U_2). For each of the three CPTs performed at the Fire Hall No.1, these values are plotted versus depth and are shown in Appendix B. The friction ratio, R_f, vs depth is also plotted in Appendix B for each CPT. The friction ratio is a percent value obtained by dividing the sleeve friction by the bearing resistance at the same depth and is an empirical indicator of soil type.

CPT is considered an effective in-situ tool to assess soil stratigraphy and soil parameters, which are based on empirical correlations. The CPT's were combined with porewater pressure dissipation tests and shear wave velocity testing at selected depths, which are used to assess the depth to the groundwater table and shear modulus of the soil.

Upon completion of drilling, a 25 mm diameter slotted standpipe was installed in BH02-1 and BH02-3. The backfill in the standpipe annulus comprised of slough material and auger cuttings. The wells were bentonite capped to prevent surface water from entering and were finished with a road box to allow vehicle traffic. Borehole locations were measured by AMEC and referenced to nearby surface features. The site was revisited on 06 December 2002 to record the groundwater level in both standpipes.

Laboratory testing was conducted in AMEC's Nanaimo laboratory and included moisture content, fine contents and one Atterberg Limit on the fines.

4.0 SUBSURFACE CONDITIONS

The borehole logs from the geotechnical drilling program are presented in Appendix A. The detailed description of the encountered subsurface conditions on the logs should be used in preference to the general description provided below.

4.1 Soil

The soils encountered on the site were relatively consistent at the three locations although the profile varied. In general, the soil profile included a surface layer of either asphalt or topsoil overlying a fill layer. Natural soil was then encountered and included either silt or sand, a variable layer of gravel and overlying layers of silt or sand.

The fill encountered underlying the surficial layer of topsoil or asphalt extended to an approximate depth of 1.3 to 1.5 m below surface. The fill was a sand material with some silt and gravel, trace clay and was loose to firm. The gravel sizes were up to 75 mm.

In BH02-1, silt was encountered underlying the fill. The silt was soft and damp with some clay and sand. A second silt layer was encountered at a depth of approximately 13.5 m. The silt had some clay was soft, grey and wet. Silt was not encountered in BH02-2, although the sand encountered had variable fines contents (% by dry weight finer than 0.074 mm)ranging from 13% to 38% of silt and clay particles.

Various methods exist by which piezocone data is used to identify the nature of the soils being penetrated. The interpretation of the CPT data agreed with the information from the boreholes, in that the soil profile consisted of a silty sand or sandy silt to a depth of about five metres. The consistency of the soil could be described as loose to compact with increasing consistency (i.e. more dense) at about the four metres.

The compact to dense soil was generally from four metres to about seven metres (in depth) on the west side of the site (BH02-1 and 2). It is possible that the interpreted dense zone was caused by spikes in the recorded data due to an increase in gravel content. Therefore, it may be appropriate to consider this layer as compact.

Sand was encountered in both boreholes. In BH02-1, the sand below the silt was fine to medium grained with little silt. Grain size analysis from a borehole sample indicated 17.4% silt and clay sizes in the sand. The sand in BH02-2 was similar in consistency and gradation but was encountered directly underlying the fill. Underlying a gravel layer, the sand encountered had trace to some silt, was loose in consistency, and wet. Trace shells were noted in this layer at a depth around 5.5 to 6 m. The fines content determined from a sample recovered at6.5 m depth in BH02-1 was 8.5%. CPT data indicated sand to about eight metres. The sand consistency varied in this layer from dense to loose with depth. The sand graded to a silty sand, sandy silt at seven metres below grade. The consistency of this soil was generally loose to compact.

The fines content of the sand increased with depth grading into a silt around 13.5 m below ground surface. BH02-1 was terminated at 15.9 m where dense soil was encountered. The soil was classified as clay till.

Gravel was encountered in all three borehole locations at around five metres below surface. The gravel layer was thickest at BH02-1 location where it was over one metre thick. The gravel at BH02-2 was only about 0.3 m thick. The gravel layer was drilled out for the CPT soundings, as the CPT probe could not penetrate this material.

A borehole was not advanced at the location of the third CPT (CPT02-3). The profile inferred from the soil behaviour type was similar in consistency in that the upper four metres was generally a silty sand or sandy silt, followed by a gravel layer. Underlying the gravel was a sand layer which graded with increasing depth to a silty sand or sandy silt.

4.2 Groundwater

The groundwater levels were measured on 06 December 2002 for the two standpipes. The water levels were 2.8 and 2.7 m below surface for BH02-2 and BH02-3, respectively.

It should be noted that groundwater levels will vary seasonally and in response to precipitation.

5.0 SEISMIC LIQUEFACTION ANALYSIS

Current seismic design aspects of construction in British Columbia are based on the 1998 British Columbia Building Code, in which seismic design criteria are provided in Section 4.1.9, entitled "Live Loads due to Earthquakes", and in Commentary J, a supplement to the 1990 National Building Code of Canada (NBCC). The objective of the earthquake resistant design requirements of the NBCC, for structures that are "post-disaster" is to remain fully functional following a major earthquake.

For design purposes, it is common to consider a "moderate" earthquake to have a peak firm ground acceleration with a 40% chance of being exceeded in 50 years (1 in 100 year earthquake). The NBCC defines a "major" earthquake as an earthquake which results in accelerations and velocities with a 10% chance of being exceeded in 50 years (1 in 475 year earthquake). Predicted peak ground acceleration values of 0.16 g and 0.47 g were used for the 1 in 100 year and 1 in 475 year earthquakes in the analysis. These acceleration values were obtained from the Geological Survey of Canada for Campbell River, BC. The peak firm ground acceleration is considered to occur at the top of bedrock in conformance with information provided by Geological Survey of Canada. Potential site-specific amplification and damping effects on the peak ground accelerations were considered (Idriss, 1990[1]). It is estimated that soil amplification will result in a peak horizontal acceleration at the ground surface of about 0.52 g (for the 1 in 475 year return period ground motion), which is based on previous studies on sites with similar geology. A magnitude 7.5 earthquake was used for the analysis which was considered a reasonable assumption based on the 1946 earthquake in the Forbidden Plateau area, west of Courtenay and Campbell River, which was a magnitude 7.3 event.

An assessment of the CPT data was undertaken to determine the seismic response of the subsoils and the potential for liquefaction and/or seismically induced ground movements. The potential for liquefaction of the site soils was evaluated using the method outlined by Robertson and Wride (1998)[2]. That method, following the same general approach as the Standard

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[1] Idriss, I.M., 1990. "Response of soft soil sites during earthquakes". *Proceedings, H. Bolton Seed Memorial Symposium*, edited by J.M. Duncan, BiTech Publishers, Vancouver, B.C., Volume **2**, pp. 273-289.

[2] Robertson, P.K. and Wride C.E. (Fear) 1998. "Evaluating cyclic liquefaction potential using cone penetration test". *Canadian Geotechnical Journal*, NRC Canada, **35**:442-459.

Penetration Test (SPT) based approach developed at the University of California (Berkeley) by the late Professor H.B. Seed and his coworkers, considers the liquefaction susceptibility of a soil for any given level of earthquake loading. That assessment considers the liquefaction resistance of a soil (as determined from CPT data and/or SPT data) and compares it against the data from a database of sites that did or did not liquefy under earthquake loading.

The liquefaction susceptibility of the foundation soils was also assessed based on the state parameter (ψ) approach for sands. The state parameter represents the difference between the void ratio of a sand and its void ratio (at the same mean normal stress) at its critical (or steady) state (Been and Jefferies 1985). The state parameter provides an index parameter for practical geotechnical engineering that is nevertheless anchored to a fundamental understanding of soil behaviour (Been and Jefferies, 1985)[3]. Where the sand is, for a given consolidation stress, loose of critical state, ψ is positive, and the soil will be contractant under shear. Soil that is contractant and sheared under undrained conditions (as exist for example during the cyclic loading imposed by an earthquake) can undergo liquefaction. Soil that is dense of critical state will be dilatant under shear, and is not susceptible to cyclic liquefaction. The state parameter approach does not consider the level of earthquake loading applied, so does not indicate if a given level of earthquake loading is sufficient to trigger liquefaction. It simply estimates in general terms the susceptibility to liquefaction for a range of potential triggering mechanisms, earthquake loading representing but one of these. Soil with a state parameter of less than –0.1 will generally not be a concern for liquefaction even under strong seismic loading.

For analysis purposes, CPT penetration resistances were normalized to equivalent Standard Penetration (SPT) resistances for driving energy and effective overburden pressures (SPT $N_{1(60)}$). This allows piezocone data to be used in liquefaction assessments, where the database developed by the late Professor H.B. Seed and his coworkers is expressed in terms of $(N_1)_{60}$ values. That database was developed based on case histories where seismic loading represented the triggering mechanism for undrained behavior.

Groundwater was assumed to be two metres below surface, as indicated by groundwater levels taken on 06 December 2002, and fines contents obtained from analysis of the sand and silt deposit were used.

The two analyses completed, generally agree with each other for the potential liquefaction susceptibility. Graphs of the analysis versus depth are provided in Appendix B. The CPT conducted at BH02-1 location indicated that the Factor of Safety (FoS) against liquefaction less than 1. A FoS of zero on the plots shown indicates that the potential for liquefaction should be evaluated using the state parameter approach. The state parameter was between 0 and -0.1 above 4 m and was less than –0.1 (negligible susceptibility to liquefaction) around 4 m. No CPT information was obtained from the gravel layer. Directly below the gravel layer a compact sand was encountered with a state parameter less than –0.1 which indicates a low susceptibility to liquefaction. This soil was encountered to about 7.5 m below ground surface where the soil again has a higher potential for liquefaction based on both the state approach and the FoS.

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[3] Been, K. and M.G. Jefferies (1985). "A State Parameter for Sands". Géotechnique, Volume 35, Number 2, pp. 99-112

The results from the second CPT, BH02-2, revealed similar results to BH02-1 except a relatively thin layer of soil highly susceptible to liquefaction was encountered in BH02-2 at about 2 m which would be around the footing elevation. Directly above and below the drilled out layer of gravel, a compact sand was encountered. This soil is consisted to have a low potential for liquefaction based on the state parameter. The compact sand extended to about 7.5 m below ground surface. Below the compact sand, both the state parameter and the FoS approaches indicated that the soil was liquefiable.

At the third location, CPT02-3, the upper 5 m had a state parameter between 0 and –0.1. The FoS against liquefaction was also less than 1 indicating the potential for liquefaction. A 0.7 m layer in thickeness was again drilled out at a depth of 5 m. Beneath the drilled out section, the state parameter indicated a low potential for liquefaction to an approximate depth of 7 m. Below 7 m, both analysis methods indicated that the soil was liquefiable with thin non-continuous soil layers with low potential for liquefaction.

In summary, the liquefaction assessment indicated that a significant portion of the soil profile underlying the site may be susceptible to liquefaction under the 1 in 475 year return period earthquake ground motions. A layer of gravel (ranging in thickness from 0.7 m to 1.3 m) was encountered in all three boreholes at an approximate depth of 5 m below ground surface, encapsulated within compact sand determined to have a negligible susceptibility to liquefaction as determined by the state approach. Based on the negligible susceptibility and the increased CPT penetration resistances encountered, it is judged that the gravel layer is not susceptible to liquefaction. This non-liquefiable layer commences at 4 m below ground surface on the west side (BH02-1 and 2) and at 5 m below ground surface on the east side. The layer extends to around 7 m below surface. The non-liquefiable zone therefore has a thickness ranging between 2 m and 3 m.

The 2 to 3 m thick layer of soil encountered at approximately 5 m below surface will provide a crust of non-liquefiable material to reduce the amount of deformation the building would undergo should the soils above and below this "crust" undergo liquefaction. The silty sand, sandy silt soil encountered above this will deform under a major seismic event. Since the layer is not consistent across the site, the potential for greater deformation on the east side exists and may result in differential liquefaction-induced settlements across the site. At present, the building is anticipated to be founded on 0.9 m of fill overlying the silty sand or sandy silt material with the natural soils at a depth of approximately 3 m below ground surface (i.e. the base of the footings is believed to be at 2 m, which is 2 m to 3 m above the top of the non-liquefiable "crust" zone). The estimate of the fill is based on the recommendations provided in the original geotechnical report prepared by Cook Pickering and Doyle in 1974. The site is relatively flat and lateral ground spreading caused by liquefaction is not considered an issue.

Additionally, as the footings are believed to be founded on a thin layer of fill material, 0.9 m, overlying the potentially liquefiable material, the potential for a different failure mechanism also exists. This other mechanism is considered a punching bearing failure in which the footing will essentially 'punch' through the soil. This failure mechanism is typical of footings founded on or near loose soils. Based on the SPT $(N_1)_{60}$ inferred from the CPT data, an estimated residual strength of the liquefied layer was obtained[4]. A FoS against 'punching' failure was calculated based on a one metre wide footing with a bearing pressure of 100 kPa (provided as the recommended bearing pressure by in the original geotechnical report). A 0.9 m 'crust' of placed fill material was also factored into the analysis. The non-liquefiable material encountered at a depth of 4 to 5 m was not considered in the analysis. The resulting FoS was below 1 indicating that the footing likely would also fail in bearing.

Based on a footing base at two metres below ground surface, potential settlements were estimated for the site using the empirical procedures developed by Tokimatsu and Seed (1987). The post seismic ground settlements ranged from 40 mm on the west side to up to 100 mm as the 'crust' of material is located deeper on the east side.

6.0 DISCUSSION

The objective for the assessment was to determine if the soil would liquefy under seismic loading conditions. The analysis undertaken indicated that a significant portion of the underlying soil profile at the site would liquefy under a major earthquake (deemed a 1 in 475 year return period event with a PGA of 0.47g). Based on the soil profile, the west side of the site would experience relatively minor deformation, about 40 mm, while the east side would experience greater deformation of about 100 mm. Additionally, the foundation would likely fail under bearing or 'punching'.

As the settlements estimated were from the footing foundation, greater settlements would be experienced outside of the building foundation. Settlement within utility trenches will also occur. Both will affect the functionality of the site to be 'post-disaster'.

In order for the Fire Hall No. 1 to meet the Building Code requirement for 'post-disaster' buildings, the site would need to have ground improvements conducted. This would involve densification and stabilization of the soil underneath the building. Several techniques have been utilized to reduce the susceptibility to liquefaction on sites with existing buildings, although, limited success has been achieved. One method of stabilization would be to inject jet grouting beneath the footings to strengthen the soils and render them non-susceptible to liquefaction. This method has been the most successful in silty soils. The grout would only require injection above the 'crust' encountered at 4 to 5 m depths. The combination of the non-liquefiable soil on site and the Jet Grouting would provide sufficient resistance to liquefaction as well as punching failure of the foundation. The ground improvement techniques would also need to be conducted on the utility lines or backup systems would need to be installed for the building. Typical costs for this are between \$35 and \$70 per cubic metre of stabilized soil. Based on the plan footing of 1100 m^2 and a full basement, the costs would range from \$180 000 to \$350 000 to stabilize the soil.

---

[4] Naesgaard, E, P. Byrne and G. Ven Huiaen, "Draft - Behaviour of Light Structures Founded on Soil 'Crust' over Liquefied Ground",

Further discussion could be provided on ground improvement techniques but the cost may be prohibitive given the limited success. Additionally, the building would need to be upgraded structurally. The structural assessment as well as estimated costs were conducted by Herold Engineering and are provided in Appendix C.

7.0 CLOSURE

Recommendations presented herein are based on a geotechnical evaluation of the findings of the site investigation noted. If conditions other than those reported are noted during subsequent phases of the project, AMEC should be notified and be given the opportunity to review and revise the current recommendations, if necessary.

This report has been prepared for the District of Campbell River for application to the project described in Section 2.0, above. Any use which a third party makes of this report, or any reliance on or decisions made based on it, are the responsibility of such third parties. AMEC accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report. It has been prepared in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made.

AMEC trusts that this report meets your present requirements and we thank you for this opportunity to have been of service. Please do not hesitate to contact the undersigned for further information.

Yours truly,

AMEC Earth & Environmental Limited

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
T.I. MacGOWAN
25486

18Mar-03

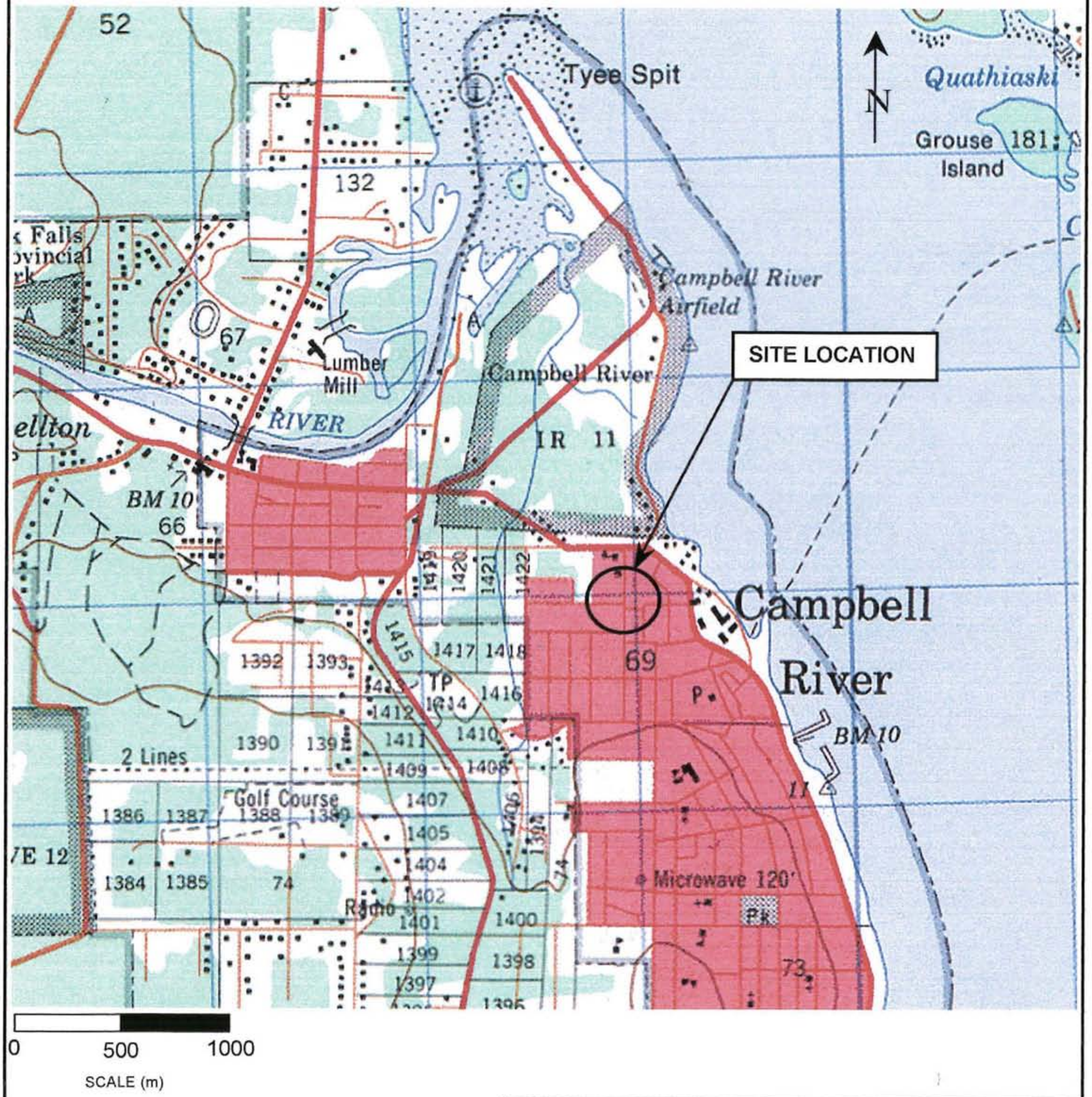
Tanys MacGowan, P.Eng.
Geotechnical Engineer

Reviewed by:

Todd Martin, P.Eng., P.Geo.,
Senior Geotechnical Engineer

APPENDIX A

Site Location Plan
Borehole Location Plan
Borehole Logs
Explanation of Terms & Symbols



Base Drawing From: BC Topographical Maps 1:50 000 – 92F14 Spectrum

amec

PROJECT:

Geotechnical Seismic Assessment

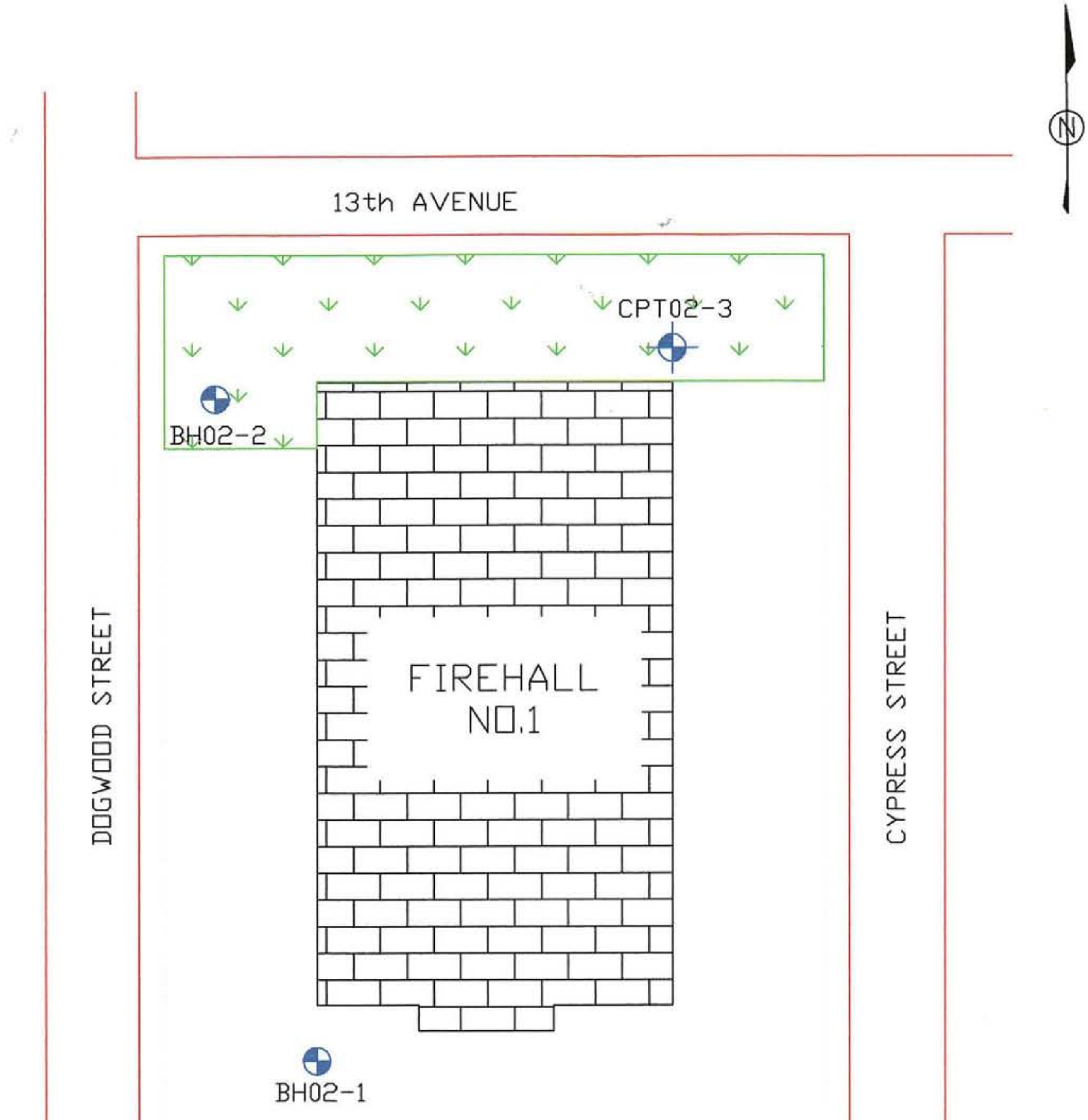
TITLE:

Site Location Plan
Fire Hall No.1
675 13th Avenue
District of Campbell River, BC

CLIENT:

District of Campbell River

DATE:	JOB No.:	SCALE:	FIGURE:	REV.
Nov 2002	NX01790-100	See Scale	FIGURE 1	A



LEGEND

⊕ Approximate Location of Cone Penetration Test (CPT) Hole

⊕ Approximate Borehole & CPT Location

Approximate property boundary

APPROXIMATE SCALE (m)

10 0 20

	PROJECT: Geotchnical Seismic Assessment
amec	TITLE: Borehole Location Plan Fire Hall No. 1 675 13th Avenue, Campbell River, B.C.
District of Campbell River	DATE: Dec 2002

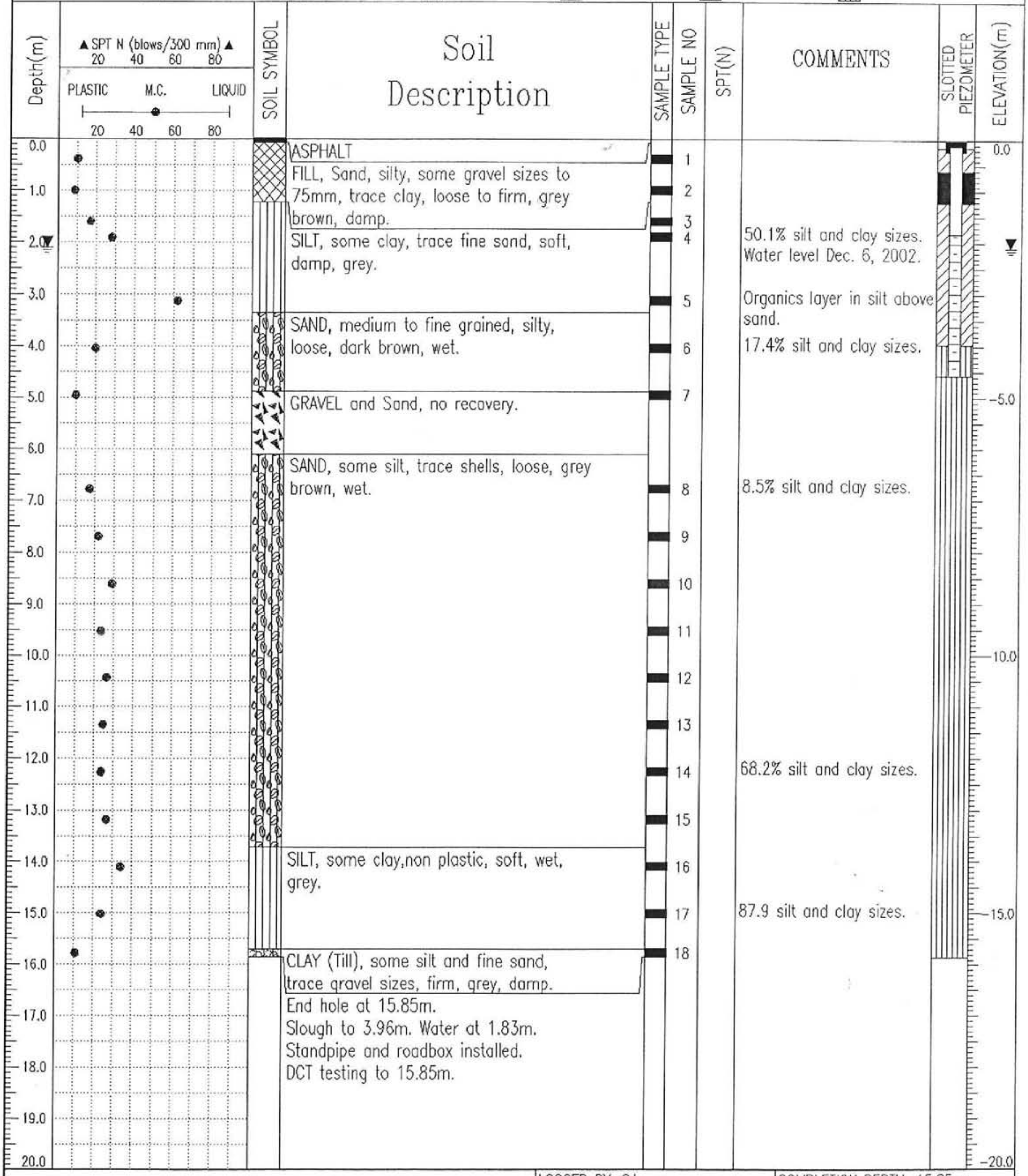
JOB No:	SCALE:	FIGURE:	REV.
NX01790	see scale bar	FIGURE 2	A

CLIENT: DISTRICT OF CAMPBELL RIVER
PROJECT: FIREHALL No. 1
BOREHOLE LOCATION: SEE FIGURE 2

CONTRACTOR: DYNAMIC DRILLING INC.
DRILL: TRUCK MOUNTED SOLID STEM AUGER

TEST HOLE NO: BH02–1
PROJECT NO: NX01790 – 100
ELEVATION:

SAMPLE TYPE ■ GRAB ▨ NO RECOVERY ☒ SPT ☰ A–CASING ▯ SHELBY TUBE ▮ PISTON TUBE
BACKFILL TYPE ■ BENTONITE ⁘ PEA GRAVEL ▥ SLOUGH ▧ GROUT ▨ DRILL CUTTINGS ⁘ SAND



Soil Description	Comments
ASPHALT	
FILL, Sand, silty, some gravel sizes to 75mm, trace clay, loose to firm, grey brown, damp.	
SILT, some clay, trace fine sand, soft, damp, grey.	50.1% silt and clay sizes. Water level Dec. 6, 2002. Organics layer in silt above sand.
SAND, medium to fine grained, silty, loose, dark brown, wet.	17.4% silt and clay sizes.
GRAVEL and Sand, no recovery.	
SAND, some silt, trace shells, loose, grey brown, wet.	8.5% silt and clay sizes. 68.2% silt and clay sizes.
SILT, some clay,non plastic, soft, wet, grey.	87.9 silt and clay sizes.
CLAY (Till), some silt and fine sand, trace gravel sizes, firm, grey, damp.	

End hole at 15.85m.
Slough to 3.96m. Water at 1.83m.
Standpipe and roadbox installed.
DCT testing to 15.85m.

AMEC Earth & Environmental Limited

LOGGED BY: SJ
REVIEWED BY: JL

COMPLETION DEPTH: 15.85 m
COMPLETE: 12/13/02

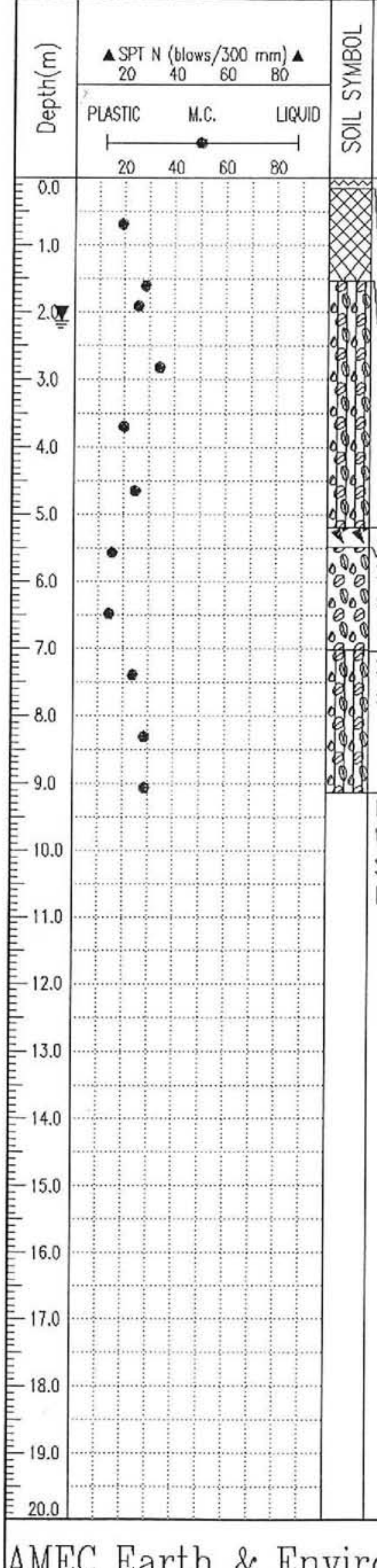
03/03/12 11:12AM (GEO20M)

DISTRICT OF CAMPBELL RIVER
FIREHALL No. 1
BOREHOLE LOCATION: SEE FIGURE 2

CONTRACTOR: DYNAMIC DRILLING INC.
DRILL: TRUCK MOUNTED SOLID STEM AUGER

TEST HOLE NO: BH02-2
PROJECT NO: NX01790 – 100
ELEVATION:

SAMPLE TYPE ■ GRAB ▨ NO RECOVERY ☒ SPT ▤ A-CASING ▥ SHELBY TUBE ▮ PISTON TUBE
BACKFILL TYPE ■ BENTONITE ⁚ PEA GRAVEL ▥ SLOUGH ▨ GROUT ▨ DRILL CUTTINGS ⁚ SAND



Soil Description	SAMPLE NO	COMMENTS
TOPSOIL, some clay and silt, organic, black.	1	
FILL, Sand, silty, some gravel sizes to 50mm, trace clay, loose, brown with rusty streaks, damp.	2 3	Trace organics and rootlets.
SAND, fine grained, silty, some rusty oxides, soft, mottled brown and grey, damp to wet.	4 5 6	13.2% clay and silt sizes.
GRAVEL and sand, no recovery.	7	
SAND, medium to fine grained, trace silt, shells, soft, grey, wet.	8	
SAND, fine grained, silty, trace clay, soft, grey, wet.	9 10 11	37.6% clay and silt sizes.

End hole at 9.14m.
Water at 2.13m.
Slough at 4.27m.
Backfilled with cuttings.

ELEVATION(m): 0.0, ▼ (2.0), –5.0, –10.0, –15.0, –20.0

AMEC Earth & Environmental Limited

LOGGED BY: SJ
REVIEWED BY: JL

COMPLETION DEPTH: 9.14 m
COMPLETE: 12/13/02
Page 1 of 1

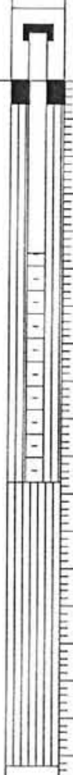
03/03/12 11:13AM (GEO20M)

DISTRICT OF CAMPBELL RIVER	CONTRACTOR: DYNAMIC DRILLING INC.	TEST HOLE NO: BH02-3
FIREHALL No. 1	DRILL: TRUCK MOUNTED SOLID STEM AUGER	PROJECT NO: NX01790 - 100
BOREHOLE LOCATION: SEE FIGURE 2		ELEVATION:

SAMPLE TYPE ■ GRAB ▨ NO RECOVERY ☒ SPT ☰ A-CASING ▯ SHELBY TUBE ▮ PISTON TUBE

BACKFILL TYPE ■ BENTONITE ⁙ PEA GRAVEL ▥ SLOUGH ⁘ GROUT ▨ DRILL CUTTINGS ░ SAND

Depth(m)	▲SPT N (blows/300 mm)▲ 20 40 60 80 PLASTIC M.C. LIQUID 20 40 60 80	SOIL SYMBOL	Soil Description	SAMPLE TYPE	SAMPLE NO	SPT(N)	COMMENTS		ELEVATION(m)
0.0			CPT testing only. No sampling.						0.0
1.0									
2.0									
▼							Water level Dec. 6, 2002.		▼
3.0									
4.0									
5.0									-5.0
6.0									
7.0									
8.0									
9.0			End ~~DCP~~ CPT testing at 9.14m. Water at 2.4m. Stanpipe installed.						
10.0									10.0
11.0									
12.0									
13.0									
14.0									
15.0									15.0
16.0									
17.0									
18.0									
19.0									
20.0									-20.0



AMEC Earth & Environmental Limited	LOGGED BY: SJ	COMPLETION DEPTH: 9.14 m
	REVIEWED BY: JL	COMPLETE: 12/13/02
		Page 1 of 1

03/03/12 11:13AM (GE020M)

EXPLANATION OF TERMS AND SYMBOLS

The terms and symbols used on the borehole logs to summarize the results of field investigation and subsequent laboratory testing are described in these pages.

It should be noted that materials, boundaries and conditions have been established only at the borehole locations at the time of investigation and are not necessarily representative of subsurface conditions elsewhere across the site.

TEST DATA

Data obtained during the field investigation and from laboratory testing are shown at the appropriate depth interval.

Abbreviations, graphic symbols, and relevant test method designations are as follows:

•C	Consolidation test	•ST	Swelling test
D_R	Relative density	TV	Torvane shear strength
•k	Permeability coefficient	VS	Vane shear strength
•MA	Mechanical grain size analysis	w	Natural Moisture Content (ASTM D2216)
	and hydrometer test	w_l	Liquid limit (ASTM D 423)
N	Standard Penetration Test (CSA A119.1-60)	w_p	Plastic Limit (ASTM D 424)
N_d	Dynamic cone penetration test	E_f	Unit strain at failure
NP	Non plastic soil	γ	Unit weight of soil or rock
pp	Pocket penetrometer strength	γ_d	Dry unit weight of soil or rock
•q	Triaxial compression test	ρ	Density of soil or rock
q_u	Unconfined compressive strength	ρ_d	Dry Density of soil or rock
•SB	Shearbox test	C_u	Undrained shear strength
SO_4	Concentration of water-soluble sulphate	•	Seepage
		▼	Observed water level

• The results of these tests are usually reported separately

Soils are classified and described according to their engineering properties and behaviour.

The soil of each stratum is described using the Unified Soil Classification System[1] modified slightly so that an inorganic clay of "medium plasticity" is recognized.

The modifying adjectives used to define the actual or estimated percentage range by weight of minor components are consistent with the Canadian Foundation Engineering Manual[2].

Relative Density and Consistency:

Cohesionless Soils		Cohesive Soils	
Relative Density	SPT (N) Value	Consistency	Undrained Shear Strength c_u (kPa)
Very Loose	0-4	Very Soft	0-10
Loose	4-10	Soft	10-25
Compact	10-30	Firm	25-50
Dense	30-50	Stiff	50-100
Very Dense	>50	Very Stiff	100-200
		Hard	>200

Standard Penetration Resistance ("N" value)
The number of blows by a 63.6kg hammer dropped 760 mm to drive a 50 mm diameter open sampler attached to "A" drill rods for a distance of 300 mm.

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[1] "Unified Soil Classification System", Technical Memorandum 36-357 prepared by Waterways Experiment Station, Vicksburg, Mississippi, Corps of Engineers, U.S. Army. Vol. 1 March 1953.

[2] "Canadian Foundation Engineering Manual", 3rd Edition, Canadian Geotechnical Society, 1992.

MODIFIED UNIFIED CLASSIFICATION SYSTEM FOR SOILS

MAJOR DIVISION			GROUP SYMBOL	GRAPH SYMBOL	COLOUR CODE	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
COARSE GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 75µm)	GRAVELS MORE THAN HALF THE COARSE FRACTION LARGER THAN 4.75mm	CLEAN GRAVELS (LITTLE OR NO FINES)	GW		RED	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	$C_U = \frac{D_{60}}{D_{10}} > 4;\ C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			GP		RED	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		DIRTY GRAVELS (WITH SOME FINES)	GM		YELLOW	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12 %	ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4
			GC		YELLOW	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE P.I. MORE THAN 7
	SANDS MORE THAN HALF THE COARSE FRACTION SMALLER THAN 4.75mm	CLEAN SANDS (LITTLE OR NO FINES)	SW		RED	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_U = \frac{D_{60}}{D_{10}} > 6;\ C_C = \frac{(D_{60})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			SP		RED	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		DIRTY SANDS (WITH SOME FINES)	SM		YELLOW	SILTY SANDS, SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12 %	ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4
			SC		YELLOW	CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE P.I. MORE THAN 7
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75µm)	SILTS BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 50\%$	ML		GREEN	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (SEE BELOW)	
		$W_L < 50\%$	MH		BLUE	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDS OR SILTY SOILS		
	CLAYS ABOVE "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 30\%$	CL		GREEN	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY OR SILTY CLAYS, LEAN CLAYS		
		$30\% < W_L < 50\%$	CI		GREEN-BLUE	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS		
		$W_L > 50\%$	CH		BLUE	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS		
	ORGANIC SILTS & CLAYS BELOW "A" LINE	$W_L < 50\%$	OL		GREEN	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	WHENEVER THE NATURE OF THE FINES CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER "F", E.G. SF IS A MIXTURE OF SAND WITH SILT OR CLAY	
		$W_L > 50\%$	OH		BLUE	ORGANIC CLAYS OF HIGH PLASTICITY		
HIGHLY ORGANIC SOILS			Pt		ORANGE	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR, AND OFTEN FIBEROUS TEXTURE	

SPECIAL SYMBOLS

LIMESTONE		OILSAND	
SANDSTONE		SHALE	
SILTSTONE		FILL (UNDIFFERENTIATED)	

SOIL COMPONENTS

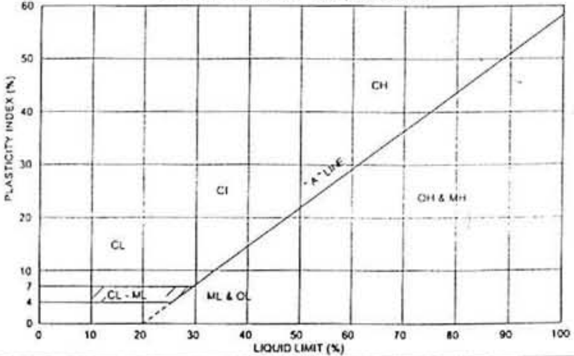
FRACTION	U.S. STANDARD SIEVE SIZE		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS	
GRAVEL	PASSING	RETAINED	PERCENT	DESCRIPTOR
COARSE	76mm	19mm		
FINE	19mm	4.75mm	35-50	AND
SAND				
COARSE	4.75mm	2.00mm	20-35	Y/EY
MEDIUM	2.00mm	425µm		
FINE	425µm	75µm	10-20	SOME
FINES (SILT OR CLAY BASED ON PLASTICITY)	75µm		1-10	TRACE

OVERSIZED MATERIAL

ROUNDED OR SUBROUNDED:
COBBLES 76mm TO 200mm
BOULDERS > 200mm

NOT ROUNDED:
ROCK FRAGMENTS > 76mm
ROCKS > 0.76 CUBIC METRE IN VOLUME

PLASTICITY CHART FOR SOILS PASSING 425 µm SIEVE



NOTES:

1. ALL SIEVE SIZES MENTIONED ON THIS CHART ARE U.S. STANDARD A.S.T.M. E.11
2. COARSE GRAIN SOILS WITH 5 TO 12 % FINES GIVEN COMBINED GROUP SYMBOLS, E.G. GW-GC IS A WELL GRADED GRAVEL SAND MIXTURE WITH CLAY BINDER BETWEEN 5 AND 12% FINES.

AMEC Earth &
Environmental Limited

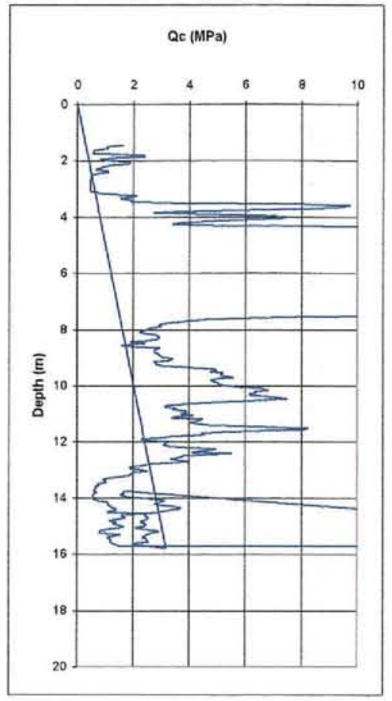
amec

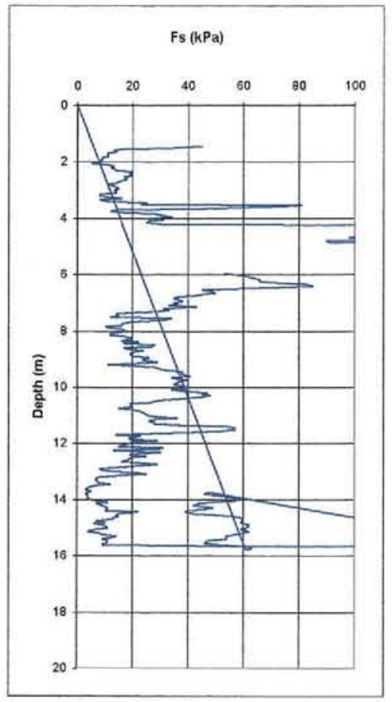
APPENDIX B

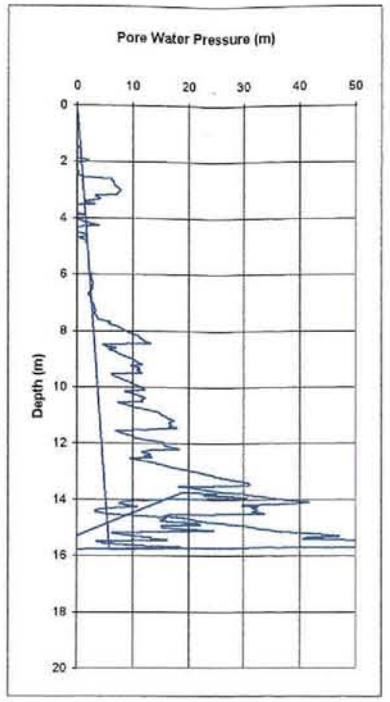
CPT Data

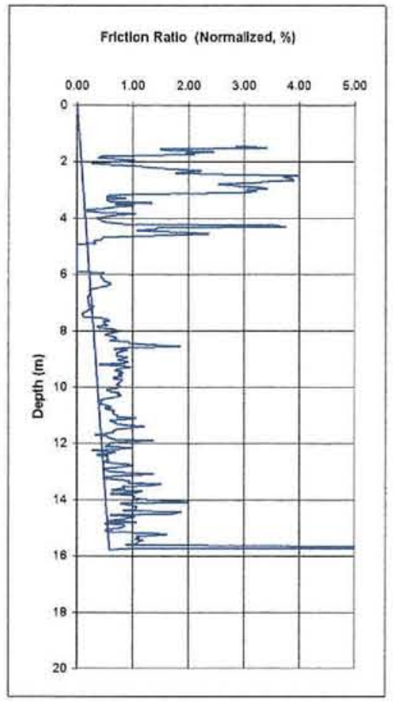
NX01790 Campbell River Fire Hall No.1
CPT 1

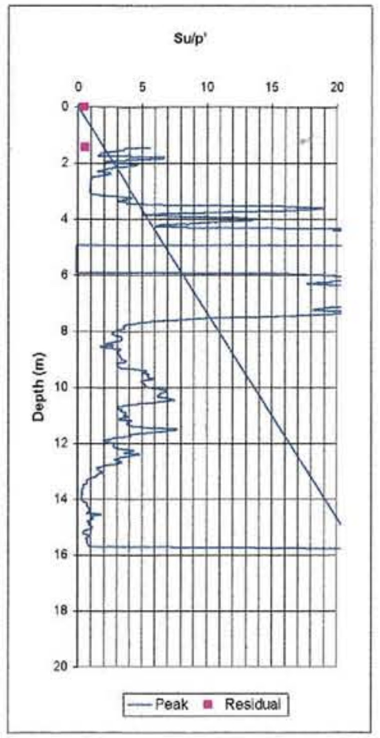
amec

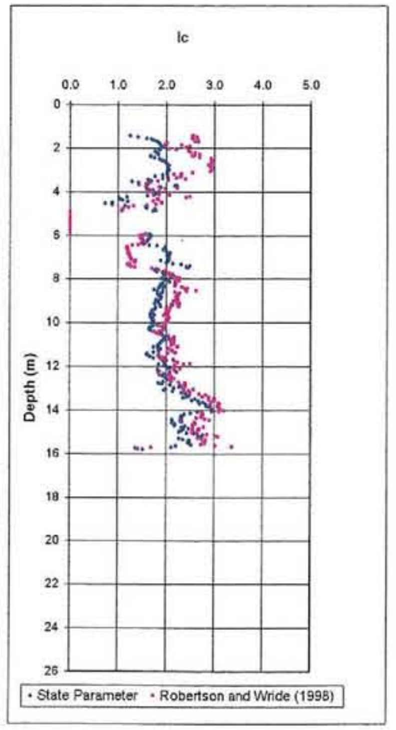


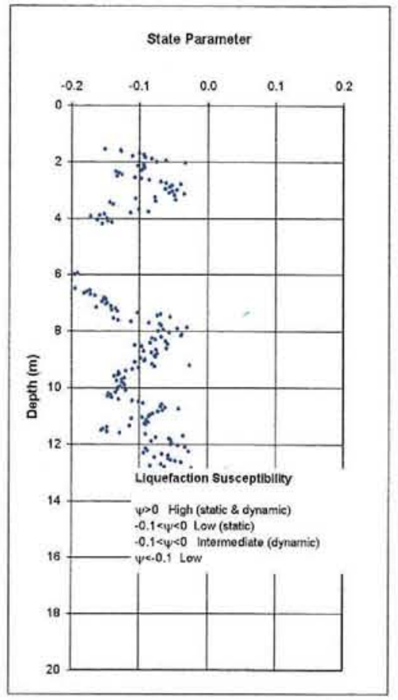


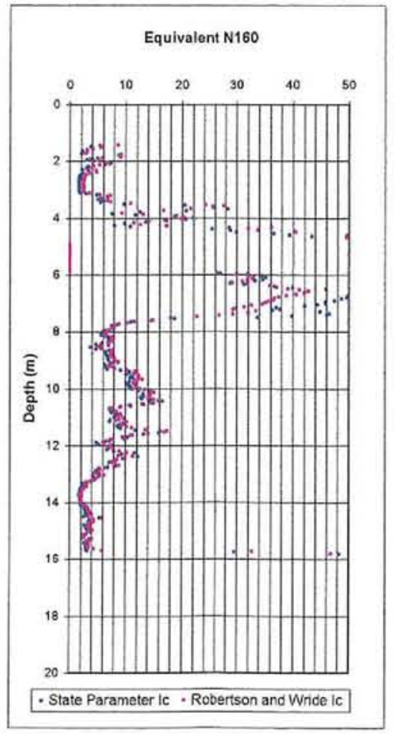


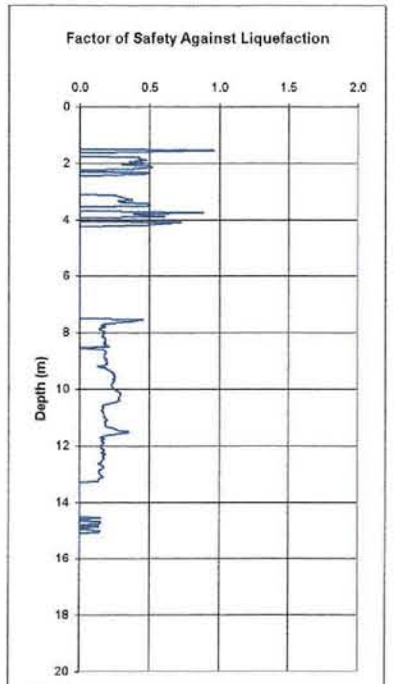












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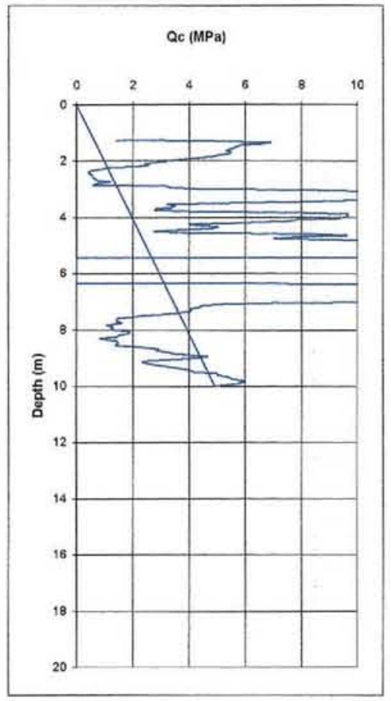
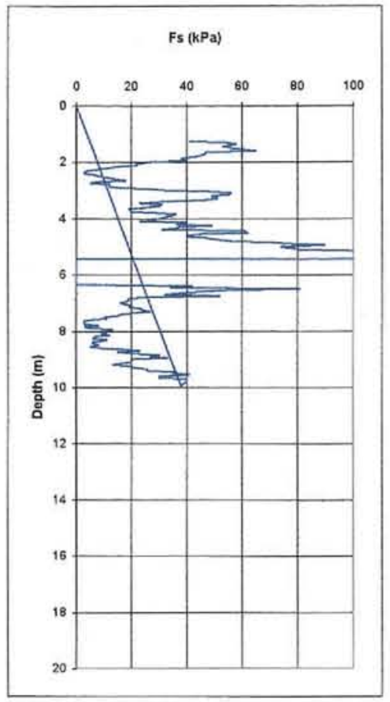
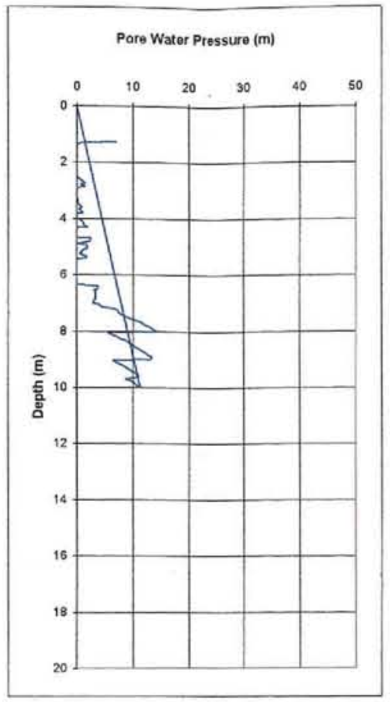
Robertson and Wride (1998)

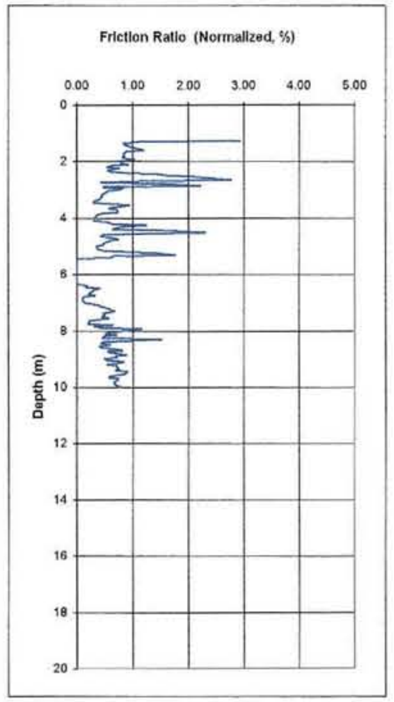
NX01790 Campbell River Fire Hall No.1
CPT 2

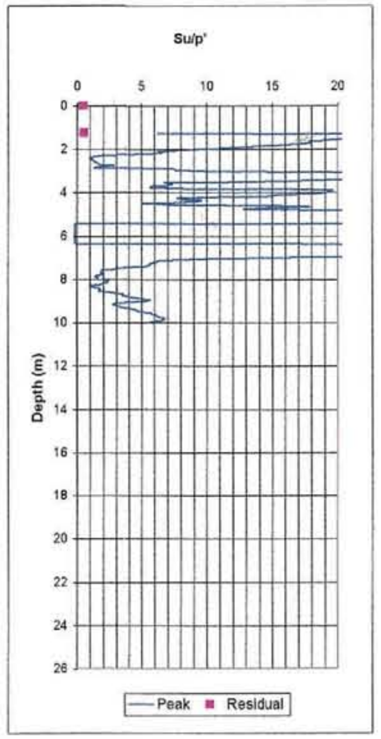
amec

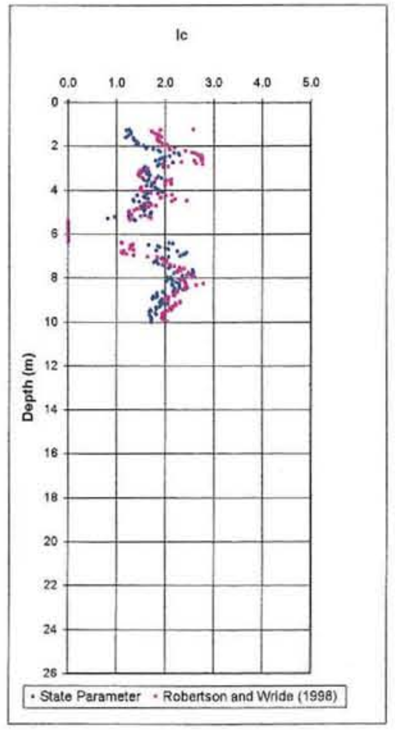


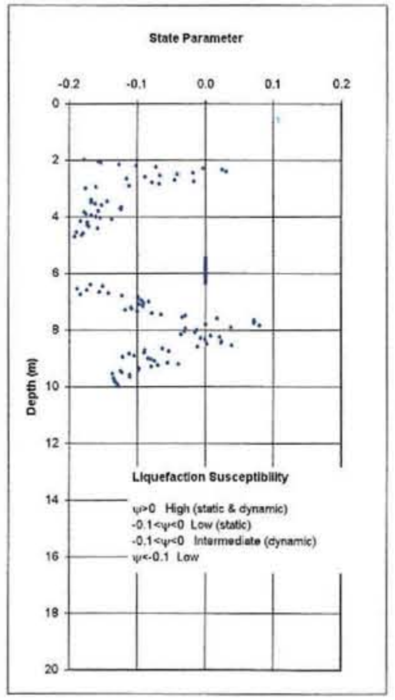


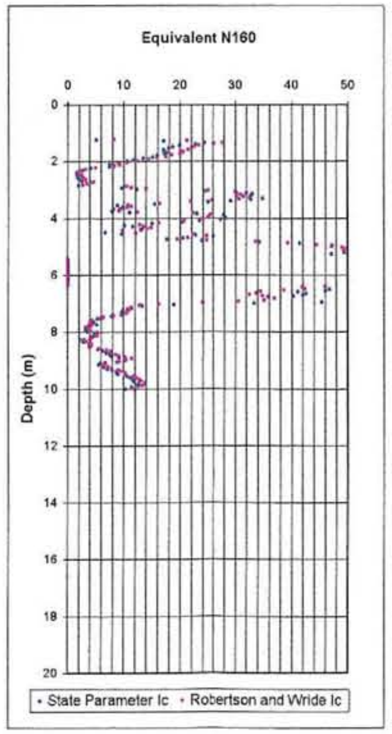


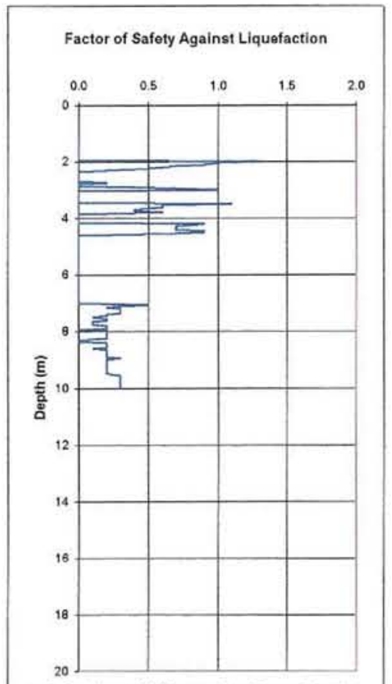












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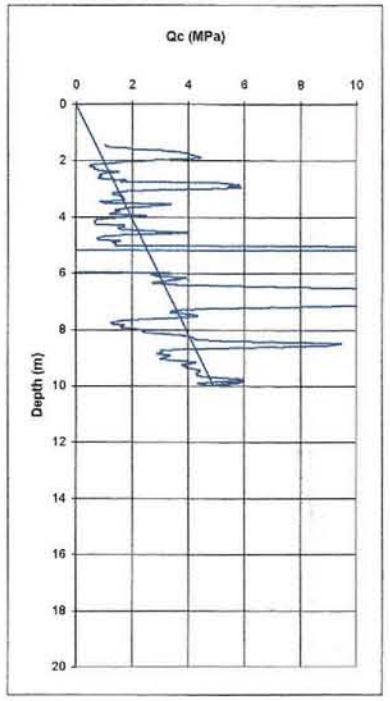
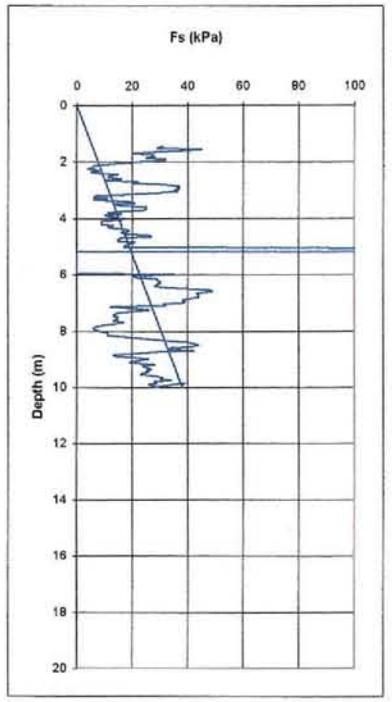
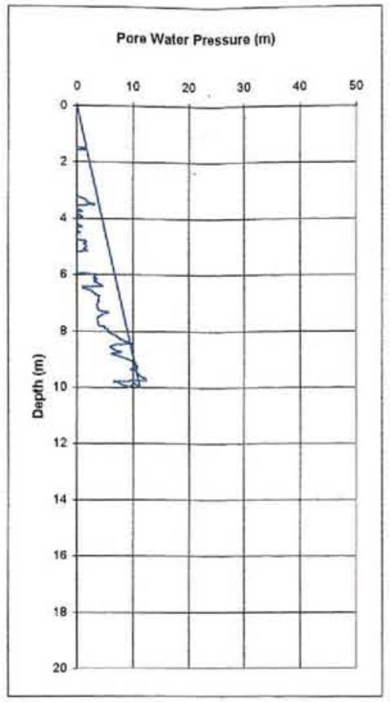
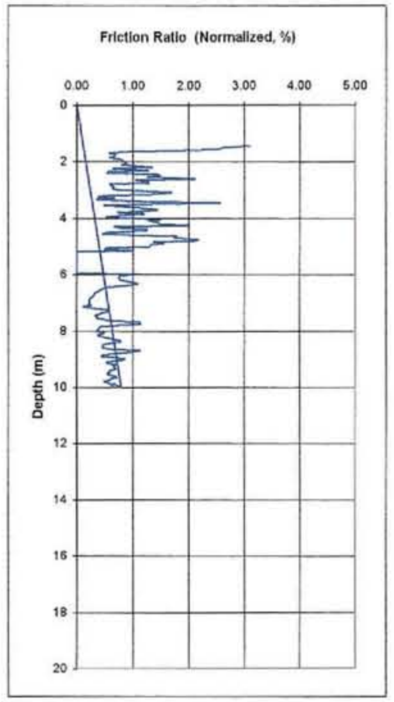
Robertson and Wride (1998)

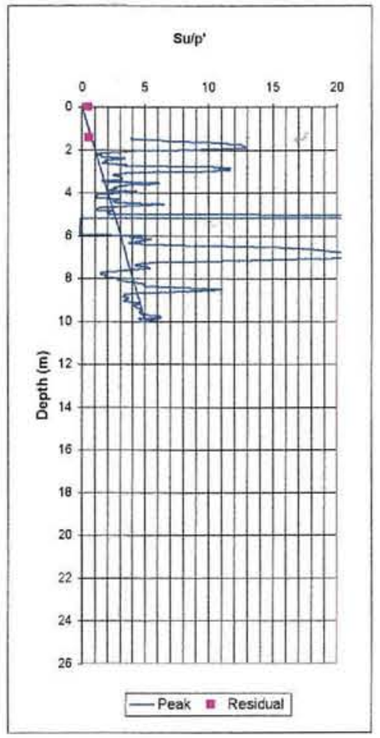
NX01790 Campbell River Fire Hall No.1
CPT 3

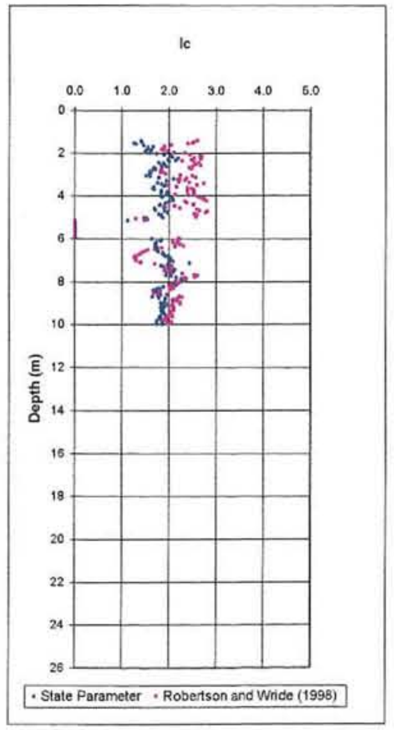


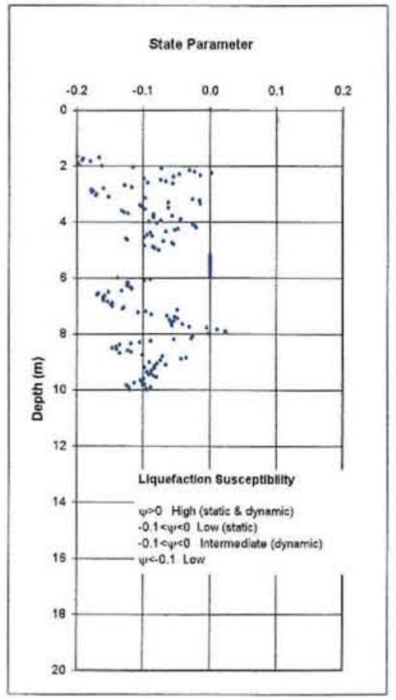


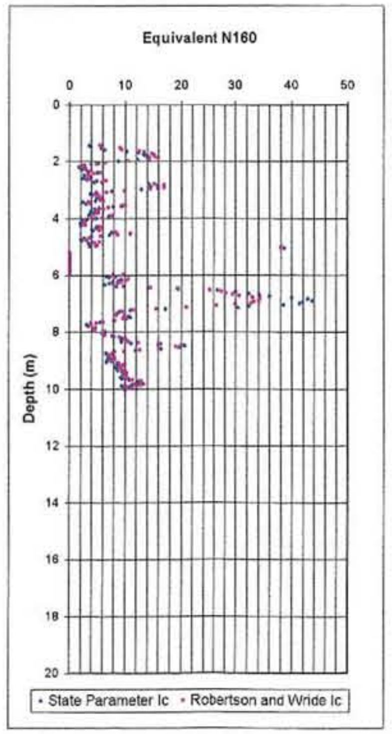


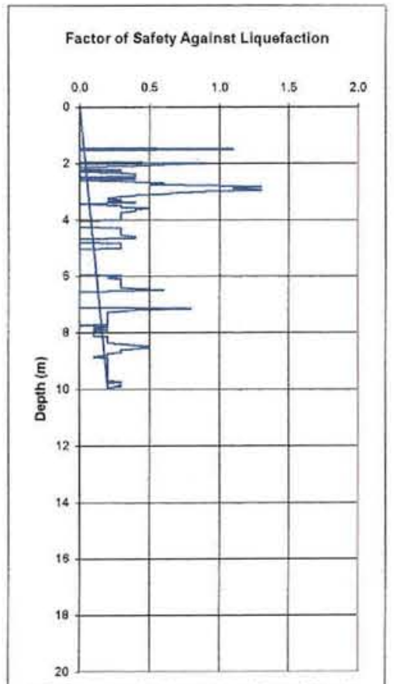












a=0.47g (value provided from geological Survey of Canada)
Robertson and Wride (1998)

APPENDIX C

Herold Engineering Structural Analysis

Structural Seismic Assessment – Fire Hall #1

Prepared by Herold Engineering Limited

Fire hall #1 is comprised of a 500m^2 two-storey office building with an attached 600m^2 single story apparatus floor. Both components of the fire hall are constructed from 200 mm concrete masonry block. The concrete block walls along the front and sides of the office portion of the building are in stack bond pattern with the remainder in running bond pattern. The floor and roof assemblies are metal deck supported by steel joists with the floor having a concrete slab poured on to the decking. The main floor has a suspended concrete slab over a partial basement. Original structural drawings, dated 1976, indicated that the masonry walls are reinforced with 15M vertical bars at 800 mm. A single course horizontal bond beam, reinforced with 2 -12mm bars, is indicated on the plans at floor and roof levels.

While the fire hall block walls do have some reinforcing, it does not meet current code requirements. The stack bond portion of the building possesses low lateral load shear resistance compared to running bond block walls. The large number of openings in the front wall coupled with the stack bond masonry produces a lateral load resisting capacity of the two-storey portion of the building estimated at less than 30% of current design requirements for a post-disaster building.

The apparatus floor portion of the building lacks significant lateral load resisting elements in the east-west direction (parallel to the main doors). The masonry pilasters are estimated to have a lateral load resisting capacity of less than 30% of current design requirements for a post-disaster building. Long-span diaphragms in this area are also under strength, but the significant deficiency is the block pilasters at the door openings.

The seismic upgrade of this building is expected to range from $270 - 325/m^2 ($25-30/ft^2) of floor space. The rehabilitation of partially reinforced masonry buildings is typically expensive. The deficient masonry must be restrained in the out-of-plane direction and a new lateral load resisting system installed. In this case, a typical upgrade would include new steel stud interior walls against the existing masonry walls, with helifix ties being installed to restrain the block walls. Concrete shear walls or steel braced frames, including new foundations, would also be required in the office and apparatus portions of the building to provide sufficient lateral strength and stiffness to resist the code level lateral loads. This type of upgrade is extremely invasive and costly as all the existing interior finishes against exterior walls must be removed and replaced with structural steel studs. The apparatus floor roof diaphragm also will require upgrading by rewelding the existing metal deck. This diaphragm upgrade will require the removal and subsequent reapplication of the roofing membrane. The connection between the walls and the floor and roof diaphragms will also have to be upgraded.

Expected cost of seismic upgrade (structural components only)..................$520,000.00