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LOCATION Fort Severn

New Airport Building

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I.-30 SEPT. 1976

FOUNDATION INVESTIGATION REPORT

CONTRACT NO 91-700



Ministry of
Transportation

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Note: For purposes of the contract, this report supersedes all other Foundation Reports prepared by, or for the Ministry in connection with the above mentioned project.

EXPLANATION OF TERMS USED IN REPORT

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N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_r	kPa	RESIDUAL SHEAR STRENGTH
τ_f	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_f}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m ³	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

FOUNDATION INVESTIGATION
NEW AIRPORT BUILDING
FORT SEVERN AIRPORT
WP. 2023-91-02
FORT SEVERN, ONTARIO

PROJECT AND SITE DESCRIPTION

We understand that a new building will be constructed at the Fort Severn Airport (see Figures 1 and 2). The proposed building is a two-storey complex with offices, garages, warehouse and workshop.

Based on published information, Fort Severn is located within the Hudson Bay Lowlands and within the continuous permafrost zone, but near the southern limit boundary with the discontinuous permafrost zone. Therefore, it is possible that the area of the site could be underlain by "warm" permafrost (viz. ground temperature slightly cooler than the freezing point) and patches/pockets of unfrozen soil. Because of the proximity to Hudson Bay, the subsoils likely consist of sediments such as fine-grained sands, silts and clays which were deposited in a marine environment; ridges of glacial till may also exist between low-lying areas.

METHODOLOGY

The fieldwork was carried out between June 11 and 13, 1991. The major components of the investigation consisted of:

- Visual examination and assessment of subsurface conditions by digging two test pits (TP1 and TP2) and advancing six probe holes (P1 to P6) to about 7 m depth.
- Drilling of three small diameter (about 30 mm) uncased sampled drill holes TC-1, TC-2 and TC-2A, about 7 m deep, to investigate the subsurface conditions and to install thermocouple strings to determine and monitor in-situ profiles of ground temperature.

The locations of test pits, drill holes, probes and thermocouple string installations are shown on Figure 3. Two test pits (2 m by 10 m) were excavated using the available on-site front-end loader to a depth of about 1 m. Deeper excavation was not possible because of the encountered frost zone at and below this depth. Salinity of the ponded water in the test pits was measured by a conductivity meter.

As outlined in our above-mentioned Proposal P11-2830, it was considered that, due to the possible "warm" permafrost at the site, conventional core drilling without the use of expensive special coolants and refrigeration would likely thaw the sample during the coring process.

Therefore, special, light weight, portable drilling equipment (Pionjar) supplied and operated by Sonic Soil Sampling Inc. from Toronto, Ontario, was used to drill the holes and to advance the probes. The Pionjar drilling unit consists of a gas driven engine, 25 mm diameter steel rods, soil sampling tubes and special drill bits.

In drill holes TC-1 and TC-2A (See Figure 3), the soil was sampled by a 33 mm outside diameter tube sampler, at an interval of about 0.9 m, down to a depth of about 7 m below ground surface. The sampler is about 300 mm in length and 25 mm inside diameter. The recovered samples were examined in the field and stored in airtight containers for further examination and testing in our geotechnical laboratory in Mississauga, Ontario. Water content, grain size distribution and Atterberg Limits (plasticity) were carried out on selected samples.

Thermocouple strings were installed in drill holes TC-1 and TC-2 at the locations shown on Figure 3. The thermocouple strings consisted of temperature sensing elements, electric wire and CPVC tubing. The electric wire was inserted into the CPVC tubing and connected to the temperature sensing elements which were mounted and protected on the exterior surface of the CPVC tubing at about 0.9 m intervals along the tubing. The pre-fabricated thermocouple strings were advanced/pushed into the pre-drilled holes to refusal, which occurred at a depth of about 4 m below ground surface. A thermocouple string could not be installed in drill hole TC-2A due to sloughing of granular material into the hole.

The fieldwork was carried out and supervised throughout by our D. E. Becker, P. Eng., who organized the field working team, selected the locations for investigation, performed visual examination of samples, directed the sampling and instrumentation installation operations, recorded the observations and measurements, and placed the samples in labelled, airtight containers.

RESULTS OF INVESTIGATION

While the Fort Severn area is generally quite flat, the site of the proposed building lies on a local topographical high consisting of a reasonably well-drained sandy ridge or possibly an esker. The proposed site area had been recently stripped of its 0.3 m to 0.4 m thick muskeg cover prior to the start of the fieldwork.

The results of the investigation are summarized in Tables 1 and 2 and on Figures 3 to 10, inclusive.

Subsurface Conditions

Two drill holes (TC-1 and TC-2A) and two test pits (TP1 and TP 2) were put down to determine the subsurface conditions at these locations. The subsoil conditions at each of the drill hole locations are summarized in Table 1 and plotted on Figure 4. The descriptions of the soil samples together with the laboratory test results are summarized in Table 2. Grain size distribution curves and Atterberg Limits test results are presented on Figures 5 to 9 and on Figure 10, respectively.

The encountered subsoils in the two drill holes consist of a layer of brown sand, about 3 m to 4.5 m thick, overlying a 1.5 m to 2 m thick layer of grey-black fine sand, sandy silt to silt material. The drill holes were terminated in a grey-black silt some clay, clayey silt to silty clay deposit. The measured water content on samples varies from about 8 per cent in the sand layer to about 20 per cent in the silty clay deposit (see Table 2).

Within the excavated test pits, groundwater seepage was observed at about 0.6 m below ground surface. The groundwater ponded within the test pit above the frost line. The salinity of the ponded water in the two test pits was measured using a portable conductivity meter. The measured salinity at the time of investigation was quite low, being about 0.03 per cent or 0.3 g/l.

Frozen Soil

The extent of the frozen soil at the test locations was estimated from the drill probes, soil sampling and test pit excavation operations. As shown on Figures 3 and 4, the inferred frozen soil thickness at the time of investigation was about 0.4 m to 1.0 m, with the depth to the top of the frost line at about 0.9 m to 1.2 m below ground surface. These depths to the frost layer are considered to be a result of the recent (i.e. May/June 1991) stripping of the muskeg layer across the site, as part of site preparation operations, to expose the natural sand deposit. In areas where the muskeg layer had been left intact (i.e. natural muskeg cover surrounding the prepared/stripped site area) frost was encountered at 0.2 m below ground surface. The removal of the muskeg

(which acts as a natural insulator on the ground surface) has apparently accelerated the thawing of the sand deposit.

Ground Temperature Measurement

The ground temperature readings were taken every week after installation by local MTO field staff. The measured temperature profiles (TC-1 and TC-2) are summarized in Table 3 and some selected profiles are presented on Figure 4.

After an elapsed time of one-half day, the measured temperatures by the thermocouple string varied from about 4°C, at depths corresponding to the frozen soil layer, to about 11°C at ground surface. The ground temperatures below the inferred frozen soil layer varied in a narrow range between 4°C and 6°C. The magnitude of, and small variation in, measured temperature below the frozen soil layer may be due to the influence of in-flow of the "warm" groundwater, above the frost line, into the open borehole. The measured air temperature was about 13°C.

During the period between June 13 and June 28, 1991, there was only a slight variation in the measured ground temperature profile. The ground temperature measurements on July 5, 1991 indicated that the ground temperatures dropped by about 2°C to 5°C, but remained above freezing. Further ground temperature monitoring results during the period of July 12 to July 19, 1991, indicated that the ground temperatures between 1.9 m and 3.8 m depth were slightly below the freezing point (0°C - see Figure 4 and Table 3). The measured air temperature was about 10°C to 12°C. On July 26, 1991, the measured ground temperatures between 1.9 m and 3.8 m were slightly above the freezing point (0°C).

Temperature readings during August 1991 were found to be quite erratic and dubious; they are, therefore, not reported in Table 3. To ascertain whether these readings were caused by a malfunctioning readout unit, we requested MTO to return the readout unit to our Mississauga laboratory for examination and calibration, if required. However, the thermocouple installations were destroyed when the actual thermocouple strings were extricated from the ground by local MTO personnel and returned to us. Therefore, further ground temperature monitoring is not possible.

While the temperature monitoring information is not conclusive, it suggests that marginal frost, within the area investigated, possibly extends to at least 4 m depth. However, the observed relative ease at which the Pionjar drilling unit penetrated the ground below the 2 m depth, and examination of the recovered fine-grained silty and clayey soil at depth, suggests that during the investigation the ground was not frozen below about 2 m depth.

Based on the above, it is considered that the 2 m depth is representative of the inferred bottom of the frost layer during the investigation; it should also closely correspond to the maximum depth of frost penetration that developed over the 1990/1991 winter season. The local native residents considered the past winter to be "very cold" and, therefore, the observed, unexpected relatively shallow depth of frost penetration cannot be attributed to an unseasonably mild winter season.

Existing correlations of frost penetration and the average annual Freezing Index (i.e. cumulative total of difference between annual daily mean air temperature and the freezing point (0°C) expressed in units of degree days), based on observations in open clear areas such as roads and aircraft runways, suggest a 3 m depth of frost penetration for the Fort Severn area (with a Freezing Index of about 3250°C degree days). However, these general correlations do not take into account soil type or insulating effect of muskeg cover. For example, the depth of frost penetration is greater in coarse-grained gravelly soils than in fine-grained clayey soils. Our calculations for a coarse-grained granular soil with a dry density of 16.5 kN/m^3 and water content of 15 per cent, under a Freezing Index of 3250°C degree days, indicate a frost penetration depth of 3.6 m. For a fine-grained soil, a calculated value of 2.9 m is obtained.

As mentioned above, it is considered that the observed relatively shallow depth of frost penetration at the site is a result of the insulating effect of muskeg, which until a month or so ago covered the area investigated. If the effect of a 0.3 m thick layer of muskeg is included in the analysis, a frost depth of about 2 m is calculated. This value agrees quite well with the inferred frost depth across the proposed building area.

NOTE: The preceding report is a copy of the factual information from the Foundation Investigation Report prepared by Golder Associates (consulting geotechnical engineers for this project), under the technical supervision of the MTO Foundation Design Section.

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TABLE 1
SUMMARY OF SUBSOIL CONDITIONS

DRILL HOLE NO.	DEPTH (m)	DESCRIPTION
TC-1	0.0 - 3.1	Brown SAND, trace to some silt, occasional fine gravel, occasional shell fragments, occasional thin layers of SAND and GRAVEL to GRAVELLY SAND. Probably frozen between 1.2 m and 1.8 m.
	3.1 - 3.9	Grey-black, fine SAND, some silt to SILTY SAND trace clay.
	3.9 - 6.1	Grey-black SANDY SILT to SILT and fine SAND trace clay becoming SILT trace fine sand, trace to some clay with depth. Occasional seams of silty fine sand.
	6.1 - 7.6	Grey-black SILT, some clay, trace fine sand, occasional thin black clayey silt to silty clay seams.
TC-2A	0.0 - 4.5	Brown SAND, trace to some silt, occasional fine gravel, occasional shell fragments, occasional thin layers of SAND and GRAVEL to GRAVELLY SAND. Probably frozen between 1.1 m and 1.6 m. At 3.1 m colour changes to grey-black with occasional small organic silt lenses/pockets.
	4.5 - 6.0	Grey-black SANDY SILT to SILT and fine SAND, trace clay becoming SILT from fine sand, trace to some clay with depth. Occasional thin seams of silty fine sand and black, clayey silt to silty clay seams.
	6.0 - 7.1	Grey-black SILT, some clay, trace fine sand to CLAYEY SILT with occasional silt, some fine sand to sandy silt seams.
	7.1 - 7.6	Grey-black SILTY CLAY, trace fine sand.

TABLE 2
SAMPLE DESCRIPTIONS

DRILLHOLE/ TEST PIT NO.	SAMPLE NO.	DEPTH (m)	DESCRIPTION	WATER CONTENT (%)	OTHER TEST
TC-1	1	0.9 - 1.21	Brown SAND & GRAVEL to GRAVELLY SAND, trace silt, occasional shell fragments.	11.2	MH
	2	1.9 - 2.2	Brown SAND, trace to some silt, occasional fine gravel.	13.9	
	3	2.9 - 3.2	Brown SAND, becoming grey-black fine SAND, some silt to SILTY SAND, trace clay at 3.1 m.	13.1	
	4	3.9 - 4.2	Grey-black SANDY SILT to SILT, some fine sand trace clay, occasional black clayey silt seams, occasional fine gravel.	15.2	
	5a	5.9 - 6.1	Grey SANDY SILT to fine SILTY SAND.	12.4	MH
	5b	6.1 - 6.2	Grey-black SILT, some clay to CLAYEY SILT.	11.5	
	6	7.3 - 7.6	Grey SILT, some fine sand, some clay occasional thin black clayey/silt seam.	13.6	
TC-2A	1	0.9 - 1.2	Brown SAND, trace silt, trace gravel to GRAVELLY SAND, trace silt, occasional shell fragments.	13.5	MH
	2	1.9 - 2.2	Brown SAND, trace silt occasional fine gravel.	10.6	
	3	2.9 - 3.2	Grey brown SAND, trace to some fine gravel, becoming black at 3.1 m with trace organics/organic silt lenses.	9.0	
	4	3.9 - 4.2	Grey SAND, trace to some fine gravel, trace silt.	7.9	
	5	4.9 - 5.2	Grey SANDY SILT to SILT and fine SAND, trace clay, occasional thin black clayey silt seams.	14.7	
	6	5.9 - 6.2	Dark grey SILT trace to some clay trace fine sand to CLAYEY SILT, occasional silt some sand to sandy silty seams.	14.5	
	7	7.3 - 7.6	Dark grey SILTY CLAY trace fine sand.	19.9	MH W _L = 33 W _p = 17 I _p = 16
TP1	1	0.9	Frozen SAND, trace fine gravel with shell fragments.	17.0	MH

Notes: M = grain size analysis by sieve
H = grain size analysis by hydrometer
W_L = liquid limit
W_p = plastic limit
I_p = plasticity index.

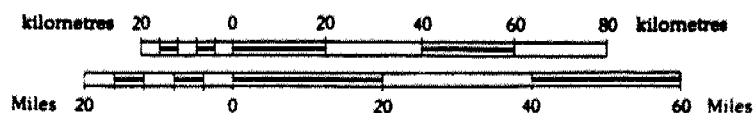
TABLE 3
GROUND TEMPERATURE MONITORING

Date	Elapsed Time (day)	THERMOCOUPLE TC-1 TEMPERATURE (°C) AT DEPTHS (m)						THERMOCOUPLE TC-2 TEMPERATURE (°C) AT DEPTHS (m)					
		Above Ground	0.1	1.0	1.9	2.8	3.8	Above Ground	0.1	1.0	1.9	2.8	3.8
June 12 6:00 p.m.	0	13.5	16.5	4.8	4.7	5.3	5.7	16.3	14.7	4.8	5.2	5.6	5.8
June 13 10:00 a.m.	1/2	13.0	11.1	5.2	5.2	5.7	6.0	13.2	9.7	4.0	4.5	5.2	5.3
June 21	9	13.3	17.5	4.2	4.6	5.7	5.6	13.6	14.9	3.9	4.6	5.4	5.0
June 28	16	19.6	19.3	4.2	4.9	5.1	5.4	21.3	20.6	3.4	4.4	4.7	5.8
July 5	23	9.4	8.2	4.4	3.1	1.3	0.7	9.7	8.5	4.1	3.4	2.2	0.9
July 12	30	10.6	6.3	4.0	-1.5	-1.2	-1.7	9.7	6.7	3.8	-1.0	-1.9	-2.0
July 16	34	12.2	9.7	3.5	-0.2	-2.1	-2.2	11.3	10.4	4.2	-0.3	-1.5	-2.0
July 19	37	11.6	7.8	4.7	0.3	-1.0	-1.5	9.8	8.1	5.1	0.3	-1.3	-2.8
July 26	44	10.8	10.1	6.8	3.5	1.1	0.4	10.1	8.9	5.2	2.3	-0.2	1.4

- Notes:
1. Temperature reading in degree Celsius (°C).
 2. Thermocouples installed on June 12, 1991.
 3. 0.61 m (2 ft.) of granular fill was placed over the building site during the week of July 2 - July 5, 1991.
 4. Thermocouple installations were destroyed in August 1991 and, therefore, further readings were not possible.

PROJECT LOCATION PLAN

FIGURE 1

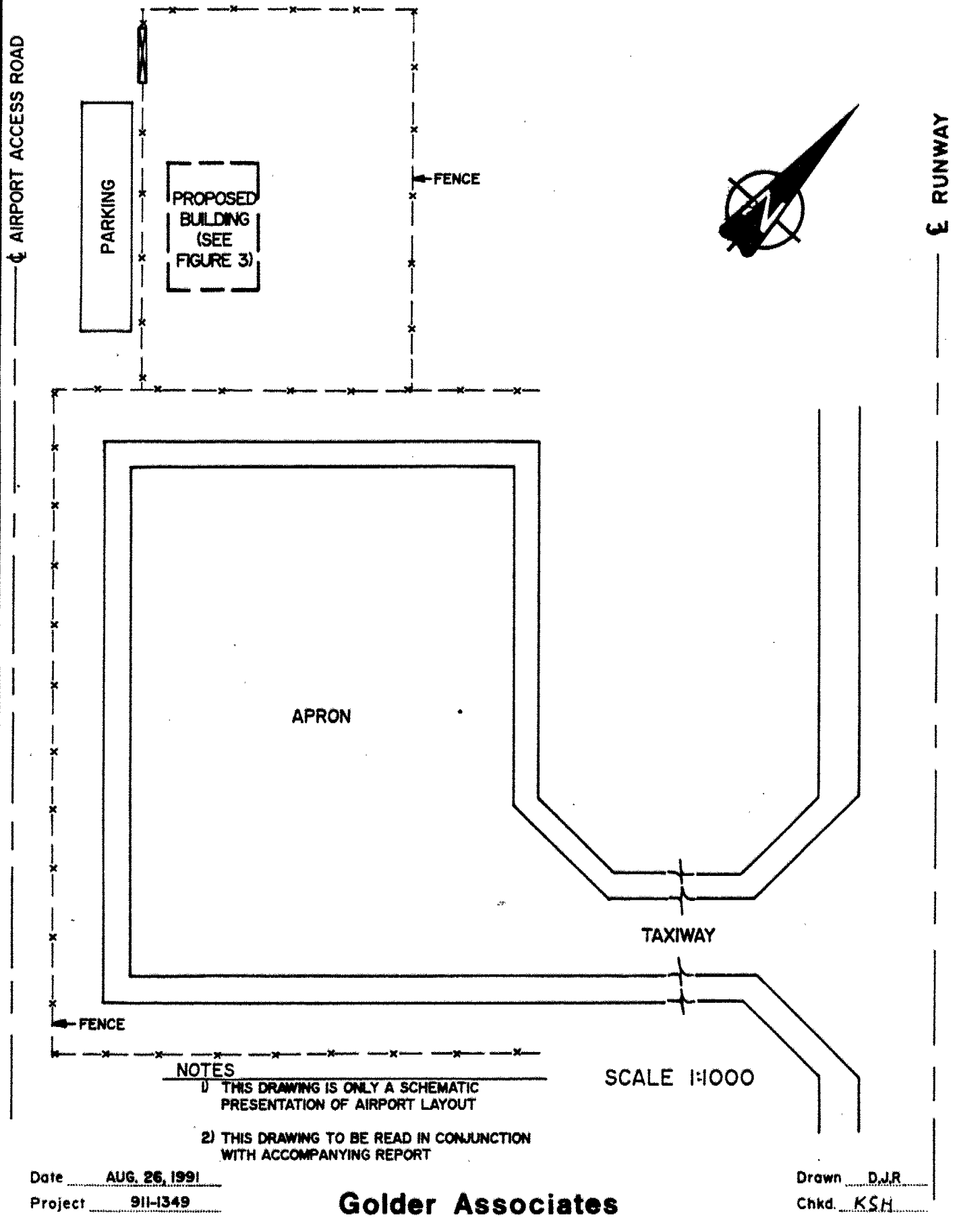


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Date JULY 3, 1991
Project 911-1349

Golder Associates

Drawn D.J.R.
Chkd. KSH



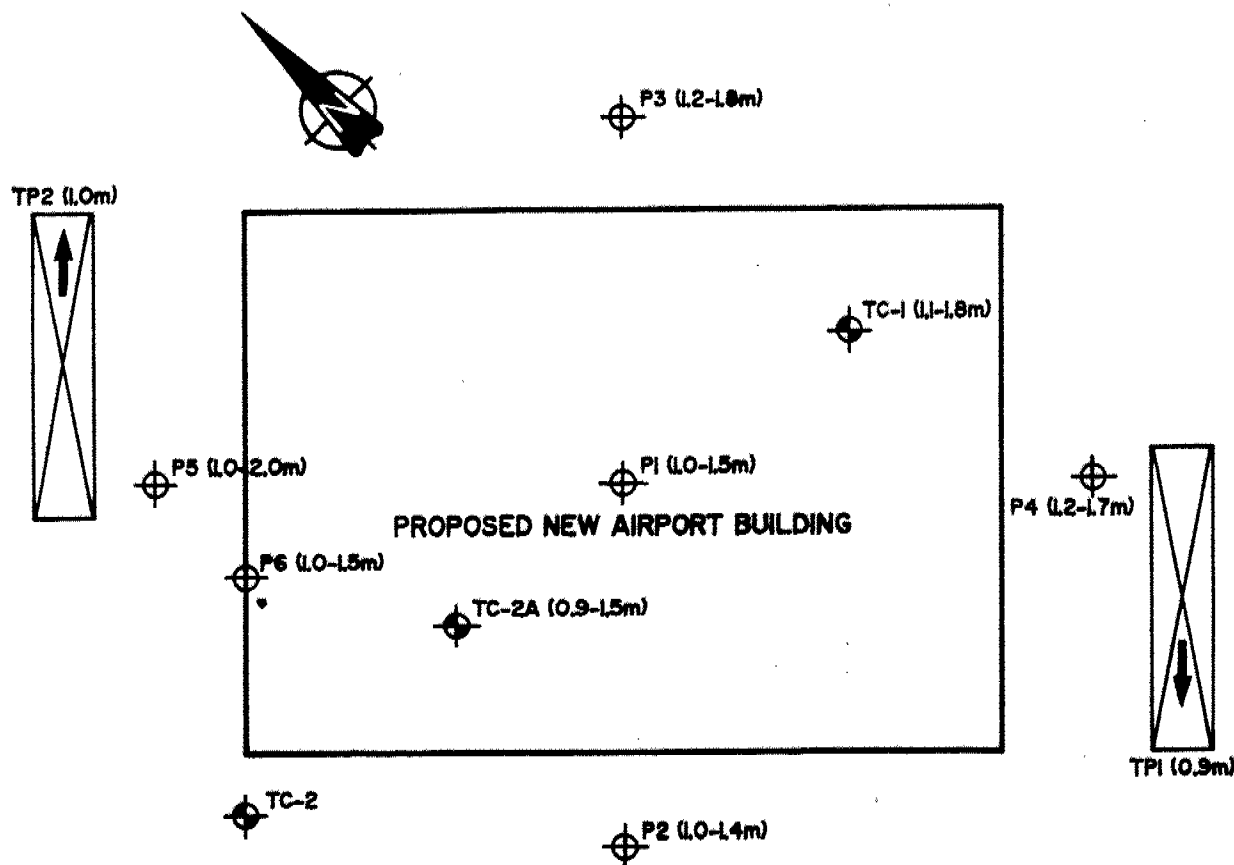
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Drawn DJR

Chkd. KSH



LEGEND

- THERMOCOUPLE AND / OR DRILL HOLE
- PROBE
- TEST PIT

NOTES

- 1) FIGURES IN BRACKETS CORRESPOND TO THE DEPTHS OF FROST INFERRED WITHIN THE HOLES
- 2) ALL THERMOCOUPLE, PROBE AND TEST PIT LOCATIONS ARE APPROXIMATE
- 3) THIS DRAWING TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

SCALE 1:250

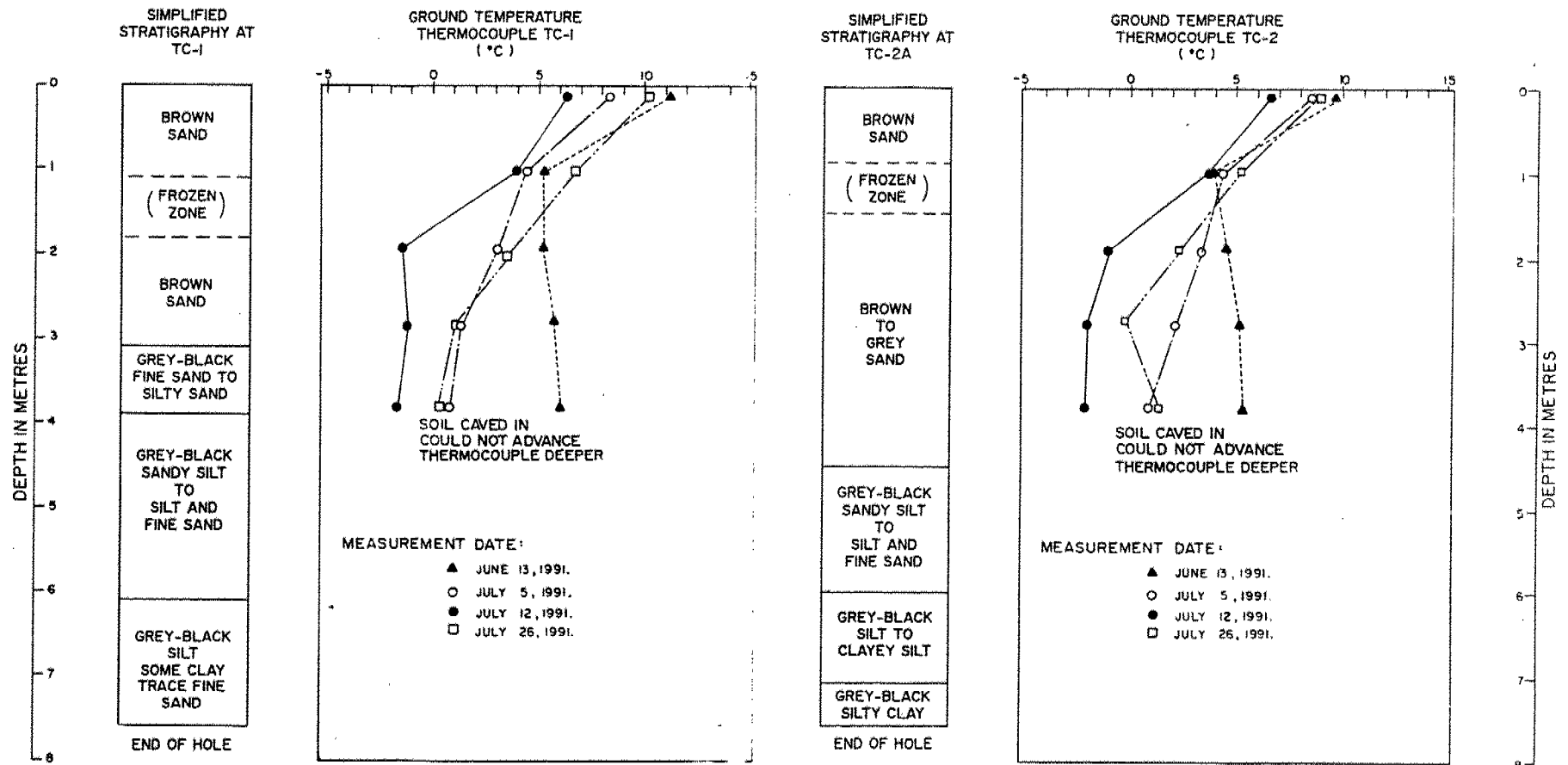
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SIMPLIFIED STRATIGRAPHY AND TEMPERATURE PROFILE

FIGURE 4



NOTES

- 1) THE SOIL STRATIGRAPHY HAS BEEN DETERMINED AT DRILL HOLE LOCATIONS FOR THERMOCOUPLE INSTALLATION
- 2) FOR LOCATIONS OF DRILL HOLE AND THERMOCOUPLE REFER TO FIGURE 3
- 3) THIS DRAWING TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

Date... JULY 2, 1991
Project... 911-1349

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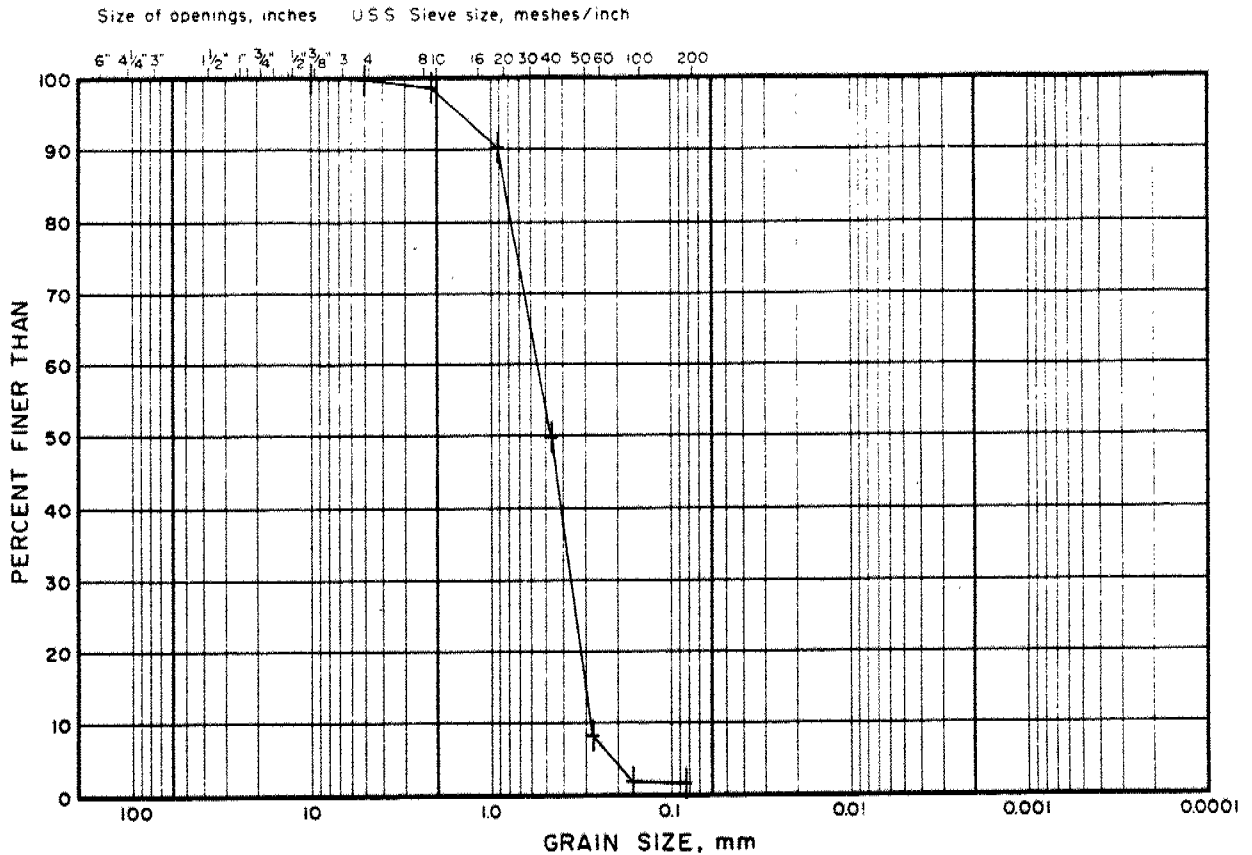
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GRAIN SIZE DISTRIBUTION

FIGURE 5

16

SAND (Frozen In-Situ)

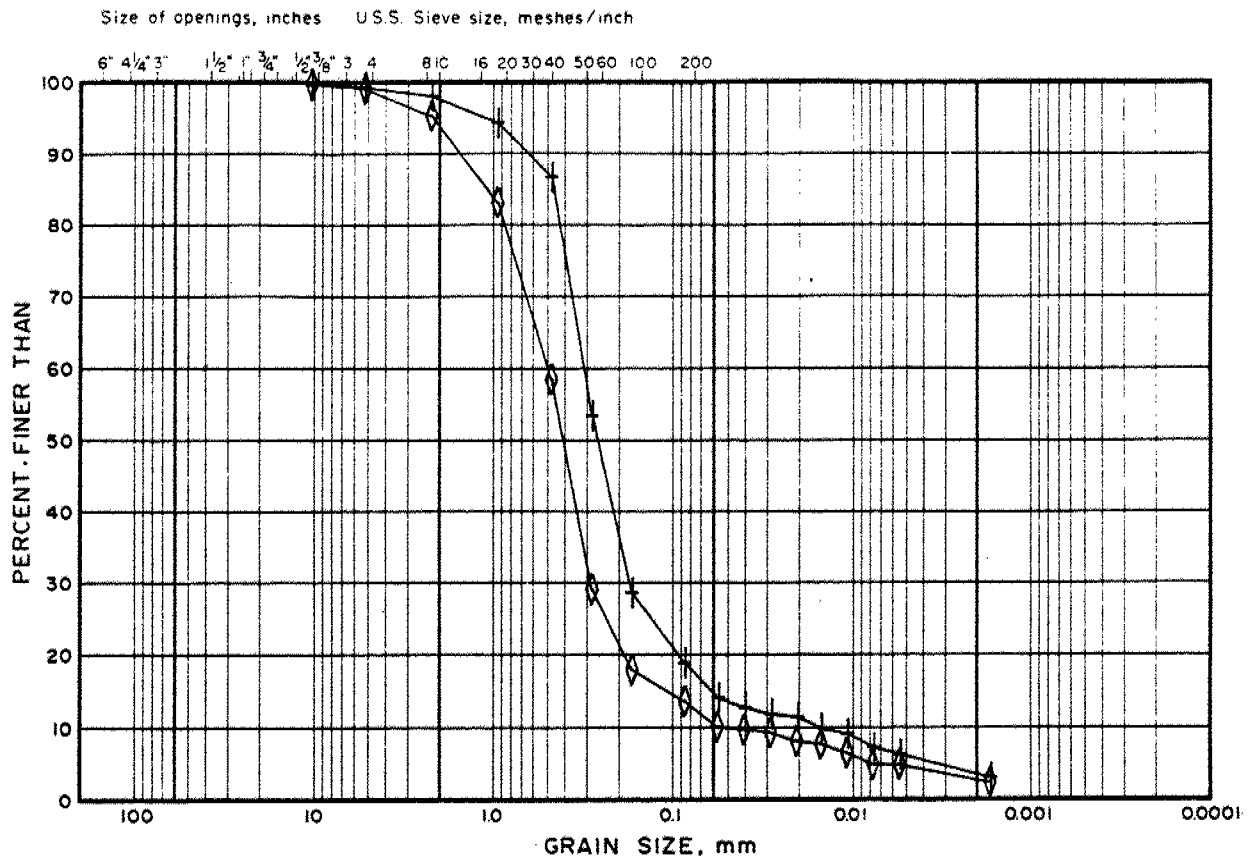


GRAIN SIZE DISTRIBUTION

FIGURE 6

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SAND, trace to some silt, trace fine gravel



COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED		

LEGEND

SYMBOL	BOREHOLE SAMPLE	DEPTH (m)
+	TC-1	2
◇	TC-2A	1

Project 911-1349

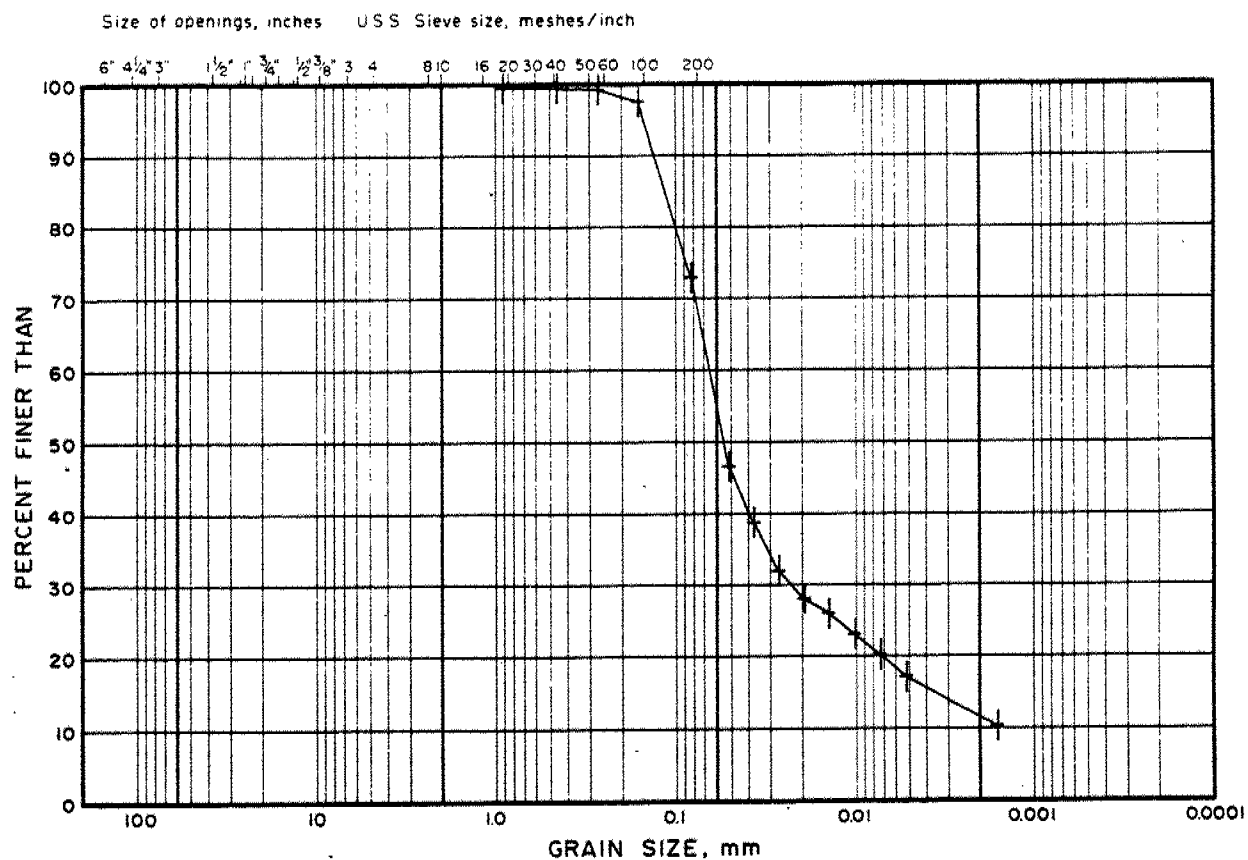
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GRAIN SIZE DISTRIBUTION

FIGURE 7

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SILT and fine SAND, trace clay



COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE SAMPLE	DEPTH (m)
+	TC-2A	5
		5.2

Project 911-1349

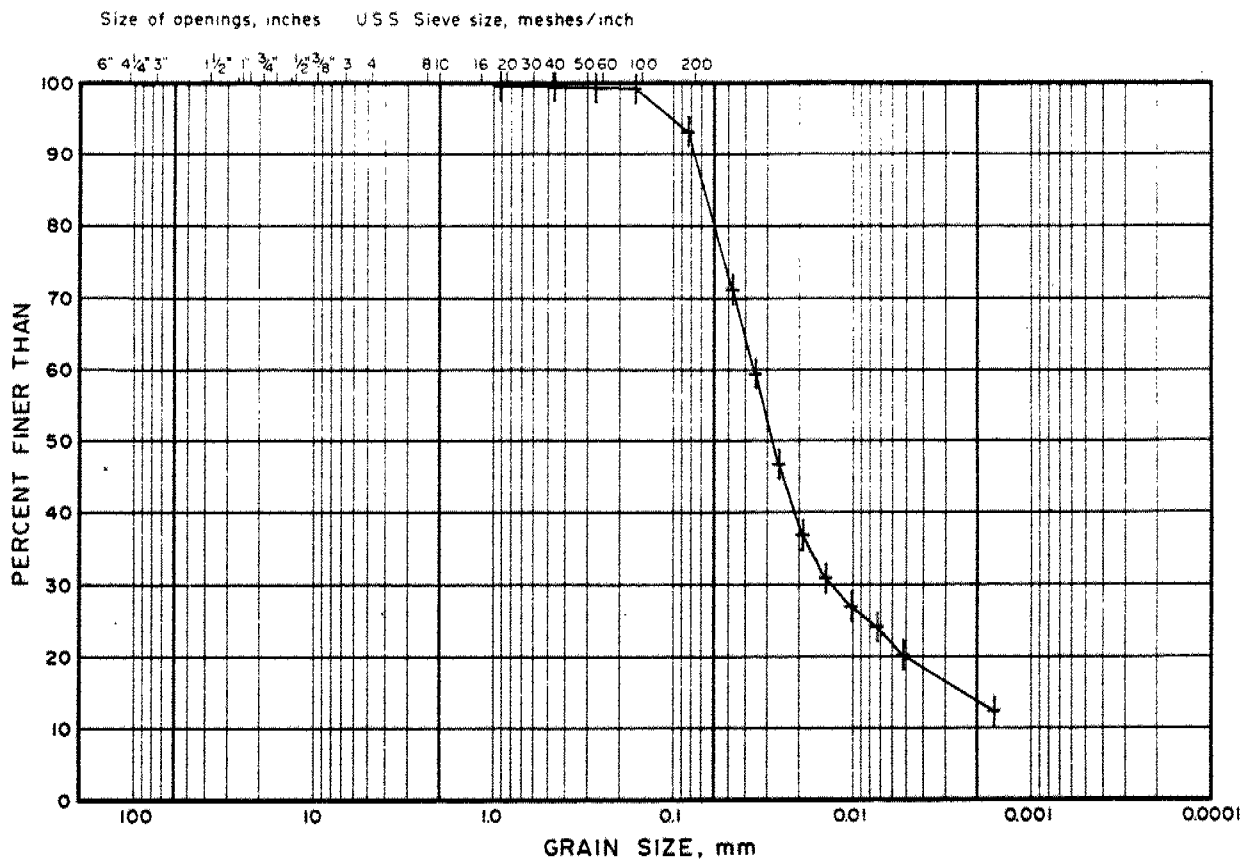
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GRAIN SIZE DISTRIBUTION

FIGURE 8

19

SILT, some clay, some fine sand

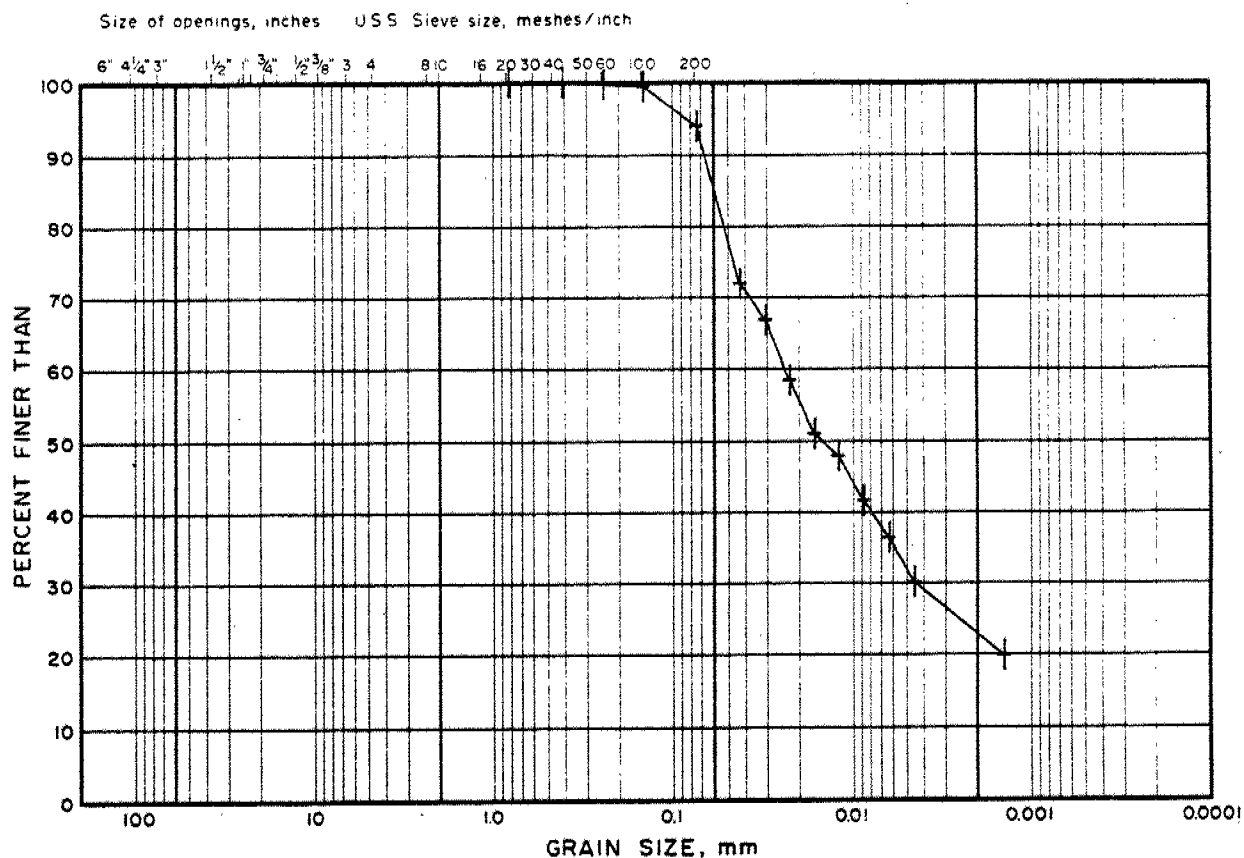


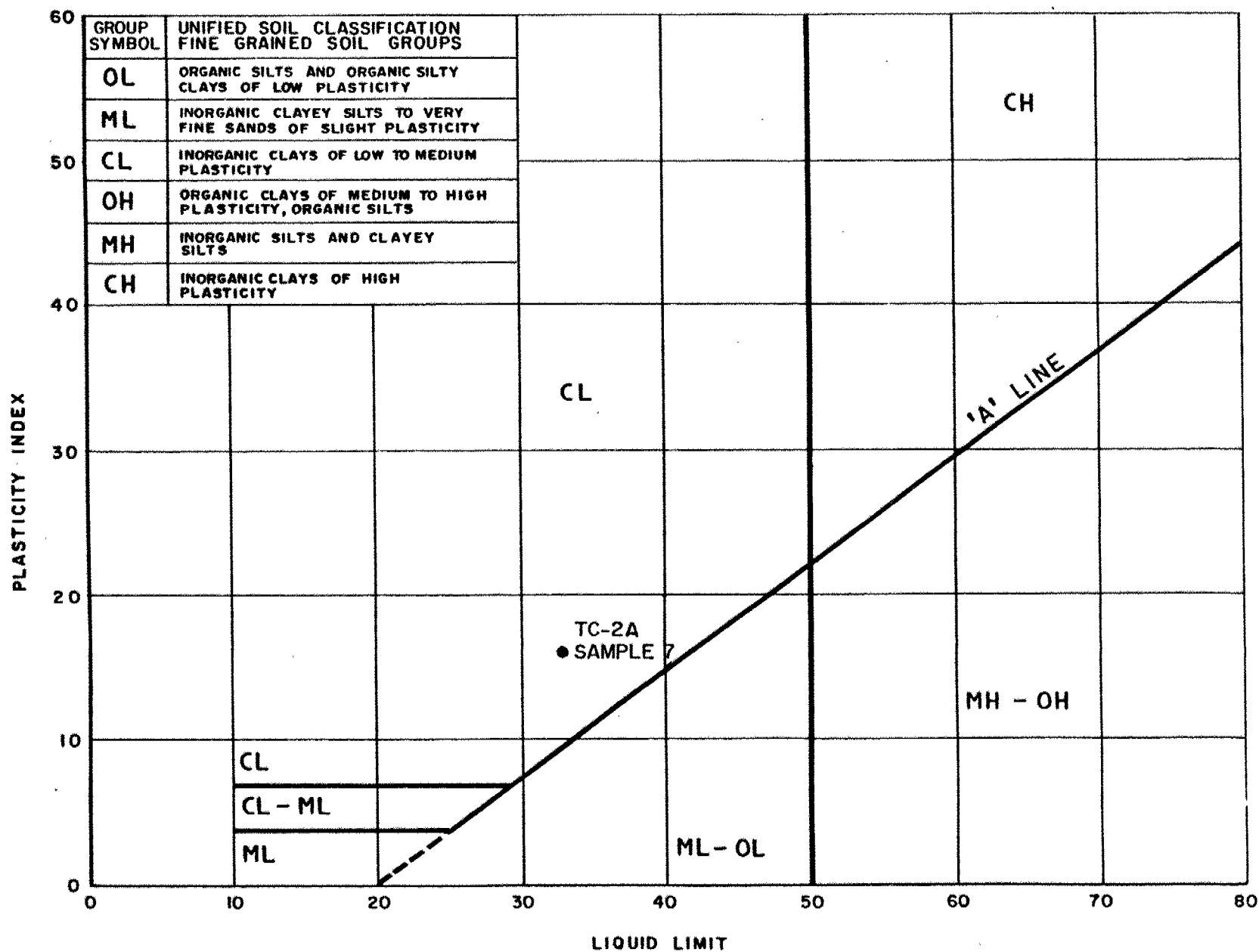
GRAIN SIZE DISTRIBUTION

FIGURE 9

20

SILTY CLAY, trace fine sand





PLASTICITY CHART



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CONSULTING ENGINEERS

REPORT ON

FOUNDATION INVESTIGATION
NEW AIRPORT BUILDING
FORT SEVERN AIRPORT
WP. 2023-91-02
FORT SEVERN, ONTARIO

CONT 91-700

Submitted to:

Ministry of Transportation of Ontario
Remote Northern Transportation
Northwestern Region
615 South James Street
P.O. Box 1177
Thunder Bay, Ontario
P7C 4X9

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October 16, 1991

911-1349

Ministry of Transportation of Ontario
Remote Northern Transportation
Northwestern Region
615 South James Street
P.O. Box 1177
Thunder Bay, Ontario
P7C 4X9

ATTENTION: Mr. L.P.J. Richard, Manager

RE: FOUNDATION INVESTIGATION
NEW AIRPORT BUILDING
FORT SEVERN AIRPORT
WP. 2023-91-02
FORT SEVERN, ONTARIO

Dear Sirs:

This report presents the final results of a geotechnical investigation and ground temperature measurements for the above project. The preliminary results of the investigation and our corresponding recommendations, concerning the geotechnical aspects of design for the proposed building, were provided to the Ministry of Transportation in our letter dated June 26, 1991.

The purpose of the investigation is to determine the subsurface conditions, thickness of active layer (viz. zone of soil that freezes and thaws each year), ground temperature and salinity of porewater. Based on our interpretation of these data, engineering recommendations for geotechnical aspects of design of the proposed new airport building are provided.

The work carried out for this investigation was completed in accordance with our proposal (Ref. No. P11-2830) to Ministry of Transportation of Ontario (MTO) dated May 24, 1991. Authorization to proceed with the work was given verbally by MTO. The terms of reference for the project are provided by Agreement No. 2321-6091-625.

PROJECT AND SITE DESCRIPTION

We understand that a new building will be constructed at the Fort Severn Airport (see Figures 1 and 2). The proposed building is a two-storey complex with offices, garages, warehouse and workshop.

Based on published information, Fort Severn is located within the Hudson Bay Lowlands and within the continuous permafrost zone, but near the southern limit boundary with the discontinuous permafrost zone. Therefore, it is possible that the area of the site could be underlain by "warm" permafrost (viz. ground temperature slightly cooler than the freezing point) and patches/pockets of unfrozen soil. Because of the proximity to Hudson Bay, the subsoils likely consist of sediments such as fine-grained sands, silts and clays which were deposited in a marine environment; ridges of glacial till may also exist between low-lying areas.

METHODOLOGY

The fieldwork was carried out between June 11 and 13, 1991. The major components of the investigation consisted of:

- Visual examination and assessment of subsurface conditions by digging two test pits (TP1 and TP2) and advancing six probe holes (P1 to P6) to about 7 m depth.
- Drilling of three small diameter (about 30 mm) uncased sampled drill holes TC-1, TC-2 and TC-2A, about 7 m deep, to investigate the subsurface conditions and to install thermocouple strings to determine and monitor in-situ profiles of ground temperature.

The locations of test pits, drill holes, probes and thermocouple string installations are shown on Figure 3. Two test pits (2 m by 10 m) were excavated using the available on-site front-end loader to a depth of about 1 m. Deeper excavation was not possible because of the encountered frost zone at and below this depth. Salinity of the ponded water in the test pits was measured by a conductivity meter.

As outlined in our above-mentioned Proposal P11-2830, it was considered that, due to the possible "warm" permafrost at the site, conventional core drilling without the use of expensive special coolants and refrigeration would likely thaw the sample during the coring process.

Therefore, special, light weight, portable drilling equipment (Pionjar) supplied and operated by Sonic Soil Sampling Inc. from Toronto, Ontario, was used to drill the holes and to advance the probes. The Pionjar drilling unit consists of a gas driven engine, 25 mm diameter steel rods, soil sampling tubes and special drill bits.

In drill holes TC-1 and TC-2A (See Figure 3), the soil was sampled by a 33 mm outside diameter tube sampler, at an interval of about 0.9 m, down to a depth of about 7 m below ground surface. The sampler is about 300 mm in length and 25 mm inside diameter. The recovered samples were examined in the field and stored in airtight containers for further examination and testing in our geotechnical laboratory in Mississauga, Ontario. Water content, grain size distribution and Atterberg Limits (plasticity) were carried out on selected samples.

Thermocouple strings were installed in drill holes TC-1 and TC-2 at the locations shown on Figure 3. The thermocouple strings consisted of temperature sensing elements, electric wire and CPVC tubing. The electric wire was inserted into the CPVC tubing and connected to the temperature sensing elements which were mounted and protected on the exterior surface of the CPVC tubing at about 0.9 m intervals along the tubing. The pre-fabricated thermocouple strings were advanced/pushed into the pre-drilled holes to refusal, which occurred at a depth of about 4 m below ground surface. A thermocouple string could not be installed in drill hole TC-2A due to sloughing of granular material into the hole.

The fieldwork was carried out and supervised throughout by our D. E. Becker, P. Eng., who organized the field working team, selected the locations for investigation, performed visual examination of samples, directed the sampling and instrumentation installation operations, recorded the observations and measurements, and placed the samples in labelled, airtight containers.

RESULTS OF INVESTIGATION

While the Fort Severn area is generally quite flat, the site of the proposed building lies on a local topographical high consisting of a reasonably well-drained sandy ridge or possibly an esker. The proposed site area had been recently stripped of its 0.3 m to 0.4 m thick muskeg cover prior to the start of the fieldwork.

The results of the investigation are summarized in Tables 1 and 2 and on Figures 3 to 10, inclusive.

Subsurface Conditions

Two drill holes (TC-1 and TC-2A) and two test pits (TP1 and TP 2) were put down to determine the subsurface conditions at these locations. The subsoil conditions at each of the drill hole locations are summarized in Table 1 and plotted on Figure 4. The descriptions of the soil samples together with the laboratory test results are summarized in Table 2. Grain size distribution curves and Atterberg Limits test results are presented on Figures 5 to 9 and on Figure 10, respectively.

The encountered subsoils in the two drill holes consist of a layer of brown sand, about 3 m to 4.5 m thick, overlying a 1.5 m to 2 m thick layer of grey-black fine sand, sandy silt to silt material. The drill holes were terminated in a grey-black silt some clay, clayey silt to silty clay deposit. The measured water content on samples varies from about 8 per cent in the sand layer to about 20 per cent in the silty clay deposit (see Table 2).

Within the excavated test pits, groundwater seepage was observed at about 0.6 m below ground surface. The groundwater ponded within the test pit above the frost line. The salinity of the ponded water in the two test pits was measured using a portable conductivity meter. The measured salinity at the time of investigation was quite low, being about 0.03 per cent or 0.3 g/l.

Frozen Soil

The extent of the frozen soil at the test locations was estimated from the drill probes, soil sampling and test pit excavation operations. As shown on Figures 3 and 4, the inferred frozen soil thickness at the time of investigation was about 0.4 m to 1.0 m, with the depth to the top of the frost line at about 0.9 m to 1.2 m below ground surface. These depths to the frost layer are considered to be a result of the recent (i.e. May/June 1991) stripping of the muskeg layer across the site, as part of site preparation operations, to expose the natural sand deposit. In areas where the muskeg layer had been left intact (i.e. natural muskeg cover surrounding the prepared/stripped site area) frost was encountered at 0.2 m below ground surface. The removal of the muskeg

(which acts as a natural insulator on the ground surface) has apparently accelerated the thawing of the sand deposit.

Ground Temperature Measurement

The ground temperature readings were taken every week after installation by local MTO field staff. The measured temperature profiles (TC-1 and TC-2) are summarized in Table 3 and some selected profiles are presented on Figure 4.

After an elapsed time of one-half day, the measured temperatures by the thermocouple string varied from about 4°C, at depths corresponding to the frozen soil layer, to about 11°C at ground surface. The ground temperatures below the inferred frozen soil layer varied in a narrow range between 4°C and 6°C. The magnitude of, and small variation in, measured temperature below the frozen soil layer may be due to the influence of in-flow of the "warm" groundwater, above the frost line, into the open borehole. The measured air temperature was about 13°C.

During the period between June 13 and June 28, 1991, there was only a slight variation in the measured ground temperature profile. The ground temperature measurements on July 5, 1991 indicated that the ground temperatures dropped by about 2°C to 5°C, but remained above freezing. Further ground temperature monitoring results during the period of July 12 to July 19, 1991, indicated that the ground temperatures between 1.9 m and 3.8 m depth were slightly below the freezing point (0°C - see Figure 4 and Table 3). The measured air temperature was about 10°C to 12°C. On July 26, 1991, the measured ground temperatures between 1.9 m and 3.8 m were slightly above the freezing point (0°C).

Temperature readings during August 1991 were found to be quite erratic and dubious; they are, therefore, not reported in Table 3. To ascertain whether these readings were caused by a malfunctioning readout unit, we requested MTO to return the readout unit to our Mississauga laboratory for examination and calibration, if required. However, the thermocouple installations were destroyed when the actual thermocouple strings were extricated from the ground by local MTO personnel and returned to us. Therefore, further ground temperature monitoring is not possible.

While the temperature monitoring information is not conclusive, it suggests that marginal frost, within the area investigated, possibly extends to at least 4 m depth. However, the observed relative ease at which the Pionjar drilling unit penetrated the ground below the 2 m depth, and examination of the recovered fine-grained silty and clayey soil at depth, suggests that during the investigation the ground was not frozen below about 2 m depth.

Based on the above, it is considered that the 2 m depth is representative of the inferred bottom of the frost layer during the investigation; it should also closely correspond to the maximum depth of frost penetration that developed over the 1990/1991 winter season. The local native residents considered the past winter to be "very cold" and, therefore, the observed, unexpected relatively shallow depth of frost penetration cannot be attributed to an unseasonably mild winter season.

Existing correlations of frost penetration and the average annual Freezing Index (i.e. cumulative total of difference between annual daily mean air temperature and the freezing point (0°C) expressed in units of degree days), based on observations in open clear areas such as roads and aircraft runways, suggest a 3 m depth of frost penetration for the Fort Severn area (with a Freezing Index of about 3250°C degree days). However, these general correlations do not take into account soil type or insulating effect of muskeg cover. For example, the depth of frost penetration is greater in coarse-grained gravelly soils than in fine-grained clayey soils. Our calculations for a coarse-grained granular soil with a dry density of 16.5 kN/m^3 and water content of 15 per cent, under a Freezing Index of 3250°C degree days, indicate a frost penetration depth of 3.6 m. For a fine-grained soil, a calculated value of 2.9 m is obtained.

As mentioned above, it is considered that the observed relatively shallow depth of frost penetration at the site is a result of the insulating effect of muskeg, which until a month or so ago covered the area investigated. If the effect of a 0.3 m thick layer of muskeg is included in the analysis, a frost depth of about 2 m is calculated. This value agrees quite well with the inferred frost depth across the proposed building area.

DISCUSSION AND RECOMMENDATIONS

This section of the report provides our interpretation of the factual data obtained during the investigation and is intended for the guidance of design engineers only. Contractors bidding on or undertaking these works should review the factual information; satisfy themselves as to the adequacy of the information; and, make their own interpretation of the factual data as it affects their construction techniques, scheduling, equipment capabilities and the like.

The results of the subsurface investigation indicate that a seasonal frost zone was encountered at about 1 m depth across the site; permafrost does not exist within the depth of investigation (viz. down to about 8 m). Further, based on the observations made during the drilling operations and the results of the thermocouple readings, it is considered unlikely that permafrost exists at depth.

Based on information provided to us by MTO, we understand that the proposed building is to consist of a concrete slab-on-grade structure which is generally placed on the surface of a compacted granular fill. The column footings are also generally placed near the surface of, or within, the compacted granular fill. The garage portion of the building will be heated. The section of the building for the radio and foreman's offices involves an crawl space between the underside of the structurally supported floor and the prepared granular fill. This section of the proposed building is usually supported by short timber posts placed on concrete pads partially embedded within the granular fill.

During future construction of the proposed building in 1992, a frost layer will exist within the natural subsoils. Because the insulating muskeg cover has been removed, the depth of frost penetration will likely be greater than that observed during the June 1991 investigation. Based on calculations, it could be of the order of 3 m deep. The actual depth of frost penetration will depend on temperature, snow cover and wind conditions at the site during the 1991/1992 winter season.

The upper part of the natural ground will probably be thawed prior to construction, depending on the construction start-up date. Following completion of the building, any remaining frozen ground beneath the heated slab-on-grade will thaw, and the continued heat source will prevent further frost development in this area. However, if the crawl space is unheated, annual freeze/thaw cycles within the ground will occur. Because of these annual cycles, a potential for

differential settlement and associated detrimental effects (deformation and cracking) could be experienced between unheated and heated portions of the building.

Consideration should, therefore, be given to providing a minimally heated crawl space beneath the building. The effects of differential settlement due to cycles of freezing and thawing beneath portions of the proposed building will thus be minimized if the crawl space is heated to a temperature modestly above freezing.

Based on our experience and established classification systems, the upper 3 m or so of the natural sandy subsoils across the proposed building area are relatively clean and well-draining. Therefore, they are considered to be thaw stable (i.e. they are reasonably stable during thawing and when thawed) and possess low frost susceptibility potential. The more silty soils below the 3 m to 4 m depth are frost susceptible and potentially unstable during thawing, especially if significant ice lenses develop during freezing.

Therefore, the design should make adequate provision to minimize the probability of the natural soils below 3 m being exposed to cycles of freezing and thawing. To minimize the effects of frost action, and to reduce potential frost penetration in the existing natural soils, we recommend that a well-compacted, clean and free-draining granular pad, 0.5 m (minimum) in thickness, be placed on the existing grade at the proposed building area. The top portion of the granular pad should be of a minimum dimension corresponding to the proposed building envelope plus 3 m on all sides. The side slopes of the granular pad should not exceed 2 horizontal to 1 vertical and to enhance erosion resistance, they should be covered with a 0.3 m minimum thick layer of coarse particles (such as coarse sand, gravel and cobbles).

The purpose of the granular pad is two-fold; it minimizes the depth of frost penetration in the natural ground and enhances positive drainage of surface water away from the proposed building.

The granular pad should be constructed using clean, free-draining and non-frost susceptible material such as OPSS Granular B material. The granular material should be placed in maximum loose lifts of 200 mm with each lift being compacted using a heavy vibratory roller to at least 95 per cent of the Standard Proctor maximum dry density. For efficient compaction operations the placement water content should be within two per cent of the optimum water content for

compaction purposes. The final 150 mm base layer beneath the proposed concrete slab should consist of OPSS Granular A material compacted to at least 100 per cent of Standard Proctor maximum dry density.

Prior to placement of the granular pad, the existing exposed natural subgrade should be clean of any organic material and proof-rolled and any softened areas sub-excavated and replaced with a well compacted clean, free-draining granular material as described above for the granular pad.

Provided the above recommendations are followed, the footings can be designed using an allowable bearing pressure of 100 kPa. For this bearing pressure, normally anticipated total and differential settlement should be of the order of 25 mm and 20 mm, respectively. However, the proposed building is quite flexible in nature and it should tolerate greater settlements without any serious structural distress. For the design of the concrete floor slab, a modulus of subgrade reaction of 60 MN/m³ (200 tons/ft³) for a 0.3 m by 0.3 m bearing area may be assumed.

To provide frost protection to perimeter footings, a combination of soil cover and insulation can be used. To minimize the amount of insulation required, it is suggested that perimeter wall footings be founded as deep as practical within the natural sand subsoils. Groundwater control may be required to keep the excavation dry, but this can be achieved by pumping from properly filtered sumps at the base of the excavation. Care will be necessary during foundation excavation and dewatering to prevent disturbance of the subsoils below subgrade level.

If possible, it is recommended that these footings be founded at a minimum depth of 1 m below the existing ground surface at the time of the June 1991 investigation. Based on observations made during this recent investigation, it is considered that thawing of the ground surface could extend to this depth by about June. Suitable insulation such as expanded polystyrene or equivalent should be, for convenience, placed on the prepared existing ground surface and carefully covered with the granular pad material. The insulation should be placed on a smooth sand bedding at a small inclination of not less than one per cent to promote drainage of surface infiltration water. The insulation should tie into the insulation running down the wall/footing and extend horizontally for a minimum distance of 2.5 m.

The required thickness of insulation depends on the embedment depth of the footing in the natural sands. If an embedment depth of 1 m below existing ground surface can be conveniently

achieved, a 75 mm minimum thickness of insulation should be adequate. If, for construction expediency, equipment and personnel availability and economic considerations, it is considered impractical to place the perimeter wall footings within the natural sands, increased thickness of insulation will be required. For an embedment depth of 0.5 m below existing ground surface, 110 mm (minimum) of insulation will be required. Alternatively, the thickness of the granular pad can be increased by 0.5 m to 1 m. However, this may cause grading problems and encroach building height (i.e. elevation) constraints relative to the nearby airstrip.

While the proposed granular pad material and the natural subsoils to 3 m depth below existing grade are reasonably thaw stable, and possess low frost susceptibility potential, differential settlements can be expected along the building foundation unit, in particular, between the offices and garage area and along the perimeter of the building. To mitigate the effects of differential settlement, special provisions such as capability for jacking and shimming of the timber posts beneath the offices and large garage door openings should be made to facilitate levelling of the building, as required, with time. Extra long anchor bolts to fasten the posts to concrete pads should be used to permit adjustments to be made. The concrete floor slab should have no structural interaction with the walls and columns (i.e. structurally independent or tied together). The wall between the garage and offices should be structurally independent or designed to accommodate differential settlement at this location.

There should be adequate frost protection provided for any sub-foundation level perimeter drains installed around the building. The perforated drain should be wrapped in an appropriately chosen filter cloth. Frost protection could consist of a combination of soil cover and insulation as previously discussed for the perimeter wall footings. Insulation should be incorporated into perimeter building walls (including the crawl space perimeter walls) and extend down to footing level.

Case records exist which demonstrate that light buildings in open exposed areas can be susceptible to strong gusts of wind and may be displaced laterally. For a sliding stability design check, a coefficient of friction between the concrete pads/footings of 0.45 may be assumed. If adequate resistance to sliding cannot be developed through "self-weight" of the building and footings, adequate anchorage will need to be provided.

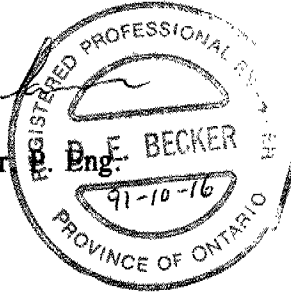
We trust that this report provides sufficient information for your immediate requirements. Should you have any questions, please do not hesitate to call us.

Yours truly,

GOLDER ASSOCIATES LTD.



Dennis E. Becker



John L. Seychuk, P. Eng.

KSH/DEB/JLS/pds

Att: Tables 1 to 3
List of Abbreviations and Symbols
Figures 1 to 10

TABLE 1
SUMMARY OF SUBSOIL CONDITIONS

DRILL HOLE NO.	DEPTH (m)	DESCRIPTION
TC-1	0.0 - 3.1	Brown SAND, trace to some silt, occasional fine gravel, occasional shell fragments, occasional thin layers of SAND and GRAVEL to GRAVELLY SAND. Probably frozen between 1.2 m and 1.8 m.
	3.1 - 3.9	Grey-black, fine SAND, some silt to SILTY SAND trace clay.
	3.9 - 6.1	Grey-black SANDY SILT to SILT and fine SAND trace clay becoming SILT trace fine sand, trace to some clay with depth. Occasional seams of silty fine sand.
	6.1 - 7.6	Grey-black SILT, some clay, trace fine sand, occasional thin black clayey silt to silty clay seams.
TC-2A	0.0 - 4.5	Brown SAND, trace to some silt, occasional fine gravel, occasional shell fragments, occasional thin layers of SAND and GRAVEL to GRAVELLY SAND. Probably frozen between 1.1 m and 1.6 m. At 3.1 m colour changes to grey-black with occasional small organic silt lenses/pockets.
	4.5 - 6.0	Grey-black SANDY SILT to SILT and fine SAND, trace clay becoming SILT from fine sand, trace to some clay with depth. Occasional thin seams of silty fine sand and black, clayey silt to silty clay seams.
	6.0 - 7.1	Grey-black SILT, some clay, trace fine sand to CLAYEY SILT with occasional silt, some fine sand to sandy silt seams.
	7.1 - 7.6	Grey-black SILTY CLAY, trace fine sand.

TABLE 2
SAMPLE DESCRIPTIONS

DRILLHOLE/ TEST PIT NO.	SAMPLE NO.	DEPTH (m)	DESCRIPTION	WATER CONTENT (%)	OTHER TEST
TC-1	1	0.9 - 1.21	Brown SAND & GRAVEL to GRAVELLY SAND, trace silt, occasional shell fragments.	11.2	MH
	2	1.9 - 2.2	Brown SAND, trace to some silt, occasional fine gravel.	13.9	
	3	2.9 - 3.2	Brown SAND, becoming grey-black fine SAND, some silt to SILTY SAND, trace clay at 3.1 m.	13.1	
	4	3.9 - 4.2	Grey-black SANDY SILT to SILT, some fine sand trace clay, occasional black clayey silt seams, occasional fine gravel.	15.2	
	5a	5.9 - 6.1	Grey SANDY SILT to fine SILTY SAND.	12.4	MH
	5b	6.1 - 6.2	Grey-black SILT, some clay to CLAYEY SILT.	11.5	
	6	7.3 - 7.6	Grey SILT, some fine sand, some clay occasional thin black clayey/silt seam.	13.6	
TC-2A	1	0.9 - 1.2	Brown SAND, trace silt, trace gravel to GRAVELLY SAND, trace silt, occasional shell fragments.	13.5	MH
	2	1.9 - 2.2	Brown SAND, trace silt occasional fine gravel.	10.6	
	3	2.9 - 3.2	Grey brown SAND, trace to some fine gravel, becoming black at 3.1 m with trace organics/organic silt lenses.	9.0	
	4	3.9 - 4.2	Grey SAND, trace to some fine gravel, trace silt.	7.9	
	5	4.9 - 5.2	Grey SANDY SILT to SILT and fine SAND, trace clay, occasional thin black clayey silt seams.	14.7	
	6	5.9 - 6.2	Dark grey SILT trace to some clay trace fine sand to CLAYEY SILT, occasional silt some sand to sandy silty seams.	14.5	
	7	7.3 - 7.6	Dark grey SILTY CLAY trace fine sand.	19.9	MH W _L = 33 W _p = 17 I _p = 16
TP1	1	0.9	Frozen SAND, trace fine gravel with shell fragments.	17.0	MH

Notes: M = grain size analysis by sieve
H = grain size analysis by hydrometer
W_L = liquid limit
W_p = plastic limit
I_p = plasticity index.

TABLE 3
GROUND TEMPERATURE MONITORING

Date	Elapsed Time (day)	THERMOCOUPLE TC-1 TEMPERATURE (°C) AT DEPTHS (m)						THERMOCOUPLE TC-2 TEMPERATURE (°C) AT DEPTHS (m)					
		Above Ground	0.1	1.0	1.9	2.8	3.8	Above Ground	0.1	1.0	1.9	2.8	3.8
June 12 6:00 p.m.	0	13.5	16.5	4.8	4.7	5.3	5.7	16.3	14.7	4.8	5.2	5.6	5.8
June 13 10:00 a.m.	1/2	13.0	11.1	5.2	5.2	5.7	6.0	13.2	9.7	4.0	4.5	5.2	5.3
June 21	9	13.3	17.5	4.2	4.6	5.7	5.6	13.6	14.9	3.9	4.6	5.4	5.0
June 28	16	19.6	19.3	4.2	4.9	5.1	5.4	21.3	20.6	3.4	4.4	4.7	5.8
July 5	23	9.4	8.2	4.4	3.1	1.3	0.7	9.7	8.5	4.1	3.4	2.2	0.9
July 12	30	10.6	6.3	4.0	-1.5	-1.2	-1.7	9.7	6.7	3.8	-1.0	-1.9	-2.0
July 16	34	12.2	9.7	3.5	-0.2	-2.1	-2.2	11.3	10.4	4.2	-0.3	-1.5	-2.0
July 19	37	11.6	7.8	4.7	0.3	-1.0	-1.5	9.8	8.1	5.1	0.3	-1.3	-2.8
July 26	44	10.8	10.1	6.8	3.5	1.1	0.4	10.1	8.9	5.2	2.3	-0.2	1.4

- Notes:
1. Temperature reading in degree Celsius (°C).
 2. Thermocouples installed on June 12, 1991.
 3. 0.61 m (2 ft.) of granular fill was placed over the building site during the week of July 2 - July 5, 1991.
 4. Thermocouple installations were destroyed in August 1991 and, therefore, further readings were not possible.

LIST OF ABBREVIATIONS

The abbreviation commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

AS auger sample
CS chunk sample
DO drive open
DS Denison type sample
FS foil sample
RC rock core
ST slotted tube
TO thin-walled, open
TP thin-walled, piston
WS wash sample

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 0.3 m (12 in.).

Standard Penetration Resistance, *N*:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 0.3 m (12 in.).

WH sampler advanced by static weight—weight, hammer

PH sampler advanced by pressure—pressure, hydraulic

PM sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) Cohesionless Soils

<i>Relative Density</i>	<i>'N'</i> Blows/0.30m or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

<i>Consistency</i>	<i>kPa</i>	<i>'Cu'</i> psf.
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1000
Stiff	50 to 100	1000 to 2000
Very stiff	100 to 200	2000 to 4000
Hard	over 200	over 4000

IV. SOIL TESTS

C consolidation test
H hydrometer analysis
M sieve analysis
MH combined analysis, sieve and hydrometer¹
Q undrained triaxial²
R consolidated undrained triaxial²
S drained triaxial
U unconfined compression
V field vane test

NOTES:

¹Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

²Undrained triaxial tests in which pore pressures are measured are shown as \bar{Q} or \bar{R} .

LIST OF SYMBOLS

I. GENERAL

π	= 3.1416
e	= base of natural logarithms 2.7183
$\log_e a$ or $\ln a$	natural logarithm of a
$\log_{10} a$ or $\log a$	logarithm of a to base 10
t	time
g	acceleration due to gravity
V	volume
W	weight
M	moment
F	factor of safety

II. STRESS AND STRAIN

u	pore pressure
σ	normal stress
σ'	normal effective stress ($\bar{\sigma}$ is also used)
τ	shear stress
ϵ	linear strain
ϵ_{xy}	shear strain
ν	Poisson's ratio (μ is also used)
E	modulus of linear deformation (Young's modulus)
G	modulus of shear deformation
K	modulus of compressibility
η	coefficient of viscosity

III. SOIL PROPERTIES

(a) Unit weight

γ	unit weight of soil (bulk density)
γ_s	unit weight of solid particles
γ_w	unit weight of water
γ_d	unit dry weight of soil (dry density)
γ'	unit weight of submerged soil
G_s	specific gravity of solid particles $G_s = \gamma_s / \gamma_w$
e	void ratio
n	porosity
w	water content
S_r	degree of saturation

(b) Consistency

w_L	liquid limit
w_P	plastic limit
I_P	plasticity index
w_s	shrinkage limit
I_L	liquidity index = $(w - w_P) / I_P$
I_C	consistency index = $(w_L - w) / I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
D_r	relative density = $(e_{max} - e) / (e_{max} - e_{min})$

(c) Permeability

h	hydraulic head or potential
q	rate of discharge
v	velocity of flow
i	hydraulic gradient
k	coefficient of permeability
j	seepage force per unit volume

(d) Consolidation (one-dimensional)

m_v	coefficient of volume change = $-\Delta e / (1+e) \Delta \sigma'$
C_c	compression index = $-\Delta e / \Delta \log_{10} \sigma'$
c_v	coefficient of consolidation
T_v	time factor = $c_v t / d^2$ (d , drainage path)
U	degree of consolidation

(e) Shear strength

τ_f	shear strength
c'	effective cohesion
ϕ'	effective angle of shearing resistance, or friction
c_u	apparent cohesion*
ϕ_u	apparent angle of shearing resistance, or friction
μ	coefficient of friction
S_f	sensitivity

$\left. \begin{array}{l} \text{in terms of effective stress} \\ \tau_f = c' + \sigma' \tan \phi' \end{array} \right\}$

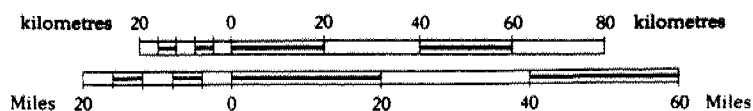
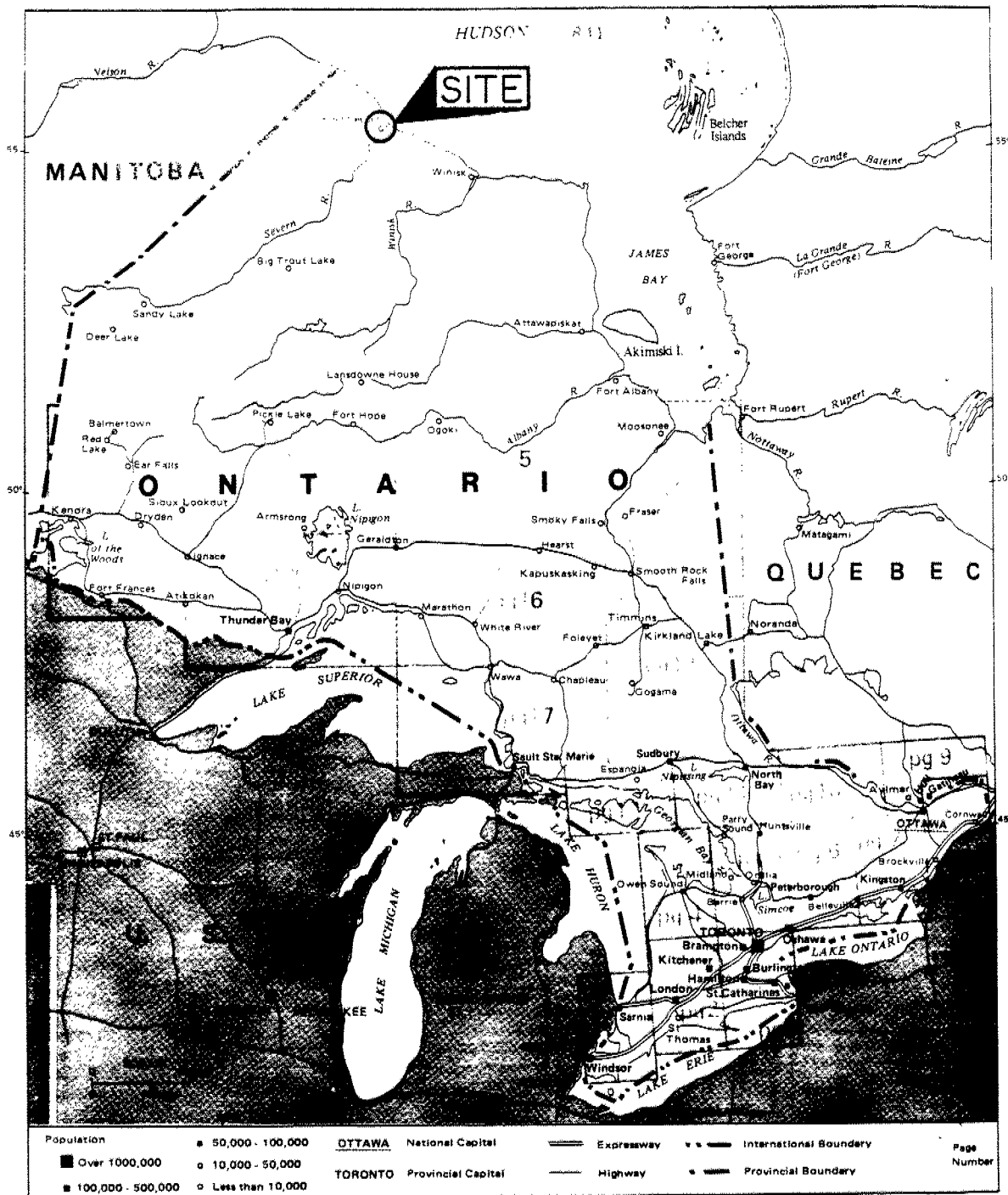
$\left. \begin{array}{l} \text{in terms of total stress} \\ \tau_f = c_u + \sigma \tan \phi_u \end{array} \right\}$

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the as half the undrained compressive strength.

strength $\tau_f = c_u$ is taken

PROJECT LOCATION PLAN

FIGURE 1

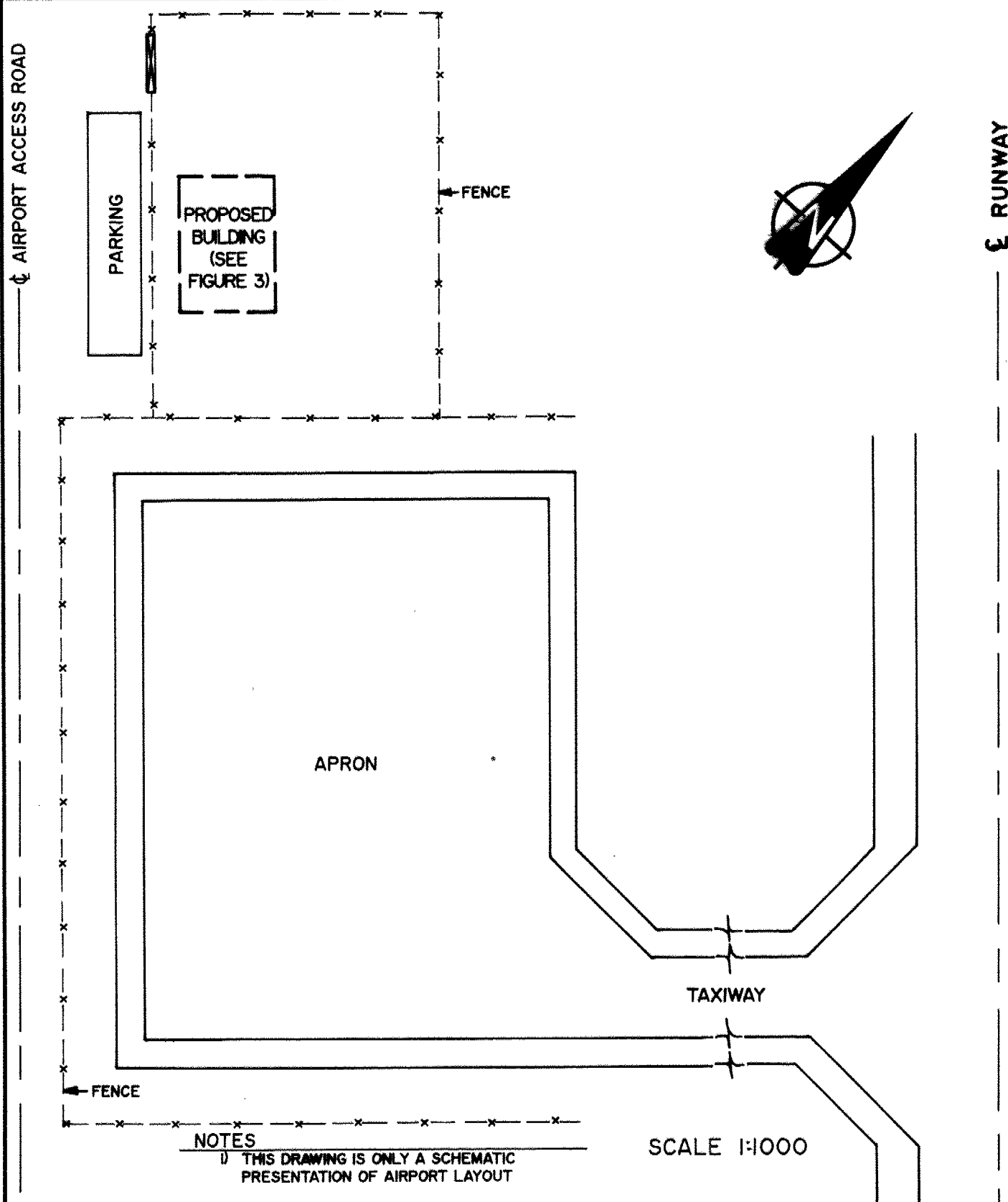


Scale 1:1,600,000

Date JULY, 3, 1991
 Project 91-1349

Golder Associates

Drawn D.J.R.
 Chkd. KSH



NOTES

- 1) THIS DRAWING IS ONLY A SCHEMATIC PRESENTATION OF AIRPORT LAYOUT
- 2) THIS DRAWING TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

SCALE 1:1000

Date AUG. 26, 1991

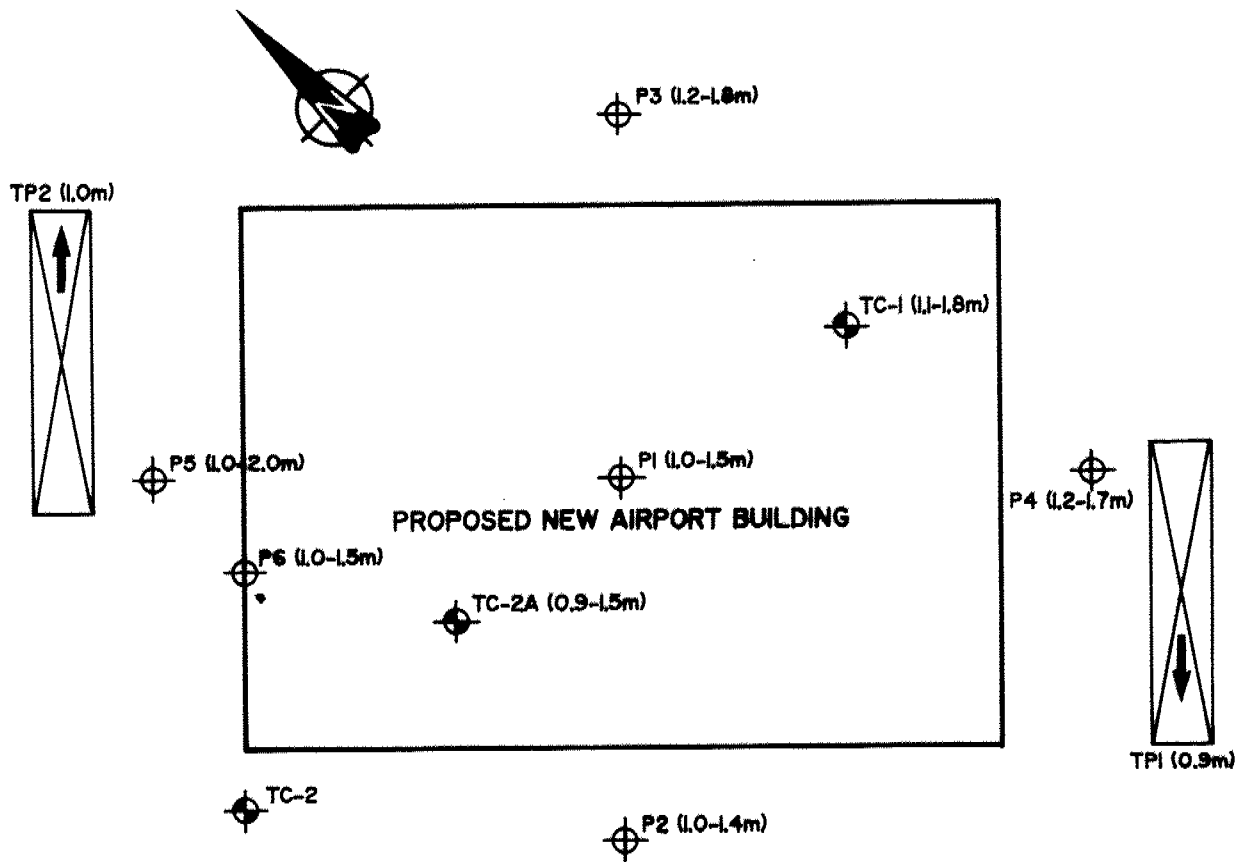
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Chkd. K.S.H.

GEO # 43M-1



LEGEND

- THERMOCOUPLE AND / OR DRILL HOLE
- PROBE
- TEST PIT

NOTES

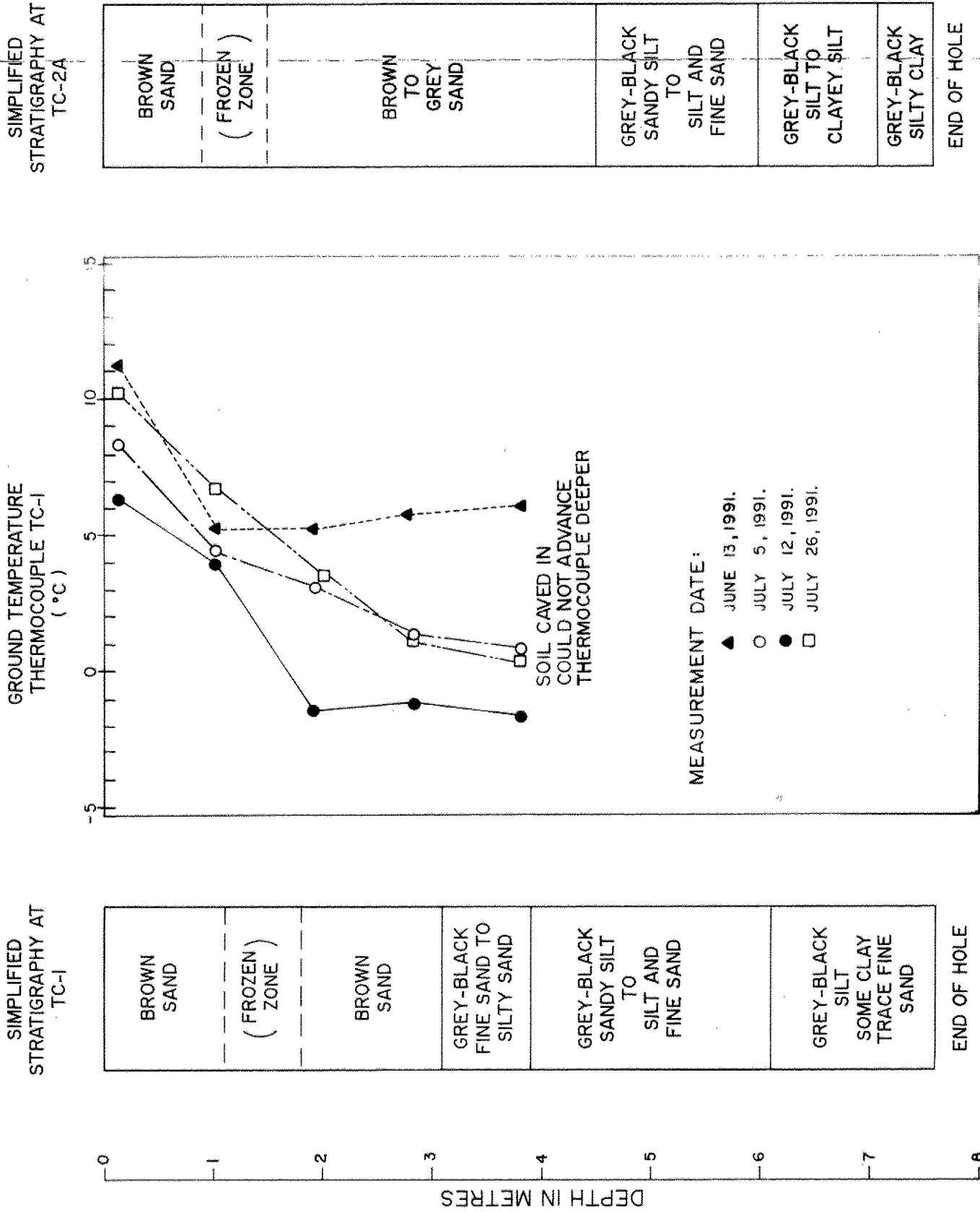
- 1) FIGURES IN BRACKETS CORRESPOND TO THE DEPTHS OF FROST INFERRED WITHIN THE HOLES
- 2) ALL THERMOCOUPLE, PROBE AND TEST PIT LOCATIONS ARE APPROXIMATE
- 3) THIS DRAWING TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT

SCALE 1:250

Date JULY 3, 1991
 Project 911-1349

Golder Associates

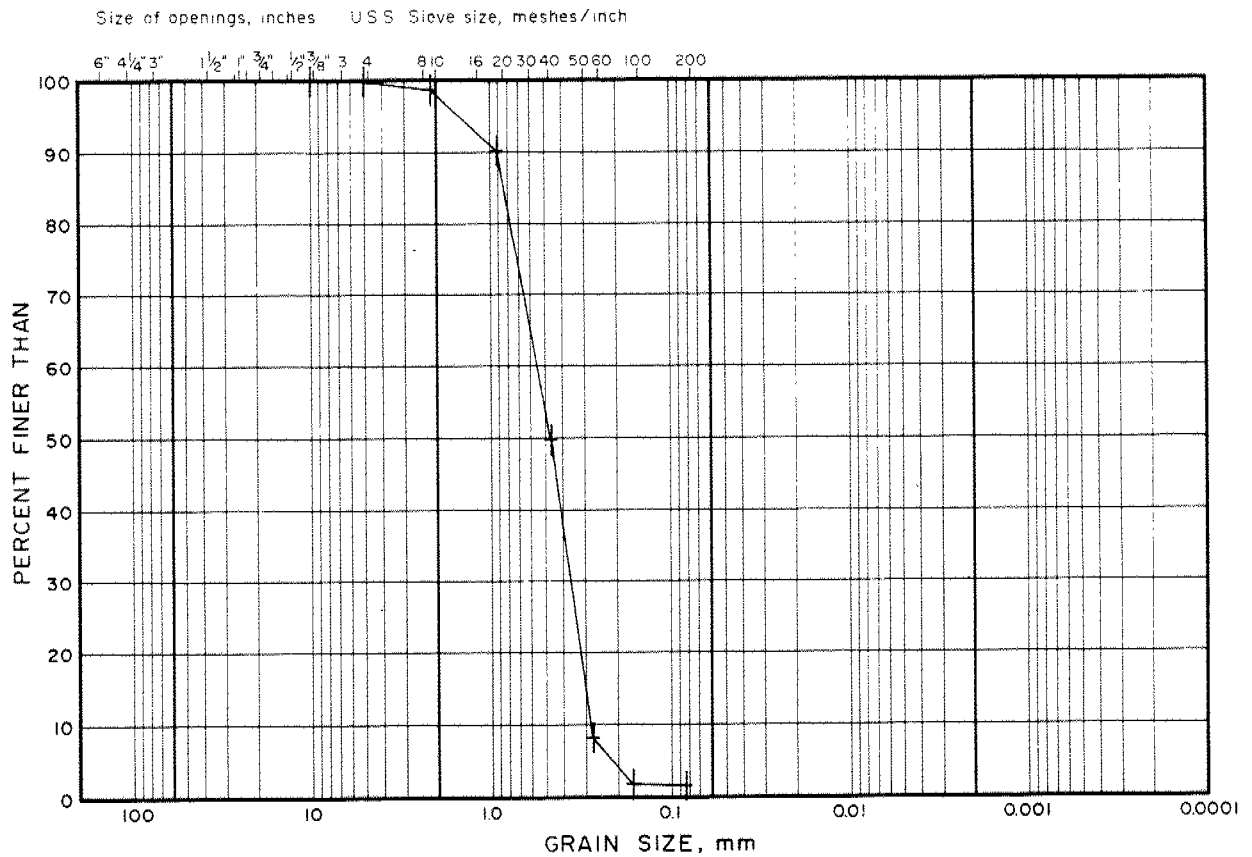
Drawn D.J.R.
 Chkd. K.S.H.



GRAIN SIZE DISTRIBUTION

FIGURE 5

SAND (Frozen In-Situ)



COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	TEST PIT	SAMPLE	DEPTH (m)
+	1		.9

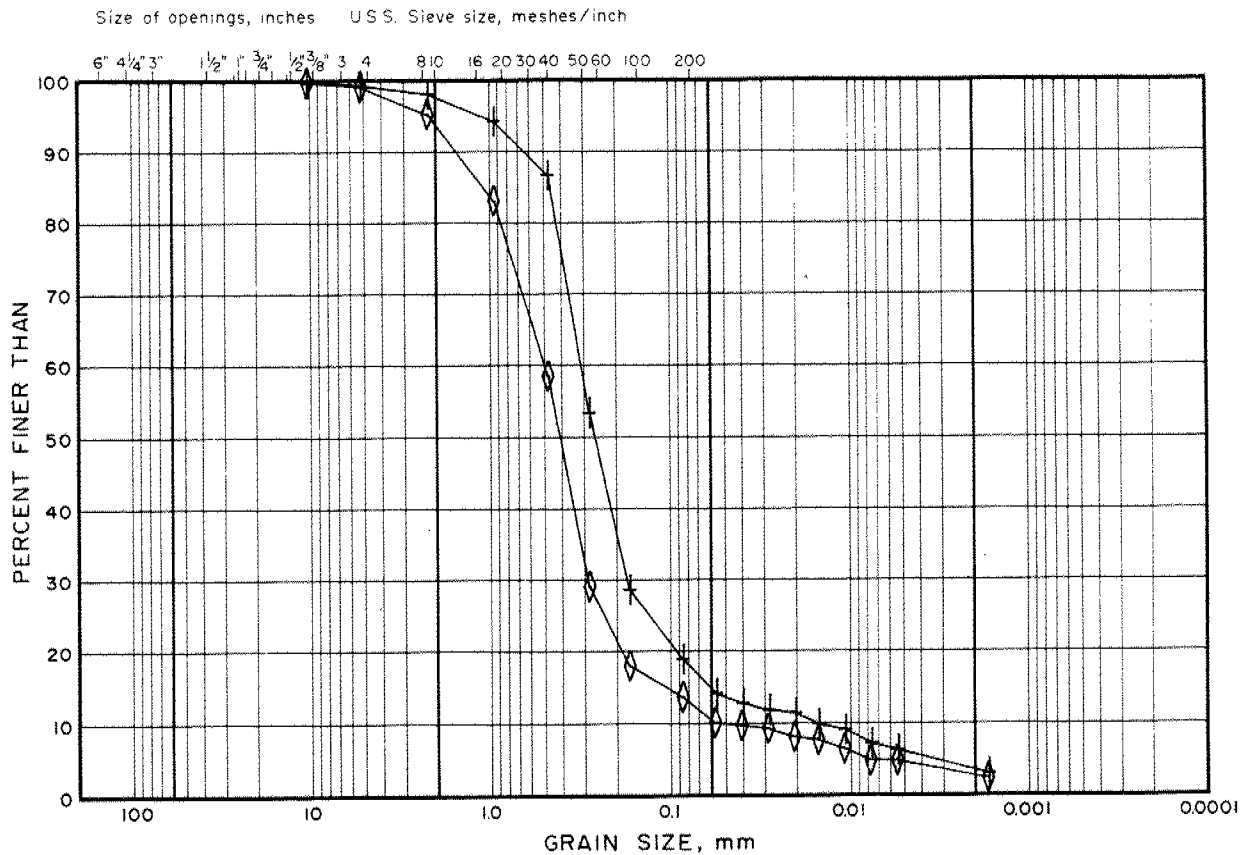
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GRAIN SIZE DISTRIBUTION

FIGURE 6

SAND, trace to some silt, trace fine gravel



LEGEND

SYMBOL	BOREHOLE SAMPLE	DEPTH (m)
+	TC-1	2
◇	TC-2A	1

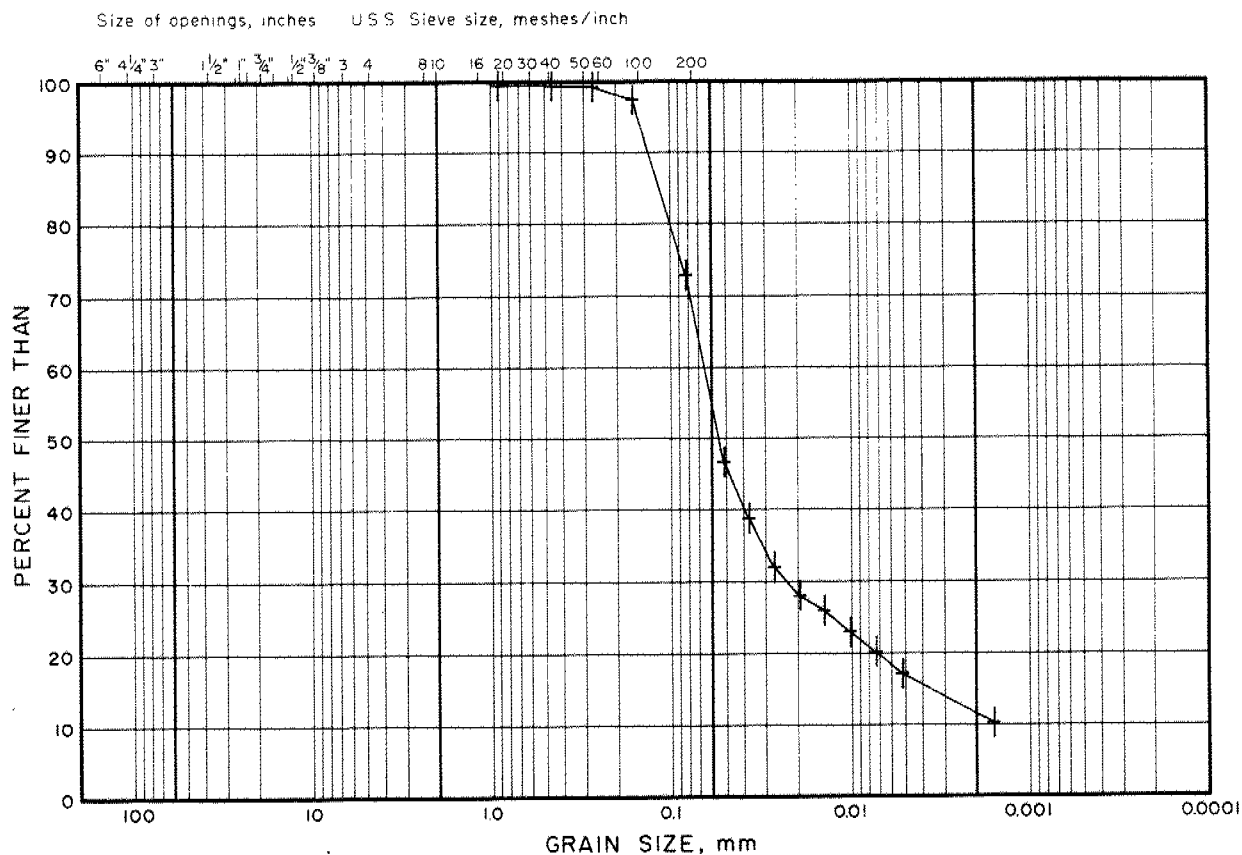
Project 911-1349

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GRAIN SIZE DISTRIBUTION

FIGURE 7

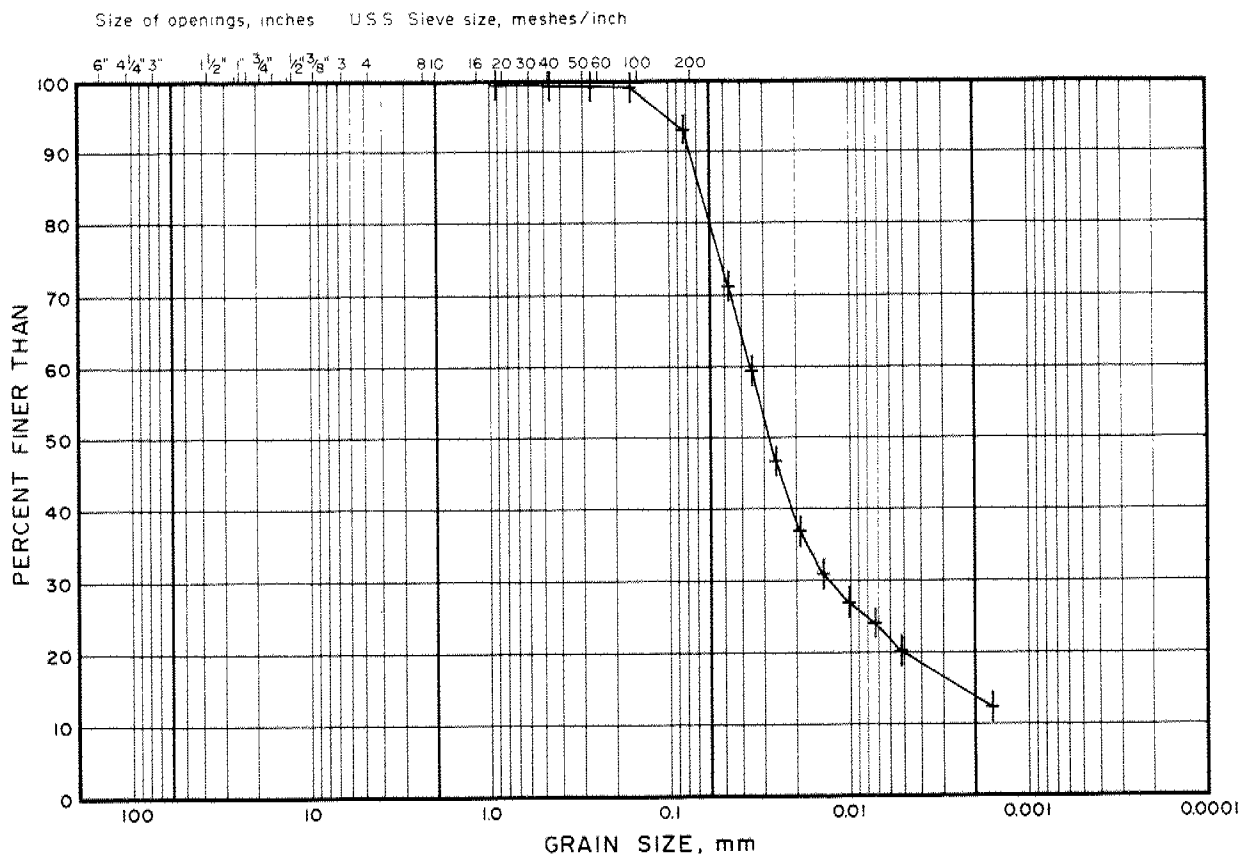
SILT and fine SAND, trace clay



GRAIN SIZE DISTRIBUTION

FIGURE 8

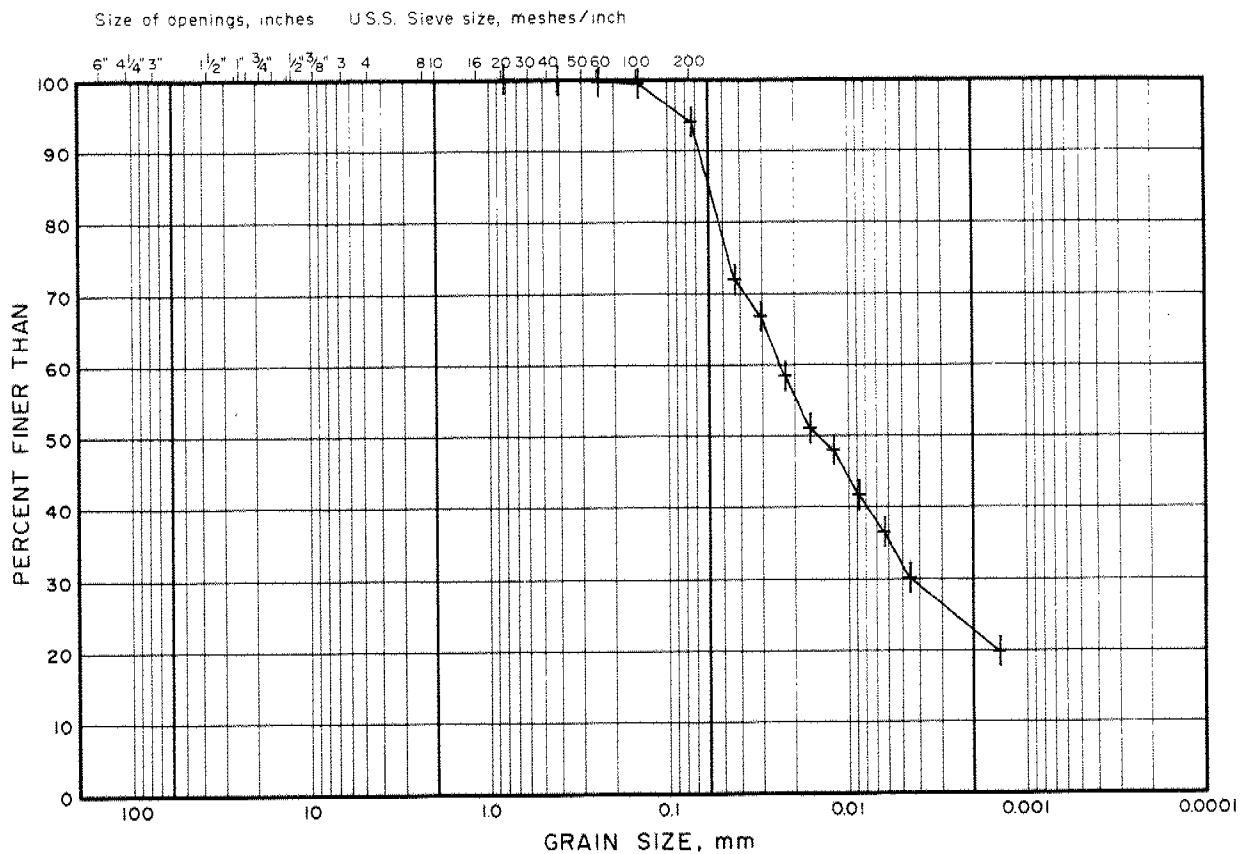
SILT, some clay, some fine sand



GRAIN SIZE DISTRIBUTION

FIGURE 9

SILTY CLAY, trace fine sand



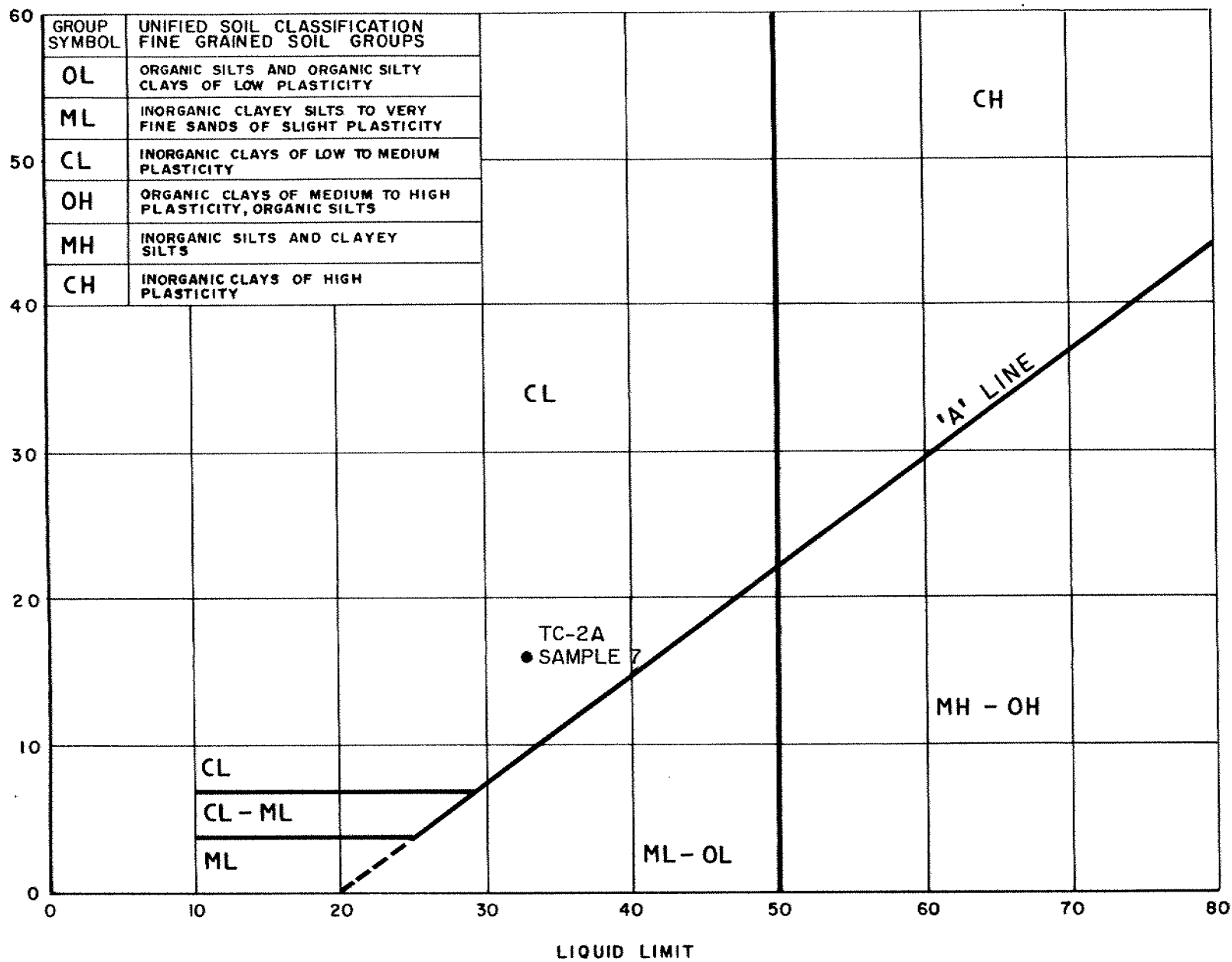
COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE	CLAY SIZE
	GRAVEL SIZE			SAND SIZE			FINE GRAINED	

LEGEND

SYMBOL	BOREHOLE SAMPLE	DEPTH (m)
+	TC-2A	7
		7.6

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PLASTICITY CHART

FIGURE 10



2321-6091-625

Golder Associates Ltd.
CONSULTING ENGINEERS



June 26, 1991

911-1349

Mr. M. Devata, P. Eng.
Chief Foundation Engineer
c/o Ministry of Transportation of Ontario
Central Building, Room 315
1201 Wilson Avenue
Downsview, Ontario
M3M 1J8

ATTENTION: Mr. P. Payer, P. Eng.
Senior Foundation Engineer

RE: W.P. 60-60329
PRELIMINARY RESULTS AND RECOMMENDATIONS
PROPOSED BUILDING AT FORT SEVERN AIRPORT
FORT SEVERN, ONTARIO

Gentlemen:

This letter presents preliminary results of the geotechnical investigation carried out at the above site. Based on observations made and factual subsoil and groundwater conditions information obtained, preliminary engineering recommendations on geotechnical aspects of building design are provided herein for the guidance of the design engineer(s). Comments on some construction aspects, which may affect design, are also provided.

We understand that a new building will be constructed at the Fort Severn Airport to house equipment and personnel. The current schedule for the building calls for the award of the contract in November, 1991. Construction is scheduled for the summer of 1992.

FIELDWORK

The geotechnical investigation fieldwork, carried out between June 11 and 13, 1991, was supervised throughout by an engineer from our staff. Six probe holes, two sampled boreholes, two thermocouple strings and two test pits were put down. The probe holes and sampled borings

were drilled to about 8 m depth, using portable Pionjar drilling and sampling equipment supplied and operated by Sonic Soil Sampling Inc. of Toronto, Ontario. In two of the borings, soil samples were obtained at regular 1.0 m intervals, for detailed visual examination and subsequent laboratory testing. Upon completion of these two holes, a thermocouple string, consisting of thermocouples at 0.9 m intervals, was installed in each hole. Because of encountered caving ground conditions, the bottom of the thermocouple string could not be installed deeper than about 4 m below the existing ground surface. The depth to frost, and its thickness, was inferred by the relative ease at which the Pionjar drilling unit could penetrate the ground. Two test pits were excavated using a front-end loader. Because of frost conditions, they could not be dug deeper than about 1 m.

Further details on the field investigation operations will be presented in our draft engineering report which is currently under preparation.

SUBSURFACE CONDITIONS

While the Fort Severn area is, in general, quite flat, the site of the proposed building lies on a local topographical high consisting of a reasonably well-drained sandy ridge or possibly an esker. The proposed site area had been recently stripped of its 0.3 m to 0.4 m thick muskeg cover prior to the start of the fieldwork.

In general, the gradation of the encountered subsoils become finer with depth. The natural subsoils encountered consist of a sand deposit, about 4 m thick, overlying an approximately 2 m thick layer of sandy silt to silt and fine sand which, in turn, overlies fine-grained silt with some clay and silty clay. With the exception of a seasonal frost layer which was encountered at about 1 m depth and was inferred to range in thickness from 0.4 m to 1.0 m, the encountered subsoils were not frozen. Within the excavated test pits, groundwater seepage was observed at about 0.6 m below ground surface. The groundwater ponded within the pit above the frost line.

Further details on subsurface conditions will be presented and discussed in our engineering report. Specific details are also discussed in the subsequent sections of this letter.

DISCUSSION AND RECOMMENDATIONS

The results of the subsurface investigation have indicated that permafrost does not exist within the depth of investigation (viz. down to about 8 m). Further, based on the initial thermocouple readings, it is unlikely that permafrost exists at depth.

A seasonal frost zone, about 1.0 m thick, was generally encountered at about 1 m depth across the site. The 1 m depth to the frost layer is considered to be a result of the recent (i.e. May/June 1991) stripping of the muskeg layer across the site, as part of site preparation operations, to expose the natural sand deposit. In areas where the muskeg layer had been left intact (i.e. natural muskeg cover surrounding the prepared/stripped site area) frost was encountered at 0.2 m below ground surface. The removal of the muskeg (which acts as a natural insulator on ground surface) has apparently accelerated the thawing of the sand deposit.

Because of the initially observed thermal gradient pattern at depth, it is considered that the inferred bottom of the frost layer, during the investigation, closely corresponds to the maximum depth of frost penetration that developed over the 1990/1991 winter season. The local native residents considered the past winter to be "very cold" and therefore, the observed, unexpected relatively shallow depth of frost penetration cannot be attributed to an unseasonably mild winter season.

The readings to date from the two thermocouple strings (installed at diagonal corners of the proposed building area) show that the minimum measured ground temperature, of about 4°C to 5°C, occurs just above and below the frost layer. Above and below the frost layer, the measured ground temperatures increase. While these initial set of readings may not necessarily reflect stabilized temperature conditions, they are sufficiently above freezing to reliably indicate that the in-situ equilibrium thermal regime lies above the freezing point (viz. except for the thin frost layer, the ground within the depth of investigation is not frozen). The relative ease at which the Pioneer drilling unit penetrated the ground below the frost layer, and examination of the recovered fine-grained silty and clayey soil at depth, also suggest that the ground is not frozen.

Existing correlations of frost penetration and the average annual Freezing Index (i.e. cumulative total of difference between annual daily mean air temperature and the freezing point (0°C) expressed in units of degree days) based on observations in open clear areas such as roads and aircraft runways, suggest a 3 m depth of frost penetration for the Fort Severn area (with a Freezing Index of about 3250°C degree days). However, these general correlations do not take into account soil type or insulating effect of muskeg cover. For example, the depth of frost penetration is greater in coarse-grained gravelly soils than in fine-grained clayey soils. Our calculations for a coarse-grained granular soil with a dry density of 16.5 kN/m^3 and water content of 15 per cent, under a Freezing Index of 3250°C degree days, indicate a frost penetration depth of 3.6 m. For a fine-grained soil, a calculated value of 2.9 m is obtained.

As mentioned above, it is considered that the observed relatively shallow depth of frost penetration at the site is a result of the insulating effect of muskeg, which until a month or so ago covered the area investigated. If the effect of a 0.3 m thick layer of muskeg is included in the analysis, a frost depth of about 2 m is calculated. This value agrees quite well with the observed maximum frost depth across the proposed building area.

Based on information provided to us by MTO, we understand that the proposed building consists of a concrete slab-on-grade structure which is generally placed on the surface of a compacted granular fill. The column footings are also generally placed on the surface of, or within, the compacted granular fill. The garage portion of the building will be heated. The section of the building for the radio and foreman's offices involves an unheated crawl space between the underside of the structurally supported floor and the prepared granular fill. This section of the proposed building is usually supported by short timber posts placed on concrete pads partially embedded within the granular fill.

During construction of the proposed building in 1992, a frost layer will exist within the natural subsoils. Because the insulating muskeg cover has been removed, the depth of frost penetration will likely be greater than that observed during the June 1991 investigation. Based on the above calculations, it could be of the order of 3 m deep. The actual depth of frost penetration will

depend on temperature, snow cover and wind conditions at the site during the 1991/1992 winter season.

The upper part of the natural ground will probably be thawed prior to construction, depending on the construction start-up date. Following completion of the building, any remaining frozen ground beneath the heated slab-on-grade will thaw, and the continued heat source will prevent further frost development in this area. However, for the end section involving an unheated crawl space, annual freeze/thaw cycles within the ground will occur. Because of these annual cycles, a potential for differential settlement and associated detrimental effects (deformation and cracking) could be experienced between the unheated and heated portions of the building.

Consideration should, therefore be given as to whether an unheated crawl space is required for the building. The effects of differential settlement due to cycles of freezing and thawing beneath portions of the proposed building will be minimized if the crawl space, if needed, is minimally heated to a temperature modestly above freezing.

Based on our experience and established classification systems, the upper 3 m or so of the natural sandy subsoils across the proposed building area are relatively clean and well-drained. Therefore, they are considered to be thaw stable (i.e. they are reasonably stable during thawing and when thawed) and possess low frost susceptibility potential. The more silty soils below the 3 m to 4 m depth are frost susceptible and potentially unstable during thawing, especially if significant ice lenses develop during freezing.

Therefore, the design should make adequate provision to minimize the probability of the natural soils below 3 m being exposed to cycles of freezing and thawing. To minimize the effects of frost action, and to reduce potential frost penetration in the existing natural soils, we recommend that a well-compacted, clean and free-draining granular pad, approximately 0.5 m (minimum) in thickness, be placed on the existing grade at the proposed building area. The top portion of the granular pad should be of a minimum dimension corresponding to the proposed building dimension plus 3 m on all sides. The side slopes of the granular pad should not exceed 2 horizontal to 1 vertical and to enhance erosion resistance, they should be covered with a 0.3 m minimum thick layer of coarse particles (such as coarse sand, gravel and cobbles).

The purpose of the granular pad is two-fold; it minimizes the depth of frost penetration in the natural ground and enhances positive drainage of surface water away from the proposed building.

The granular pad should be constructed using clean, free-draining and non-frost susceptible material such as OPSS Granular B material. The granular material should be placed in maximum loose lifts of 200 mm with each lift being compacted using a heavy vibratory roller to at least 95 per cent of the Standard Proctor maximum dry density. For efficient compaction operations the placement water content should be within two per cent of the optimum water content for compaction purposes. The final 150 mm base layer beneath the proposed concrete slab should consist of OPSS Granular A material compacted to at least 100 per cent of Standard Proctor maximum dry density.

Prior to placement of the granular pad, the existing exposed natural subgrade should be clean of any organic material and proof-rolled and any softened areas sub-excavated and replaced with a well compacted clean, free-draining granular material as described above for the granular pad.

Provided the above recommendations are followed, the footings can be designed using an allowable bearing pressure of 100 kPa. For this bearing pressure, normally anticipated total and differential settlement should be of the order of 25 mm and 20 mm, respectively. However, the proposed building is quite flexible in nature and it can probably tolerate greater settlements. For the design of the concrete floor slab, a modulus of subgrade reaction of 60 MN/m³ (200 tons/ft³) for a 0.3 m by 0.3 m bearing area may be assumed.

To provide frost protection to perimeter footings, a combination of soil cover and insulation can be used. To minimize the amount of insulation required, it is suggested that perimeter wall footings be founded as deep as practical within the natural sand subsoils. Groundwater control will be required to keep the excavation dry, but this can be achieved by pumping from properly filtered sumps at the base of the excavation. Care will be necessary during foundation excavation and dewatering to prevent disturbance of the subsoil below subgrade level.

If possible, it is recommended that these footings be founded at a minimum depth of 1 m below the existing ground surface at the time of the investigation. Based on observations made during the recent investigation it is considered that thawing of the ground surface could extend this deep by about June. Suitable insulation such as expanded polystyrene or equivalent should be, for convenience, placed on the prepared existing ground surface and carefully covered with the granular pad material. The insulation should be placed on a smooth sand bedding at a small inclination of not less than one per cent to promote drainage of surface infiltration water. The insulation should tie into the insulation running down the wall/footing and extend horizontally for a minimum distance of 2.5 m.

The required thickness of insulation depends on the embedment depth of the footing in the natural sands. If an embedment depth of 1 m below existing ground surface can be conveniently achieved, a 75 mm minimum thickness of insulations should be adequate. If, for construction expediency, equipment and personnel availability and economic considerations, it is considered impractical to place the perimeter wall footings within the natural sands, increased thickness of insulation will be required. For an embedment depth of 0.5 m below existing ground surface, 110 mm (minimum) of insulation will be required. Alternatively, the thickness of the granular pad can be increased by 0.5 m to 1 m. However, this may cause grading problems and encroach building height (i.e. elevation) constraints relative to the nearby airstrip.

While the proposed granular pad material and the natural subsoils to 3 m depth below existing grade are reasonably thaw stable, and possess low frost susceptibility potential, differential settlements can be expected along the building foundation unit, in particular, between the offices and garage area and along the perimeter of the building. To mitigate the effects of differential settlement, special provisions such as capability for jacking and shimming of the timber posts beneath the offices and large garage door openings should be made to facilitate levelling of the building, as required, with time. Extra long anchor bolts to fasten the posts to concrete pads should be used to permit adjustments to be made. The concrete floor slab should also be structurally independent of walls and columns. The wall between the garage and offices should be structurally independent or designed to accommodate significant differential settlement at this location.

There should be adequate frost protection provided for any sub-foundation level perimeter drains installed around the building. The perforated drain should be wrapped in an appropriately chosen filter cloth. Frost protection could consist of a combination of soil cover and insulation as previously discussed for the perimeter wall footings. Insulation should be incorporated into perimeter building walls (including the crawl space perimeter walls) and extend down to footing level.

Case records exist which demonstrate that light buildings in open exposed areas can be susceptible to strong gusts of wind and may be displaced laterally. For a sliding stability design check, a coefficient of friction between the concrete pads/footings of 0.45 may be assumed. If adequate resistance to sliding cannot be developed through "self-weight" of the building and footings, adequate anchorage will need to be provided.

We trust that this letter provides sufficient information for your immediate requirements. Should you have any questions, or if you consider that some of the comments and recommendations of this letter should be revised for incorporation into the draft report, please do not hesitate to call us. We will be pleased to discuss any aspect of this letter, or the project, with you.

Yours truly,

GOLDER ASSOCIATES LTD.



Dennis E. Becker, P. Eng.

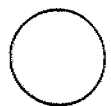



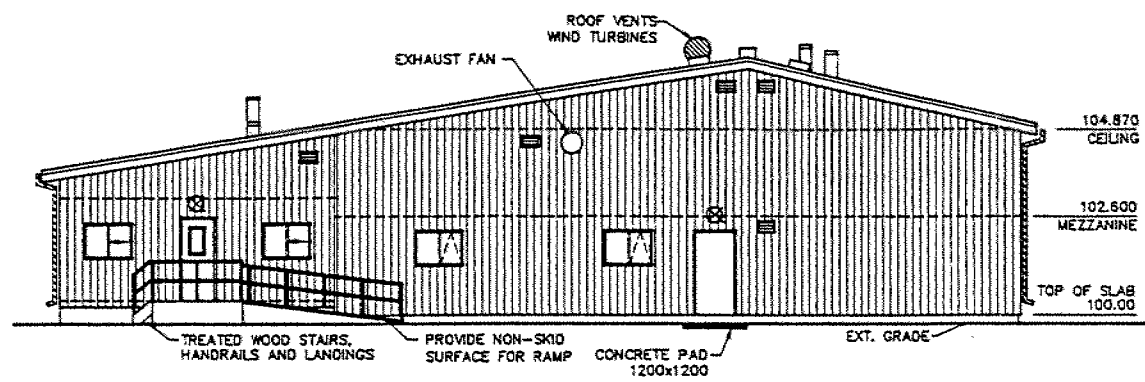
John L. Seychuk, P. Eng.

DEB/JLS/pds

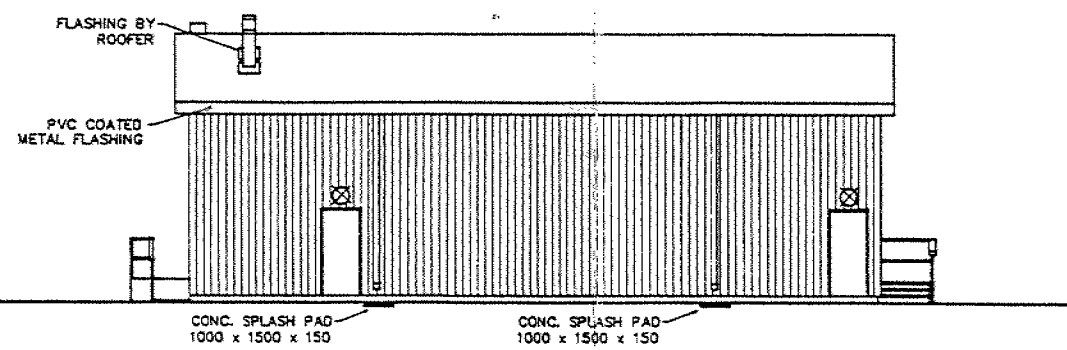
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METRIC
DIMENSIONS ARE IN METRES
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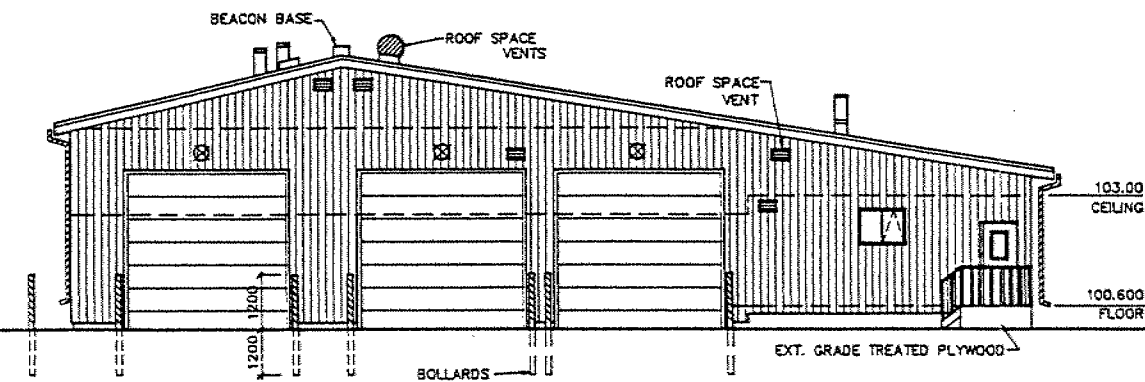
ONT No 90-701 WP No 2009-89-02	
WUNNUMIN AIRPORT GARAGE/RESIDENCE BUILDING ELEVATIONS AND SECTION	SHEET 3
 C.D. HOWE CENTRAL LTD. CONSULTING ENGINEERING SERVICES OTTAWA THUNDER BAY MONTREAL	DESIGN W.V.R. DRN L.L.L. DATE 10/90 DWG A3
C. D. HOWE REFERENCE #80097-03	



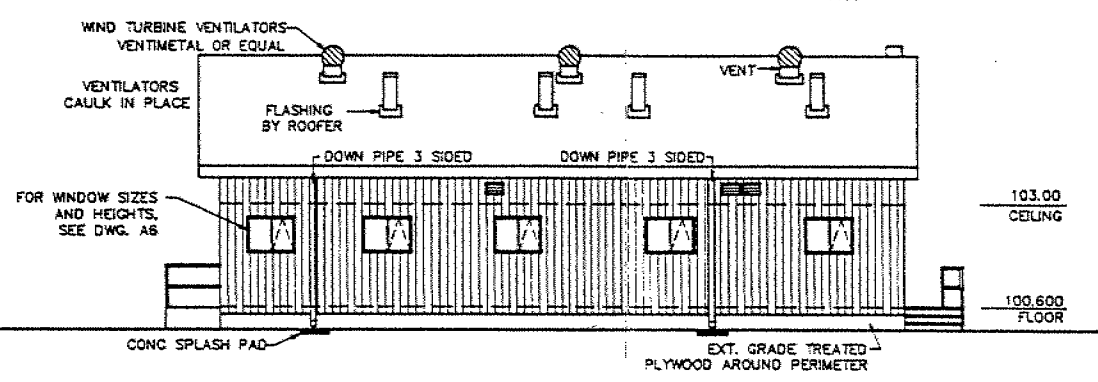
NORTH ELEVATION
1:100



WEST ELEVATION
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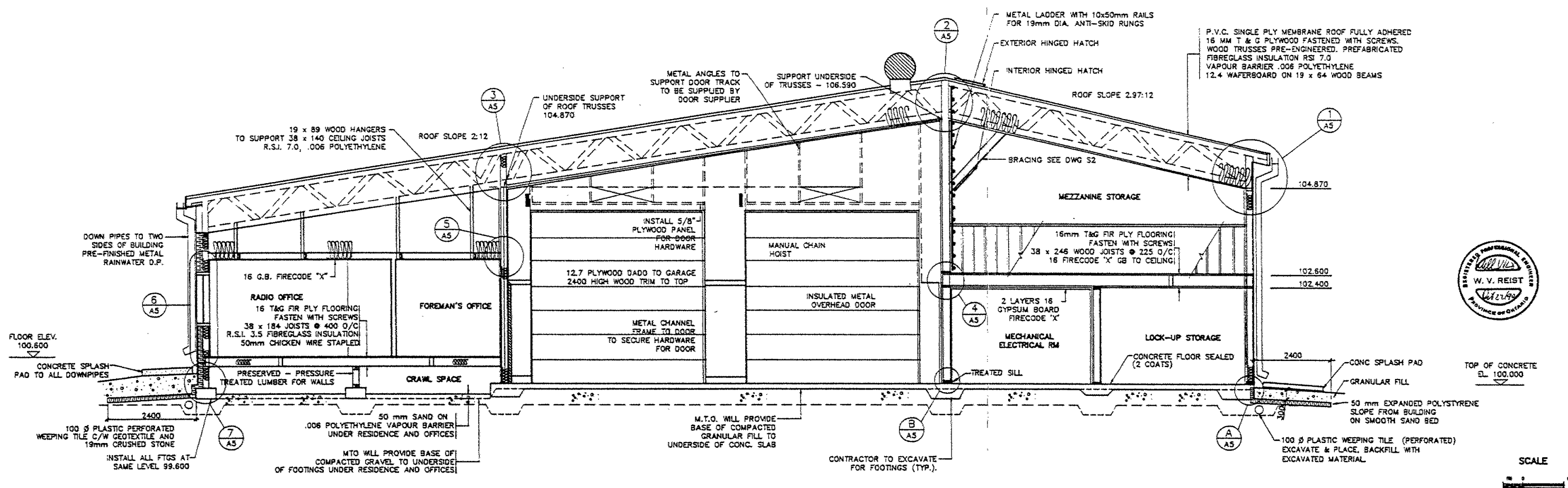


SOUTH ELEVATION
1:100



EAST ELEVATION
1:100

REFERENCE ELEVATION:
TOP OF NEW CONCRETE FLOOR SLAB IN GARAGE 100.00

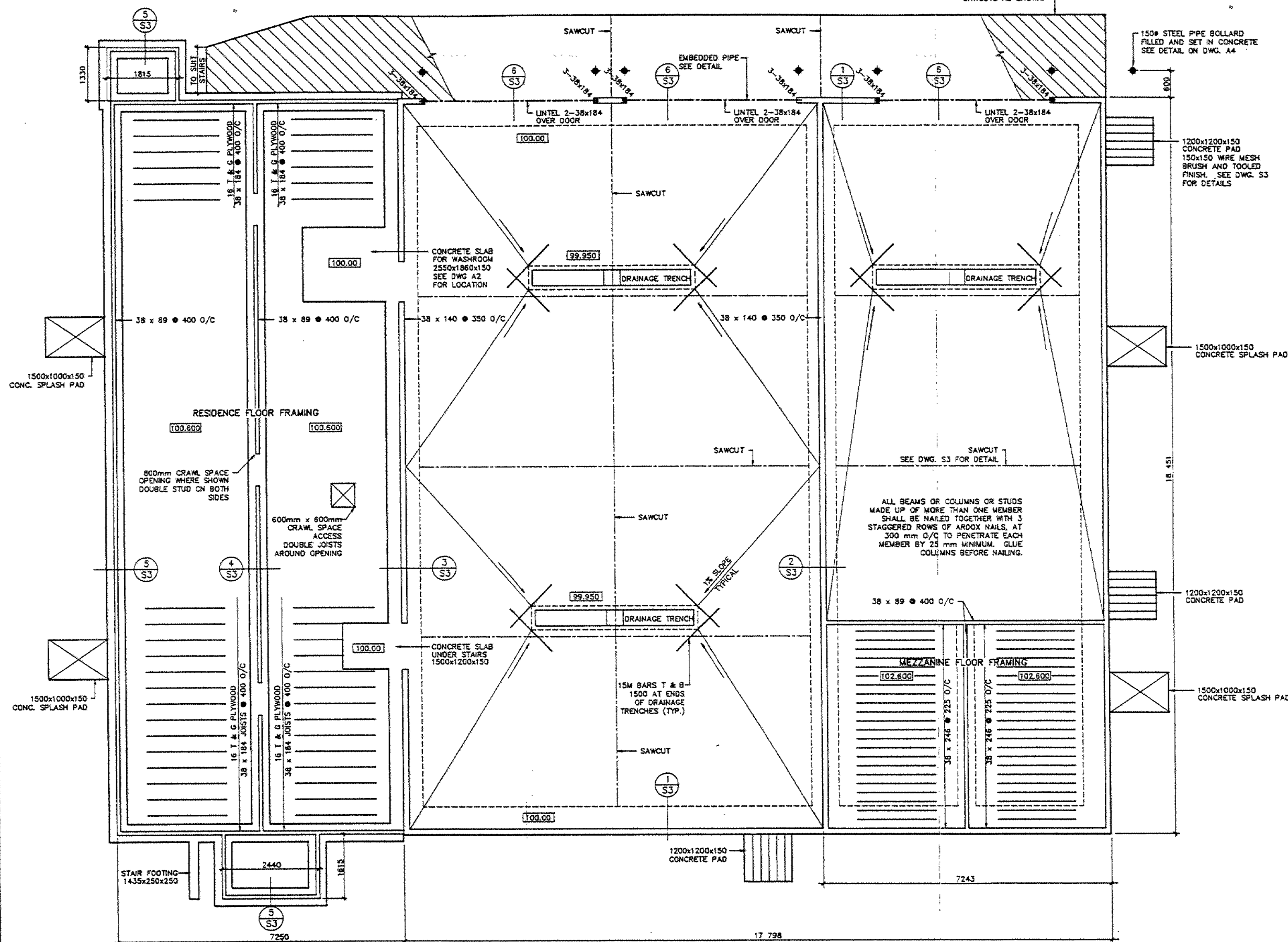


CROSS SECTION A-A
SCALE 1:50



SCALE
1:50

PLAN, SECTION, ELEVATION OR DETAIL NO.
NO. OF DRAWING WHERE ABOVE IS DRAWN



GENERAL NOTES

1. CHECK ALL DIMENSIONS ON S1 WITH THE ARCHITECTURAL DRGS. REPORT ANY INCONSISTENCIES DO NOT SCALE DRGS.
2. PROTECT FOOTINGS, WALLS, SLAB ON GRADE AGAINST FREEZING AND FROST ACTION.
3. BARS NOTED CONTINUOUS SHALL BE DEVELOPED BY LAPPING. MIN. LAP = 1000 mm
4. MINIMUM COVER TO REINFORCING AGAINST EARTH = 75 mm
5. DESIGN GROUND PRESSURE $95 \frac{\text{KN}}{\text{M}^2}$
6. CONCRETE COMP. ST. 25 MPa @ 28 DAYS
7. M.T.O. WILL PROVIDE BASE FOR FOOTINGS AND FLOOR SLAB. THIS BASE WILL EXTEND TO UNDERSIDE OF FLOOR SLAB. CONTRACTOR TO EXCAVATE IN BASE FOR FTGS. UNDER RESIDENCE BASE PROVIDE TO UNDERSIDE OF FOOTINGS.
8. FOOTINGS TO BE POURED WITH CONCRETE FLOOR SLAB.
9. INSTALL 1--LAYER OF 15 FELT BARRIER UNDER ALL STUDS ON CONCRETE. IN GARAGE AREA & STORAGE AREA WOOD SILL TO BE PRESSURE TREATED WOOD.
10. PERIMETER INSULATION TO BE 50 mm x 2800. INSTALL CAREFULLY DOUBLE LAYER AT CORNERS.
11. INSTALL EXTERIOR PRESSURE TREATED PLYWOOD WHERE INSTALLED IN SUB-GRADE CONDITIONS
12. CONCRETE FLOOR SLAB 150mm THICKNESS.
13. FOR SAWCUT CONTROL JOINT DETAIL, SEE DWG. S3.
14. INSTALL CONCRETE RAMP AT GARAGE DOORS, SLOPE AWAY FROM BUILDING.
15. REINFORCING SHALL BE DEFORMED BARS, GRADE 400.



SCALE

0.5m 1.0m