#13

GROUND ENGINEERING

CIVIL & GEOTECHNICAL ENGINEERS

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October 07, 1980

File GE-620

DeLCan → STANTEC CONSULTING LTD.
2149 Albert Street 301 - 1919 ROSE ST
REGINA, Saskatchewan 781-6400
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ATTENTION: Mr. George Taylor, P. Eng.

Dear Sir:

SUBJECT: Geotechnical Investigation
Proposed Transit Operations Centre for the City of Regina
REGINA, Saskatchewan

I INTRODUCTION

This report presents the results of a subsurface soils investigation and geotechnical analysis carried out at the site of the proposed Transit Operations Centre to be constructed for the City of Regina. The completed facility is to include administration, operations, maintenance, servicing, inside vehicle storage and site areas. Authorization to proceed with this work was given verbally by Mr. Dale Turvey at the site meeting of July 30, 1980

The objectives of this investigation were to provide the following information:

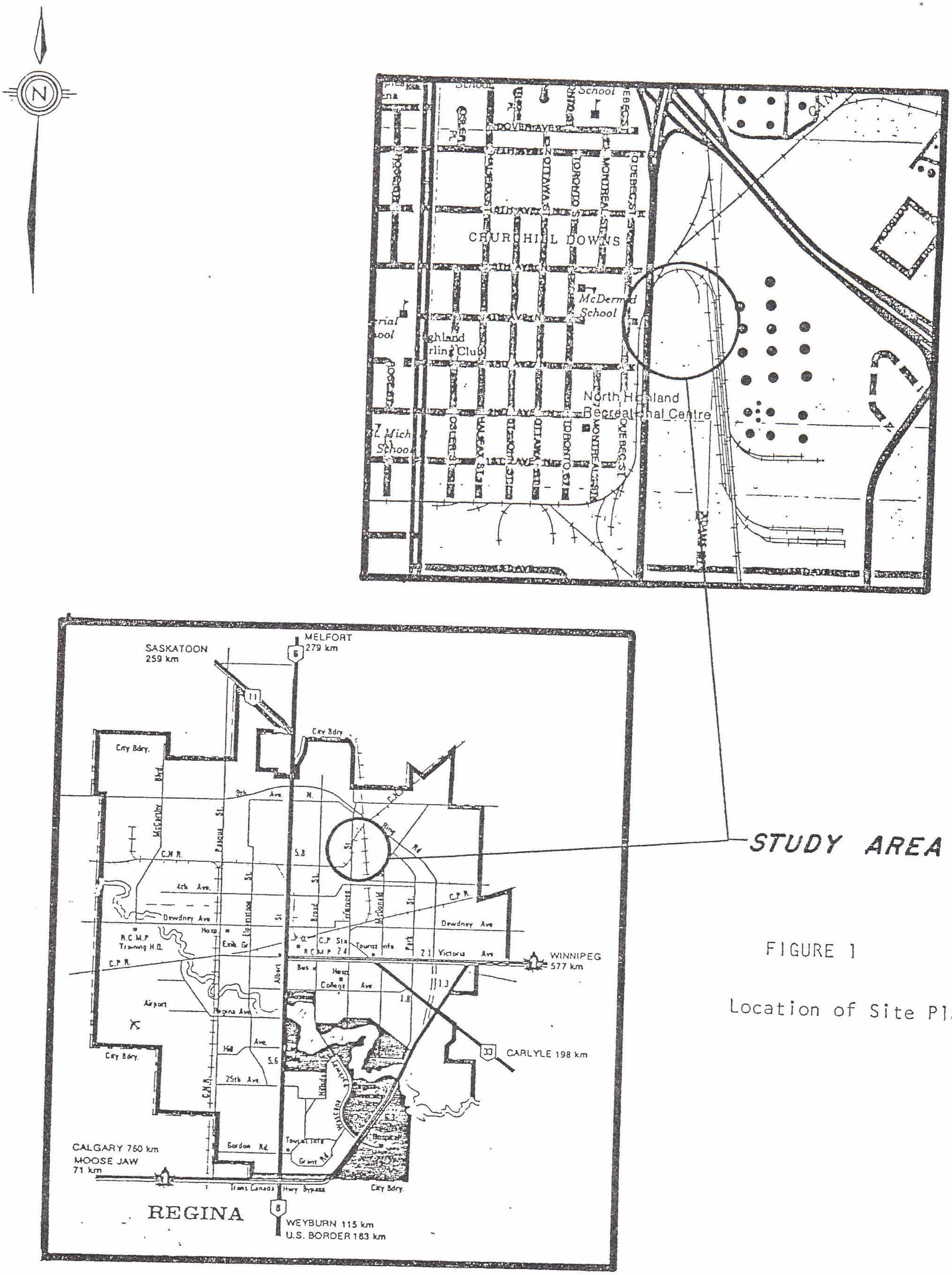


FIGURE 1

Location of Site Plan

1. Subsurface soil conditions existing at the building site.
2. The most suitable method of foundation support for the building structure.
3. Possible excavation and construction problems with particular reference to groundwater.
4. Pavement structure design.
5. Design considerations for underground fuel storage tanks.

This report has been prepared for the exclusive use of DeLCan for specific application to the design and construction of the foundations to support the proposed Transit Operations Centre for the city of Regina. It has been prepared in accordance with generally accepted soil and foundation engineering practices.

II DESCRIPTION OF THE SITE

The study area as shown in Figure 1 is to be located east of the 200 to 300 Block of Winnipeg Street in northeast Regina, Saskatchewan. The proposed building site is part of the property which had formerly been completely developed as the site of the Imperial Oil Refinery. The structures have been removed however most of the foundations consisting of concrete slabs, partially filled basements, underground storage tanks and reservoirs, roadways and paved areas are still intact though partially demolished and covered with debris. There is also an extensive system of abandoned underground utilities such as municipal services and industrial piping. An air photo enlargement showing the structures that were located on this property in 1975 is shown on Drawing No. GE-620-1. The topography is relatively flat with the ground surface varying less than about 1 metre across the entire property.

Considerable contamination from oil and gasoline is present in most areas. Generally, the soils are contaminated with hydro carbons to a depth of about 7 to 10 metres.

III. FIELD AND LABORATORY INVESTIGATION

The subsurface soil and groundwater conditions were investigated by a total of seven (7) test hole borings drilled at the locations shown on Drawing No. GE-620-1. Test Holes 101, 104 and 107 were drilled on August 8, 11 and 12, 1980 using a Brat model B24 digger equipped with a 230 mm diameter continuous flight, hollow stem auger and a 150 mm diameter continuous flight auger to depths ranging from 14.3 to 33.5 metres below existing ground surface. Test Holes 102, 103, 105 and 106 were drilled on August 29, 1980 using a Hughes Model LDH 80 digger equipped with a 610 mm diameter auger.

Standard Penetration tests were performed at selected intervals during test drilling. In addition, representative disturbed and undisturbed soil samples were recovered from the test borings and taken to our laboratory for analysis. Each soil sample was visually classified and a natural moisture content test was performed on each soil sample. An estimate of the shear strength of the undisturbed soil samples was made using both a pocket penetrometer and a laboratory vane shear apparatus. In addition, Atterberg Limits, unconfined compression, soluable sulphate content and dry density tests were performed on selected representative soil samples. Details of the soil profile, samples taken, laboratory test results and stratigraphic interpretation of the subsoils are appended to this report on Drawings No. GE-620-9 to -16 inclusive.

IV GEOTECHNICAL ANALYSIS

A. Stratigraphy

The approximate boundaries of the various stratigraphic units encountered are illustrated in the cross-sections (Drawings GE-620-8 and -9) and the test hole logs. Test Hole No. 104 (Drawing GE-620-13) gives the best illustration of all the sediments penetrated.

The drilling information indicates that the site is overlain with an expansive, silty clay which extends to a depth of 7.0 to 9.7 metres below exisitng ground surface. This material is primarily highly plastic clay with numerous silt layers near its base. It is stiff, laminated, and moist with a water content close to the plastic limit. The clay is highly jointed with numerous clearly developed slickenslides and prominent iron and manganese staining. The number of silt layers increases with depth. A surficial topsoil and/or fill material consisting of asphalt and granular base was encountered at all test hole locations. The depth of the topsoil and/or fill varies from about negligible to about 200 mm, however, there maybe areas on this property where the depth of fill material may be considerably greater.

The surficial expansive, silty clay is underlain by a laminated, clayey, sandy silt which extends to a depth of about 16.7 to 18.2 metres below existing ground surface. The clayey silt stratum is damp near the surface becoming moist to wet near the base. It is low plastic and medium dense. In Test Hole 101 an interbedded till layer was encountered from a depth of 8.2 to 9.0 metres. In Test Hole 103 a water bearing gravel seam was encountered at a depth of 11.6 to 14.6 metres and in Test Hole 107 a saturated sand layer was encountered at a depth of 18.2 metres.

The stratified drift (clay and silt) is underlain by an unoxidized, firm to very stiff till unit which extends to the maximum depth penetrated in Test Borings, 101, 102 and 104. The till is a heterogeneous mixture of gravel, sand, silt and clay with numerous cobblestones and boulders. In Test Hole 102 a saturated sand and gravel layer was encountered at a depth of 17.5 to 18.0 metres.

The analysis and recommendations submitted in this report are based in part on the data obtained in the test hole logs. The boundaries between soil strata have been established only at the bore hole locations. Between the bore holes, the boundaries are assumed from geological evidence and may be subject to considerable error. If variations are in evidence during construction of the foundation it will be necessary to re-evaluate the recommendations of this report.

B. Groundwater

All soils encountered are moist and the silt stratum progressively increased in moisture content with depth becoming saturated below a depth of about 10.0 metres. All soils at the clay/silt contact were saturated with hydro carbons. Water seepage was noted in the interbedded sand and gravel layers encountered at various depths below about 12 metres. Sloughing was also noted in the interbedded sand and gravel layers. No piezometers were installed at this site to monitor the piezometric levels.

V DISCUSSION

6

The surficial, highly plastic clay encountered to a depth of about 7.0 to 9.7 metres at this site is an expansive type soil that is desicated near the ground surface. It is anticipated that this material will exhibit a considerable increase in volume as it increases in moisture content following construction of the building and pavement structure. Because of the montmorillonite minerology and drier intitial moisture contents, their usual reaction to changed environment conditions is to swell or exert large pressures against non-yielding structures. These soils are also subject to a high degree of shrink-swell volume change. A classification (Figure 2) based on percentage of clay fraction and plasticity index can be used to categorize probable volume change severity.* A soil having a clay content in excess of 25 percent and a plasticity index greater than 30 percent would be suspected of a very high potential for shrinking and swelling. This volume change or swelling causes surface deformation of pavements, movement of footings, and creates swelling pressure on walls and foundations.

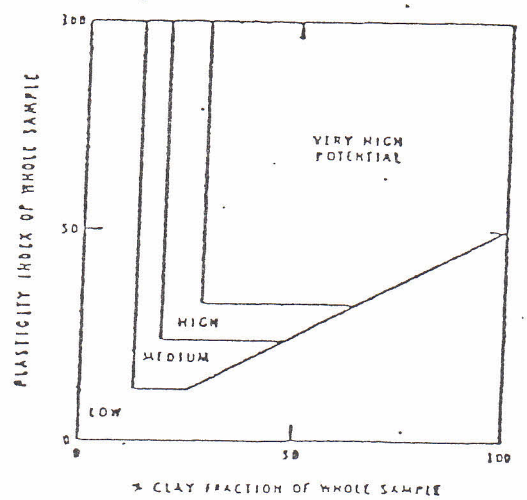


FIGURE 2.
Potential severity of volume change of clay soils.
(After Williams (2))

* Foundations on Swelling and Shrinking Subsoils
by J.J. Hamilton, Canadian Building Digest - CBD 184

Construction and landscaping activities can have great impact on the magnitude and depth of influence of ground movements. The introduction of deep rooted vegetation in areas where it has not grown previously, or the removal of mature vegetation which has depleted subsoil moisture, has resulted in surface settlement or heaving of the order of 150 mm extending for considerable horizontal distances. Heavy irrigation or changed ground surface covers have caused similarly large surface movements. Relatively small reductions in total stresses due to lowered grades or excavation have also induced large rebound swelling. Rapid heat flow to or from un-insulated surfaces has also caused spectacular changes in soil moisture and volume.

Structures founded in the "active" zone which is a term used to describe the dynamic environment around structures on or in potentially active subsoils often exhibit rapid deterioration of structural elements. The active zone is considered to encompass all of the subsoil mass around and below a structure which is or will be appreciably affected by the presence of the structure. Included in these effects ar cyclic or long-term changes in soil moisture contents, soil volume changes, ground water levels, effective stresses, shear strength, soil temperatures, soil chemistry, and frost action. The rational engineering design approach commonly taken for foundations of larger buildings is to utilize deeper foundation units which induce little or no differential movements in the superstructure. Usually these foundations are designed to develop the bearing capacity in stable ground conditions below the active zone (Figure 3).

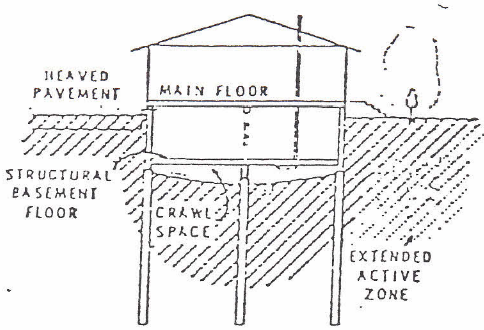


FIGURE 3.

Typical long-term deep foundation performance on a deep deposit of expansive subsoil, including effects of tree growth.

Trouble free performance for these foundations still require strict attention to many design and construction details: sufficient tensile strength in bearing walls and piles to resist uplift forces and in foundation beams at the walls to resist horizontal and vertical forces; void spaces maintained between the soil and all the grade beams, pile caps, footings and structural floor systems; and special attention to connections and transitions between the main structure and all ground supported appurtenances, such as doors, steps, sidewalks, driveways, tunnels, planters, water, sewer, gas, power and communication conduits. The large differential movements of the latter are usually sharply contrasted against the stationary structure unless adequate transitions or flexible junctions are provided.

The damp to wet clayey, sandy silt stratum which extends to a depth of about 18.2 metres is medium dense and compressible. The wet silt is subject to dilatency (reaction to shaking) when disturbed. It is not considered to be a competent foundation bearing material.

The till formation encountered below a depth of 16.7 to 18.2 metres is a stable soil type. It is stiff to very stiff in consistency and is considered to be the most competent foundation bearing material encountered at this site.

VI FOUNDATION CONSIDERATIONS

It is understood that the structure is to be one storey in height and basementless. The type of construction is still under consideration at the time of writing this report. The typical structural loads on the interior columns will be 530 kilonewtons; on the exterior columns 265 kilonewtons and continuous wall loading of 87.6 meganewtons per metre. We recommend that the walls and columns of the building structure be supported on a drilled shaft concrete pile and grade beam type of foundation system. Our specific design recommendations for this type of foundation unit are presented below:

1. It is recommended that relatively light foundation loads (550 (125 Kips) kilonewtons or less) be supported on straight shaft piles designed to develop load carrying capacity on the basis of side friction only. An average allowable skin friction carrying capacity of 29.0 (600 psf) kilopascals of contact area between the pile surface and the

surrounding soil may be used for the clay, silt and till. The upper 1.8 metres of pile length should be discounted insofar as skin friction is concerned. The formation level for piles should be a minimum of six (6) metres below finished grade elevation and the minimum pile shaft diameter should be 400 mm. 16"

2. If additional bearing capacity is required, the piles could be belled out or expanded at the base. An allowable end bearing value 6.06 kip/sq ft. of 290 kilopascals is available in the clay at a depth of 7.5 metres. If the bells are excavated mechanically and not hand cleaned and inspected, the allowable end bearing value should be reduced by one-third (1/3).

3. The test holes remained open to a depth of about 14.6 metres during drilling indicating that the soils possess sufficient stand-up time to this depth to complete construction of drilled shaft piles. It should be possible to construct drilled shaft piles designed as straight shafts to this depth without using temporary steel casing provided that the steel reinforcement and concrete are placed immediately following completion of excavation to minimize the danger of sloughing and/or ingress of groundwater into the bottom of the pile hole.

4. Caissons with loads in excess of 550 kilonewtons should have a minimum shaft diameter of 28" 710 mm in order to allow the base to be properly cleaned and inspected prior to placing of the concrete

and steel reinforcement. IT MUST BE FULLY APPRECIATED THAT IF THE ABOVE LOADING INTENSITIES ARE TO BE DEVELOPED, VERY STRINGENT SITE CONTROL AND INSPECTION BY A QUALIFIED SOILS ENGINEER WILL BE REQUIRED DURING THE FINAL PREPARATIONS OF THE BELL AND THE PLACING OF THE CONCRETE IN THE CAISSON BASE AND SHAFT.

5. The foundation soils at this site are very heavily contaminated with gasoline and fuel oils. It is recommended that the foundation piles be designed as straight shaft piles so that it would not be necessary to lower workmen and/or inspectors into the holes to clean and inspect the foundation units. Very strict safety precautions would be required to protect workmen and/or inspectors that would have to go down into the pile holes against accumulation of gases. The necessary safety precautions can be provided if required.

6. To minimize fallout, the side slopes of the caisson bells should not be greater than 45 degrees from the vertical (or approximately 1 horizontal to 1 vertical) The vertical side of the base of the bell should be no less than 150 mm in height. The bell diameter should be no greater than approximately two and one-half ($2\frac{1}{2}$) with a maximum of three (3) times the shaft diameter.

7. Pile shafts carrying little or no bending moment should be reinforced with nominal vertical reinforcement in the form of intermediate grade deformed bars, composing about one-half (1/2) of one (1) percent of the cross-sectional area. The cage should be projected or dowels set into the top of the caisson to tie into the foundation walls or columns.

8. Recommended design, installation and inspection procedures are outlined in Appendix A.

9. Spandrel grade beams should be reinforced continuously at both top and bottom to resist possible negative bending stresses at mid span. To eliminate the possibility of upheaval, the grade beams should be constructed with a void space underneath to prevent contact with the underlying soil. Alternately, they must be adequately reinforced to resist swelling pressures of 380 kilopascals on horizontal surface in contact with the clay soils.

10. A minimum of 75 mm rigid insulation should be placed on the inside of all perimeter grade beams to reduce the heat losses.

VII UNDERGROUND FUEL TANKS

It is understood that underground fuel tanks are to be installed. Our design recommendations for this facility are intended to make the tank as leakproof as possible and to remove any fuel which might accumulate around the tank. The detailed recommendations are as follows:

1. The tank may be supported on a raft foundation. An average allowable bearing capacity of 145 kilopascals is available in the clay at a depth of 2.4 metres below ground surface. Considerable care must be exercised to ensure that the bearing stratum is not disturbed or softened by excavation activity, or allowed to dry following

exposure. To prevent drying, the granular backfill and/or a layer of lean concrete should be placed as soon as possible after as the excavation reaches its final grade.

2. After the base of the excavation has been inspected, the foundation for the raft should be prepared by placing approximately 400 mm of well graded gravel with less than five (5) percent of its dry weight passing the No. 200 sieve. The granular base should be compacted to 95 percent of Standard Proctor Density (ASTM D-698).

3. It is imperative that water not be allowed to collect around the foundation of this structure. An effective subdrainage system as described in Section IX above should be installed to keep the foundation area dewatered. This system should drain to a sump which should discharge into the municipal storm drainage system, or into another separate collection system for disposal if it is * contaminated with fuel. The subdrainage system should be protected against sewer back-up through use of directional check-valve system.

4. The walls should be backfilled with clean (less than 5 percent passing the No. 200 sieve) free-draining sand or gravel for the full depth of the wall.

5. Rigid inspection during construction is very important. Drainage tile placement and backfill are very critical operations.

6. A leakage test should be conducted before the tank is backfilled.

that a combination of floor systems be considered depending on the use of the building areas.

A. Structurally Supported Floor System

If movement of any of the floor slabs is critical, they should be designed to be structurally supported with a crawl space below to permit expansion of the clay without movement of the floor slab. This method gives a superior floor system and allows placement of ducts and mechanical systems below the floor. For a structurally supported floor sytem, we recommend the following:

1. The ground surface in the crawl space should be covered with six (6) mil polyethylene pressed down with 75 mm of sand to prevent drying of the soil beneath the crawl space.

2. There should be some provision made to ventilate the crawl space during the summer months.

3. The crawl space should be graded to drain to a sump with a connection to the sewer system to remove water which may inadvertently pond in the crawl space.

B. Grade Supported Floor Slabs

Floors where differential vertical movement is not critical, may be designed to be grade supported. For a grade supported floor slab, we recommend the following:

VIII EXCAVATION CONSIDERATIONS

Excavation at the site will be in the rubble and expansive clay. Conventional excavation procedures should be applicable to the materials at this site. No special problems are forseen. For stability, the minimum temporary backslope of excavations up to five (5) metres deep should be specified at two (2) horizontal to one (1) vertical. Some local sloughing can be expected in these temporary slopes.

IX. FLOOR SLAB CONSIDERATIONS

The completed facility including the building structure and asphalt pavement will cover considerable ground area. This will have the effect of providing a barrier against further evaporation of moisture from the ground that is covered.

The moisture from the lower levels will gradually migrate towards the ground surface and adjust to the change in the physical environment brought about by the construction of the proposed facility. A gradual increase in the moisture content of the subgrade clay will take place and this will cause the highly swelling clay to expand. As much as 50 to 75 mm of heaving could occur over the long term (5 to 10 years). This heaving will produce warping and cracking of the supported floor slab.

A structurally supported floor system would be most desirable insofar as overcoming the problems associated with the highly swelling clay, however this type of construction is relatively expensive. It is recommended

1. For uniform bearing conditions under the floor slab, a minimum thickness of 200 mm of granular base should be provided. Any soft or loose or organic soil below the floor area should be removed and replaced with granular material compacted to 95% of Standard Proctor as determined by ASTM D-698. Recommended gradation limits for fill materials are appended to this report, Appendix B.

2. The concrete floor slab in heavily loaded areas should have a minimum thickness of 125 mm and the 28 day concrete compressive strength should be specified to be 25 MPa made with Type 10 (Standard Portland) cement. The concrete slab in areas where only light floor loads are to be supported may have a minimum thickness of 100 mm and the 28 day concrete compressive strength may be specified to be 20 MPa with Type 10 (Standard Portland) cement. A Guide for Concrete Floor and Slab Construction Report No. ACI 302.1 R:80 is included in Appendix C.

3. The minimum reinforcement for the concrete floor slab should be specified to be 10 M reinforcement at 300 mm center to center.

4. All grade supported concrete slabs should be designed as floating slabs completely independent of the foundation walls and/or columns. Isolation joints should be provided at columns and walls to separate the grade supported floor slabs from any connection with the building or appurtenances. Control joints (sawn or premolded) should be provided at a maximum spacing of 8 to 12 metres.

5. A layer of robust polyethylene sheeting should be placed between the granular base and the concrete slab to deter the migration of moisture through the floor.

6. Attention is drawn to the presence of shale which is commonly found in concrete aggregates in Regina. This may produce "pop-outs" on concrete floors. These small holes are detrimental under tiled or on smooth finished floors. If finish is critical on the floors, the aggregate should be carefully checked to ensure its acceptability.

X PAVEMENT STRUCTURE

1. The pavement around the buildings should be designed to slope in order to provide adequate drainage of water away from the perimeter of the buildings and from the surface of paved areas. The need for adequate drainage cannot be overstressed. To ensure fast runoff, the surface of the pavement should have a slope of at least two (2) percent either to the outer perimeter of the paved areas, or to suitably located catch basins leading to underground drains. The contour of the finished pavement at all points should prevent water from standing on the surface, and surface water should not be permitted to seep back under the outer edges of the pavement. Subsurface drains should be installed in locations where subsurface water may accumulate within the pavement structure or where it is necessary to intercept water that would tend to make its way into the pavement structure.

2. After the areas are paved, a soil moisture regime will develop that is somewhat different than the condition immediately after construction. Consequently, expansion and shrinkage almost certainly will take place. These volume changes will result in differential movements in the form of transverse ridges and surface deformation which is

highly undesirable. The problem can be minimized by modifying the upper 300 mm of subgrade with two and one-half ($2\frac{1}{2}$) percent (by weight of soil) lime. The lime will markedly increase the strength and bearing capacity and decrease the water sensitivity and volume change during wet/dry cycles of the subgrade clay.

3. The subgrade should be well compacted by proofrolling using a heavy sheepsfoot roller prior to placement of granular base or fill material. Any soft, wet or organic soil should be removed and replaced with select granular material and compacted to 95 percent of Standard Proctor Density. Suggested specifications for proof-rolling are included in Appendix B. All fill materials, should be compacted to 95% of Standard Proctor Density, ASTM D-698.

4. Pavement structures have been analyzed for heavy truck loading and for light service areas such as parking lots for automobiles. These are given in Table 1.

TABLE 1

Alternate No.	Asphaltic Concrete Surface Course (mm)	Base Course Thickness (mm)	Sub-base Thickness (mm)	Lime Modified Clay Thickness (mm)
Heavy Structure				
1	75	225	150	300
Light Structure				
2	50	225	0	300

5. It is recommended that, where a pavement structure is used, in conjunction with a granular base, that paving be postponed until the area has been well trafficed if possible. Top lift pavement for heavy structures should be placed after the area has been well trafficed and has experienced one (1) winter and spring season of exposure to the ambient conditions at this location. Any surface defects should then be corrected before paving.

XI OTHER

1. Adequate drainage away from the buildings in nonpaved areas should be provided and maintained to minimize infiltration of water into the subgrade.

2. Test results indicate that the soluble sulphate content in the soil of this site ranges from 0.07 to 0.41 percent by dry soil weight. Therefore, Class 2 concrete as specified in the Guide For Use of Sulphate Resistant Cement (Appendix A) should be used for all concrete in contact with native soils. A minimum 28-day compressive strength of 20 MPa should be specified for all concrete.

3. In the event that changes are made in design, location or nature of the project, the conclusions and recommendations included in this report would not be deemend valid unless the changes in the project were reviewed by our firm. Modifications to this report would then be made if necessary.

Futhermore, it is recommended that this firm be allowed an opportunity for a general review of the final design and specificiations in order to ensure that the recommendations made in this report are properly interpreted and implemented. If this firm is not allowed the opportunity for this review we assume no responsibility for the mis-interpretation of any of the recommendations.

4 It is recommended that this geotechnical firm be retained to provide inspection services during construction of the foundations for this project. This is to observe compliance with the design concepts, specifications or recommendations and to allow design changes in the event that the subsurface conditions differ from what was anticipated.

5. The samples obtained from this site will be retained in our laboratory for 90 days following the date of this report. Should no instructions be received to the contrary, these samples will then be discarded.

We trust the above comments will permit a decision to be made regarding the method of foundation support for the proposed City of Regina Transit Operations Centre to be constructed at this site. Should you require further information, please contact this office.

Yours very truly,

GROUND ENGINEERING LTD.

Prepared By: for WALTER BUETTNER, E.I.T.

Reviewed By: PAUL KOZICKI, P. ENG.

WB/kln

Distribution: DeLCan (4)
Office (1)

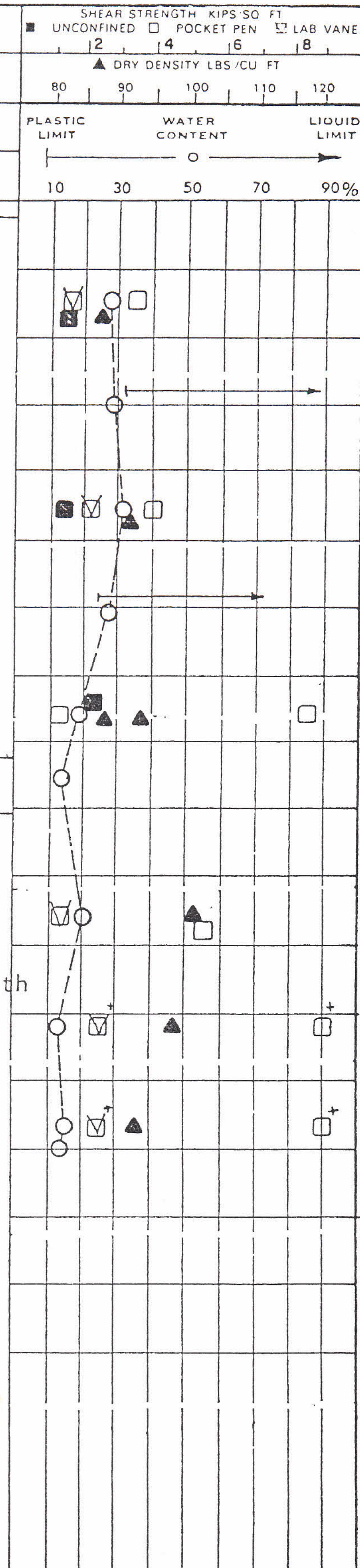
ASSOCIATION OF PROFESSIONAL
ENGINEERS OF SASKATCHEWAN
CERTIFICATE OF AUTHORIZATION NUMBER 8

TEST HOLE LOG

DATE August 12, 1980

HOLE NO. 101

SAMPLE DATA				SYMBOL	ELEV. COLLAR
WEIGHT HAMMER					ELEV. GROUND
HEIGHT DROP					CO-ORD. LOCATION
DEPTH ELEV.	NO TYPE	UNIF PI	% SO.		DESCRIPTION OF MATERIAL
				200mm	FILL - granular, sandy - old roadway
1.5M	A458 Sy				
3.0M	A459 Bg	CH 55			CLAY - silty, highly plastic - calcareous - oxidized - olive brown (2.5Y 4/4) - stratified - moist, firm - soft at 6.1 M - glauber salts - Iron stains
4.6M	A460 Sy				
6.1M	A461 Bg	CH 47			
7.6M	A462 Sy				
8.5M	A463 SPT			8.2M N=17 9.0M	TILL - clay - calcareous - oxidized - dark grayish brown (2.5Y 4/2) - massive - moist, firm - Iron stains
10.6M	A464 Sy				SILT - clayey, becoming more sandy with depth - calcareous - unoxidized - olive (5Y 4/3) - stratified - damp stiff - some sloughing
12.2M	A465 Sy				
13.7M	A466 Sy				
14.0M	A467 SPT			N=10 14.3M E.O.H.	



NOTES:

1. Drilled using a Brat Model B24 digger equipped with a 230 mm diameter auger.
2. No water seepage
3. Soil contaminated with hydro-carbons to a depth of 9.0 M

GROUND ENGINEERING LTD.
GEOTECHNICAL ENGINEERS/Soil Mechanics & Foundations

PROJECT Proposed City of Regina Transit Operations Centre

LOCATION Regina, Saskatchewan

TEST HOLE LOG

DATE August 20, 1980

HOLE NO. 102

SAMPLE DATA	
WEIGHT HAMMER	
HEIGHT DROP	
ELEV. COLLAR	
ELEV. GROUND	
CO-ORD. LOCATION	

SHEAR STRENGTH KIPS/SQ FT: ■ UNCONFINED □ POCKET PEN ☒ LAB VANE — 2, 4, 6, 8

▲ DRY DENSITY LBS/CU FT — 80, 90, 100, 110, 120

PLASTIC LIMIT — WATER CONTENT O — LIQUID LIMIT — 10, 30, 50, 70, 90%

DEPTH ELEV.	NO TYPE	UNIF PI	% SO4
3.0M	A548 Bg		
6.1M	A549 Bg		
8.4M	A550 Bg		
12.2M	A551 Bg		
15.8M	A552 Bg		
16.8M	A553 Bg		
17.4M	A554 Sy		
18.3M	A555 Bg		

DESCRIPTION OF MATERIAL

76mm

TOPSOIL - organic

CLAY - silty, highly plastic
- calcareous
- oxidized
- olive brown (2.5Y 4/4)
- laminated, fissured
- moist, stiff
- glauber salts

7.0M

SILT - clayey, sandy
- calcareous
- oxidized
- olive brown (2.5Y 4/4) becoming unoxidized, dark gray (5Y4/1) below 12.2M
- stratified
- moist becoming wet below 15.8 M
- medium dense
- iron, manganese stains

16.7M

TILL - calcareous
- unoxidized
- dark gray (5Y 4/1)
- massive
- moist, stiff to very stiff
- dense
- water bearing sand and gravel layer 17.5M to 18.0M

17.5M

18.0M

18.3M E.O.H.

128.4

NOTES:
1. Drilled using a Hughes Model LDH 80 digger equipped with a 610 mm diameter auger.
2. Some water seepage at 15.8M
3. Test hole sloughed in to a depth of 17.8M
4. Soils contaminated with hydrocarbons to a depth of 6.1M

GROUND ENGINEERING LTD.

GEOTECHNICAL ENGINEERS/Soil Mechanics & Foundations

PROJECT Proposed City of Regina Transit Operations Centre

LOCATION Regina Saskatchewan

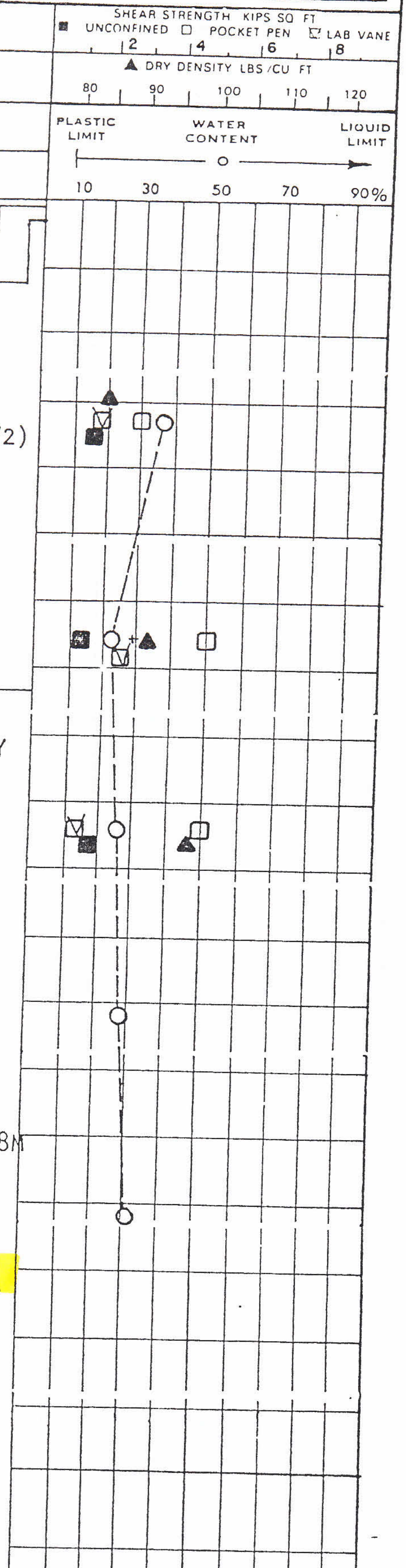
TEST HOLE LOG

DATE August 29, 1980

HOLE NO. 103

SAMPLE DATA				SYMBOL	
WEIGHT HAMMER					ELEV. COLLAR
HEIGHT DROP					ELEV. GROUND
					CO-ORD. LOCATION
DEPTH / ELEV.	NO / TYPE	UNIF / PI	% / SO.		DESCRIPTION OF MATERIAL
				76mm / 220mm	Asphalt surface Granular base
3.3M	A543 Sy				CLAY - silty, highly plastic - calcareous - oxidized - gray (2.5Y 5/0) becoming dark grayish brown (2.5Y4/2) at 6.7 M - laminated - moist, firm to stiff
6.7M	A544 Sy			7.2M	SILT - sandy, clayey - calcareous - oxidized, olive brown (2.5Y 4/4) becoming unoxidized, gray (5Y 5/1), below 12.2M - stratified - moist becoming wet below 11.6M - water bearing gravel layer at 15.8 M to 16.2M
9.4M	A545 Sy				
12.2M	A546 Bg				
15.2M	A547 Bg			15.8M 16.2M 17.1M E.O.H.	

NOTES:
1. Drilled using a Hughes Model LDH 80 digger equipped with a 610 mm diameter auguer.
2. Water seepage encountered at 15.8M
3. Sloughing encountered at 15.8M
4. Not able to recover soil samples below a depth of 16.2M
5. Soil contaminated with hydro-carbons to a depth of 7.2 metres.



GROUND ENGINEERING LTD.
GEOTECHNICAL ENGINEERS/Soil Mechanics & Foundations

PROJECT Proposed City of Regina Transit Operations Centre

LOCATION Regina, Saskatchewan

GE-620

TEST HOLE LOG

DATE August 11, 1980

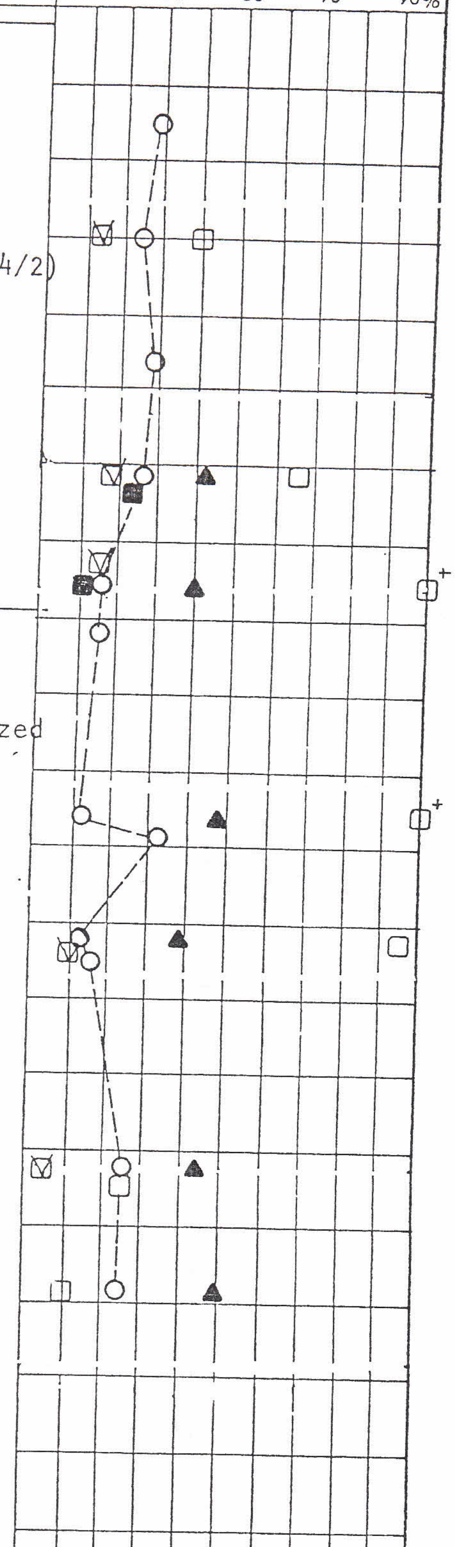
HOLE NO. 107

SAMPLE DATA	SYMBOL	ELEV. COLLAR
WEIGHT HAMMER		ELEV. GROUND
HEIGHT DROP		CO-ORD. LOCATION

SHEAR STRENGTH KIPS SQ FT: ■ UNCONFINED □ POCKET PEN ☒ LAB VANE (2, 4, 6, 8)

▲ DRY DENSITY LBS/CU FT (80, 90, 100, 110, 120)

PLASTIC LIMIT — WATER CONTENT o — LIQUID LIMIT (10, 30, 50, 70, 90%)



DEPTH ELEV.	NO TYPE	UNIF PI	% SO.	DESCRIPTION OF MATERIAL
				200mm — TOPSOIL - organic
1.5 M	A446 Bg			CLAY - silty, highly plastic - calcareous - oxidized - dark grayish brown (2.5Y 4/2) - stratified - moist, stiff - Iron, manganese stains - selenite crystals - glauber salts
3.0 M	A447 Sy			
4.6 M	A448 Bg			
6.1 M	A449 Sy			
7.6 M	A450 Sy			
8.2 M	A451 Sy			7.9 M N=16 — SILT - clayey, fine sand - calcareous - oxidized, light olive brown (2.5Y 5/4) becoming unoxidized gray (5Y 5/1) below 10.6M - stratified - moist, very stiff to 12.5M - moist, becoming wet below 15.2 M - sand seam at 18.2 M - some sloughing
10.6 M	A452			
10.9 M	Sy A453 SPT			N=9
12.2 M	A454			
12.5 M	Sy A455 SPT			N=16
15.2 M	A456 Sy			
16.8 M	A457 Sy			
				18.2 M E.O.H.

NOTES:

1. Drilled using a Brat Model B24 digger equipped with a 230 mm diameter auger.
2. Water seepage noted at 15.2 M
3. Soil contaminated with hydro-carbons to a depth of 7.9 M

GROUND ENGINEERING LTD.

GEOTECHNICAL ENGINEERS/Soil Mechanics & Foundations

PROJECT Proposed City of Regina Transit Operations Centre

LOCATION Regina, Saskatchewan