APPENDIX B

GEOTECHNICAL SOILS INFORMATION

M. M. DILLON LTD.,

REPORT ON THE GEOTECHNICAL INVESTIGATION OF THE PORTAGE AVENUE OVERPASS REHABILITATION PROJECT AT OMANDS CREEK

PROVINCE OF MANITOBA
A. D. GOULD
REGISTERED ENGINEER

MAY 1992

A. DEAN GOULD AND ASSOCIATE
GEOTECHNICAL CONSULTANTS

GEOTECHNICAL REPORT

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1.0 Introduction

This report summarizes the results of a geotechnical investigation undertaken by A. Dean Gould and Associates on behalf of the engineering Consultant M. M. Dillon Ltd., for the proposed Portage Avenue widening and pedestrian underpasses at Omands Creek, the CPR railway overpass and Empress Street overpass, and presents foundation and slope treatment proposals for design consideration.

The site encompasses three structures, namely the CPR Bridge of the La Riviere subdivision, the Empress Street Overpass and the Portage Avenue culvert over Omand's Creek. The Omands Creek waterway is classified a provincial waterway. The project is intended to add two additional east - west lanes to Portage Avenue and will require widening of the Omand's Creek culvert deck slab and the installation of two pedestrian underpass walkways through the CPR abutments.

The purpose of the geotechnical investigation was (a) to determine the backfill materials of the CPR Bridge abutments and adjacent embankments to provide parameters for design of the pedestrian underpass tunnels, and (b) to evaluate the impact upon existing and long term Omand's creek riverbank stability of the proposed widening of the Portage Avenue culvert and associated erosion protection.

This report is intended to address the engineering requirement of the Rivers and Streams Authority No.1 under whose authority and control zone the project is located, and present the impact of the project design upon the Omand's Creek provincial waterway and riverbanks.

2.0 Field Investigation

A subsurface investigation consisting of nine, 125 mm augured test holes was undertaken on March 9-10, 1992. The test holes, located as shown on the attached plan SK-1 in Appendix A, were drilled by Paddock Drilling Ltd., through the use of 5 -inch flight auger using a track mounted drilling rig.

The test holes were logged and sampled by Mr. R. Deighton P.Eng of A. Dean Gould and Associate, and the logs are shown in Appendix A of this report. All test holes were backfilled upon completion with excavated material.

Water level inflow to each hole was recorded, however silt zones encountered in TH-1, TH-6 and TH-9 were found unstable, causing caving and thus limited the final water levels to be obtained.
Photographs of the site condition, drilling operation, creek water flow and topography were obtained but not included in this report.

3.0 Laboratory Testing

Soil samples were subjected to moisture content determination, Atterberg limit classification testing, grain size analysis, unconfined compression strength testing and Direct Shear test. From this testing, parameters for design of the pedestrian walkway structure, riverbank stability analysis and cofferdam design were obtained. The data is presented in Appendix A of this report.

4.0 Subsurface Conditions

4.1 Soil Profile

The soil profile was found to consist of brown and grey clays overlying till. The basement soil is a dense, consolidated till found in TH-8 and TH-5 at elevations of 221.57 and 220.03 respectively. In test hole 1A, located at the Clifton Street storm sewer outfall, downstream of the site approximately 150m, the elevation of the dense till was found to be 221.9. Standard penetration test values obtained in TH-5 at the elevation 219.9 produced allowable bearing capacities in the order of 670 kPa.
Overlying the dense till is a strata of moist, soft clay till transition (soft till). This mixture has a moisture content which varies from 27.9% to 11.6% in TH-5. The dense till in comparison has a moisture content in the 8.6% range.

Grey clay strata overlying the tills appears to contain a large quantity of till materials such as stone, silt and sand. One sample at elevation 223.5 was subjected to Direct shear testing. This sample contained significant amounts of silt, sand and stone well embedded into a clay matrix. The results of that test are shown appended. The grey clay was found to underlay the entire site and was identified in TH-1,TH-6,TH-5,TH-9,TH-7 andTH-8.

Brown silty clay comprises surface soils throughout the riverbank and the area beyond the railway and road embankment. Within the brown clays were found significant silt lensing. These silts were predominate in TH-1,TH-6 and TH-9 located on both south and north CPR embankment at elevation 231.55 - 231.40, 231.8 - 230.76 and 232.22 - 231.0 respectively. In all locations, the silts are at high moisture content and in the south embankment were unstable and the source of water inflow.

Railway embankment at the abutment area consists of well graded granular material ranging in size from boulders to clay. Boulders were encountered in both abutment areas preventing penetration by drilling equipment utilizing flight augers. TH-4 and TH-2 wherein penetration was accomplished were sampled and those samples subjected to grain size analysis. Grading charts are shown appended. Material in all cases was well compacted, and standard penetration tests performed, produced N values which averaged 16 blows per foot.

At the south abutment, a distinct soil change occurs from TH-1 located in the embankment 3m south of the abutment to TH-2 located within the abutment. Soils of the south embankment from the 2m depth (elev. 233.4) are native soils consisting of silts and clays. Soils of the Abutment are imported boulder gravels extending from the rail grade to at least elevation 232.7 (refusal) and probably to the abutment base at elevation 229.9.

At the north abutment, boulder gravels extend north of the abutment limits into the embankment. TH-4 located 4m north of the abutment encountered boulder gravels to elevation 232.4

Appended to this report, SK-2, SK-3 profiles of the north and south embankments show the soil logs, elevations of soil stratigraphy and Omand's creek water levels as of March 12, 1992.

4.2 Groundwater Conditions

Inflow to the test holes was noted on the south embankment in TH-1 and TH-6 through a silt lense located at approximately elevation 231.5 and 230.91 respectively. The source of this water is questionable since it occurs well above Portage avenue and the adjacent Omands Creek. This high elevation suggests a perched water level produced through water main leakage or bridge drainage. Area ground water levels are considerably lower, controlled by the major drainage channels of Omands Creek and by storm sewer systems.

A standpipe installed during the UMA investigation of October 1990 indicated piezometric levels at elevation 221.7m (re:EMP-1) or 1.3m below the glacial till surface. This low phreatic surface will have little impact upon slope stability and the perched water , emenating from the silt zones has been applied as having greater influence.

5.0 Historical Review

The geotechnical history of the site is complex. Early accounts suggest that considerable rerouting of Omand's Creek has occurred. The original location passed through Polo Park upstream of the site, instead of the current location parallel to Empress Street and the CPR line. It is unknown if relocation work was extended downstream of the CPR bridge and across Portage Avenue. Similar instability of riverbank is evident upstream of the culvert site and along Empress Street suggesting some relocation through this area may have occurred.

The Omand's Park Area to the south of Portage Avenue has experienced historic instability, and has been stabilized through resloping and the addition of internal drainage features. Flows of Omand's Creek fluctuate widely with spring runoff and heavy precipitation periods. Erosion of the riverbank does not appear to present a serious problem as evidenced by the vegetation growing near the waters edge. The steep creek gradient would suggest high velocities are possible and consequently the vegetation must be providing high erosion resistance.

The CPR bridge and Omand's Creek culvert appear stable and do not appear to have experienced significant movement. These structures are reportedly founded on driven timber pilings probably extending to the dense till.

6.0 Discussion and Recommendations

6.1 General

The proposed project will involve the construction of pedestrian underpass walkways through the CPR abutments and substructure plus substructure works for the culvert deck widening. This section will discuss the results of slope stability analysis and provide parameters for the design of the walkway structures and substructures. In addition, it deals with temporary works such as excavation slopes, cofferdams and provides guidelines for the design of erosion control works of the creek bed at the Portage Avenue crossing.

6.2 Pedestrian Underpass Walkway

6.2.1 Design Considerations

The proposed pedestrian underpass walkway is to be constructed through the existing CPR abutments on both sides of Portage Avenue. The material through which excavation must be made will be a compact, well graded granular which was found to exist within the confines of the concrete abutment. This material contains approximately 12% silt and clay binder which is sufficient to provide a high degree of strength for excavation slope stability and still allow drainage.

From direct correlation to the effective strength parameter (phi) the backfill coefficient for wall design should be 0.31

The existence of water bearing silts at elevations 232 - 230 in the south embankment (TH-1) may present problems in excavation should an alternative be considered extending excavation beyond the abutment limits. Provision of free draining backfill should be made and adequate pipe drainage leading to storm sewer facilities on both north and south walkway outer walls. This drainage could be developed through;

(a) Well graded gravel and a granular filter toe drain.

or alternatively

(b) The use of filter fabric and a prefabricated drainage composite material such as Miradrain placed between native backfill or low quality gravel backfill and the wall.

In both cases a pipe drain, protected against freezing should be installed leading to a storm water system or to Omand's Creek through a well designed flexible joint, drainage system.

Dense glacial till has been identified from all test holes at shallow depth throughout the site. Some variability in elevation has been noted in test holes from this investigation and those of previous work suggesting the dense till surface is irregular but does not exhibit an identifiable slope.

Soft glacial till, overlying dense till is relatively variable in thickness, strength is low and this material can be expected to flow into large diameter augered pile excavations. Driven pilings therefore are recommended at this site for deep foundations to support the retaining wall design. Recommended values for end bearing driven pilings are;

Timber Pilings

Capacity - governed by structural strength and pile diameters
Normal Capacity = 100 kN (10-12 tons)
Estimated Elevation of end bearing 221.0-219.5 m
Final set should be 6 blows/ft with a hammer having a 10,000 ft-lb driving energy.

Concrete Precast Units

Pile Diameter	Design End Bearing Capacity	Final Set * blows/in.
300mm	450 kN (50 tons)	6
350mm	625 kN (70 tons)	10
400mm	800 kN (90 tons)	14

* Final set with a pile driver have an energy rating of 30,000 ft-lbs

** Preboring for pilings is recommended in the vicinity of services and existing structures to prevent soil displacement and damage to service.

6.2.2 Excavation Slope Stability

Excavation slopes for the placement of the pedestrian walkway will be governed primarily by the granular embankment soils currently in place. The lower limit of the granular re: TH-1 and TH-9 is 233.7 and 232.5 at the south and north embankments respectively whereas the proposed excavation level of the base of the walkway is 229.9. The total 6m excavation could be performed in a number of ways some of which are:

(a) Open Cut

Side slopes required to support railway loadings are governed by the strength of the brown silty foundation clays with both the granular fill and surface loading contributing to instability. Since rail traffic involves public safety and potential for damage claim through out of service delay, a much higher Factor of Safety against sliding will be required than ordinarily applied in temporary construction slopes. The required "Safe" slope (F.S.= 1.25) is computed as 3.5H:1V up to slope heights of 3 metres and 4:1 for slope heights to 6 metres in native clay materials. Within the abutment area where a boulder fill was found to exist, excavation slopes can be considerably steeper at 2.0H:1V. The character of probable failures in clay vs granular is important since the clay failure may be deep seated passing through the base and involving a large volume of material, whereas the failure in granular will be shallow surface sloughing.

Since maintenance of rail traffic will require extensive bridging using an open cut alternative, costs may prove prohibitive.

(b) Shored Excavations

Excavation with the use of shoring is a practical method of achieving depth of cut providing the shoring can be adequately supported. Three methods of support can be used;

(i) Braced shoring utilizing opposite trench walls and the passive soil pressure to resist active pressure acting upon the opposite wall.

(ii) An anchored shoring which is supported through soil anchorage systems. Soil anchor design capacity can be based upon the strength of the brown clays of 444 psf or 21.5 kPa (Factor of Safety = 2). Soil anchors must be installed a minimum distance of 6 metres from the face of the shoring (see SK-7) in order to be beyond the influence limit of surface loading.

(iii) The use of steel sheet piling driven below the footing level may be considered for areas beyond the concrete abutments. Depth of penetration will depend upon the piling section and structural capability. The Rankin pressure coefficients for design are; Passive = 2.68 Active = 0.37 with drainage provided.

6.2.3 Final Grade Slopes

Slopes from Portage Avenue curb to the pedestrian walkway for space reasons will be maximized, consequently material selection and placement will be highly important. Options available should these slopes be constructed of local materials are presented in the Table 1;

TABLE 1

FINAL GRADE SLOPES FOR VARIOUS EMBANKMENT MATERIALS

Slope Height metres	Material Type	Slope	FACTOR OF SAFETY
1	Limestone Gravel Clay phi=26°	1H : 1V 2.0H : 1V 3.0H : 1V	1.34 SL 1.36 SL 1.65 B
2	Limestone Gravel Clay phi=26°	1.5H : 1V 2.0H : 1V 4.0H : 1V 3.5H : 1V	1.30 SL 1.36 SL 1.48 B 1.36 B
3	Limestone Gravel Clay phi=26°	1.5H : 1V 2.0H : 1V 4.0H : 1V	1.30 SL 1.36 SL 1.33 B
4	Limestone Gravel Clay phi=26°	1.5H : 1V 2.0H : 1V 4.0H : 1V	1.30 SL 1.36 SL 1.23 B
5	Limestone Gravel Clay phi=26°	1.5H : 1V 2.0H : 1V 4.0H : 1V	1.30 SL 1.36 SL 1.17 B
6	Limestone Gravel Clay phi=26°	1.5H : 1V 2.0H : 1V 4.0H : 1V	1.30 SL 1.36 SL 1.12 B

Note; B indicates a probable Base Failure condition
SL indicates a probable Slope Failure condition

6.3 Sub Structures for Deck Slab Widening

6.3.1 Design Considerations

The widening of Portage Avenue will require the lateral extension of the deck slab box culvert over Omand's Creek. The construction of the base extension will impact upon existing embankment slopes upstream and downstream of the culvert and upon the design of cofferdams and erosion control at stream level.

6.3.2 Omand's Creek Riverbank Stability

To obtain design parameters for culvert rehabilitation works, an analysis was made of the existing condition of stability of the Omand's Creek riverbanks. Based upon the 1992 subsurface investigation, peak and residual direct shear strength testing and the work of other investigators, the strength parameters were established and tested through computer analysis of existing riverbank topography. Residual soil strength parameters are shown in Table 2.

TABLE 2

SOIL STRENGTH PARAMETERS FOR EXISTING RIVERBANK

Material	Unit Weight kg/cu.m.	Cohesion kPa	Angle of Internal Friction
Brown Clay	17.0	0	24 degrees
Grey Clay	16.93	2.0	14 (test)
Soft Till	17.67	0	26.1 (test)
Dense Till	20.0	0	32 (SPT)

Applying the parameters of Table 2 in the computer program G-Slope which utilizes the Bishop Modified method of analysis a near unity (1.0) Factor of Safety was determined for the existing slopes of Sections 1 & 2 shown on SK-1 Location Plan and in the appended 2 dimensional slope profiles with probable failure surfaces. The computer analysis enabled a rapid testing of the slope for the position of the phreatic surface and local failure conditions. It was found that;

- The slope downstream of the bridge is sensitive to the position of the phreatic surface and thus drainage incorporated into approach embankments provides a benefit to slope stability.

- The position of the most probable deep seated failure surface which would present the greater potential damage to the structure and adjacent riverbank area is controlled by the elevation of the till and the strength parameter of the grey clay. Drainage control in the form of drainage trenches installed into the existing slopes upstream and downstream of the structure should effectively produce a level of stability wherein the computed Factor of Safety against sliding will be 1.4. The impact of drainage control on the Factor of Safety against sliding is shown on SK-5 appended. The proposed arrangement, shown on SK-6 includes armour rip rap for the drainage trench near the Creek bed and to maximum flood level of elevation 229 (100 yr. event). This rip rap will form an upstream and downstream erosion control wall , connected to the channel protection.

Other alternatives such as provision of rock toes have been explored and found to produce minimum benefits to sliding stability other than that of toe erosion control. Existing slopes are controlled by property limits, installed water, sewer, gas services and structures, consequently little potential for improving stability through flattening slopes exists. Active stabilization measures which improve the soil strength through drainage are therefore recommended in lieu of sloping.

A summary of the computed miminum Factors of Safety against sliding of both circular and non circular sliding surfaces for cross- sections 1 and 2 shown on Plan SK-1 and on cross sections SK- 2,3,4 and 5 are as follows;

Section 1	Existing (Upstream)	1.03
	With Rock Toe	1.14
	With Drainage to 230	1.14
	With Drainage to 229	1.24
	With Drainage to 228	1.37
Section 2	Existing (Downstream)	0.99
	With Drainage to 228	1.37

o Local toe failures, although a concern as an erosion initiated slide, are less likely if sufficient creek channel protection is provided than a deep seated failure involving a large volume of material, possibly impacting upon the bridge structure. These toe failures can and often do present a hazard to a larger riverbank failure should they be allowed to progress. An analysis has been performed to investigate the risk associated with progressive failure. From this, plus the hydraulic stream analysis summarized below, the potential for erosion is high and erosion protection as shown on the M. M. Dillon detail drawings, is an intergral part of this project.

o The increase in computed stability evident with internal drainage is considered essential for the Omands creek abutment design and is shown on the appended detail SK-6 incorporated into existing gas, water and sewer line positions. It is of paramount importance that these service lines be located accurately prior to the installation of the drains to avoid damage and disruption of service.

6.3.3 Substructure Foundations

The occurrence of competent glacial till at a shallow depth below the Omands Creek water level has been established through this investigation and confirmed by others as being a general condition. The capacity of the till as determined through the Standard Penetration test exceeds 14 ksf or 670 kPa.
The surface of the consolidated till appears to vary approximately 1.5m in elevation as from 221.6 to 220.0 indicated from test holes 8 and 5 located upstream and downstream of the existing structure. This till slope is confirmed by UMA investigation of 1990. Further downstream, the dense till again rises to elevation 222 according to 1979 data for the Clifton Storm sewer project. The sloping dense till elevation will present a minor problem for any additional pilings that are requires to support the bridge extension.

The soft glacial till overlying dense till is relatively unstable and can be expected to slough when exposed in large diameter augured caisson units. Driven pilings, either precast concrete, timber or steel are therefore recommended. Design should be based upon the following;

Timber Pilings

Capacity - governed by structural strength and pile diameters
Normal Capacity = 100 kN (10-12 tons)
Estimated Elevation of end bearing 221.0-219.5 m
Final set should be 6 blows/ft with a hammer having a 10,000 ft-lb driving energy.

Concrete Precast Units

Pile Diameter	Design End Bearing Capacity	Final Set * blows/in.
300mm	450 kN (50 tons)	6
350mm	625 kN (70 tons)	10
400mm	800 kN (90 tons)	14

* Final set with a pile driver have an energy rating of 30,000 ft-lbs

** Preboring for pilings is recommended in the vicinity of services and existing structures to prevent soil displacement and damage to service.

6.3.4. Excavation Slopes

Excavation slopes for the substructure construction works will be short term. For temporary slopes, the selection can utilize Factors of Safety against sliding of 1.2 which is lower than recommended for permanent slopes.

Should steeper slopes be required for reasons of space, shoring, braced internally or with soil anchors may be a reasonable solution. The active soil pressure coefficient for the clays, based upon a phi parameter for the brown clays (26 degrees) is 0.39 for a drained backfill.

6.3.5 Final Grade Slopes

Final graded slopes will be selected from many factors including: slope stability, maintenance operations, sodding stability, erosion and hydraulic stream requirements. Table 1 presents the sensitivity of reshaped slopes to stability and soil strength. Long term slopes should have Factors of Safety in the order of 1.4 where public safety is of little concern and soil movements would not endanger structure.

6.4 Cofferdam Design

To install the river diversion, necessary to enable the substructure construction work to proceed, a form of cofferdam or diversion works will be required to pass Omand's Creek flow during the construction season. Should construction proceed during winter, the flow requirement will be minimal, however hydrology study by M. M. Dillon indicates that spring and summer flow may produce water levels of 227.7 (10 year flood frequency) at flow velocities of 6 - 7 fps at low Assiniboine river levels.

Various cofferdam approaches have been applied successfully and would have application for this work. Some of these are:

(a) Steel Sheet cofferdam driven through the structure embedded into the clays and soft till above the 220 elevation. Sufficient clay exists below the creek level (Elevation 226.5) to provide adequate embedment to resist hydraulic heads of 1.2 m. This type of cofferdam would occupy the least space and could serve as formwork for the base extension.

(b) Embankments placed parrellel to the stream flow, constructed of clay and wrapping into the riverbanks on each end would restrict flow but could be used to minimize cost. Side slopes as steep as 1.5:1 could be used for short term purposes against a head of 1.5m. Erosion could be anticipated under high flow velocity, consequently a high risk factor applies to this alternative.

(c) A combination of the two above alternatives may be feasible utilizing soil embankments at each end with the steel sheet piling through the bridge within confined space.

(d) Other alternatives which may warrant consideration are open flumes or pipes constructed of culvert material into which flow is directed by means of soil embankments.

In any cofferdam where temporary flow restriction is made, erosion control at the inlet and outlet must be provided in order to prevent riverbank instability. Placement of limestone rip rap over the stream bed area is advisable prior to installation of the cofferdam.

6.5 Hydraulic Considerations by Mr. L.A. Buhr P.Eng

Assiniboine river flood levels have a significant impact on Omand's Creek levels at the Portage Avenue site. Data compiled on the Assiniboine river flood stages are as follows;

Flood Frequency	Water Elevation
160 year	229.5 to 230.3
100 year	228.9 to 229.6
10 year	227.5 to 227.7
5 year	226.7 to 227.1

The roadway elevation on Portage avenue is 231.0 +\- and the underside of the deck is 229.0 +\-. Therefore the crossing is relatively secure from flood staging.

The existing culvert barrel is approximately 11.6m wide and 4.6m high from crown to fill for a total opening of approximately 53 sq.m. Flow capacity of this crossing may be influenced by backwater from the Assiniboine river, however, the hydraulic opening is more than adequate to handle normal design flows from the 80 + sq. km drainage area.

Only a limited amount of flow data is available for Omand's creek. Analysis of this data in conjunction with the Sturgeon Creek flow parameters indicates that the 50 year design flow is 16 cu.m./sec. (565 cfs) and the 100 year design flow is 20 cu.m./sec (700 cfs). Under higher Assiniboine river levels, these flows can pass under Portage Avenue with velocities of 0.6 to 0.9 m/s (2-3 ft/sec). However, under more normal low Assiniboine river levels, velocities in the range of 1.9 - 2.2 m/s (6-7 ft/sec) could result.

Considerable downward erosion of the Omand's creek channel bottom is likely to occur at the above velocities. Site conditions do reflect this erosion e.g. the channel bottom within the Portage Avenue crossing has been lowered significantly in the last two decades.

Continued erosion could expose the structure to frost heave and create stability problems. Therefore it is essential that a form of erosion protection be incorporated into the current structural modifications. Suitable erosion protection could be grouted rip rap or concrete slab. Erosion protection is required for at least 20m of the downstream channel complete with a shallow rip rap termination berm. Upstream and/or downstream wiers could be incorporated within the structure confines to disipate hydraulic energy and to limit future erosion.

Consideration has been given to creating a pedestrian underpass crossing of Portage Avenue through the culvert barrel. The pedestrian underpass would link a proposed linear parkway along Omand's creek. Such a walkway can be built on one or both sides of the creek. Head room requirements suggest a walkway elevation of 227.300. That elevation is above the 5 year flood level on the Assiniboine river and should normally be available for pedestrian traffic during the spring and summer months.

Installation of a 2.4m wide walkway on both sides of the creek would leave a vertically walled channel approximately 6.4m wide to pass spring and summer runoff. This channel would be able to pass a 10 year frequency Omand's creek flood event with a water level at the walkway of 227.300. A 50 year frequency flood event would raise the water level to an elevation between 228.000 and 228.300. In both cases velocities would exceed 2.2

m/sec (7 ft/sec). Therefore it is considered essential to control the hydraulic energy by means of wiers and erosion protection to prevent scour and structure/ riverbank instability if the walkways are constructed.

6.6 Creekbed Erosion Control

Based upon the above hydraulic considerations and projected flow velocities of 6- 7 fps, which are well above the allowable limits of 2.5 - 3 fps normally considered as erosion limits for a weathered brown clay, protection of the streambed is recommended. Brown clay not subjected to desiccation through freeze thaw cycles would erode at higher velocities, however within the stream bed of Omand's Creek seasonal freeze depths are in the order of 1 metre consequently the erosion potential is high.

Streambed protection should be provided through and both above and below the bridge to prevent scour and slope instability. The protection proposed is a 300mm layer of 200 mm nominal size, limestone rip rap placed upon a 150 mm thick layer of gravel transition material or alternatively geofabric. Geofabric supplied in non woven material has a tendency to mud pack thus limit through flow. This limitation may produce hydraulic pressure on the lower surface under high velocity hydraulic flows and dislodge the stone armour. Gravel transition material is recommended as it will develop layer filter grading which, with time, is self healing and may offer improved long term performance.

Respectfully Submitted,

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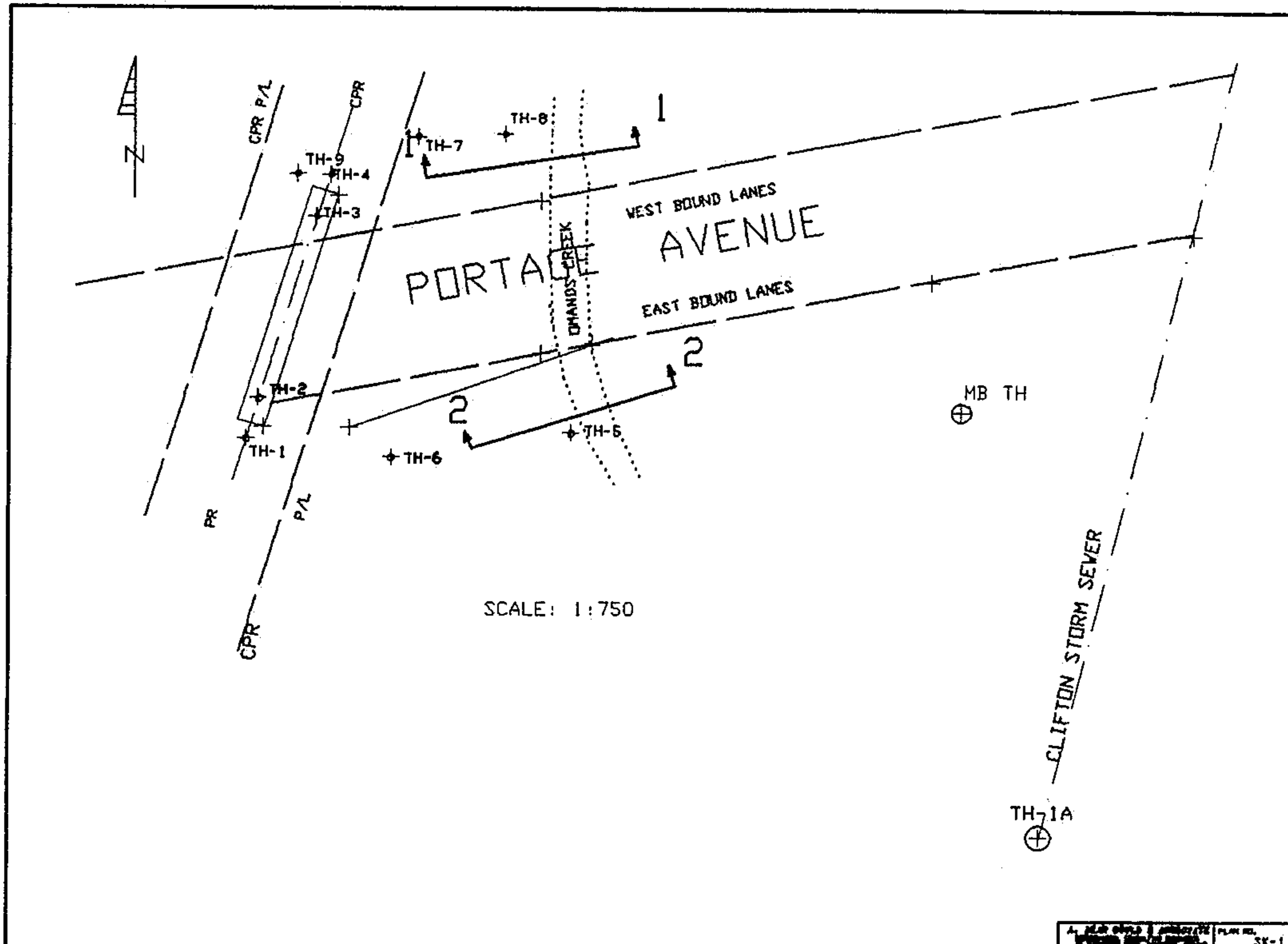
APPENDIX A

PLANS , PROFILES AND CROSS SECTIONS



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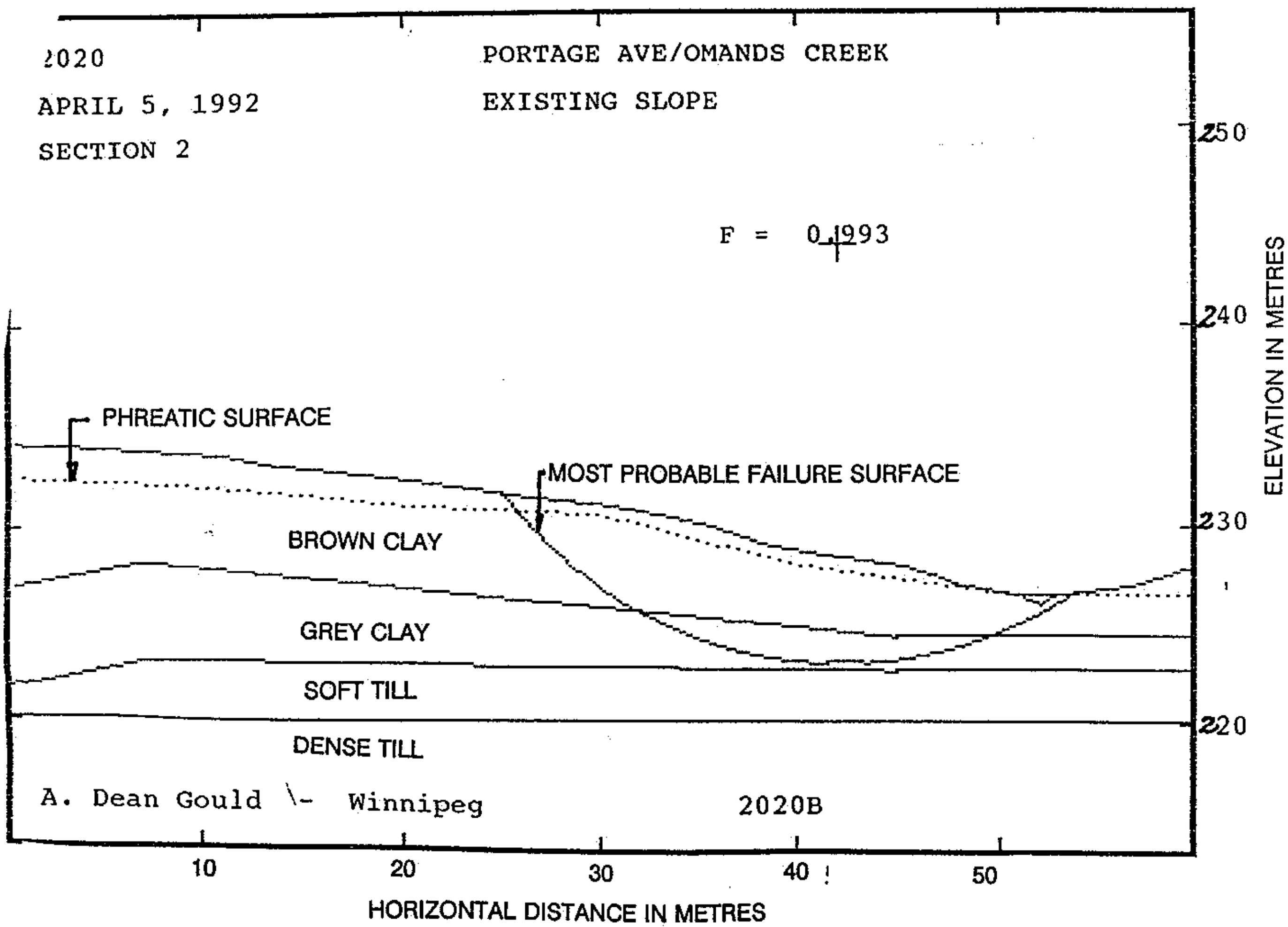
PORTAGE AVENUE REHABILITATION PROJECT AT DHAND'S CREEK

LOCATION PLAN OF TEST HOLES

PLAN NO. SK-1

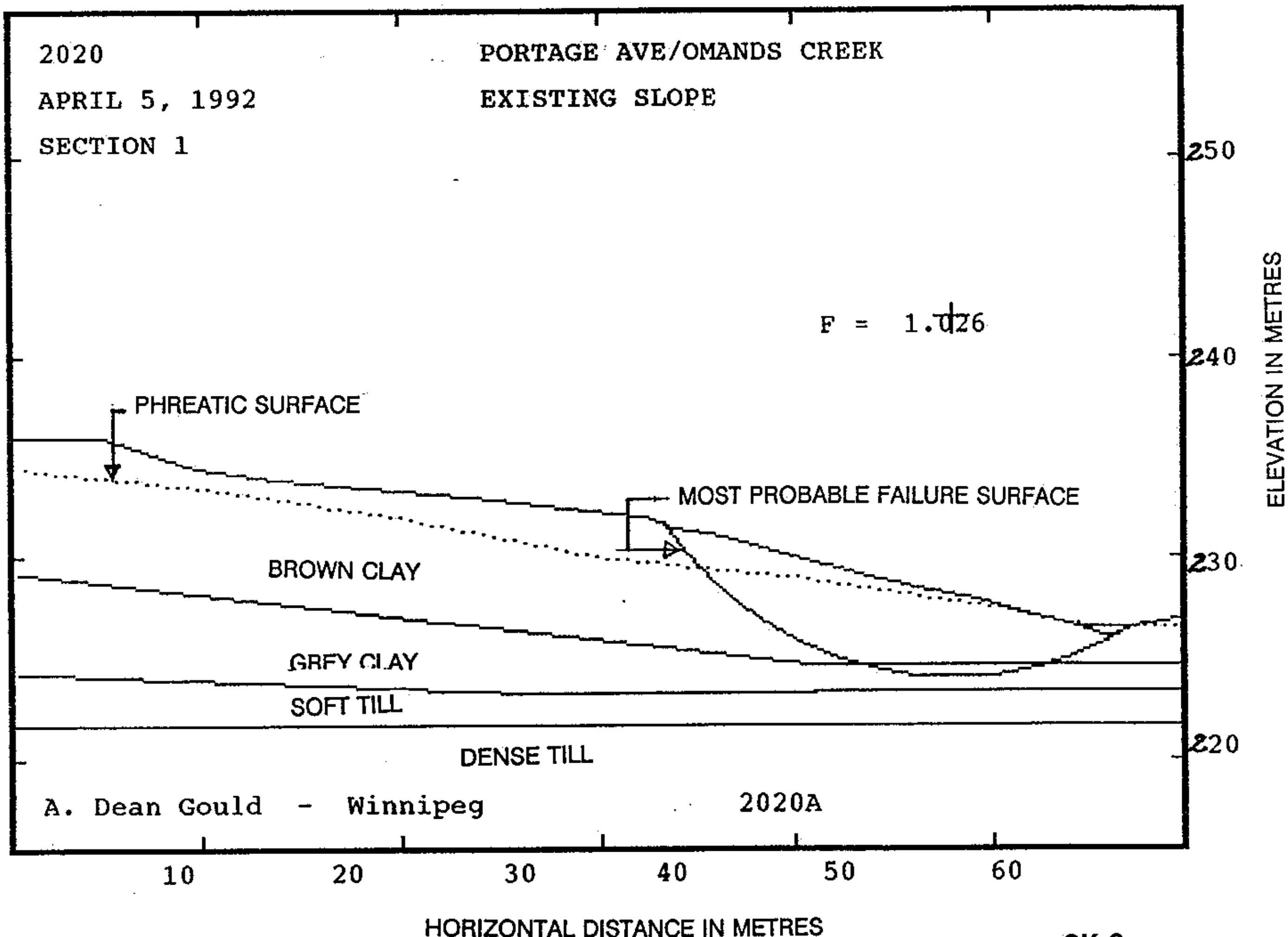
SK-1

	Unit Wt	Cohesion	Friction Angle	Piezo Surface	Ru Value
EEK WATER	9.8	0	0	0	0
₍OWN CLAY	17	0	24	1	0
REY CLAY	16.93	2	14	1	0
;OFT TILL	17.67	0	26.1	1	0
DENSE TILL	20	0	40	0	0



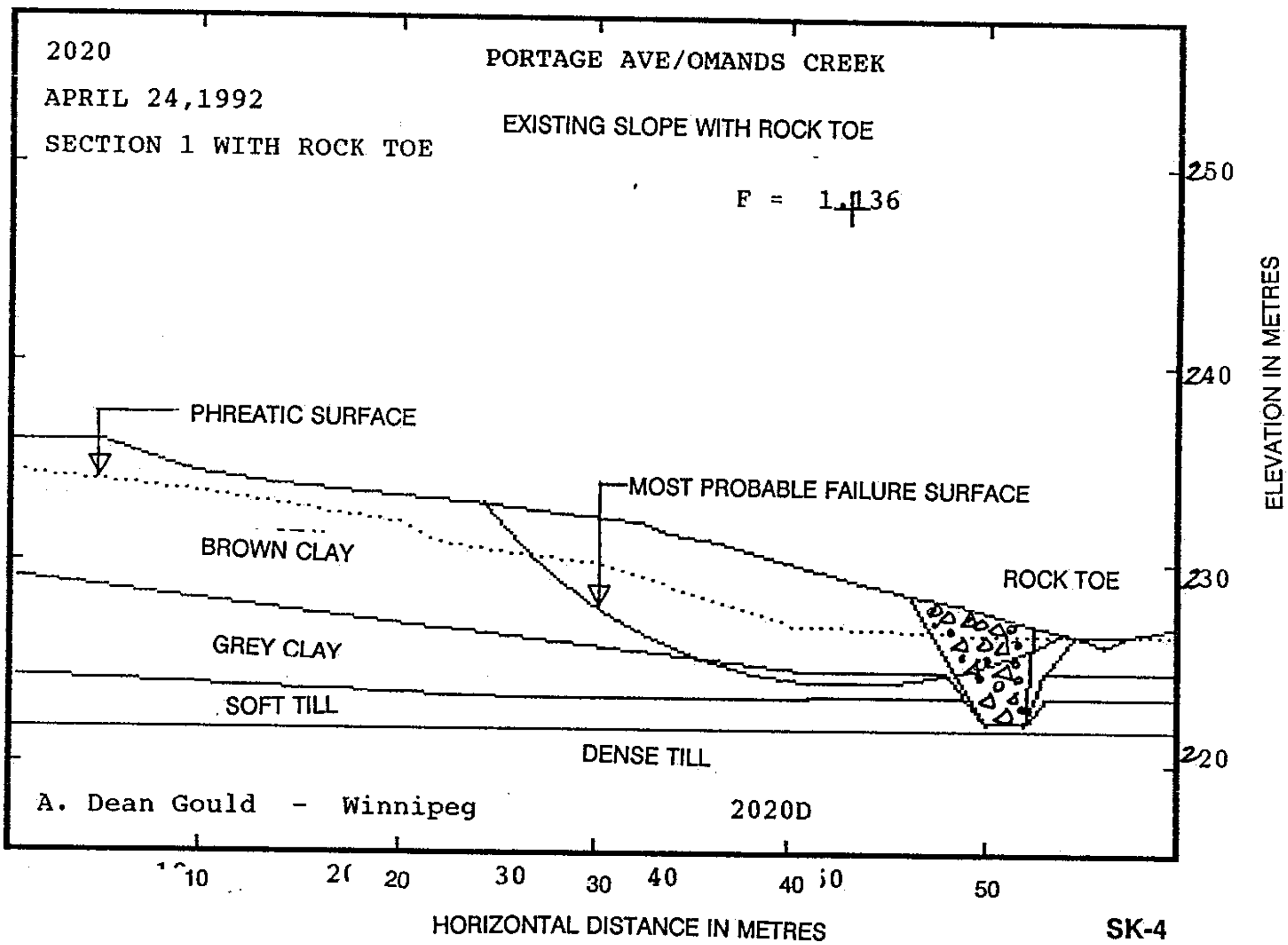
SK-2

Material	Unit Wt	Cohesion	Friction Angle	Piezo Surface	Ru Value
# 1 -CREEK WATER	9.8	0	0	0	0
# 2 -BROWN CLAY	17	0	24	1	0
# 3 -GREY CLAY	16.93	2	14	1	0
# 4 -SOFT TILL	17.67	0	26.1	1	0
# 5 -DENSE TILL	20	0	40	0	0



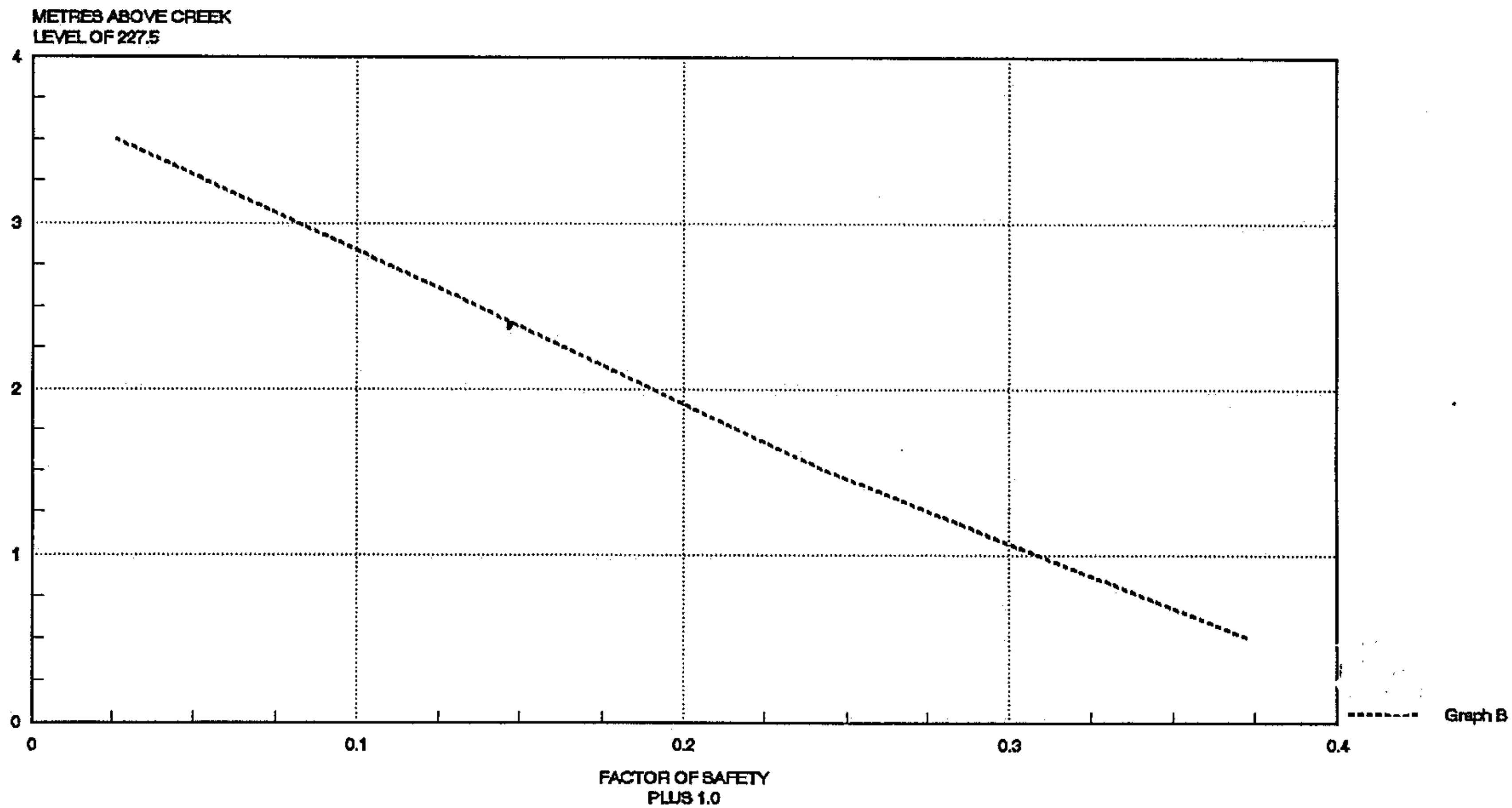
SK-3

Material	Unit Wt	Cohesion	Friction Angle	Piezo Surface	Ru Value
# 1 -CREEK WATER	9.8	0	0	0	0
# 2 -ROCK TOE	15	0	45	1	0
# 3 -BROWN CLAY	17	0	24	1	0
# 4 -GREY CLAY	16.93	2	14	1	0
# 5 -SOFT TILL	17.67	0	26.1	1	0
# 6 -DENSE TILL	20	0	40	0	0



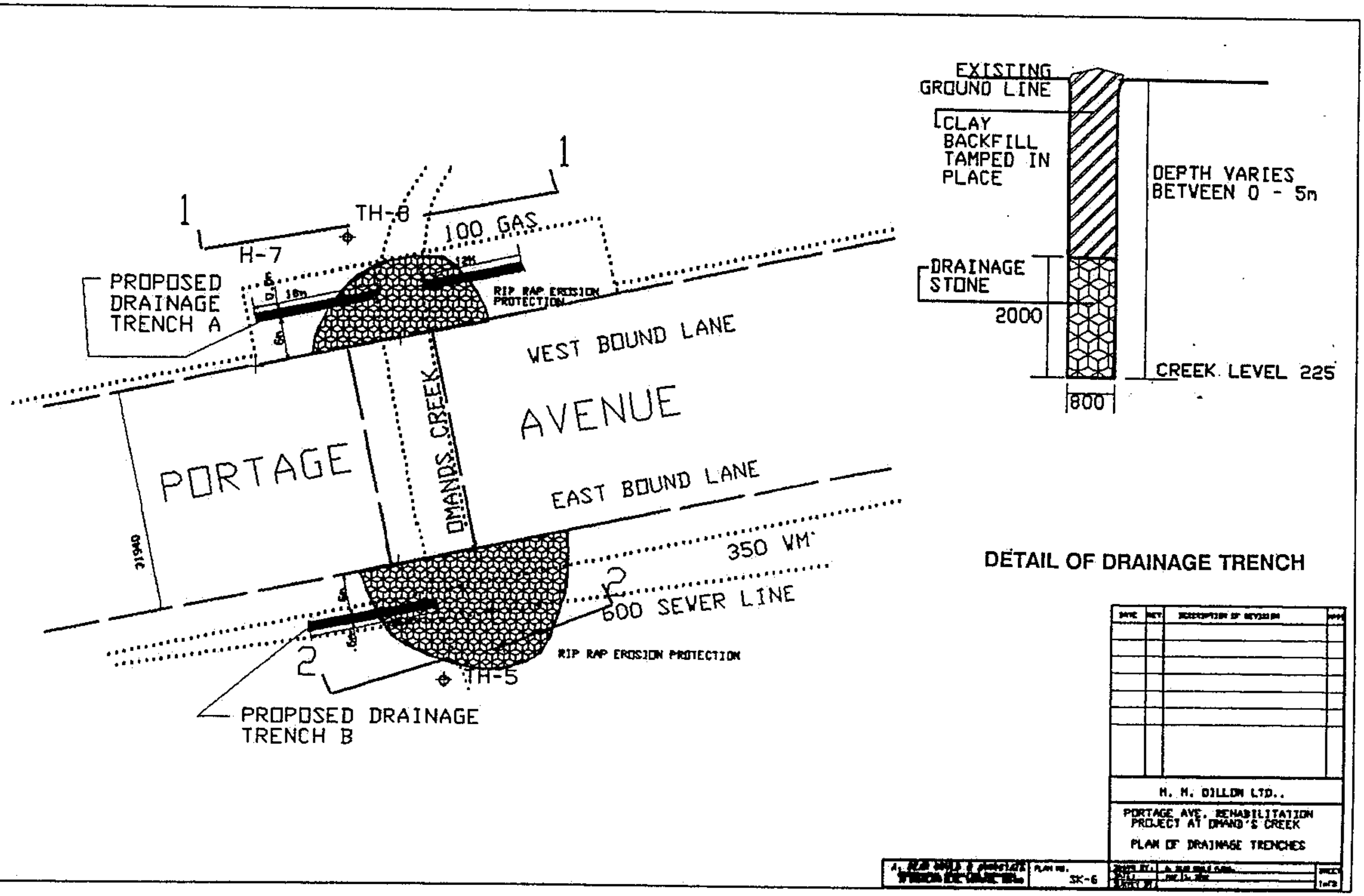
SK-4

M. M. DILLON LTD.,
PORTAGE/OMAND CR. REHABILITATION PROJECT
SAFETY FACTOR VS PHREATIC SURFACE
SECTION 1– NORTH WEST SLOPE

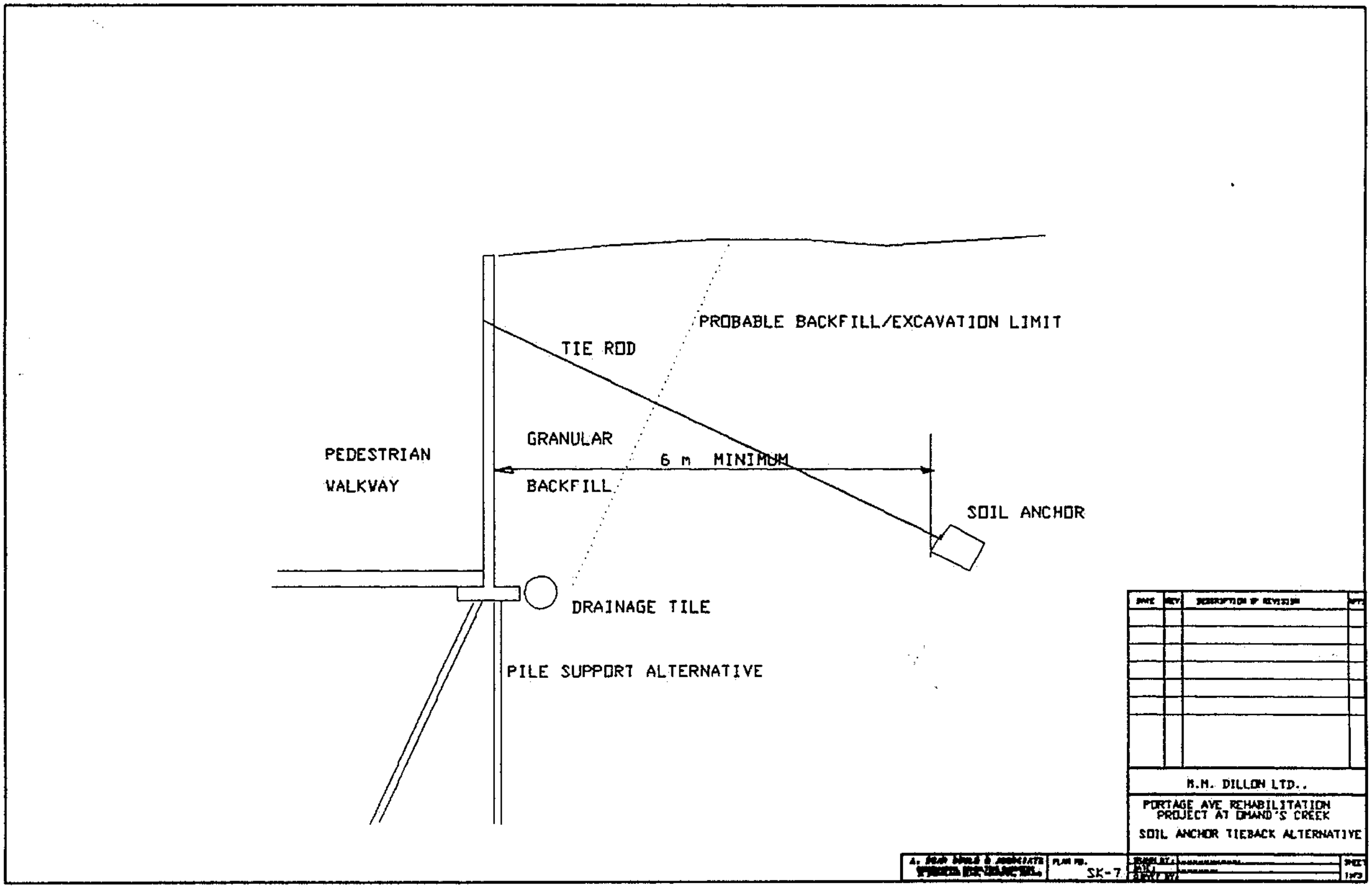


KeyChart 2000

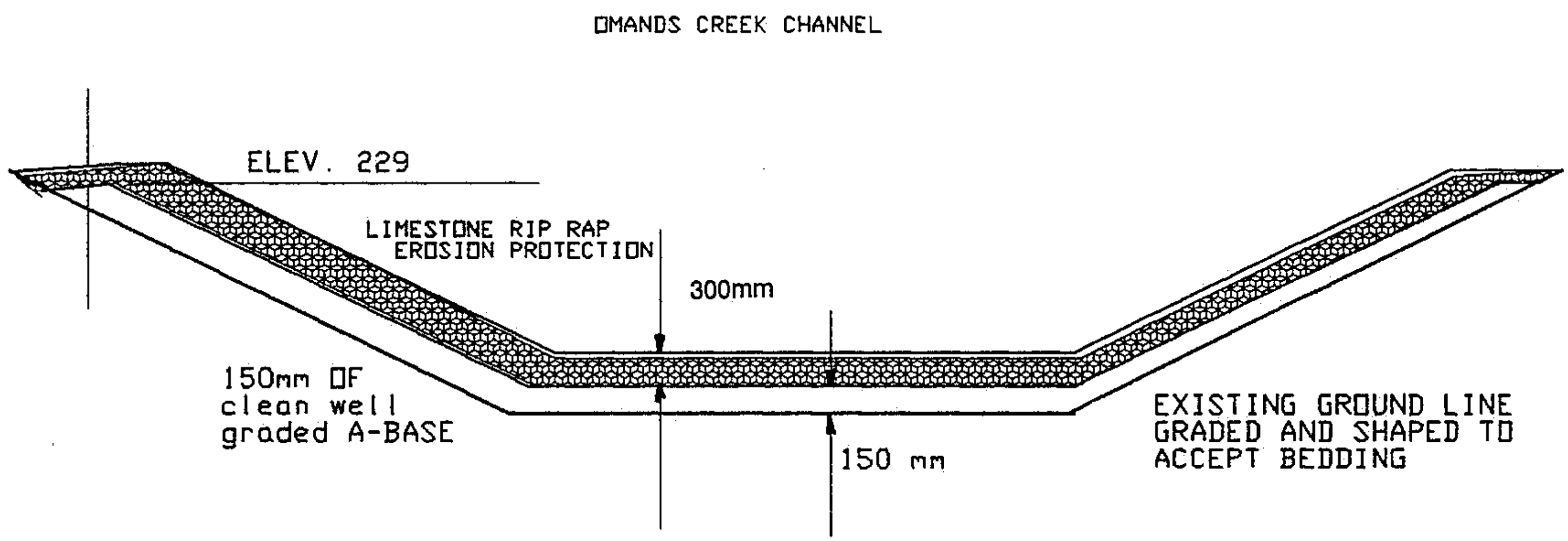
SK-5


PROPOSED DRAINAGE TRENCH A
H-7
TH-8
100 GAS
RIP RAP EROSION PROTECTION
WEST BOUND LANE
PORTAGE
AVENUE
OMANDS CREEK
EAST BOUND LANE
350 WM
600 SEWER LINE
RIP RAP EROSION PROTECTION
TH-5
PROPOSED DRAINAGE TRENCH B
31940
EXISTING GROUND LINE
CLAY BACKFILL TAMPED IN PLACE
DRAINAGE STONE
2000
800
DEPTH VARIES BETWEEN 0 - 5m
CREEK LEVEL 225
DETAIL OF DRAINAGE TRENCH
M. M. DILLON LTD.,
PORTAGE AVE. REHABILITATION PROJECT AT OMAND'S CREEK
PLAN OF DRAINAGE TRENCHES
SK-6

SK-6


PROBABLE BACKFILL/EXCAVATION LIMIT
TIE ROD
GRANULAR
BACKFILL
6 m MINIMUM
PEDESTRIAN
WALKWAY
SOIL ANCHOR
DRAINAGE TILE
PILE SUPPORT ALTERNATIVE
DATE
REV
DESCRIPTION OF REVISION
M.M. DILLON LTD.,
PORTAGE AVE REHABILITATION
PROJECT AT OMAND'S CREEK
SOIL ANCHOR TIEBACK ALTERNATIVE
PLAN NO.
SK-7

SK-7



NOTE: LIMESTONE EROSION PROTECTION TO HAVE MAXIMUM SIZE OF 150 mm AND CONSIST OF DENSE, HARD WHITE STONE QUARRIED FRAGMENTS.

DATE	REV.	DESCRIPTION OF REVISION	APP'D

M. M. DILLON LTD

PORTAGE AVE. REHABILITATION PROJECT AT OMAND'S CREEK

DETAIL OF EROSION PROTECTION

A. DEAN GOULD & ASSOCIATE

PLAN NO. SK-8

DRAWN BY: A. DEAN GOULD P.ENG.

DATE: MAY 11,

SURVEY BY:

SK-8

APPENDIX B

LABORATORY TEST DATA

PROJECT: PORTAGE AVE./OMAND'S CREEK

CLIENT: M.M. DILLON LTD.

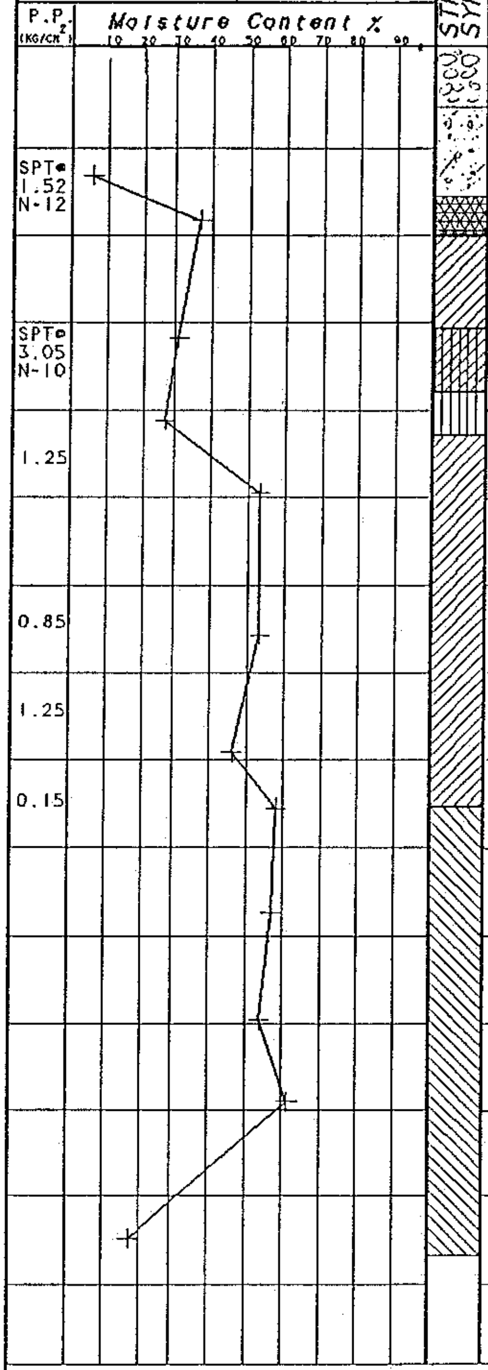
TEST HOLE LOG 1

DATE DRILLED: 03/09/92

P-PACKAGE SAMPLE | T-TUBE SAMPLE

GROUND ELEV. 235.36 DATE: 3/9/92

GROUND WATER LEVEL: DATE:



P.P. (KG/CM²)	DEPTH	SAMPLE NO.	UN. CLASS
	1m		
SPT@ 1.52 N-12		P-1	
	2m	P-2	
			CH
	3m	P-3	
SPT@ 3.05 N-10			CL-CH
	4m	P-4	ML
1.25		T-1	
	5m		
	6m		
0.85		T-2	CH
	7m		
1.25		T-3	
	8m		
0.15		P-5	
	9m	T-4	
	10m		
	11m	P-6	
	12m	P-7	CH
	13m	P-7B	
	14m		
	15m		

SOIL DESCRIPTION

0.00 - 0.61 BALLAST

0.61 - 1.68 SUB-BALLAST - gravel, sand, clay

1.68 - 2.00 TOPSOIL - black
- moist
- trace of organics

2.00 - 3.05 CLAY - brown
- firm
- moist
- trace of gravel and sand

3.05 - 3.81 CLAY & SILT - brown
- firm
- moist
- trace of gravel and sand

3.81 - 4.27 SILT - light brown
- wet
- uniform
- oxidized

4.72 - 8.53 CLAY - brown
- moist
- firm to soft
- trace of light brown silt pockets
- at 7.77m slicken-sided surface at approx. 60 degrees from horizontal

8.53 - 13.71 CLAY - grey
- soft to very soft
- trace of light brown silt pockets
- till inclusions below 11.58 m

END OF HOLE AT 13.71m IN VERY SOFT GREY CLAY/LIGHT BROWNISH GREY TILL MIXTURE

NOTE: SILT LAYER AT 3.81 - 4.27m WAS SUBJECT TO CAVING DUE TO WATER SEEPAGE IN OPEN BOREHOLE

Logged by: R. Deighton

A. Dean Gould P.Eng.
Geotechnical Consultant

PROJECT: PORTAGE AVE./OMAND'S CREEK

CLIENT: M.M. DILLON LTD.

TEST HOLE LOG 2

DATE DRILLED: 03/09/92

GROUND ELEV. 235.36 DATE: 3/9/92

GROUND WATER LEVEL: DATE:

P-PACKAGE SAMPLE | T-TUBE SAMPLE

P.P. (KG/CM²)	Moisture Content % (10 20 30 40 50 60 70 80 90)	STRATA SYMBOL	DEPTH	SAMPLE NO.	UN. CLASS	SOIL DESCRIPTION
			0.25m			0.00 - 0.30 BALLAST
			0.5m			
			0.75m			
			1.0m			0.30 - 2.67 GRANULAR FILL - compact - well graded gravelly sand
			1.25m	P-8		
			1.5m			
			1.75m	P-9		
			2.0m	P-10		
			2.25m			AUGER REFUSAL AT 2.67m
			2.5m	P-11		
			2.75m			
			3.0m			

NOTE:

SPT @ 1.52m N = 14

SPT @ 2.13m N = 17

* auger refusal assumed on coarse gravel or cobbles

Logged by: R. Deighton

A. Dean Gould P.Eng.
Geotechnical Consultant

PROJECT: PORTAGE AVE./OMAND'S CREEK

CLIENT: M.M. DILLON LTD.

TEST HOLE LOG 3

DATE DRILLED: 03/09/92

GROUND ELEV. 235.36 DATE: 3/9/92

GROUND WATER LEVEL: DATE:

P-PACKAGE SAMPLE | T-TUBE SAMPLE

P.P. (KG/CM²)	Moisture Content % (10 20 30 40 50 60 70 80 90)	STRATA SYMBOL	DEPTH	SAMPLE NO.	UN. CLASS	SOIL DESCRIPTION
			0.25m			0.00 - 0.30 BALLAST
			0.5m			
			0.75m			
			1.0m			0.30 - 1.83 GRANULAR FILL - compact - well graded gravelly sand
			1.25m			
			1.5m			
			1.75m	P-12		
			2.0m			
			2.25m			AUGER REFUSAL AT 1.83m
			2.5m			
			2.75m			NOTE:
			3.0m			* auger refusal assumed on coarse gravel or cobbles

Logged by: R. Deighton

A. Dean Gould P.Eng.
Geotechnical Consultant

PROJECT: PORTAGE AVE./OMAND'S CREEK

CLIENT: M.M. DILLON LTD.

TEST HOLE LOG 4

DATE DRILLED: 03/09/92

GROUND ELEV. 235.36 DATE: 3/9/92

GROUND WATER LEVEL: DATE:

P-PACKAGE SAMPLE | T-TUBE SAMPLE

P.P. (KG/CM²)	Moisture Content % (10 20 30 40 50 60 70 80 90)	STRATA SYMBOL	DEPTH	SAMPLE NO.	UN. CLASS	SOIL DESCRIPTION
			0.25m			0.00 - 0.61 BALLAST
			0.5m			
			0.75m			
			1.0m			0.61 - 3.05 SUB-BALLAST & GRANULAR FILL - compact - well graded gravelly sand
			1.25m			
			1.5m			
			1.75m	P-13		
			2.0m			
			2.25m			AUGER REFUSAL AT 3.05m
			2.5m			
			2.75m			
			3.0m			
			3.25m			

NOTE:

SPT @ 1.52m N - 23

3 other test holes were attempted within a 2m radius of TH4. auger refusals were 3.05. 1.22. & 0.91m

* auger refusal assumed on coarse gravel or cobbles

Logged by: R. Deighton

A. Dean Gould P.Eng.
Geotechnical Consultant

PROJECT: PORTAGE AVE./OMAND'S CREEK

CLIENT: M.M. DILLON LTD.

TEST HOLE LOG 5

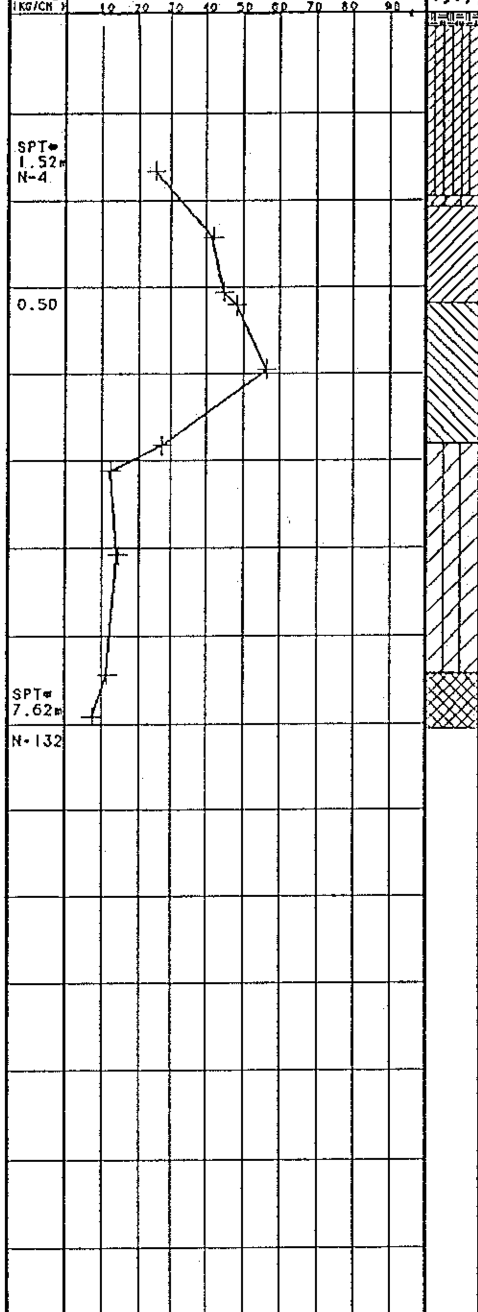
DATE DRILLED: 03/09/92

P-PACKAGE SAMPLE | T-TUBE SAMPLE

GROUND ELEV. 227.50 m DATE: 3/9/92

GROUND WATER LEVEL: DATE:

P.P. (KG/CM²) | Moisture Content % | STRATA SYMBOL | DEPTH | SAMPLE NO. | UN. CLASS



Depth	Sample No.	Un. Class
1m		CL-CH
2m	P-15	
3m	P-16	CH
	T-5	
4m	P-17	CH
5m	T-6	
	P-18	
6m	P-19	
7m		
	P-20	
8m	P-21	
9m		
10m		
11m		
12m		
13m		
14m		
15m		

SOIL DESCRIPTION

0.00 - 0.15 TOPSOIL - black
- silty/sandy
- trace of organics

0.15 - 1.98 SILT & CLAY - brown
- oxidized silt
- soft

1.98 - 2.04 CLAY - black/brown
- silty
- trace of organics

2.04 - 3.20 CLAY - brown
- soft
- trace of light brown silt pockets

3.20 - 4.88 CLAY - grey
- very soft
- trace of gravel & sand
- trace of light brown silt pockets

4.88 - 7.47 TILL - light brownish grey
- soft
- comprised of cobbles, gravel, silt, sand, clay
- moist

7.47 - 8.08 TILL - light brownish grey
- very dense
- dry

END OF HOLE AT 8.08m IN VERY HARD LIGHT BROWNISH GREY TILL

NOTE:

SPT @ 1.52m N = 4

SPT @ 7.62m N = 132

Logged by: R. Deighton

A. Dean Gould P.Eng.
Geotechnical Consultant

PROJECT: PORTAGE AVE./OMAND'S CREEK

CLIENT: M.M. DILLON LTD.

TEST HOLE LOG 6

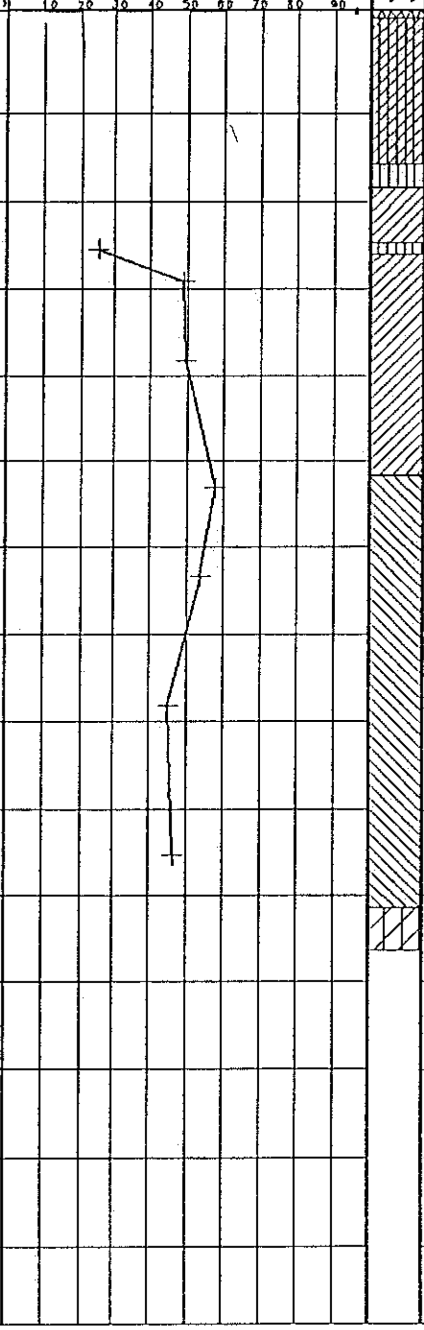
DATE DRILLED: 03/10/92

GROUND ELEV. 233.35 m DATE: 3/10/92

GROUND WATER LEVEL: DATE:

P-PACKAGE SAMPLE | T-TUBE SAMPLE

P.P. (KG/CM²) | Moisture Content % (10 20 30 40 50 60 70 80 90)



P.P. (KG/CM²)	DEPTH	SAMPLE NO.	UN. CLASS
	1m	P-26	CL-CH
		P-27	ML
	2m	P-28	CH
		P-29	ML
0.75	3m	P-30	
0.70	4m	P-31	CH
		P-32	
	5m		
0.50		P-33	
	6m	P-34	
0.95		T-7	
	7m	P-35	
		P-36	CH
0.75	8m	T-8	
		P-37	
	9m	P-38	
0.35		T-9	
	10m	P-39	
		P-40	
	11m		
	12m		
	13m		
	14m		
	15m		

SOIL DESCRIPTION

0.00 - 0.08 TOPSOIL - black
- moist
- trace of organics

0.08 - 1.52 CLAY & SILT - brown
- moist
- firm to soft

1.52 - 1.83 SILT - light brown
- wet
- uniform

1.83 - 2.44 CLAY - brown
- moist
- firm
- trace of light brown silt pockets

2.44 - 2.59 SILT - light brown
- wet
- uniform

2.59 - 5.18 CLAY - brown
- soft to firm
- moist
- trace of light brown silt pockets

5.18 - 10.21 CLAY - grey
- moist
- firm to very soft with increasing depth
- some light brown silt pockets

10.21 - 10.67 TILL - light brownish grey
- soft
- comprised of gravel sand, silt and clay

END OF HOLE AT 10.67m IN SOFT TILL

NOTE: SILT LAYERS AT 1.52 - 1.83m AND 2.44 - 2.59m WERE SUBJECT TO SLIGHT CAVING DUE TO WATER SEEPAGE INTO OPEN BOREHOLE.

Logged by: R. Deighton

A. Dean Gould P.Eng.
Geotechnical Consultant

PROJECT: PORTAGE AVE./OMAND'S CREEK

CLIENT: M.M. DILLON LTD.

TEST HOLE LOG 7

DATE DRILLED: 03/10/92

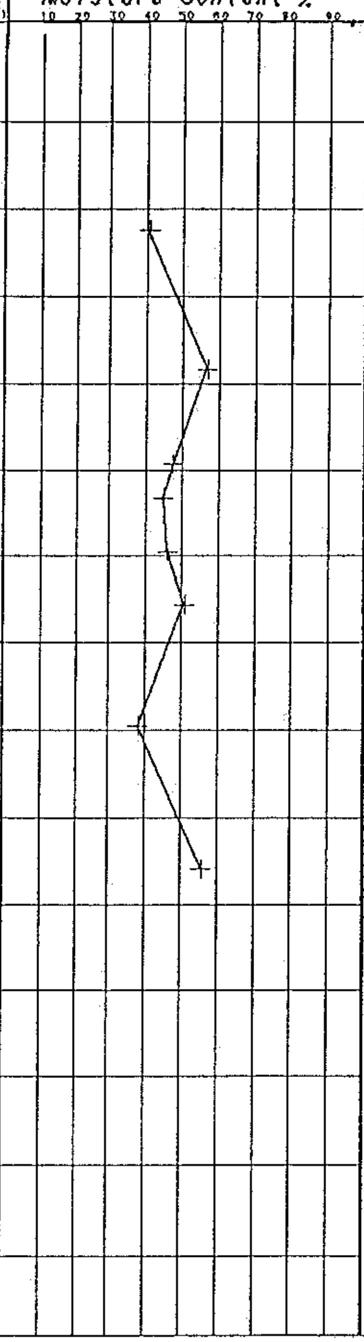
P-PACKAGE SAMPLE | T-TUBE SAMPLE

GROUND ELEV. 233.09 m DATE: 3/10/92

GROUND WATER LEVEL: DATE:

P.P. (KG/CM²)	Depth	Sample No.	Un. Class
	1m	P-41	
		P-42	CL-CH
	2m		
1.60		P-43	
	3m	P-44	
0.65	4m	P-45	CH
		P-46	
1.05	5m	T-10	
0.65		P-47	
0.50	6m	P-48	
1.25		T-11	
	7m	P-49	
		P-50	
		P-51	
	8m	T-12	
0.75		P-52	CH
	9m	P-53	
0.45		T-13	
	10m		
		P-54	
	11m		
	12m		
	13m		
	14m		
	15m		

Moisture Content %



STRATA SYMBOL

SOIL DESCRIPTION

0.00 - 0.91 TOPSOIL - black/brown
- silty/sandy
- moist
- trace of organics

0.91 - 1.52 CLAY & SILT - light brown
- some sand
- trace of black organics

1.52 - 7.00 CLAY - brown
- moist
- firm to soft with increasing depth
- trace of light brown silt pockets
- trace of wood (black) at 6.40m

7.00 - 9.91 CLAY - grey
- soft to very soft with increasing depth
- trace of light brown silt pockets
- trace of wood (black) at 7.62m

9.91 - 10.67 TILL - light brownish grey
- soft
- comprised of gravel sand, silt & clay

END OF HOLE AT 10.67m IN SOFT TILL

Logged by: R. Deighton

A. Dean Gould P.Eng.
Geotechnical Consultant

PROJECT: PORTAGE AVE./OMAND'S CREEK

CLIENT: M.M. DILLON LTD.

TEST HOLE LOG 8

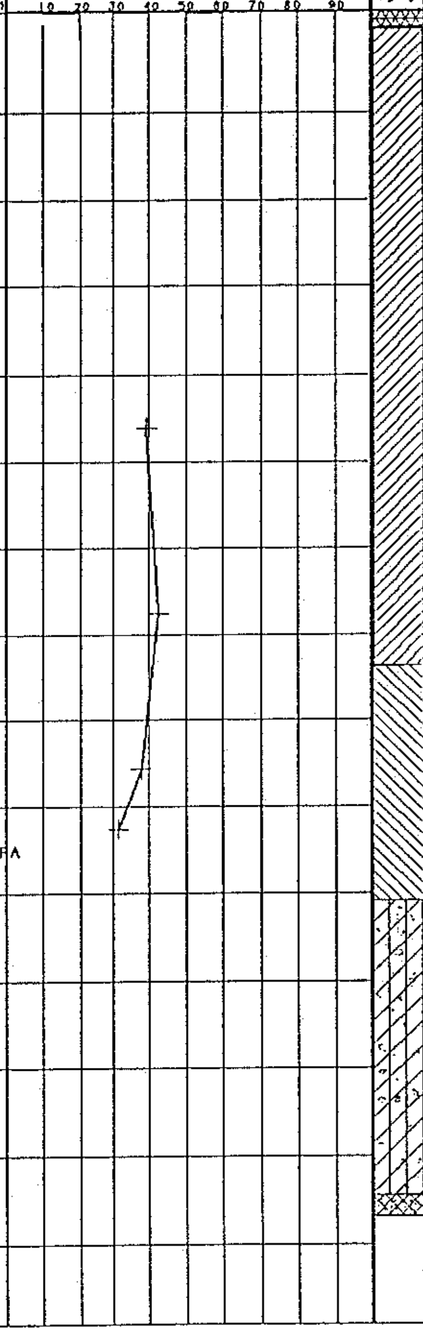
DATE DRILLED: 03/10/92

GROUND ELEV. 228.28 m DATE: 3/10/92

GROUND WATER LEVEL: DATE:

P-PACKAGE SAMPLE | T-TUBE SAMPLE

P.P. (KG/CM²) | Moisture Content % (10 20 30 40 50 60 70 80 90)



Depth	Sample No.	N. Class	P.P.
0.5m			
1.0m	P-55		
1.5m	P-56		
2.0m		CH	
2.5m	P-57		
3.0m	P-58		
3.5m	T-14		0.85
4.0m	P-59		
4.5m	P-60		
5.0m	T-15	CH	0.40FA
5.0m	P-61		
5.5m			
6.0m	P-62		
6.5m			
7.0m	P-63		

SOIL DESCRIPTION

0.00 - 0.08 TOPSOIL - black
- moist
- trace of organics

0.08 - 3.66 CLAY - brown
- moist
- firm to soft
- trace of light brown silt pockets

3.66 - 5.03 CLAY - grey
- moist
- soft to very soft
- trace of light brown silt pockets

5.03 - 6.71 TILL - light brownish grey
- soft
- sandy
- moist to wet
- comprised of gravel sand, silt & clay

6.71 - 6.86 TILL - light brownish grey
- very dense

END OF HOLE AT 6.86m IN VERY DENSE LIGHT BROWNISH GREY TILL

Logged by: R. Deighton

A. Dean Gould P.Eng.
Geotechnical Consultant

PROJECT: PORTAGE AVE./OMAND'S CREEK

CLIENT: M.M. DILLON LTD.

TEST HOLE LOG 9

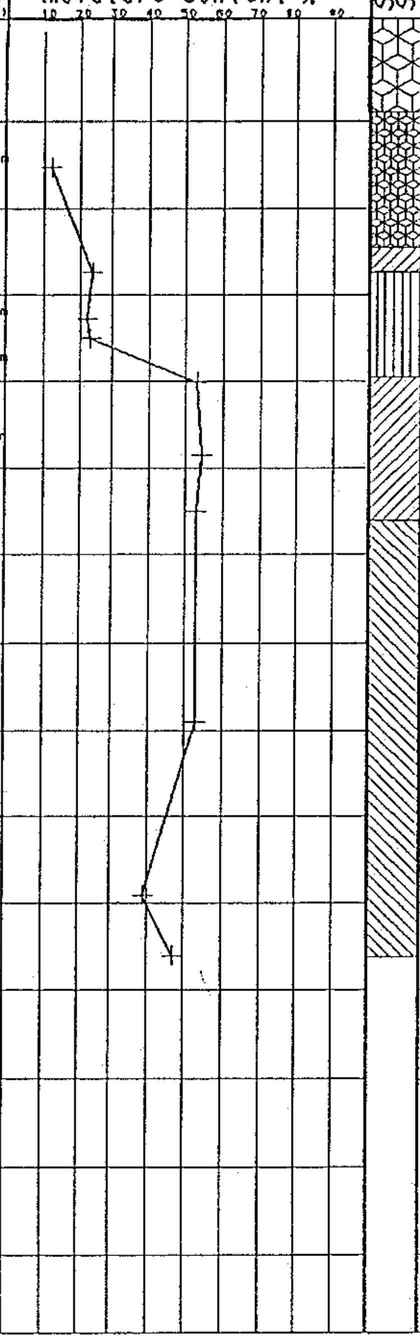
DATE DRILLED: 03/10/92

GROUND ELEV. 234.96 m DATE: 3/10/92

GROUND WATER LEVEL: DATE:

P-PACKAGE SAMPLE | T-TUBE SAMPLE

P.P. (KG/CM²) | Moisture Content %



P.P. (KG/CM²)	DEPTH	SAMPLE NO.	UN. CLASS
	1m	P-64	
SPT @ 1.52m N-11	2m	P-65	
	3m	P-66	CL-CH
SPT @ 3.05m N-7		P-67	
SPT @ 3.81m N-8	4m	P-68	ML
0.75		P-69	
SPT @ 4.57m N-4	5m	P-70	CH
1.10		P-71	
1.45	6m	T-16	
	7m	P-72	
		P-73	
0.65	8m	T-17	CH
	9m	P-74	
	10m	P-75	
0.25		P-76	
	11m		
	12m		
	13m		
	14m		
	15m		

SOIL DESCRIPTION

0.00 - 0.91 FILL - sandy silty clay
- brown
- moist
- firm

0.91 - 2.44 FILL - brown
- gravel, silt, sand and clay
- moist
- firm

2.44 - 2.74 CLAY - brown
- moist
- firm
- layered

2.74 - 3.96 SILT - light brown
- wet
- oxidized
- uniform

3.96 - 5.64 CLAY - brown
- moist
- firm
- trace of light brown silt pockets

5.64 - 10.67 CLAY - grey
- firm to very soft with increasing depth
- moist
- trace of light brown silt pockets
- traces of till inclusions below 9.5m

END OF HOLE AT 10.67m IN VERY SOFT GREY CLAY/TILL INCLUSIONS

NOTE:

SPT @ 1.52m N = 11
SPT @ 3.05m N = 7
SPT @ 3.81m N = 8
SPT @ 4.57m N = 4

** SILT LAYER AT 2.74 - 3.96m WAS SUBJECT TO SLIGHT CAVING DUE TO MINOR WATER SEEPAGE.

Logged by: R. Deighton

A. Dean Gould P.Eng.
Geotechnical Consultant

TABLE 1

MOISTURE CONTENTS

CLIENT: M.M. DILLON LTD.		
PROJECT: PORTAGE AVE./OMAND'S CREEK OVERPASS		
SAMPLES OBTAINED: MARCH 9-10, 1992		
MOISTURE CONTENT DETERMINATIONS		
TEST HOLE #1		
SAMPLE NO.	**DEPTH(m)**	**% MOISTURE**
P-1	1.37	6.1
P-2	1.52-1.98	36.8
P-3	3.05-3.51	30.7
P-4	4.11	27.0
T-1	4.57-5.18	53.7
T-2	6.10-6.71	53.7
T-3	7.62-8.23	46.9
P-5	8.53	58.0
T-4	9.14-9.75	57.4
P-6	10.97	53.4
P-7	11.89	60.8
P-7b	13.41	18.8
TEST HOLE #5		
P-15	1.68	25.4
P-16	2.44	42.3
T-5	3.05-3.51	48.8
P-17	3.96	57.1
T-6	4.57-5.18	27.9
P-18	5.18	11.8
P-19	6.10	13.7
P-20	7.47	11.6
P-21	7.62-8.08	8.6

TABLE 1

MOISTURE CONTENTS

SAMPLE NO.	DEPTH(m)	% MOISTURE
TEST HOLE #6		
P-29	2.51	25.4
P-30	2.90	49.6
P-31	3.81	49.7
P-33	5.33	57.5
T-7	6.10-6.71	53.8
T-8	7.62-8.23	45.0
T-9	9.14-9.75	46.9
TEST HOLE #7		
P-43	2.29	40.8
P-45	3.81	57.3
T-10	4.57-5.03	47.5
P-47	5.33	45.0
P-48	5.94	46.1
T-11	6.10-6.55	50.6
T-12	7.62-8.23	38.9
T-13	9.14-9.75	56.0
TEST HOLE #8		
P-57	2.29	39.7
T-14	3.05-3.66	42.8
P-60	4.27	38.5
T-15	4.57-5.03	31.9

TABLE 1

MOISTURE CONTENTS

TEST HOLE #9		
SAMPLE NO.	DEPTH(m)	% MOISTURE
P-65	1.52-1.98	11.6
P-66	2.74	23.8
P-67	3.05-3.51	21.2
P-68	3.51	22.9
P-69	3.81-4.27	52.9
P-70	4.57-5.03	54.4
T-16	5.33-5.79	52.9
T-17	7.62-8.23	52.9
P-75	9.91	39.6
P-76	10.67	47.6

TABLE 2

UNCONFINED COMPRESSIVE STRENGTH RESULTS

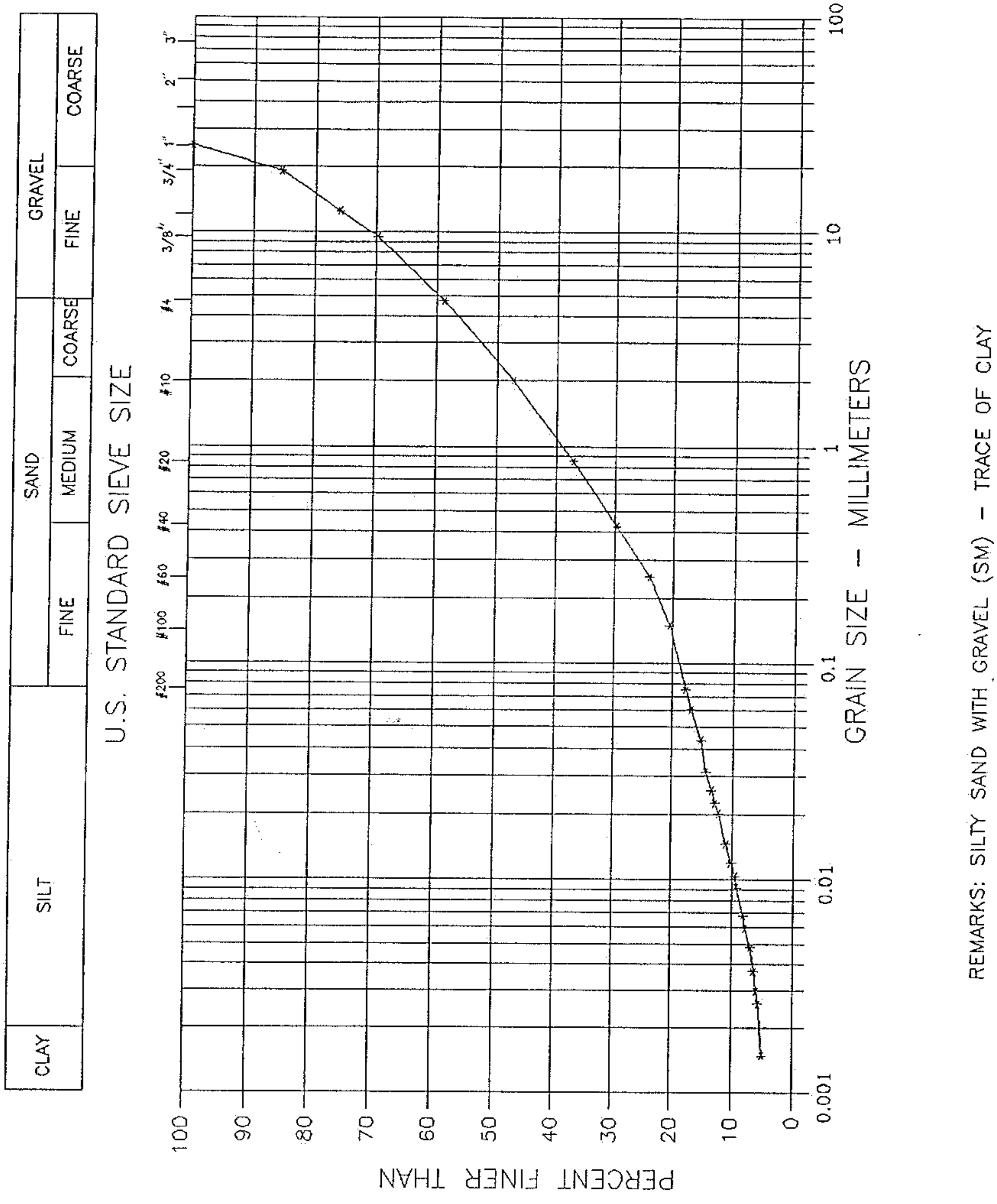
TEST HOLE	DEPTH (m)	NET UNIT WT. (kN/m^3)	MOISTURE CONTENT (%)	UNCONFINED COMPRESSION (kPa)
1	5.03	16.9	52.4	79.4
1	6.40	18.5	54.0	117.7
1	8.08	16.9	54.9	113.6
1	9.45	16.9	53.7	80.7
6	6.55	16.7	61.2	103.5
6	7.77	17.4	55.0	97.9
7	4.88	17.1	52.3	76.6
7	6.25	17.6	50.8	79.0
7	8.08	17.7	47.6	90.8
9	5.49	16.2	53.9	90.8
9	7.92	16.6	54.8	97.6

TABLE 3

ATTERBERG LIMITS

ATTERBERG LIMITS					
TEST HOLE	SAMPLE NO.	DEPTH (m)	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX
1	T-2	6.10-6.71	101.8	27.1	74.7
6	P-30	2.90	81.2	21.6	59.6
6	T-7	7.62-8.23	82.6	20.0	62.6
7	T-10	4.57-5.18	89.9	22.9	67.0
8	T-15	4.57-5.03	48.2	13.2	35.0
9	P-68	3.51	19.8	16.9	2.9

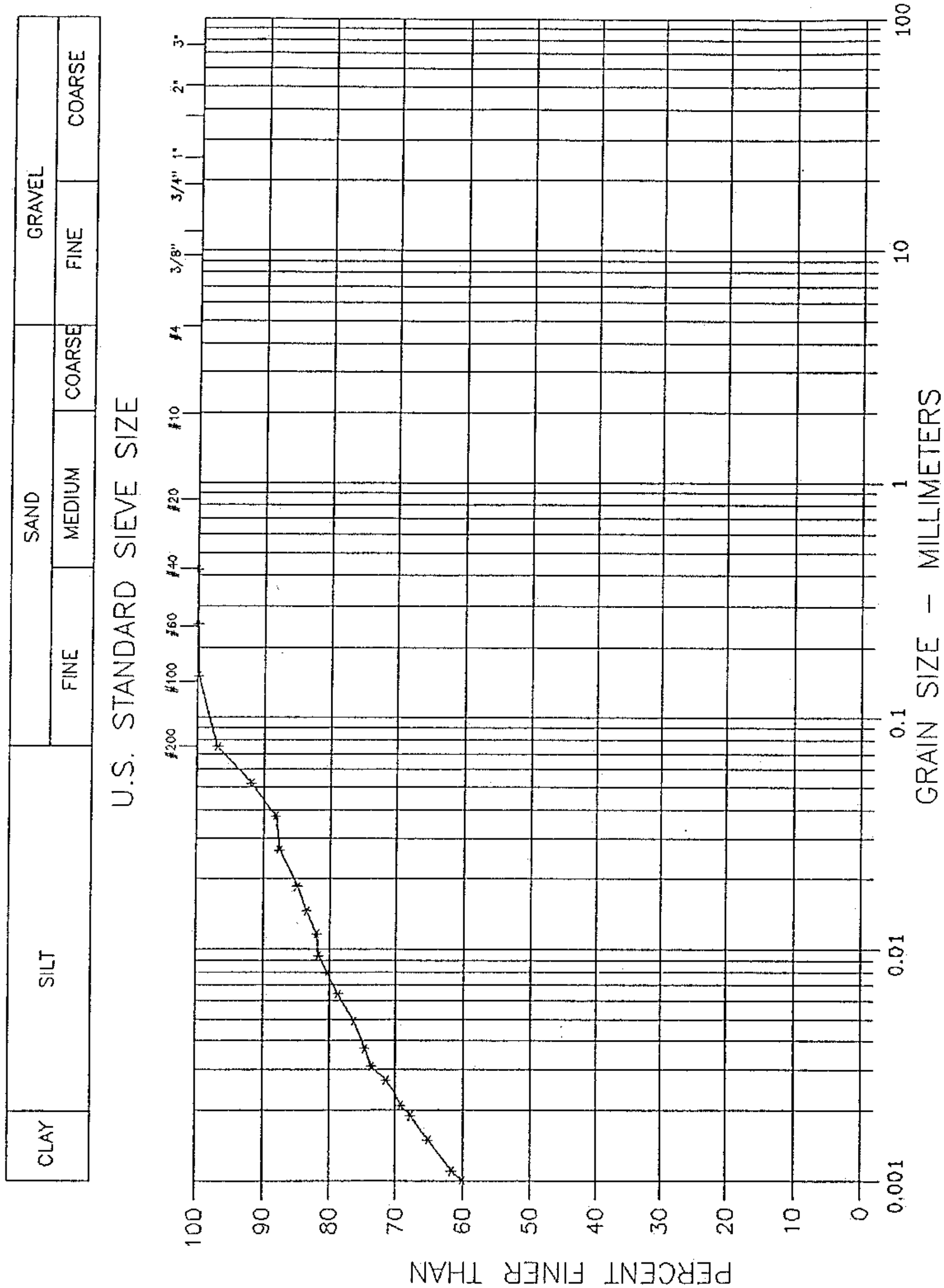
GRAIN SIZE CURVE



A.D. Gould & Associates
Geotechnical Consultants

JOB NO.: 2020	DATE: MARCH 1992
CLIENT: M.M. DILLON LTD.	
PROJECT: PORTAGE AVE./OMAND'S CREEK OVERPASS	
LOCATION: WINNIPEG, MANITOBA	
HOLE NO.: 1	SAMPLE NO.: P-1
DEPTH: 1.37 m	

GRAIN SIZE CURVE

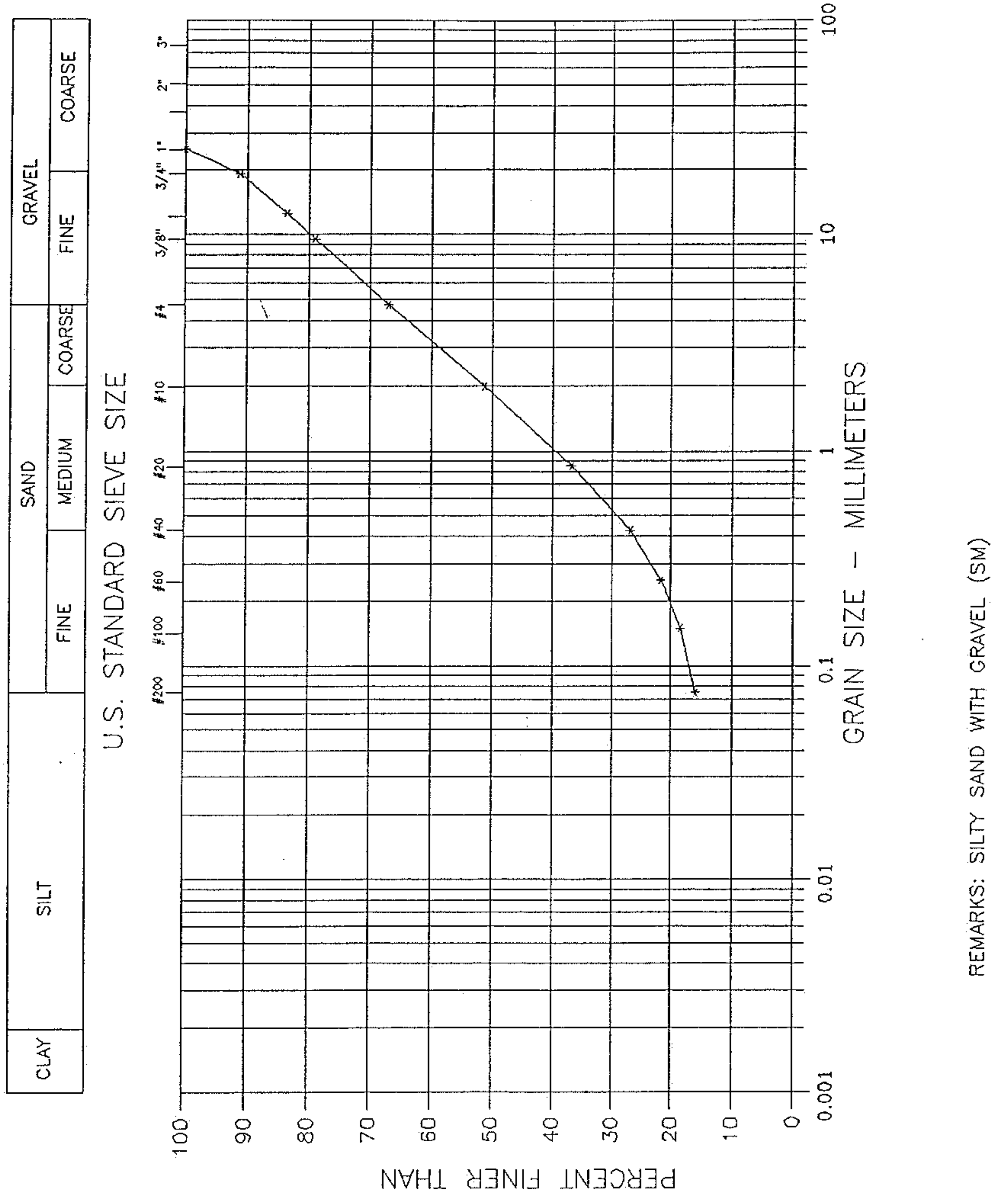


REMARKS: LIGHT BROWN/BROWN SILTY CLAY - FIRM - MOIST

A.D. Gould & Associates
Geotechnical Consultants

JOB NO.: 2020	DATE: MARCH 1992
CLIENT: M.M. DILLON LTD.	
PROJECT: PORTAGE AVE./OMAND'S CREEK OVERPASS	
LOCATION: WINNIPEG, MANITOBA	
HOLE NO.: 1	SAMPLE NO.: P-3
DEPTH: 3.05 - 3.51 m	

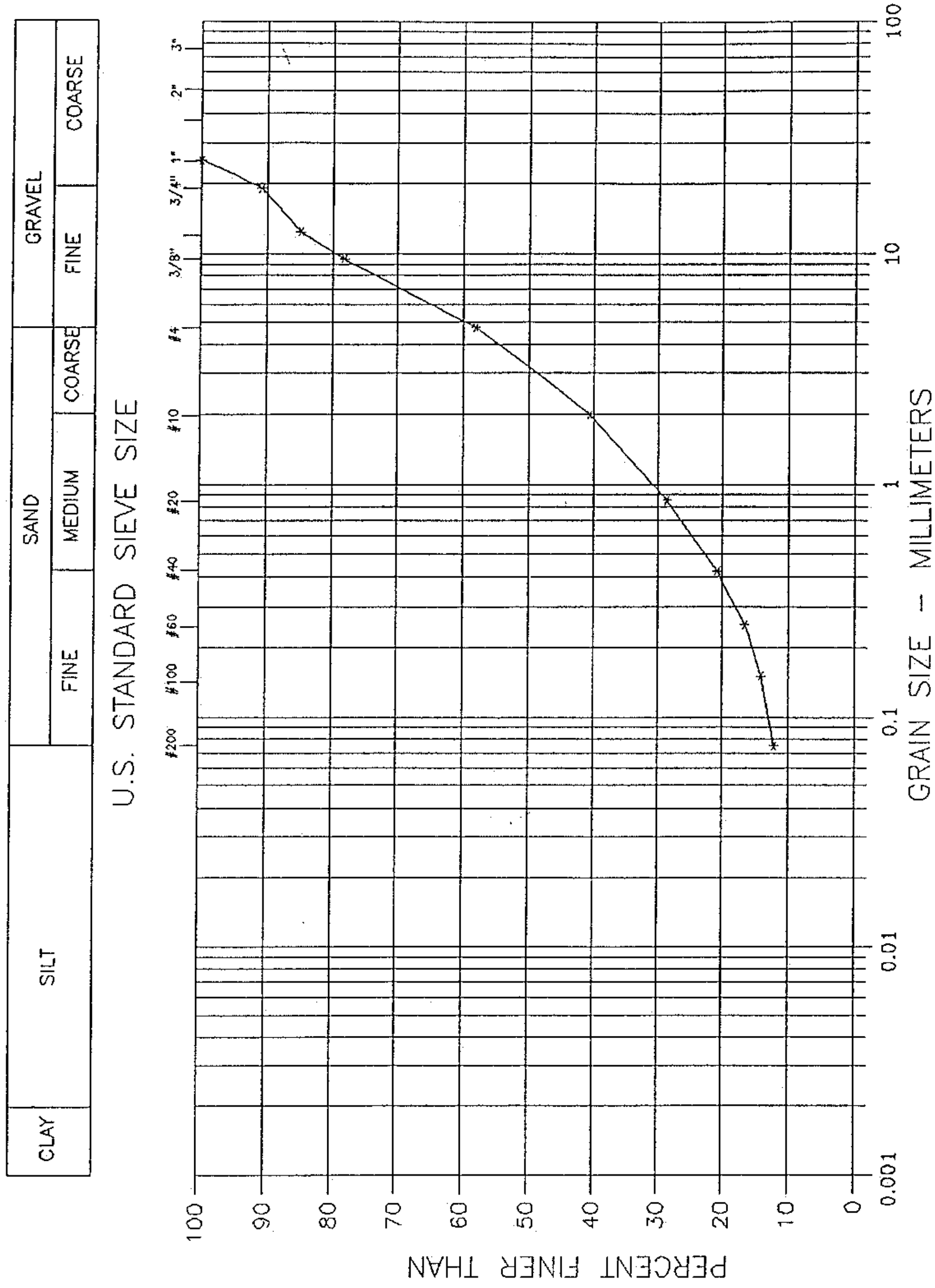
GRAIN SIZE CURVE



A.D. Gould & Associates
Geotechnical Consultants

JOB NO.: 2020	DATE: MARCH 1992
CLIENT: M.M. DILLON LTD.	
PROJECT: PORTAGE AVE./OMAND'S CREEK OVERPASS	
LOCATION: WINNIPEG, MANITOBA	
HOLE NO.: 2	SAMPLE NO.: P-9
DEPTH: 1.52 - 1.98 m	

GRAIN SIZE CURVE

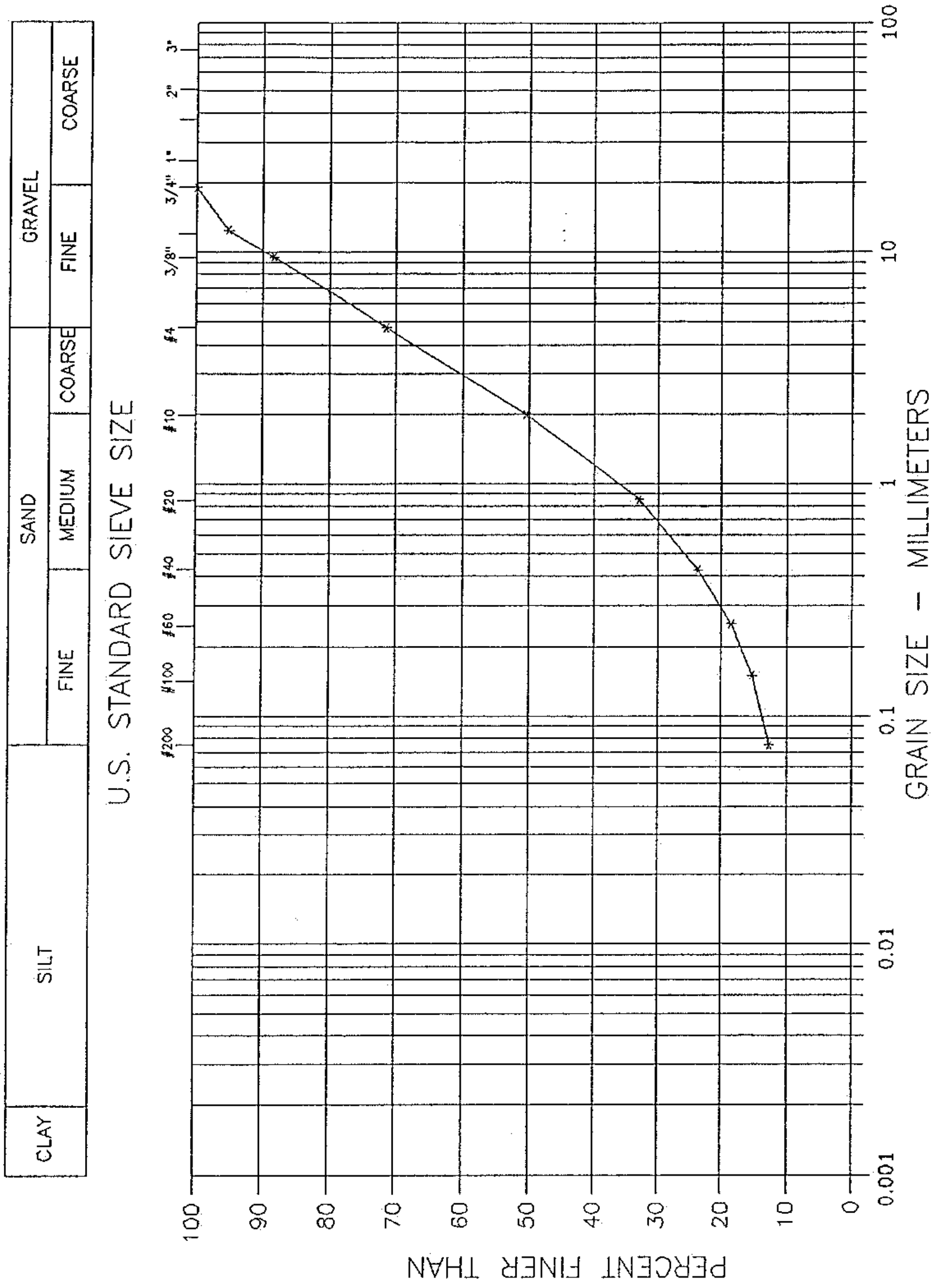


REMARKS: WELL GRADED SAND WITH SILT AND GRAVEL (SW-SM)

A.D. Gould & Associates
Geotechnical Consultants

JOB NO.: 2020	DATE: MARCH 1992
CLIENT: M.M. DILLON LTD.	
PROJECT: PORTAGE AVE./OMAND'S CREEK OVERPASS	
LOCATION: WINNIPEG, MANITOBA	
HOLE NO.: 2	SAMPLE NO.: P-11
DEPTH: 2.13 - 2.59 m	

GRAIN SIZE CURVE

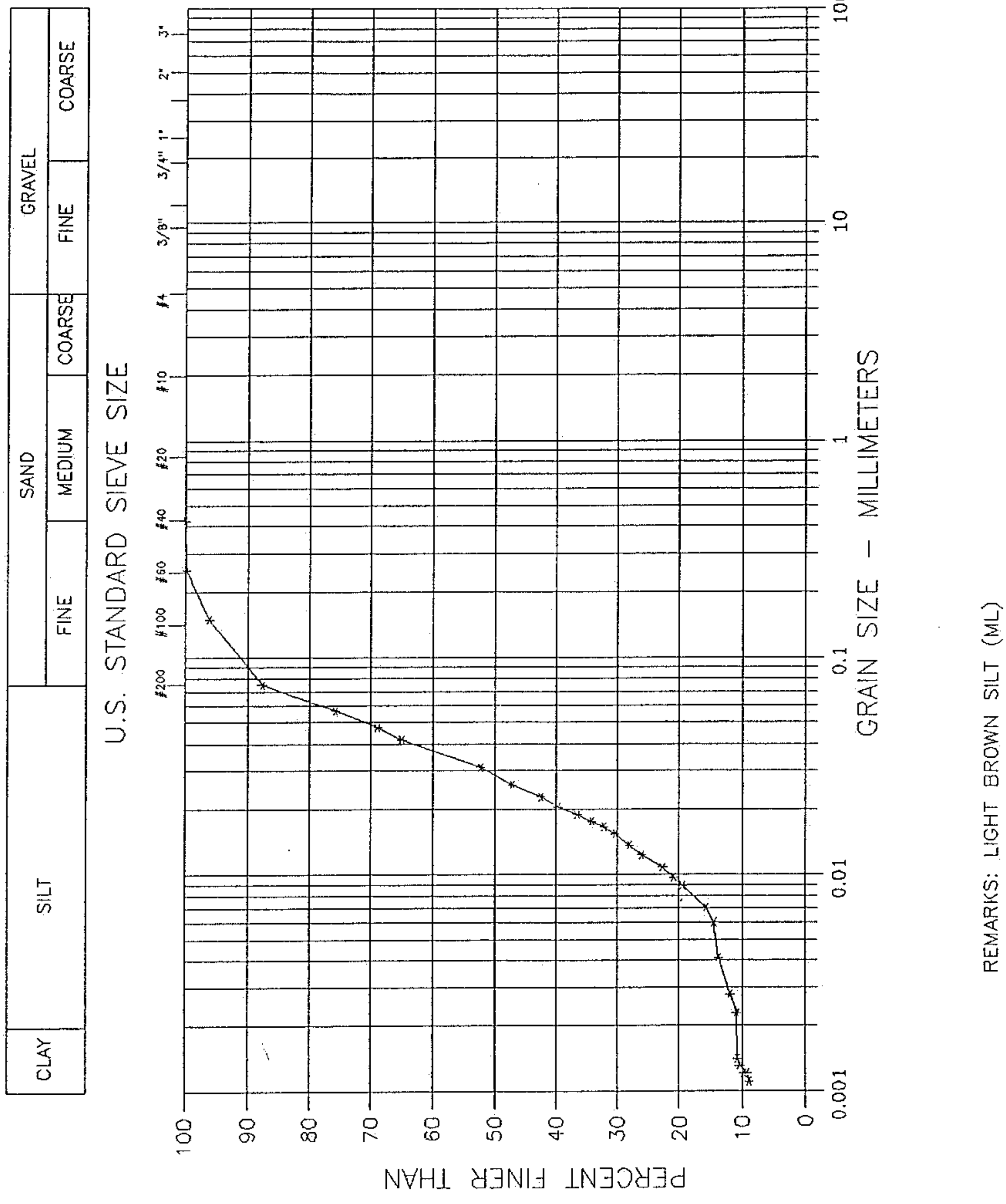


REMARKS: WELL GRADED SAND WITH SILT AND GRAVEL (SW–SM)

A.D. Gould & Associates
Geotechnical Consultants

JOB NO.: 2020	DATE: MARCH 1992
CLIENT: M.M. DILLON LTD.	
PROJECT: PORTAGE AVE./OMAND'S CREEK OVERPASS	
LOCATION: WINNIPEG, MANITOBA	
HOLE NO.: 4	SAMPLE NO.: P-13
DEPTH: 1.52 - 1.98 m	

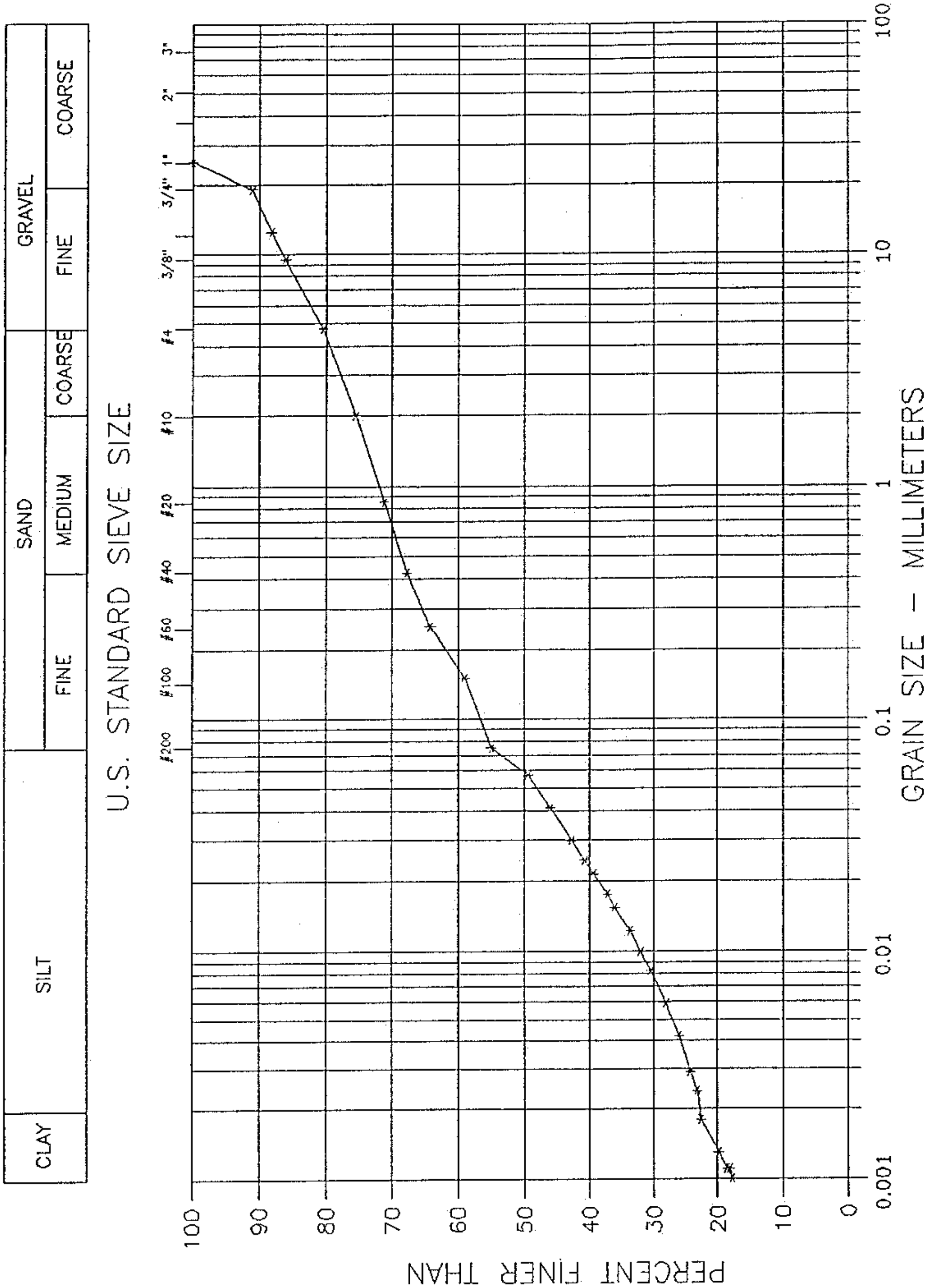
GRAIN SIZE CURVE



A.D. Gould & Associates
Geotechnical Consultants

JOB NO.: 2020	DATE: MARCH 1992
CLIENT: M.M. DILLON LTD.	
PROJECT: PORTAGE AVE./OMAND'S CREEK OVERPASS	
LOCATION: WINNIPEG, MANITOBA	
HOLE NO.: 6	SAMPLE NO.: P-29
DEPTH: 2.51 m	

GRAIN SIZE CURVE



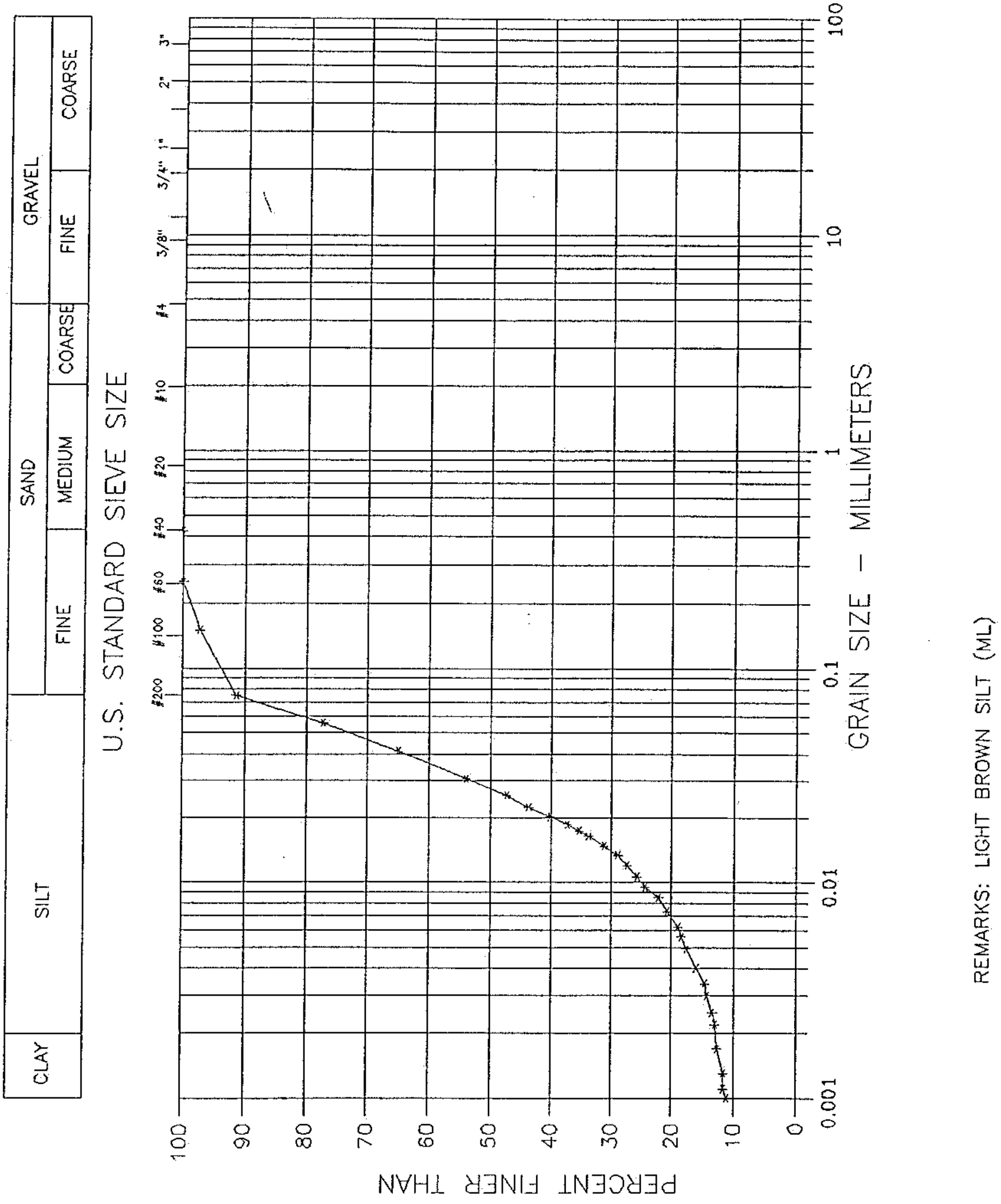
REMARKS: SANDY CLAYEY SILT WITH GRAVEL

A.P. Gould & Associates

Geotechnical Consultants

JOB NO.: 2020	DATE: MARCH 1992
CLIENT: M.M. DILLON LTD.	
PROJECT: PORTAGE AVE./OMAND'S CREEK OVERPASS	
LOCATION: WINNIPEG, MANITOBA	
HOLE NO.: 9	SAMPLE NO.: P-65
DEPTH: 1.52 - 1.98 m	

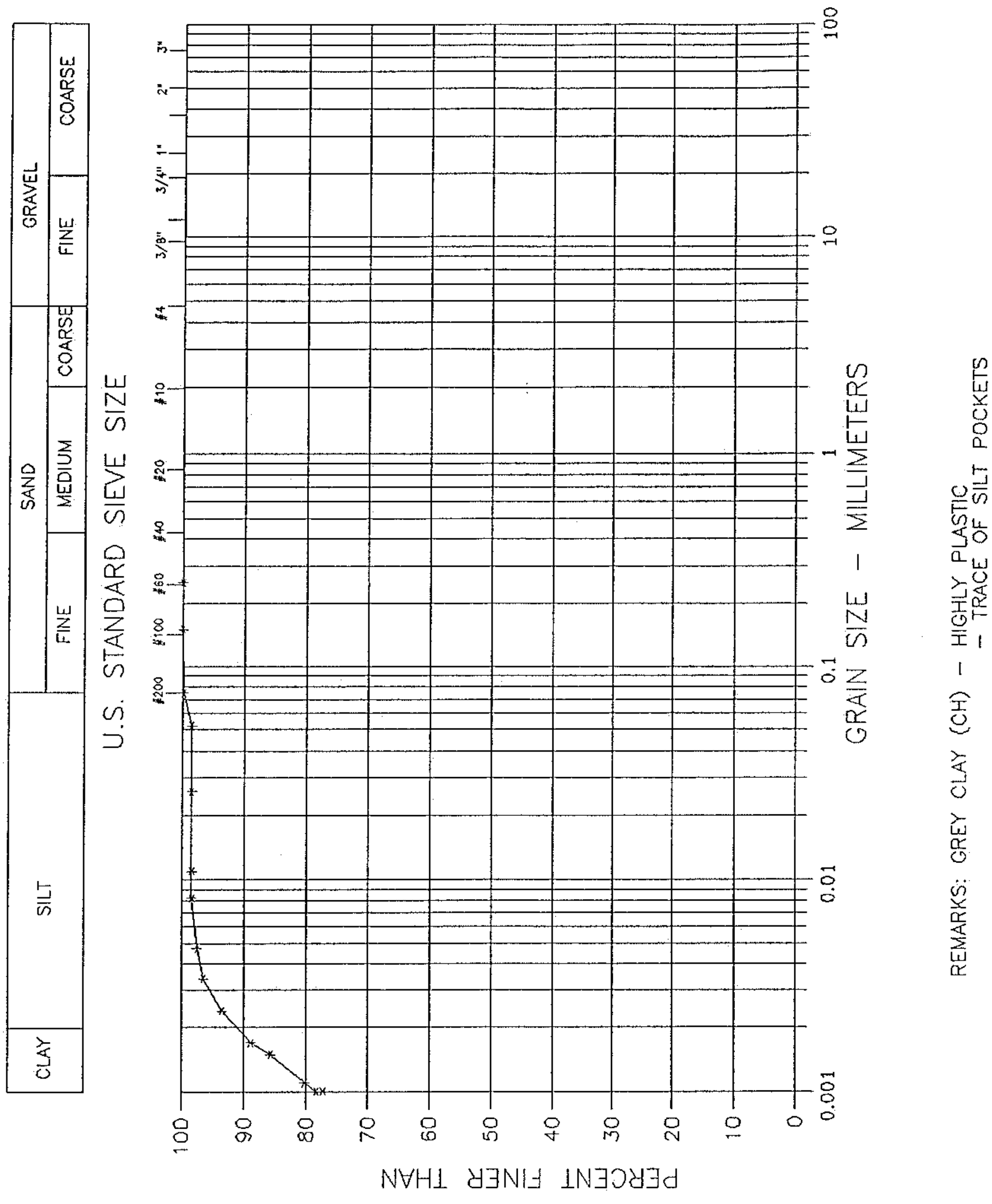
GRAIN SIZE CURVE



A.D. Gould & Associates
Geotechnical Consultants

JOB NO.: 2020	DATE: MARCH 1992
CLIENT: M.M. DILLON LTD.	
PROJECT: PORTAGE AVE./OMAND'S CREEK OVERPASS	
LOCATION: WINNIPEG, MANITOBA	
HOLE NO.: 9	SAMPLE NO.: P-68
DEPTH: 3.51 m	

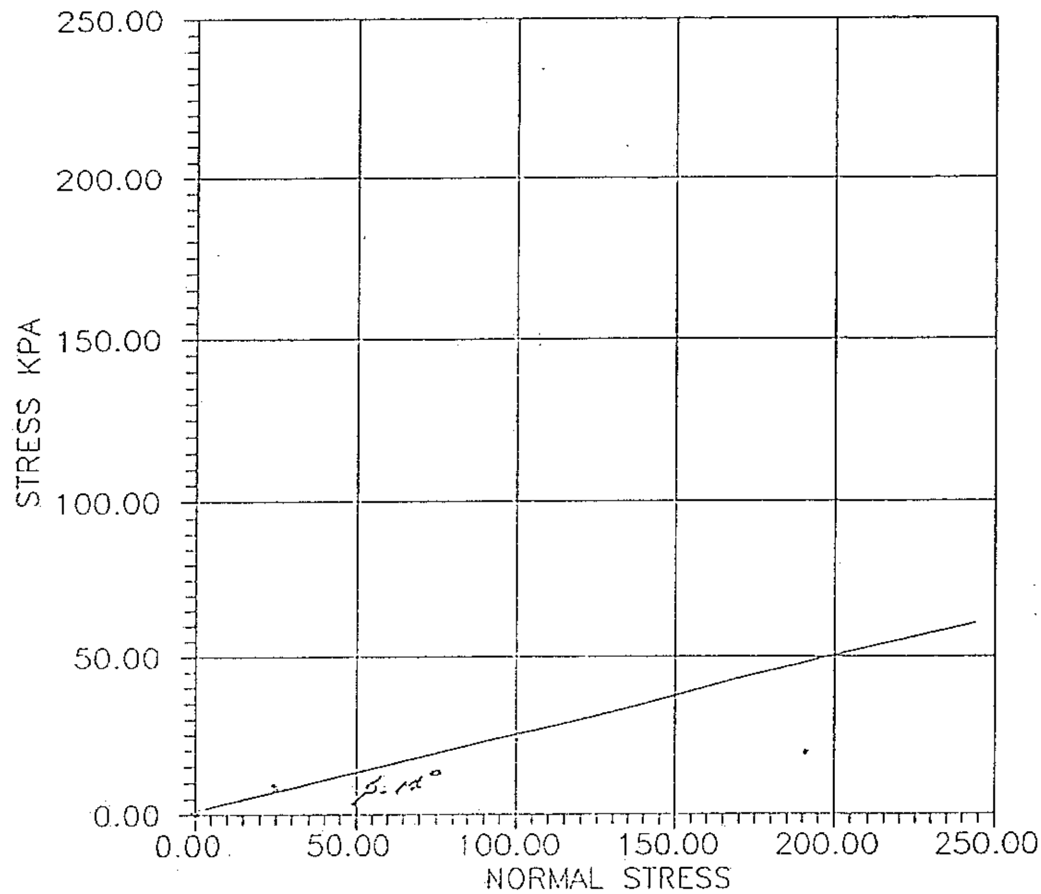
GRAIN SIZE CURVE



A.D. Gould & Associates
Geotechnical Consultants

JOB NO.: 2020	DATE: MARCH 1992
CLIENT: M.M. DILLON LTD.	
PROJECT: PORTAGE AVE./OMAND'S CREEK OVERPASS	
LOCATION: WINNIPEG, MANITOBA	
HOLE NO.: 9	SAMPLE NO.: P-70
DEPTH: 4.57 - 5.03 m	

DIRECT SHEAR TEST



Soil Characteristics; Grey Clay with till inclusions, numerous tan silt pockets and small (5mm diam) stones, occasional large stone to 25mm diam, soft, plastic consistency

Strain Rate; 0.0061 mm/m Peak
0.1191 mm/m Residual

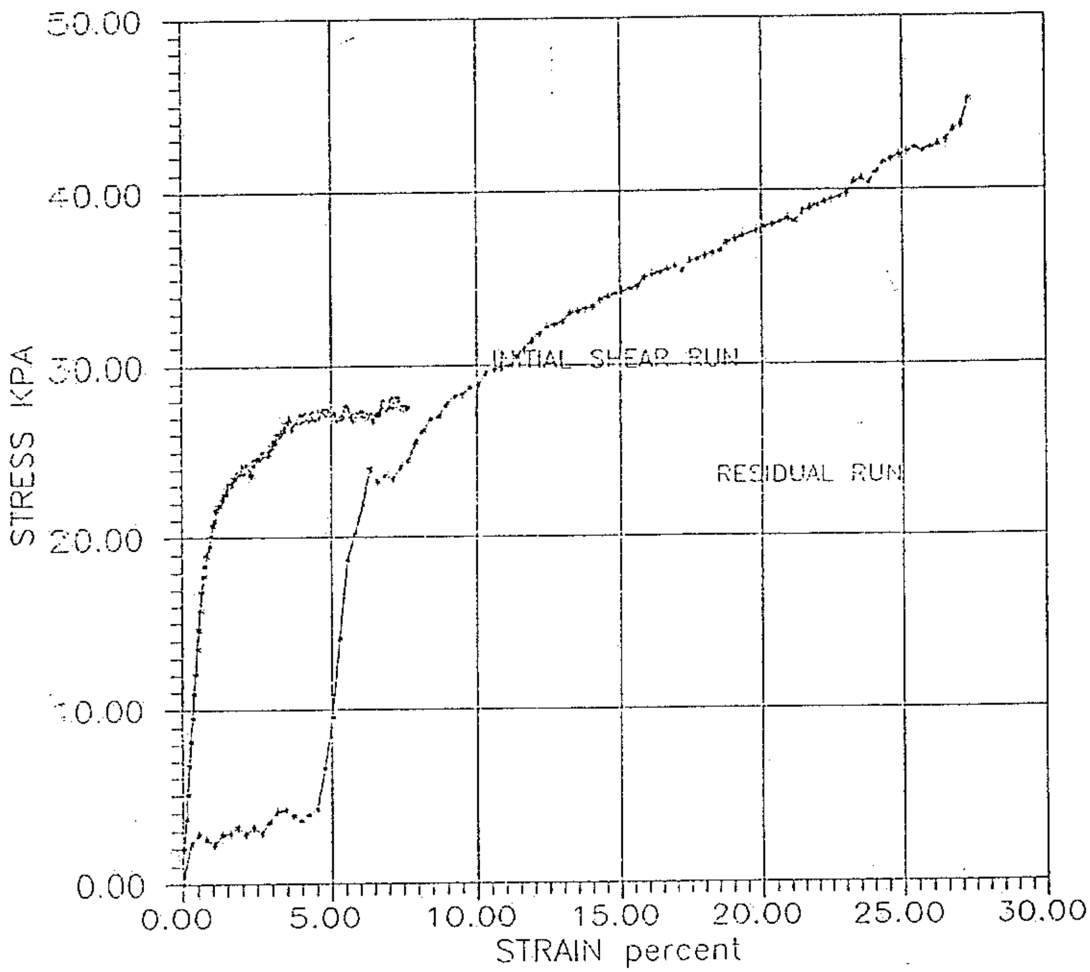
Moisture Content; 39.4 - 26. %
Unit Weight; 18.03-16.73 kN/cu.m.

Normal Load; 190.95 kPa
100.1 kPa
24.36 kPa

Initial Void Ratio; 0.915
Deg. of Sat. 88.8 %

A. DEAN GOULD & ASSOCIATE *Geotechnical Consultants*	JOB. No. 2020	DATE: 03/25/92
	CLIENT: M. M. DILLON LTD.,	
	PROJECT: Portage Avenue Overpass	
	LOCATION: Lower Slope of Omands Creek	
	HOLE No. 8	SAMPLE No. T-15
	DEPTH: 4.7m	TEST No. 2020 B

DIRECT SHEAR TEST



Soil Characteristics; Grey Clay with till inclusions, numerous tan silt pockets and small (5mm diam) stones, occasional large stone to 25mm diam, soft, plastic consistency

Strain Rate; 0.0061 mm/m Peak, 0.1191 mm/m Residual

Moisture Content; 31.4 %

Normal Load; 100.1 kPa

Unit Weight; 18.08 kN/cu.m.

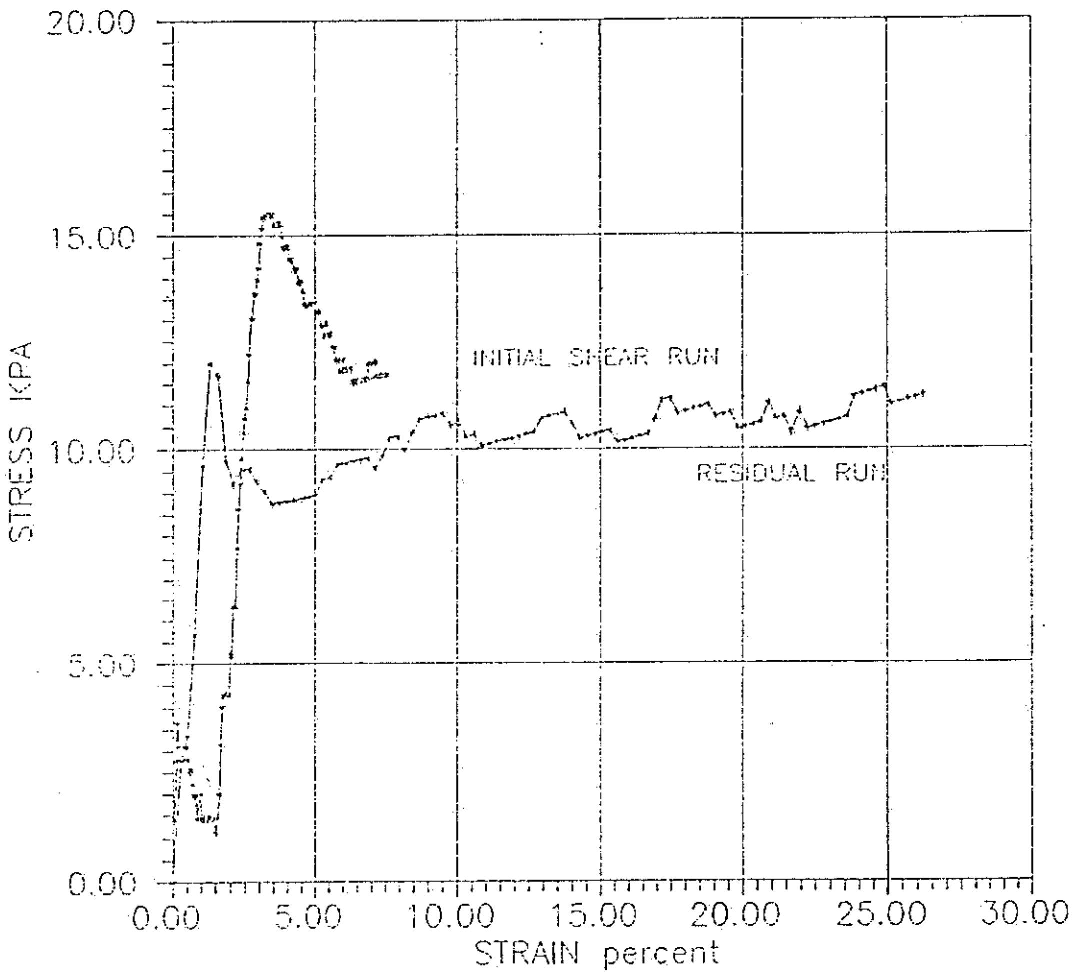
Initial Void Ratio; 0.915

Deg. of Sat. 88.8 %

A. DEAN GOULD & ASSOCIATE
Geotechnical Consultants

JOB. No. 2020	DATE: 03/25/92
CLIENT: M. M. DILLON LTD.,	
PROJECT: Portage Avenue Overpass	
LOCATION: Lower Slope of Omands Creek	
HOLE No. 8	SAMPLE No. T-15
DEPTH: 4.7m	TEST No. 2020 B

DIRECT SHEAR TEST



Soil Characteristics; Grey Clay with till inclusions, numerous tan silt pockets and small (5mm diam) stones, occasional large stone to 25mm diam, soft, plastic consistency

Strain Rate; 0.0061 mm/m Peak
0.1191 mm/m Residual

Moisture Content; 39.4 %

Unit Weight; 16.73 kN/cu.m.

Normal Load; 24.36 kPa

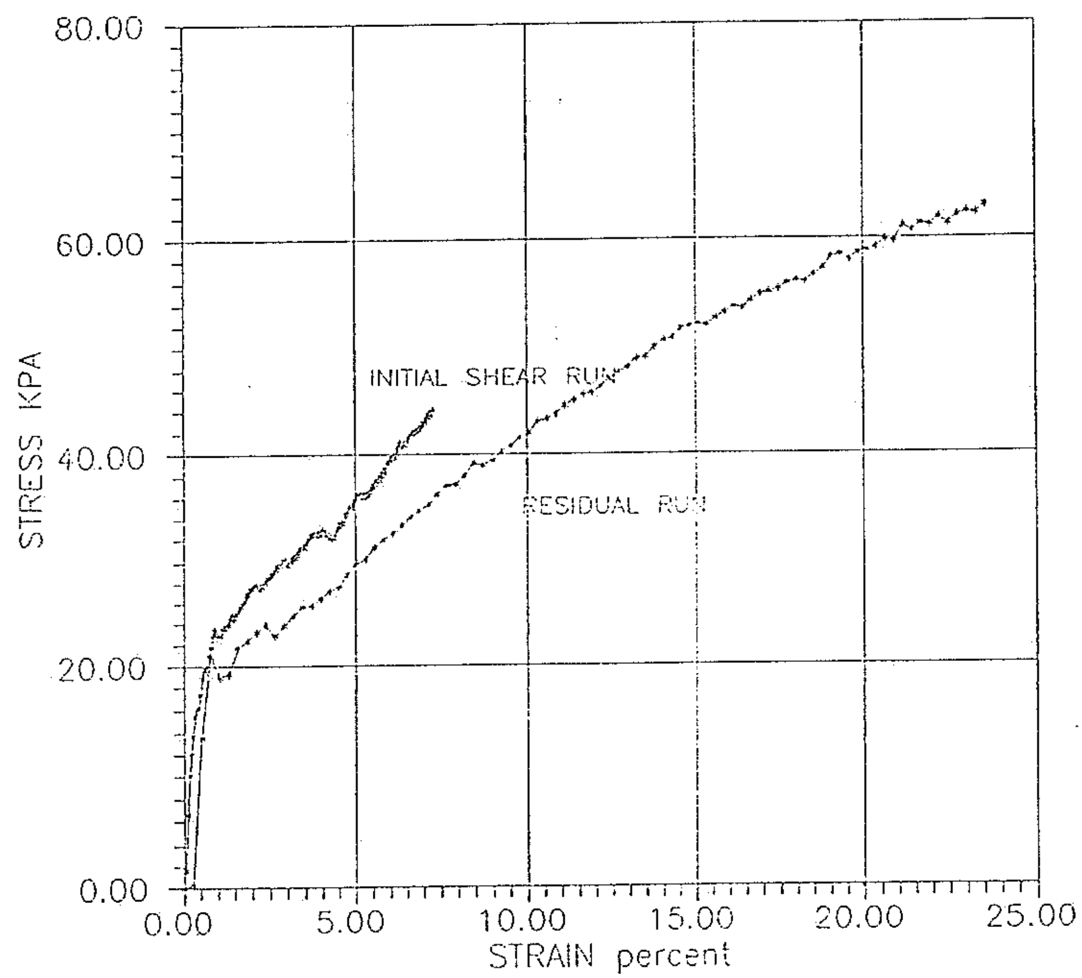
Initial Void Ratio; 1.393

Deg. of Sat. 97.1 %

A. DEAN GOULD & ASSOCIATE
Geotechnical Consultants

JOB. No. 2020	DATE: 03/25/92
CLIENT: M. M. DILLON LTD.,	
PROJECT: Portage Avenue Overpass	
LOCATION: Lower Slope of Omands Creek	
HOLE No. 8	SAMPLE No. T-15
DEPTH: 4.7m	TEST No. 2020 A

DIRECT SHEAR TEST



Soil Characteristics; Grey Clay with till inclusions, numerous tan silt pockets and small (5mm diam) stones, occasional large stone to 25mm diam, soft, plastic consistency

Strain Rate; 0.0061 mm/m Peak
0.1191 mm/m Residual

Moisture Content; 26.2 %
Unit Weight; 17.43 kN/cu.m.
Deg. of Sat. 83.2 %

Normal Load; 190.95 kPa

Initial Void Ratio; 1.393

A. DEAN GOULD & ASSOCIATE
Geotechnical Consultants

JOB. No. 2020	DATE: 03/25/92
CLIENT: M. M. DILLON LTD.,	
PROJECT: Portage Avenue Overpass	
LOCATION: Lower Slope of Omands Creek	
HOLE No. 8	SAMPLE No. T-15
DEPTH: 4.7m	TEST No. 2020 C

38/3-0

A. Dean Gould, P. Eng. & Associates
Geotechnical Consultant

September 10, 1992

RECEIVED
SEP 11 1992
M. M. DILLON LTD.

File No. 2020

M. M. Dillon Ltd.,
6 Donald Street South,
Winnipeg, MB
R3L 0K6

Attention; Mr. Sital Rihal P.Eng.

Dear Sir;

Re; Portage Omands Creek - Pedestrian Access Ramp

You have informed me that the support for the approach structures for the pedestrian walkway has been modified from a pile supported structure to a reinforced slab on grade with the following specifications;

Height of Wall(Fill surface to base of footing)		= 4.7 m
Footing Width		= 4.1 m
Contact Pressure	max	= 1500 p.s.f. (71.8 kPa)
	min	= 1000 p.s.f. (47.9 kPa)

Based upon Test Hole 1 laboratory test results;

Unconfined Compression strength	Depth	Elevation
79.4 kPa	5.03 m	230.33
117.7	6.40 m	228.96
113.6	8.08 m	227.28

From this test data, the <u>ultimate</u> load intensity the footing could sustain is 277.1 kPa Applying a Factor of Safety of 3 the <u>allowable</u> load intensity of 92.1 kPa is well above the 71.8 kPa maximum design load intensity applied. The footing is therefore considered safe against base failure.

Settlement predictions, based upon Atterberg limit correlations (consolidation data not available) suggest maximum footing design pressures(71.8 kPa) may produce long term settlement as high as 2.2 inches. The local foundation clays however are known to have been subjected to pre consolidation stress levels in excess of 145 kPa (2000 p.s.f.), therefore predicted settlements under design loading will be substantially less. In fact, rebound and swelling may develop if moisture is allowed to pond within the clay subbase. Swelling pressures can be expected to exceed the 71.8 kPa design pressure applied. To limit swelling subbase drainage is highly recommended.

Minor differential movement can be anticipated between the structural passageway through the bridge abutment and the approach ramps due to the difference in foundations. The base of footing will be within the seasonal freeze thaw zone following construction . In order not to impart structural stress to the rigid passageway, potential for differential movement should be accommodated through ramps, slip joint or corbels.

In summary, adequate bearing capacity exists in the clay soils to support the proposed ramps. Settlement is anticipated a minimal, however a joint system should be provided to accommodate differential movement resulting from swelling and seasonal moisture changes.

Should you have questions or require additional assistance, please do not hesitate to call.

Respectfully Submitted,

A. Dean Gould P.Eng.
Geotechnical Consultant