FILE COPY

REPORT

ON

WESTERN BEAUFORT REGION CONCRETE AGGREGATE STUDY

PREPARED FOR

INDIAN AND NORTHERN AFFAIRS CANADA
OTTAWA, ONTARIO

PA 2291

APRIL 1988

KLOHN LEONOFF
CONSULTING ENGINEERS

REPORT

on

WESTERN BEAUFORT REGION CONCRETE AGGREGATE STUDY

prepared for

INDIAN AND NORTHERN AFFAIRS CANADA
OTTAWA, ONTARIO

PA 2291

April 1988

KLOHN LEONOFF

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

LIST OF APPENDICES

LIST OF TABLES

LIST OF FIGURES

SECTION A - FIELD PROGRAM

1.0 INTRODUCTION

Klohn Leonoff Ltd. were successful in their response to a request for proposal received from Department of Supply and Services Canada in early September 1986. The work was carried out for the Department of Indian Affairs and Northern Development (DIAND) under the direction of the Scientific Authority (Mr. R. Gowan, P.Geol.) for the project.

The necessary field work required for sampling the six designated aggregate sources on the western shores of the Mackenzie Delta and the Beaufort Sea, generally between the Aklavik area, N.W.T. and King Point, Yukon Territory, was conducted in late September 1986 immediately after notification to proceed with the proposal.

The intent of the field program was to sample up to 6 sources of gravel in or close to the designated source areas which are described as follows in the Terms of Reference:

> "The study will consider a number of previously-identified granular sources, (Hardy, 1976, or as indicated) including, but not necessarily limited to, the following six areas located west of the Mackenzie Delta:
>
> 1) Willow River, west of Aklavik, N.W.T.:
> Deposit 467 - 68° 12'N, 135° 27'W
>
> 2) Moose Creek, Yukon, just west of Coal Mine Lake, N.W.T.:
> Deposit Y-102 - 68° 45'N, 136° 30'W.
>
> 3) Blow River, west of Whitefish Station, Yukon:
> Deposit T-93 - 68° 53'N, 137° 00'W
> Consider also: Y-86 - active flood plain
>
> 4) Shingle Point - Running River, Yukon:
> Deposit Y-85 - 68° 56'N, 137° 12'W.
> Consider also: Y-78 - scarp west of river
> Y-79 - river terraces
> Y-80 - active flood plain
>
> 5) Jacob's Lake Ridge, south of Sabine Point, Yukon:
> Deposit Y-74: 68° 59'N, 137° 38'W
> Includes part of site 7 (Klohn Leonoff, 1975)

6) King Point, Yukon:
Deposit Y-70: 60° 06'N, 138° 04'W
Originally designated site 11 (Klohn Leonoff, 1975)
Consider also: Deposit Y-62 - northwest of King Point
Deposit Y-71 - costal scarp
Y-72 - bay mouth bar"

2.0 INVESTIGATIONS

The investigations of the six sources were completed between September 25 and 28, 1986 using a Bell 206 helicopter chartered from Sunrise Helicopters in Inuvik. Each of the sources listed above were inspected and photographed from the helicopter and from the ground. Exposures were logged and selected for sampling on each source. A copy of the photographs were provided to DIAND in our report of November 21, 1986.

Sampling of the sources was done by excavating shallow test pits in exposures using pick and shovel. Between 160 and 180 kg of gravel and sand were taken from one or more of the exposures in each source. Our proposal of August 28, 1986 indicated that 50 to 100 kg of material would be taken from each source. After discussions with the Scientific Authority on September 23, 1986 in our office, the quantity of materials to be retained for sampling was raised to 160 - 180 kg per source. Frozen soil conditions during the period of the investigation limited the extent and depth of the sampling in each of the exposures. However, samples taken were judged to be representative of the gravel and sand soils that existed in each exposure. All samples were bagged, taken by helicopter to Inuvik, and shipped by truck to our Calgary laboratory.

The locations of all sources are shown on the 1:250,000 map which is attached in Appendix I. Air photographs of the 6 source areas are also attached in Appendices II to VII inclusive along with other information pertaining to that particular source.

During the reconnaissance and sampling, each of the sources were also assessed with respect to possible access, extent, future exploration and development of the source for a concrete casting plant operation.

3.0 SITE DESCRIPTION

3.1 Willow River

This source consists of 2 kame terraces or deltas located on the north side of the Willow River about 20 km directly west of Aklavik. The kame terraces or deltas are located on a generally flat bench 50 to 70 m above the level of the Willow River flood plain. The kame features were probably formed during the Early Wisconsin Glaciation by streams flowing from the northern Richardson Mountains which were blocked by the edge of the glacier covering the Mackenzie Delta and adjacent areas (Rampton, 1982[1]).

Numerous exposures of gravel and sand are present along the escarpment which forms the southern limit of the deposits. Some minor extractions of gravel have been made on and immediately adjacent to the escarpment and at the east end of the source. The thickness of overburden is generally 30 cm or less all along the escarpment and appears to be well drained. Back from the escarpment, patterned ground is present, indicating the presence of ground ice, and the thickness of fine grained overburden soils is expected to be thicker. On the west end of the source, which is slightly higher in elevation, the overburden soils observed in exposures are thicker (2 to 3 m) and probably increase in thickness away from the escarpment.

The source area is delineated on the air photograph that is attached in Appendix II. Logs of exposures taken from previous reports (Hardy 1977[2]) are included in the appendix along with Exposures No. 1a and 1b which were logged as part of this study. A photograph of Exposure No. 1a is also attached in the appendix. Exposure No. 1b was located in a bulldozer trench. Photographs of this exposure are shown in the photographic record dated November 21, 1986. See Frame Nos. 1-11 to

---

[1] Rampton, V.N., 1982 "Quaternary Geology of the Yukon Coastal Plain", Bulletin 317, Geological Survey of Canada.

[2] R.M. Hardy and Associates Ltd., January 1977. "Granular Materials Inventory, Yukon Coastal Plain and Adjacent Area"

1-15 inclusive, Frame No. 1-18 and 2-3. Samples were taken from the 2 locations for testing.

3.2 Moose Creek

This source is located along the escarpment adjacent to the MacKenzie Delta and just west of the Yukon/Northwest Territories Boundary. It extends along the escarpment for a distance of approximately 3.5 km. The gravel and sand which are exposed at several locations along the escarpment are preglacial in origin. These deposits were covered with moraine deposits during the Early Wisconsin glacial advances. Back from the escarpment, the overlying moraine deposits are gently rolling and covered with tundra containing some polygonal ground.

At the time of the first inspection on September 25, 1987, the complete area was covered by snow and gravel exposures could not be seen. Test pits were attempted in the same general location as shown in the Hardy (1977) report but were stopped in frozen clay soil at 0.3 to 0.6 m depths. On September 28, 1987, a second inspection of the site was made and a gravel and sand exposure was located about 1 mile southeast of the previously located test pits and samples were taken.

The source area is delineated on an air photograph that is attached in Appendix III. Exposure No. 2a located near Hardy source Y 102B was not sampled. Exposure No. 2b was sampled. Logs of Expsosures No. 2a and No. 2b are included in the Appendix III along with logs of test pits Y 102A and B of the Hardy (1977) report. A photograph of Exposure No. 2b is also attached in Appendix III.

3.3 Blow River

Two sources, previously investigated by Hardy (1977), were designated as potential sources for gravel and sand materials. The first Hardy (1977) source (Y-93A) is located along the escarpment just south of the area known as Whitefish Station. The escarpment in this area is 10 to 20 m in height and shows evidence of past slumping all along the potential source area. Snow cover on the ground masked all of the exposures and no suitable locations for sampling could be located. Based on the helicopter inspection and ground traversing, it was judged that most of the deposit would be covered with a thick mantle of fine grained silt and clay and not suitable for development without a detailed investigation. Thus, this source area was not sampled.

The second Hardy (1977) Source (Y-86) is located all along the active flood plain of the Blow River. While large volumes of granular materials are present within the flood plain, extraction and processing of large volumes of material is not recommended because of high water table, possibility of flooding and shallow extraction depth above the water table. The source area was not sampled.

3.4 Shingle Point - Running River

This source area was previously investigated by Hardy (1977). For this report, the sources investigated along the escarpment (Y-78 and Y-85) will be described as Shingle Point. The sources along or adjacent to the Running River (Y-79 and Y-80) will be described as Running River.

3.4.1 Shingle Point

The first source checked was the one designated as Y-78 by Hardy (1977). It is located along the coast escarpment extending 2 to 3 km northwest from the mouth of the Running River. Preglacial fluvial gravels are exposed along the escarpment under overburden which varied between 3 and 10 m in thickness. Numerous fine grained silt and sand layers were noted in all the exposures. Because of these fine grained soils, and based on the visual assessment of this source, it was judged not suitable for the production of large volumes of granular material for concrete.

The next source checked was the one designated as Y-85 in the Hardy (1977) report. Gravel outcrops along the source were poorly defined with excessive silt layers and representative samples could not be obtained due to steep slope and frozen ground conditions. This source was judged not to be suitable for development.

The area sampled was located along the escarpment opposite the Shingle Point DEW Station. This area has been developed to provide granular materials for the original and continuing maintenance construction at the station. The materials are preglacial in origin and are exposed in a 20 m face in the area developed. The moraine overburden of the top of the slope is 2 to 4 m in thickness and probably increases in thickness back from the escarpment.

The approximate source area is delineated on the air photograph that is attached in Appendix IV. The exposure is designated as Exposure No. 3. A log of the exposure and a photograph are attached in Appendix IV.

3.4.2 Running River

The Hardy (1977) report delineated potential sources along the active flood plain and in the adjacent glaciofluvial terraces of the Running River. As discussed previously for the Blow River Source in Section 3.3 above, the development of the source in the active flood plain (Hardy Y-80) is not considered to be practical. Accordingly the glaciofluvial river terraces along the Running River and source Y-79 Hardy (1977) were investigated. Apart from the exposures and test pits logged by Hardy, the best gravel exposure observed by us was up to 13 m thick and extended along the east side of the Running River flood plain on both sides of a junction with a small tributary stream. Thick gravel and sand underlie up to 5 m of moraine overburden. This exposure has been designated as Exposure No. 4.

The locations of the river terrace sources along the Running River is shown on the air photograph attached in Appendix V. Logs of test pits and exposures from Y-79 Hardy (1977), Exposure No. 4, and a photograph showing a cross section through the exposure are attached in Appendix V.

3.5 Jacobs Lake Ridge

This source is an outwash fan formed at the edge of the retreating glacier during the Early Wisconsin glacial period. The source is up to 10 km in length and approximately 0.5 m in width and runs parallel to, and 3 to 4 km from, the Beaufort Sea shore line.

Numerous exposures of gravel are present along the northern and southern edges of the narrow deposit. The top of the ridge is generally flat to rolling and contains some patterned ground probably indicating high ice contents.

The Hardy (1977) investigation excavated a series of test pits along the length of the area designated as Y-74. Klohn Leonoff sampled exposures at widely spaced intervals in the same general area which was designated as Potential Source Area 7. Logs of exposures were not required as part of that assignment. An additional gravel exposure up to 15 m in height (Exposure No. 5) was logged and sampled as part of the present investigation.

The approximate source area is delineated on the air photograph that is attached in Appendix VI. The air photograph outlines the Hardy (1977) Y-74 source and part of the Klohn Leonoff Potential Source Area 7.

Logs of test pits taken from the 1977 Hardy report are included in Appendix VI along with Exposure No. 5 which was logged on the shore of Jacobs Lake. All data relating to Potential Source Area 7 is also included in the appendix. A photograph of Exposure No. 5 is also attached.

3.6 King Point

Four sources, previously investigated by Hardy (1977) and Klohn Leonoff 1975[3] were designated as potential sources for gravel and sand materials. The Hardy (1977) source Y-71, was located along the coast escarpment and consisted of preglacial gravels and sands. Inspection showed this deposit was extremely erratic and contained layers of silt organics and clay within the clean gravel and sand deposits. Overburden thickness was variable and extensive slumping in the overburden soils was also present. This source was not considered to be developable.

Hardy Source Y-72 was a long narrow bar along the coast at King Point. Development of this source to provide large volumes of gravel and sand on a consistent basis is not judged to be practical.

Hardy Source (Y-70) and Klohn Leonoff (No. 11) were closely inspected. Both reports describing this source discuss the variability of the deposit, existence of massive ground ice and thick fine grained overburden soils. The source is not judged to be suitable for large scale development.

Hardy Source Y-62 consists of a series of coalescing outwash fans 4 to 5 km in length and up to 0.4 km in width. The source runs parallel to, and 1 to 2 km from the coast. It is about 10 km northwest of King Point. This source area was judged to have the best potential for future development.

The source area is delineated in the air photograph attached in Appendix VII. Logs of test pits taken from the Hardy (1977) report for the source are included in the appendix along with Exposure No. 6 which was logged on the north slope of a lake bordering the south end of the source. Exposure No. 6 was also sampled. A photograph showing the exposure described as Y-62D(e) in the Hardy (1977) report is attached in the appendix.

---

[3] Klohn Leonoff Consultants Ltd. 1975 "Gravel Search Program Yukon Coastal Plan, King Point Area, Phase II, Imperial Oil Limited.

4.0 EXPLORATION

All of the data relating to the granular sources investigated for this project consisted of visual inspections, logging of exposures and excavations of shallow test pits. The review of "Rampton's" (1981) report suggests that the Moose Creek, Shingle Point and the east side of Running River Sources are preglacial in origin and covered with glacial moraine or lacustrine deposits. The remaining sources; Willow River, west side of Running River, Jacobs Ridge, and King Point are glaciofluvial in origin and were formed during the early Wisconsin glaciation period.

To our knowledge, no detailed test hole drilling investigations have ever been completed in any of the source areas. Prior to development, a detailed investigation program will be required to determine the following:

- thickness of overburden
- thickness of gravel and sand deposits
- permafrost conditions (active layer thickness, ice content, degree of ice bonding) in overburden soils and in the granular deposits. The presence of massive ground ice should also be known
- the quality and quantity of the gravel

The investigation will require a rotary air drill to drill a series of holes in a grid pattern (not more than 50 m) over the source area. Samples would be taken on a 1 m spacing to the bottom of the drill hole. Disturbed samples should be taken from the air return and visually classified to estimate ice content and material type. Coring of gravel samples should be attempted to confirm ice content at selected intervals. Selected samples would be retained for additional testing. The size of the area to be explored is dependent upon the volumes of the granular materials required and the depths of usable materials that are confirmed during the drilling program.

The extent of a drilling program to confirm the quality and quantity of materials at the selected sources would have to be throughly planned prior to the start of the work.

The results of the laboratory testing program (See Table 1 Summary of Test Results in Section B.) suggests an order of ranking from best to worst. Apart from location and distance from the Beaufort Sea, the results generally show that Willow River and Moose Lake are the worst of the six sources tested. Thus, additional exploration at these 2 sources is not recommended. The remaining 4 sources appear to be suitable for production of concrete aggregates. It is recommended that additional exploration be carried out at these 4 sources.

The source outlines are based on air photo interpretation, inspection of exposures and excavation of shallow test pits. Whether the source outlines are correct or exposures are representative of the gravel materials to be encountered within the source area can only be determined by detailed investigation programs.

Two stage programs for investigation are recommended. The first stage would include the drilling of test holes in each of the 4 sources to confirm that the required quantity and quality could be obtained from the area. The first stage test holes would be widely spaced on a random basis and would cover an area large enough to confirm that sufficient volumes of aggregate are available in the source.

Based on the results of the first stage program, the best of the 4 sources could be identified and a detailed second stage program with test holes at a 50 m interval could be initiated in the selected source.

The first stage investigation would start in the area of the major exposure checked during this program. These are:

Shingle Point	Exposure No. 3
Running River *	Exposure No. 4
Jacobs Ridge *	Exposure No. 5
King Point *	Exposure No. 6

* If the results of initial test holes at each of these exposures are visually assessed to be not suitable, the exposures listed in the Hardy (1977) report would then be checked.

Sketches of the suggested First and Second Stage program are shown on Plate No. 2 and 3, Appendix I.

Timing of the first or second stage programs would be the decision of the client. It is our suggestion that separate programs should be conducted for the first and second stage. In the first stage the widely spaced holes would confirm the quantity of gravel. Laboratory testing would provide additional data on quality of the aggregate. Based on the results of the first stage, the best source could be identified and the second stage investigation and laboratory testing could be then carried out.

Regardless of the stages in the field program the best time for the investigation would be in March-April when winter roads would probably be located close to the areas. The drilling would be done on a 24 hour basis and would have to be supported by bulldozer and a self sufficient camp.

5.0 DEVELOPMENT

Some of the factors which must be considered in the development of a processing facility to produce concrete aggregates in this region are volumes of material, the time frame and the permafrost conditions that will be present. The fact that the natural gravel and sand soils will be frozen probably dictates that most of the excavations will have to be ripped or blasted, placed in stockpiles and allowed to thaw before using.

It is likely that the production of aggregate and operation of the concrete plant would be restricted to the summer months.

The scale of the gravel pit area, the aggregate processing facility, concrete batch plant, precast plant curing and storage areas, etc. will be extensive. A typical set up would have to provide for some or all of the following:

i) In the Gravel Pit Area

- defined limits of gravel area to be worked;
- area for stockpiling of overburden soils. If overburden soils are ice rich, siltation ponds may be required to collect water from thawing permafrost;
- provision for drainage within the pit.

ii) In the Aggregate Processing Area

- stockpile of unprocessed gravel and sand;
- thawing of frozen material before processing;
- crushing, screening of aggregates into different sizes, conveyors and stockpiles of processed aggregates.

iii) In the Concrete Batch Plant Area

- cement storage;
- storage bins for aggregates;
- scale, mixer and discharge.
- water supply

iv) Precast Plant

- precasting area;
- curing;
- storage of completed units;
- loading area.

v) Miscellaneous

- camp, equipment servicing and maintenance facilities.

The land area required for the complete processing plant could be in the order of 5 Ha depending on the scale of the operation. The site would have to be located on generally flat and thaw stable terrain close to the gravel pit or close to a loading facility on the Beaufort Sea shore. From visual inspections, it is judged that these conditions are present at the Running River, Jacobs Ridge or King Point sources. At Shingle Point, terrain is more rolling and patterned ground is present. Helicopter flights around the DEW station were severely restricted. The Shingle Point site was therefore selected from air photographs.

The area of the gravel pit to be developed depends on the results of the exploration program. The locations suggested in each of the four prospective sources for the initial pit development area, gravel stockpiles, aggregate and concrete production, fabrication, curing and stockpiling of completed units are shown on the 1:50,000 plans contained in Appendices IV to VII inclusive.

A typical conceptual layout for the stockpile area, aggregate and concrete production, and fabrication area is shown on Plate I-4 in Appendix I.

6.0 ACCESS

Assuming a summer operation for the concrete plant, the precast units will be transported by barge to the off-shore locations in the Beaufort Sea during the shipping season. Barge loading points could be established as close as possible to the concrete processing plant. Barges are presently off loaded at the existing Shingle Point DEW Radar Station.

Access from the individual sources to the nearest barge loading point would be as follows.

Willow River

Access from this source to the Beaufort Sea during the summer would be difficult. The road would be located across the gently sloping terrain to the east of the site as far as the MacKenzie Delta, thence it would cross the Willow River and travel south along the edge of the delta to the Husky Channel for loading on the barge and transporting to the Beaufort Sea. The approximate length of the access road would be 16 km.

The alternative would be to transport the units via winter road to Aklavik and thence by barge during the summer months along the MacKenzie River to the sites on the Beaufort Sea. The locations of the suggested summer and winter access roads are shown on Drawing B-2291-1 in Appendix I and on Plate II-1, Appendix II.

Moose Creek

Access from the source to the Beaufort Sea during summer would be difficult unless some of the channels adjacent to the escarpment were navigable by barge. It not, transportation by winter access roads (35 to 40 km) to the nearest barge loading facility at Shingle Point for loading on to barges during the shipping season would be necessary. See Drawing B-2291-1 in Appendix I for location of the winter road.

Shingle Point

This source is located on the shoreline of the Beaufort Sea. Barges are presently loaded and unloaded from the shore at this DEW Radar Station. See Plate IV-1 in Appendix IV.

Running River

Access from this site to the existing Shingle Point loading facility would be along a road (7 km long) crossing the gently rolling terrain in a north easterly direction to join existing roads serving the DEW Radar Station. See Plate V-1, Appendix V.

Jacobs Ridge

This site is about 4 km from the ocean shore. The closest road access from the concrete plant would cross gently sloping terrain and thence along a drainage course to the shoreline. Barge loading facilities could probably be established along the ocean shore. The alternatives would be to use existing facilities at Shingle Point Dew Station (15 km southeast) or at the proposed facilities to be established at King Point (20 km northwest). See Plate VI-1, in Appendix VI.

King Point

Road access from the source directly to the coast (1-2 km) would have to cross high unstable banks to reach the ocean shore. The suggested road access from this source to the ocean would be in a southeasterly direction to the proposed facility at King Point. The access road would parallel the coast and cross gently rolling terrain to King Point. The length of this road would be in the order of 10 to 12 km. See Plate VII-1 in Appendix VII.

From the point of view of access to the Beaufort Sea, it appears that it would not be practical to develop the sources at Willow River or Moose Creek particularly since the remaining four sources investigated have better quality aggregates and are closer to the Beaufort Sea. Regarding the Shingle Point source, there may be restrictions on the use of the source and the existing barge loading area. If so, and depending on where loading facilities are established, the source with the closest access to the Beaufort Sea would be King Point, followed by Jacobs Ridge and Running River.

SECTION B - LABORATORY PROGRAM

7.0 INTRODUCTION

The laboratory analyses proceeded through the proposed tests which were considered to be in two categories - standard or basic tests as well as some less commonly conducted tests referred to as non standard. Some of the tests, by definition, require an extended time frame as detailed in the proposal.

This section of the report is intended to present the data available to complete the project up to the time of writing in accordance with the terms of our proposal. Additionally, based on results disclosed by this testing program a proposal for further required testing is presented.

8.0 PRESENTATION OF TEST RESULTS

8.1 Overview

The results of all tests, both basic and non standard, are summarized in the following Table 1 on a Pass-Fail basis in an attempt to present an overview.

The table was based in turn on the summaries of the individual tests as noted in the following three subsections.

8.2 Basic Tests

The field samples arrived in the laboratory as pit run material. The results of the first analyses on the basis of grading alone are listed in Table 2.

Included in Appendix VIII are the actual detailed gradings plotted for the pit run aggregate. Appendix IX includes plots of the conformance of coarse and fine portions of each of the samples, with the respective CSA specifications for coarse and fine concrete aggregate. The fine and coarse portions were prepared by splitting on a No. 4 (4.75 mm) sieve.

In the case of the gradings for the coarse fraction in terms of concrete use we have plotted conformance with two standard grading designations - nominal 38 mm and nominal 25 mm. In each case the same coarse aggregate sample was used except that all +38 was removed to show the gradings for each of the two maximum allowable sizes.

Table 3 lists the test results of the basic tests conducted on the fine aggregate portion while Table 4 summarizes the results of the same tests conducted on the coarse fraction.

TABLE 1
SUMMARY OF TEST RESULTS

Description of Test	Test I.D.	SOURCE IDENTIFICATION					
		1 Shingle Point	2 Running River	3 King Point	4 Jacobs Ridge	5 Moose Creek	6 Willow River
Soundness	A	P	P	P	P-B	P	P-W
L.A. Abrasion	B	P	P	P	P-B	P	P-W
Petrographic	C	F	F	F	F	P-B	F-W
Density	D1	P	P	P-B	P-B	P	P-W
Absorption/Coarse	D2	P	P	P	P	P-B	P-W
Absorption/Fine	D3	P	P	P-B	P-B	P	P-W
Durability Absorption Ratio	E1	P	P	P	P-B	P	P-W
Durability Index	E2	P	P	P	P-B	P	F-W
Organic	E3	P	P	F	F-W	P	F-W
Cleanness Coarse	F1	P	P	P	P-B	F-W	F-W
Cleanness Fine	F2	P	P	P	F	F-W	F
3 mos. Expansion Fine	G1	P	P	P	P-B	F-W	P
6 mos. Expansion Fine	G2	P	P-B	P	P	P-W	P
3 mos. Expansion Coarse	H	P	F	P-B	F-W	P	P
Expansion Brine	I	F-W	P-B	F	P	F	P
3 mos.		0.03	0.007	0.025	0.019	0.027	0.019
6 mos.		0.04	0.015	0.030	0.024	0.036	0.026

NOTE: P denotes pass — P-B denotes pass - best performance
P-W denotes pass - worst performance

F denotes fail — F-W denotes fail - worst performance

TABLE 2

SUMMARY OF PIT RUN GRADING ANALYSES

Sample No.	Source Name	% Sand Minus 5 mm (No. 4)	% Oversize Plus 40 mm	% Coarse Aggregate (40 mm - 5 mm)
1	Shingle Point	33.5	3.5	63.0
2	Running River	26.1	16.7	57.2
3	King Point	36.9	11.3	51.8
4	Jacobs Ridge	40.8	5.9	53.3
5	Moose Creek	26.3	17.4	56.3
6	Willow River	29.3	20.8	49.9

TABLE 3

FINE AGGREGATE TEST SUMMARY

Sample No .	Source Name	% Sand Finer Than 80um (#200)	F.M.	Organic Plate No.	% Low Density Material	Relative Density at S.S.D.	Absorp-tion %
1	Shingle Point	3.1	3.48	2	-	2.55	2.2
2	Running River	4.7	3.54	2	-	2.51	3.0
3	King Point	4.6	3.20	5 (4)*	0.42	2.54	2.0
4	Jacobs Ridge	8.2	2.84	5 (5)*	0.13	2.55	2.0
5	Moose Creek	16.5	2.94	1	-	2.55	4.1
6	Willow River	12.2	2.67	5 (5)*	0.10	2.48	5.1

* After removal of lightweight pieces

TABLE 4
COARSE AGGREGATE TEST SUMMARY

Test No.	Description of Test	Reported Value	SOURCE IDENTIFICATION						Allowable Value by Specification
			1 Shingle Point	2 Running River	3 King Point	4 Jacobs Ridge	5 Moose Creek	6 Willow River	
A	Soundness Test by MgSO4 CSA A23.2-9A	% weight loss	1.95	0.65	2.84	0.40	3.69	7.75	Maximum of 12%
B	Los Angeles Abrasion Test CSA A23.2-16A	% weight loss	22.7	21.8	21.2	19.8	20.3	26.2	up to 50% (best wearing characteristics if up to 35%)
C	Petrographic Analysis CSA A23.2 Appendix 'B'	Petrographic Number (PN)	171	165	202	181	109	250	100-110 Excellent 111-125 Good 126-140 Fair 141-155 Poor
D	Relative Density and Absorption CSA A23.2-12A	(Sp Gravity) R Density at SSD	2.60	2.61	2.62	2.62	2.59	2.57	no formal limits
		Absorption %	1.21	1.11	1.26	0.99	0.93	1.83	no formal limits
E	Durability Index	Durability Absorption Ratio	28	34	27	40	28	11	Minimum 10; best if greater than 24
		Durability Index	62	73	62	80	54	32	52 minimum
F	Cleaness of Aggregate California Test Method 224		48	24	24	79	3	3	Higher the better but no specified limits.

8.3 Standard Alkali-Reactivity Tests

The standard tests for coarse aggregate were conducted according to CSA A23.2-14A Supplement No. 2 - 86 "Alkali-Aggregate Reaction".

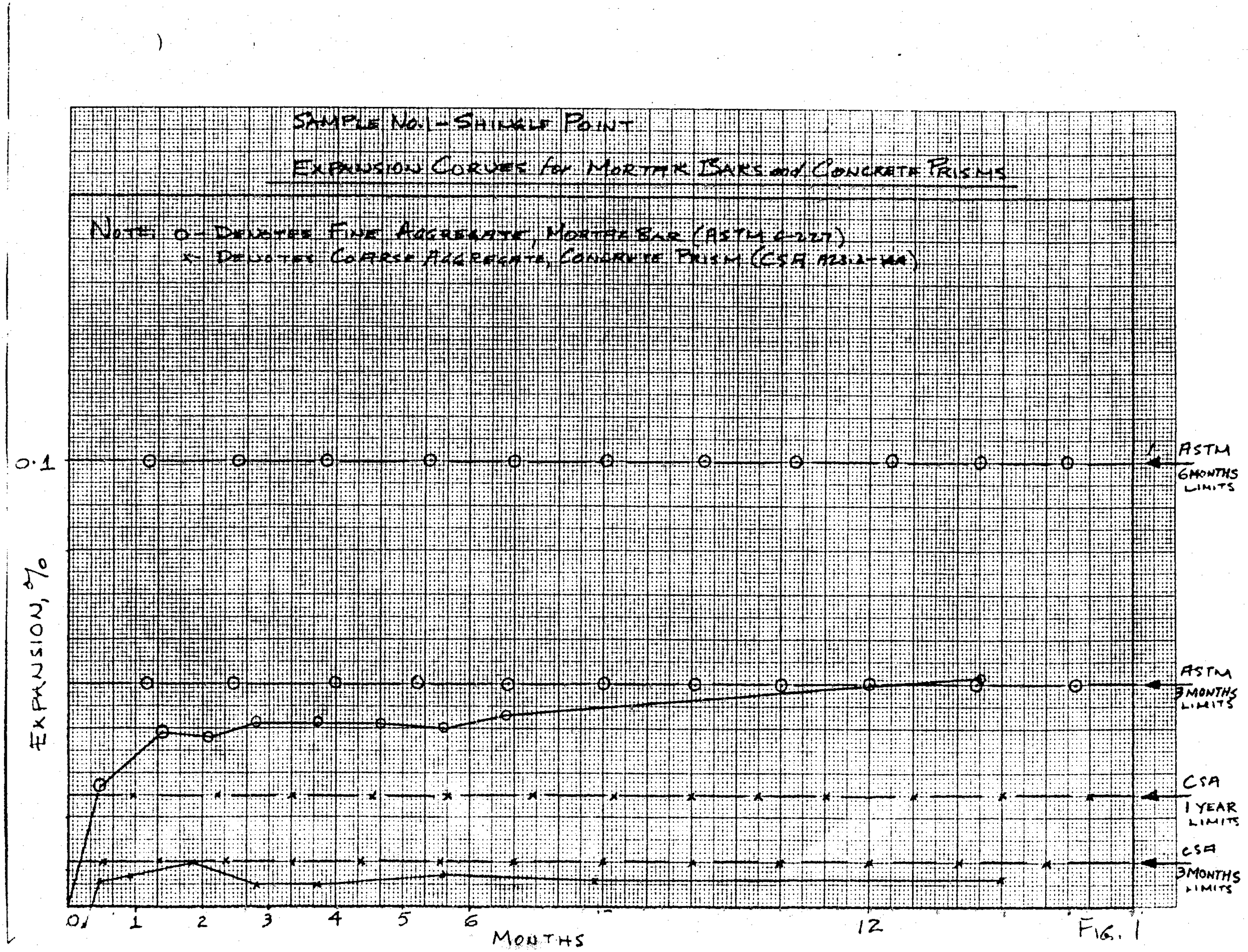
The mortar bar tests for fine aggregate were conducted according to ASTM C-227 "Standard Test Method for Potential Alkali Reactivity of Cement - Aggregate Combinations (Mortar Bar Method)", since there is no comparable CSA method at present.

Maximum allowable expansion limits for both the fine (ASTM C-227) and coarse (CSA A23.2) aggregates for each of the six sources on the attached rough graphs, Figures 1 to 6 inclusive. This is the format recommended in the new CSA Supplement. Also, on each graph we plot the expansive readings to date for each of the six sources of aggregate sample.

Specifically, ASTM limits expansion of fine aggregates to 0.05% after 3 months and 0.10% after 6 months.

Coarse aggregate, with a lesser specific surface than fine aggregate, has lower limits since there is not the same amount of particularly expansive material exposed chemically to the alkalis or other materials causing expansion. For that reason, CSA limits the three month expansion of coarse aggregate to only 0.01% and 0.025% after one year.

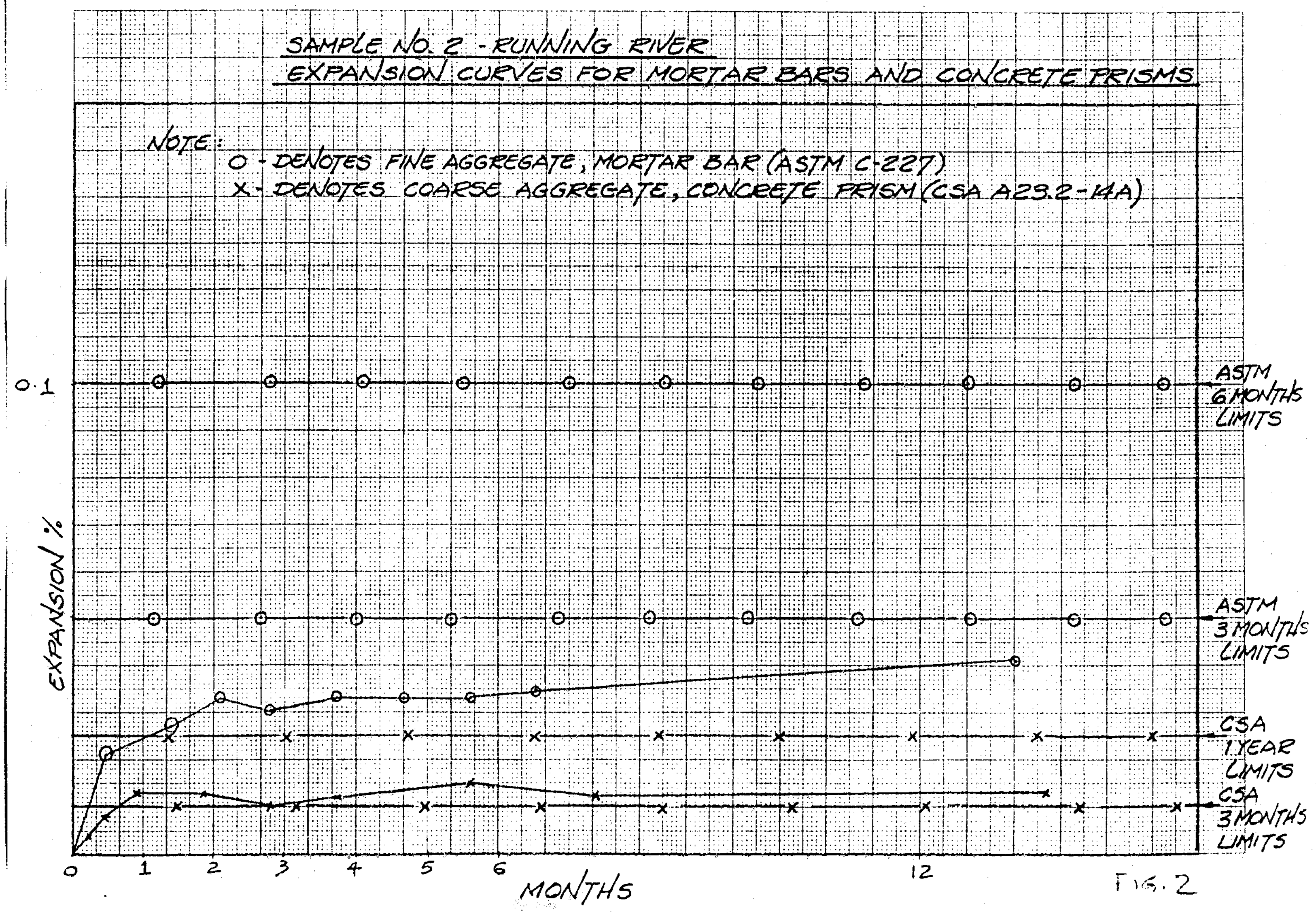
That is to say aggregate expansion causes disruption in concrete by causing excessive strain and hence excessive stress beyond the ultimate tensile strength of only the mortar fraction of concrete. The aggregate itself is not fractured, only the cement paste portion of the mortar which by definition involves only the sand fraction of aggregate and not the coarse portion. The mortar fraction comprises about one quarter to one fifth of the total concrete volume and it is therefore for this reason that maximum expansion of concrete prisms is limited to only about one quarter to one fifth of the limiting expansion of mortar bars.



SAMPLE No. 1 – SHINGLE POINT

EXPANSION CURVES for MORTAR BARS and CONCRETE PRISMS

FIG. 1



SAMPLE NO. 2 - RUNNING RIVER
EXPANSION CURVES FOR MORTAR BARS AND CONCRETE PRISMS

NOTE:
O - DENOTES FINE AGGREGATE, MORTAR BAR (ASTM C-227)
X - DENOTES COARSE AGGREGATE, CONCRETE PRISM (CSA A23.2-14A)

FIG. 2

SAMPLE NO. 3 - KING POINT

EXPANSION CURVES FOR MORTAR BARS AND CONCRETE PRISMS

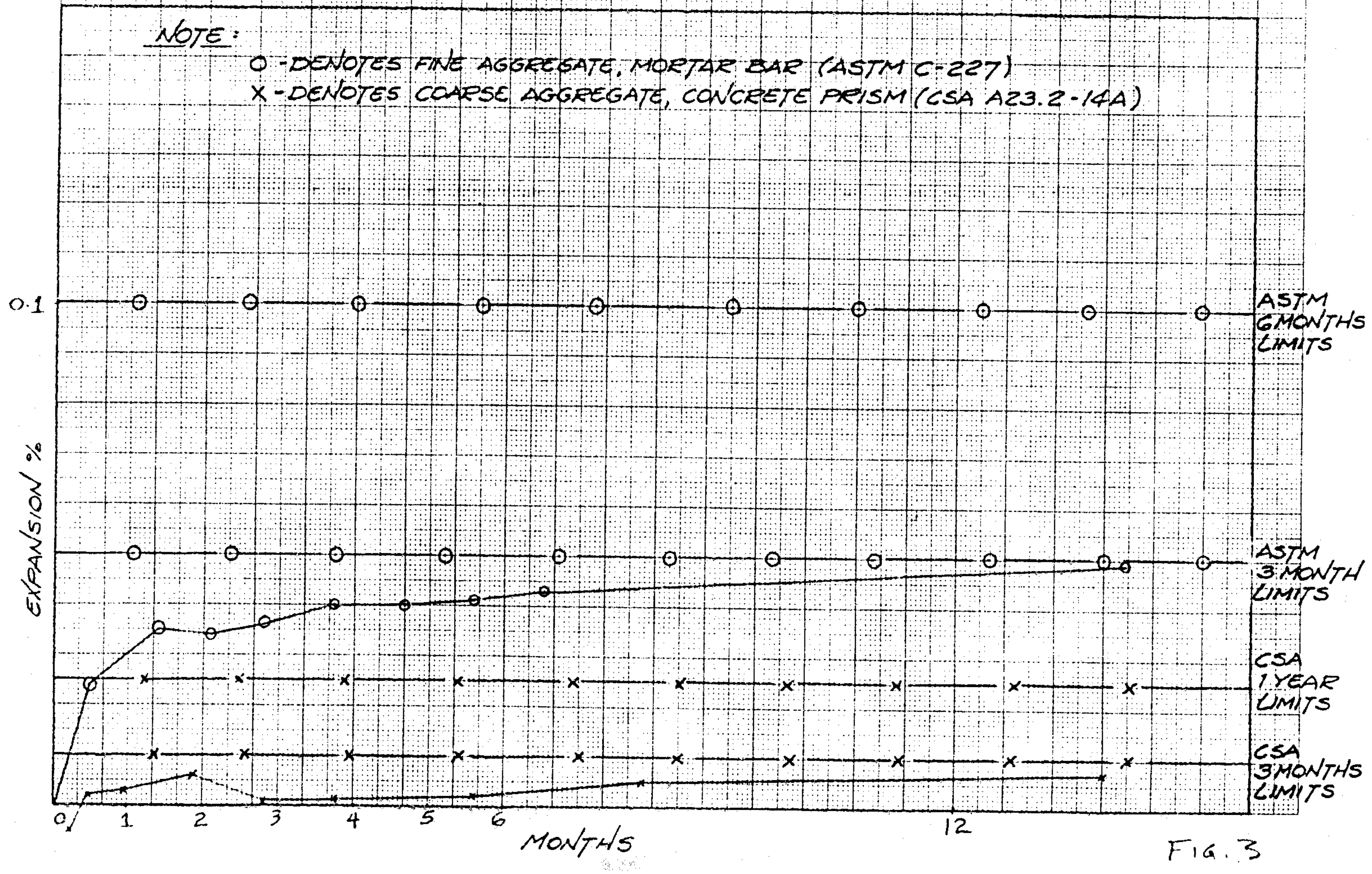
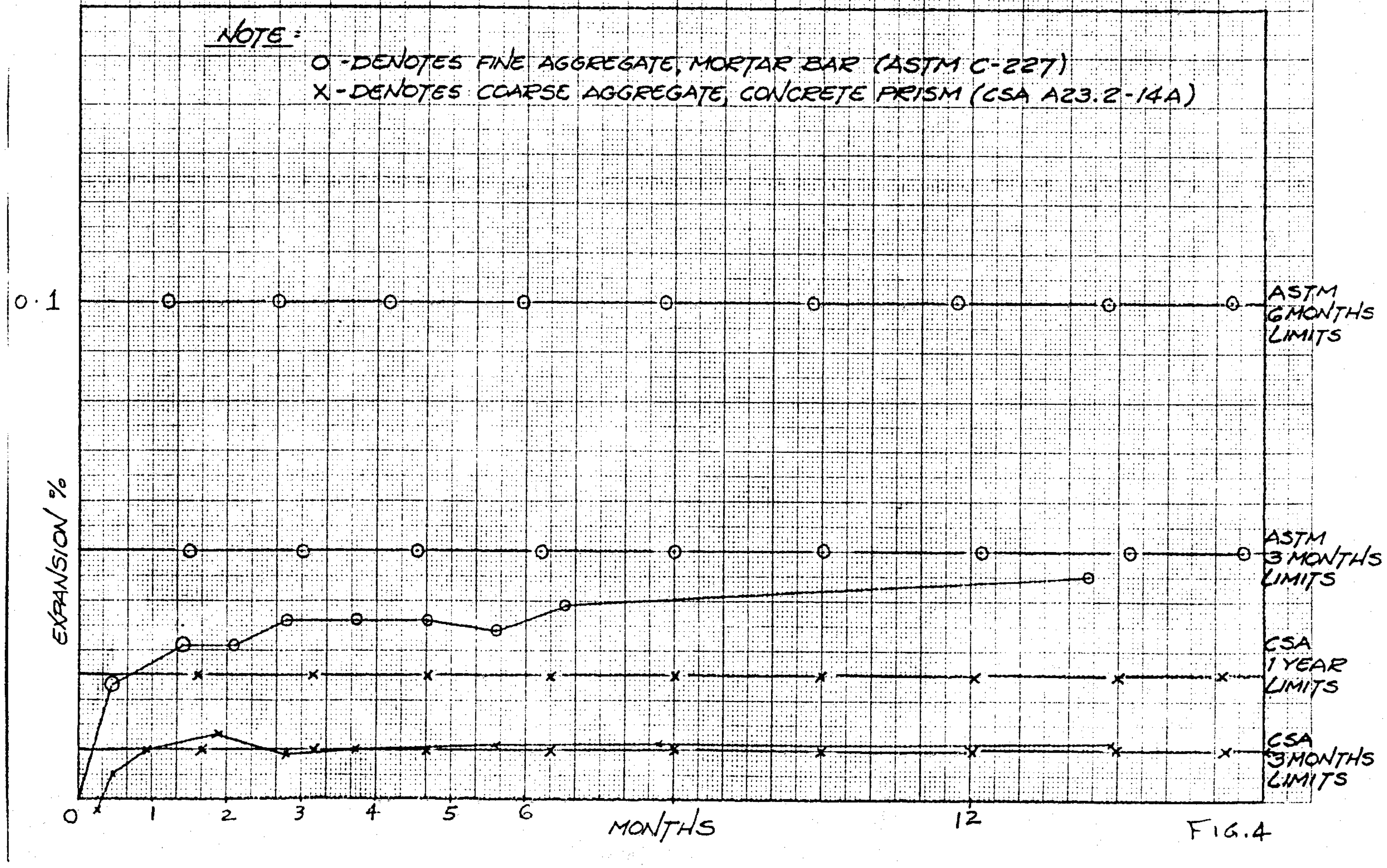


FIG. 3

KLOHN LEONOFF

SAMPLE NO. 4 - JACOBS RIDGE
EXPANSION CURVES FOR MORTAR BARS AND CONCRETE PRISMS



NOTE:
O - DENOTES FINE AGGREGATE, MORTAR BAR (ASTM C-227)
X - DENOTES COARSE AGGREGATE, CONCRETE PRISM (CSA A23.2-14A)

FIG. 4

SAMPLE NO. 5 COAL MINE LAKE - Moose Creek
EXPANSION CURVES FOR MORTAR BARS AND CONCRETE PRISMS

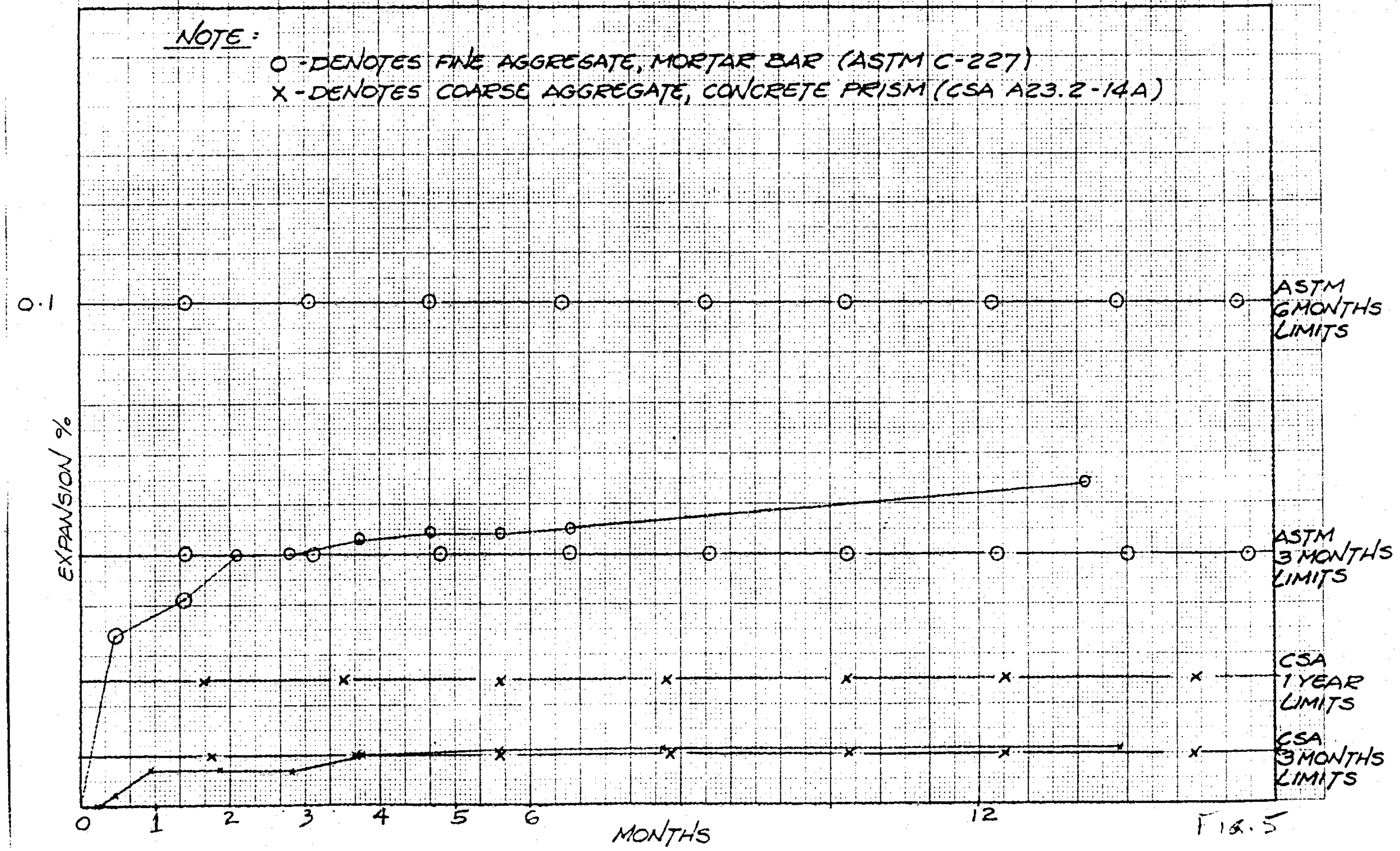


Fig. 5

SAMPLE NO. 6 - WILLOW RIVER

EXPANSION CURVES FOR MORTAR BARS AND CONCRETE PRISMS

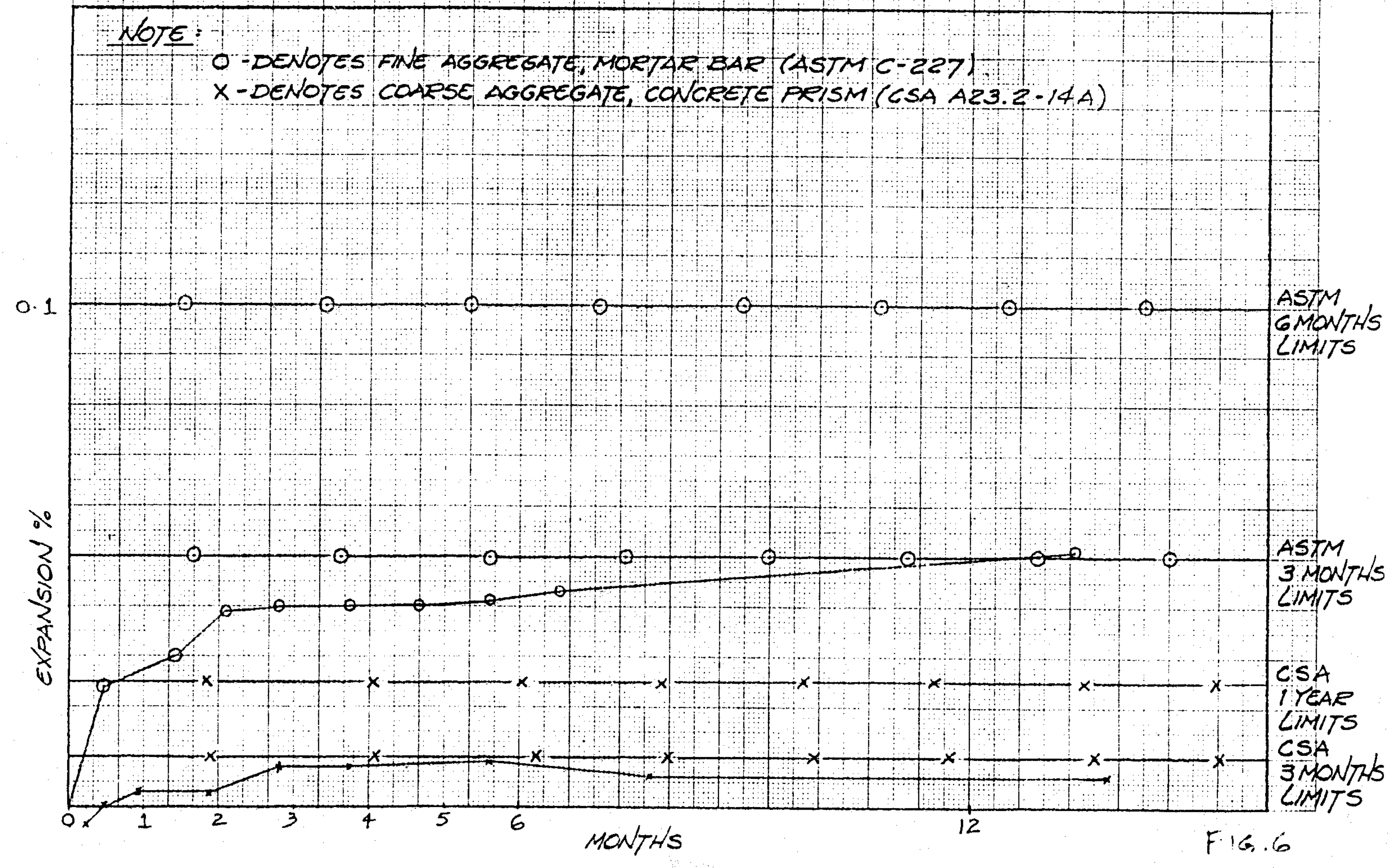


FIG. 6

8.4 Non Standard Brine Reactivity Tests

This study is concerned with the potential use for aggregate from this area on the edge of the Beaufort Sea for concrete immersed in sea water. This project therefore included the immersion of cement-aggregate prisms in a brine solution containing ice. In other words an attempt has been made to simulate subsurface exposure conditions for that eventuality. The brine was purchased from a marine life retail store and is purported to simulate sea water salinity.

By the same token the use of ice in the brine was intended to approximate Beaufort Sea conditions when in an other than frozen state. That is to say there are many periods when shore ice and pack ice are floating in the Beaufort presumably with the saline water at a temperature near 0°C.

The brine ice and samples were , and are therefore continuing to be kept in a laboratory freezer maintained at 0°C.

By definition a non standard test of this nature has no formally set standards for the allowable length change of concrete prisms subjected to this artificially created environment. At the same time the results are of particular interest in this case. The samples were prepared and measurements taken in the same manner as the standard test.

Data available to date on aggregate from each source is presented in graphical form on the following pages. See Figures 7 to 12 inclusive.

Since there are no standard limits to results of a non standard test procedure one must rely on first principles in interpreting the results. Therefore, although no limits are shown on the graphs of Figures 7 to 12 we have used the basic criteria used in establishing both the ASTM and CSA limits in setting our unofficial criteria for the Pass-Fail evaluation listed for this test in Table 1.

Our Pass-Fail criteria for this test was thus arbitrarily taken as an upper limit of expansion in three months in the order of 0.016% being related to CSA's 0.01% and ASTM 0.05/4=0.012%. That is to say because we can't be precise we must be less rigid in our limit than is the case with more extensively documented standard tests.

SAMPLE NO 1 – SHINGLE POINT
EXPANSION CURVE – NON STANDARD* ALKALI REACTIVITY TESTING

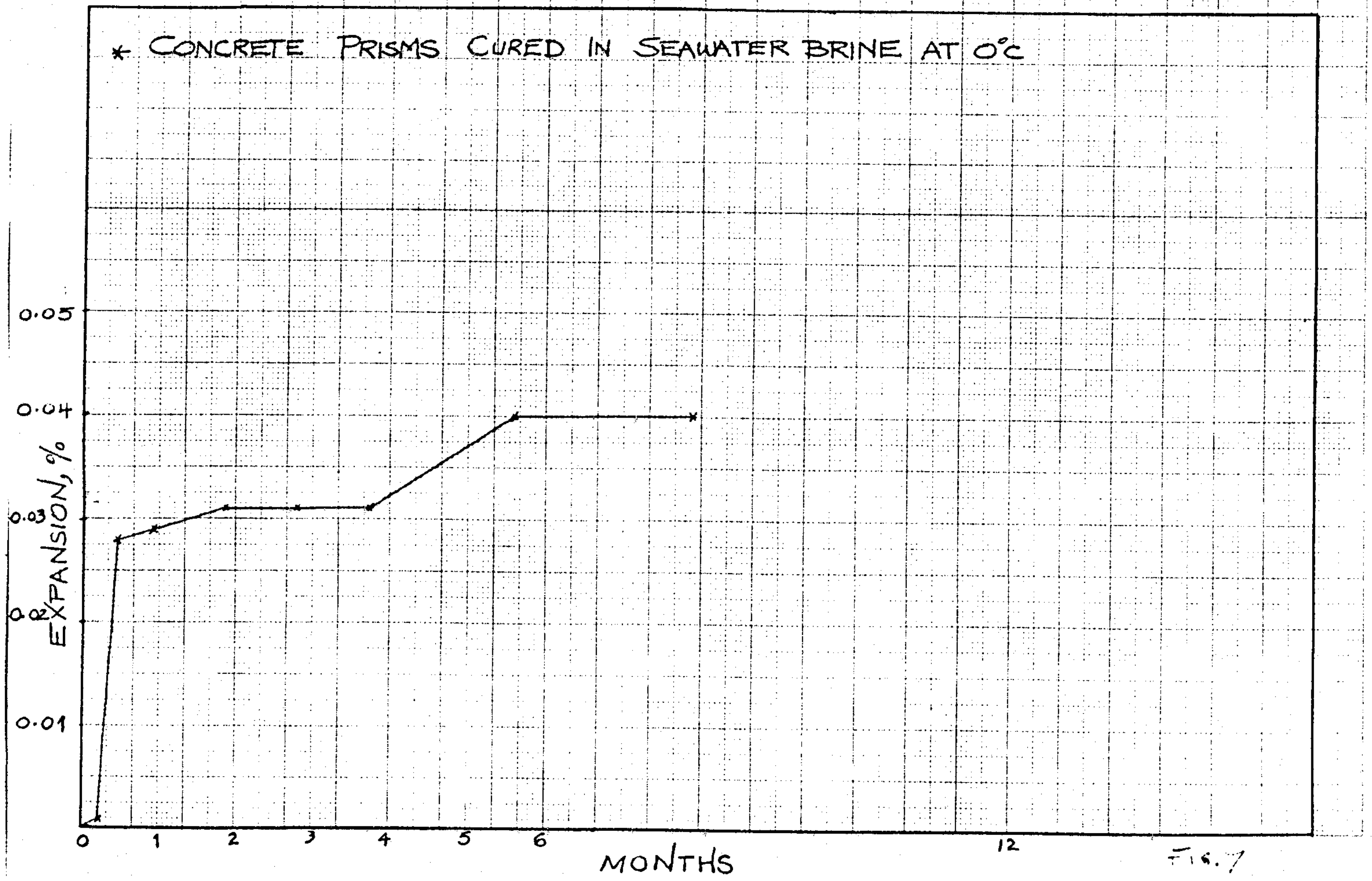
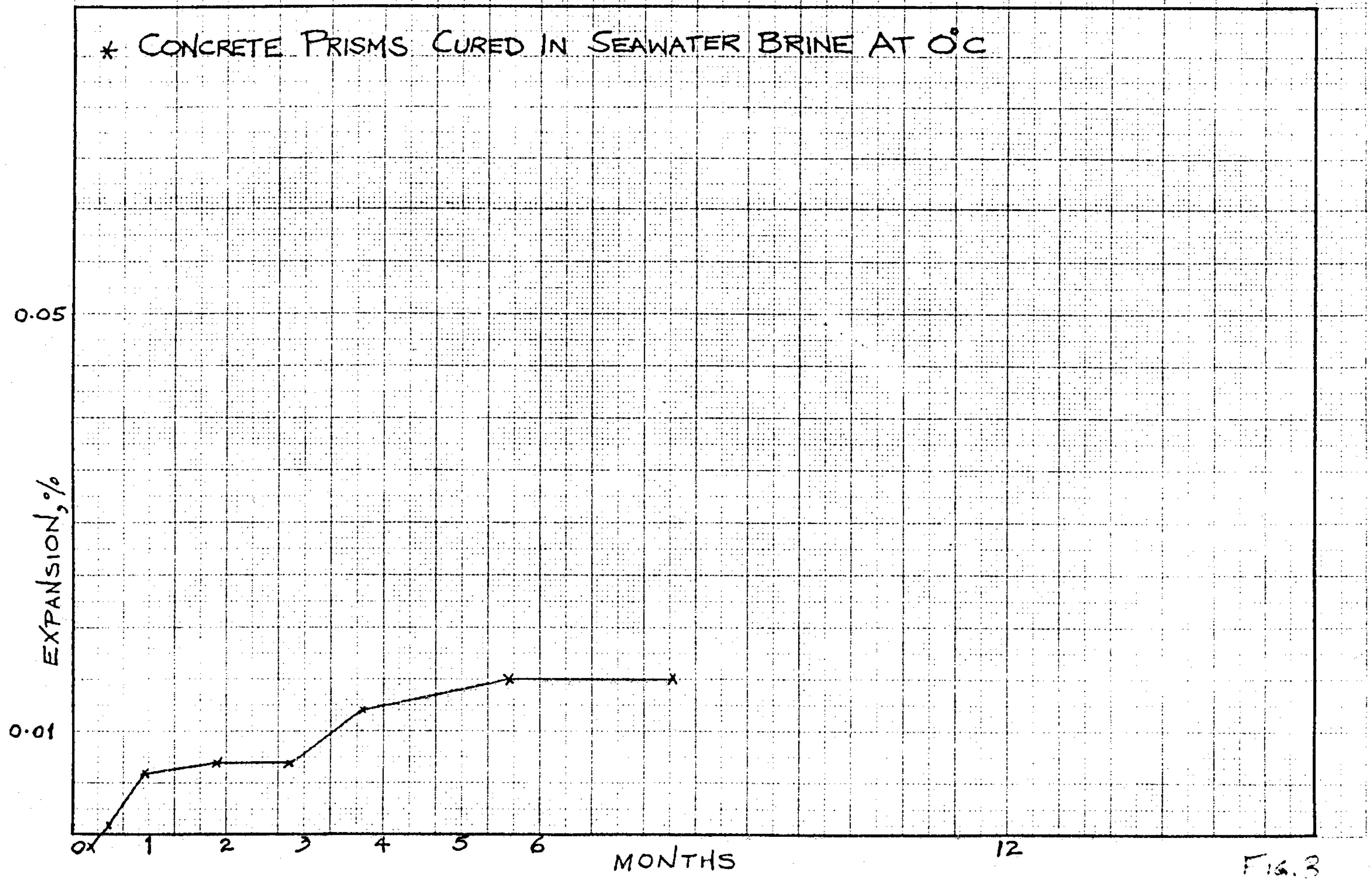


FIG. 7

SAMPLE NO 2 – RUNNING RIVER

EXPANSION CURVE – NON STANDARD* ALKALI REACTIVITY TESTING



* CONCRETE PRISMS CURED IN SEAWATER BRINE AT 0°C

FIG. 3

SAMPLE NO. 3 – KING POINT

EXPANSION CURVE – NON STANDARD* ALKALI REACTIVITY TESTING

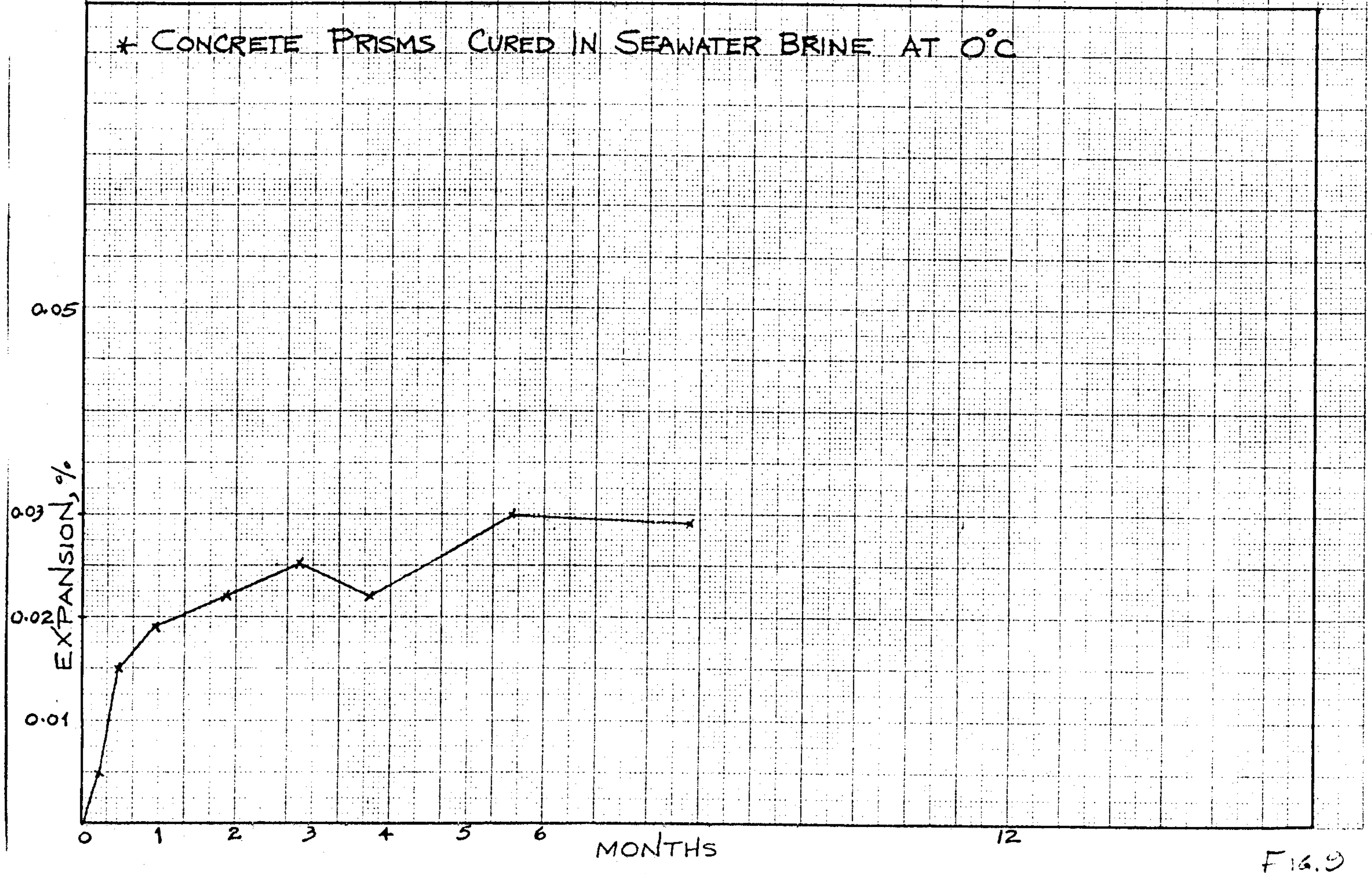


FIG. 9

SAMPLE NO. 4 - JACOBS RIDGE
EXPANSION CURVE - NON STANDARD* ALKALI REACTIVITY TESTING

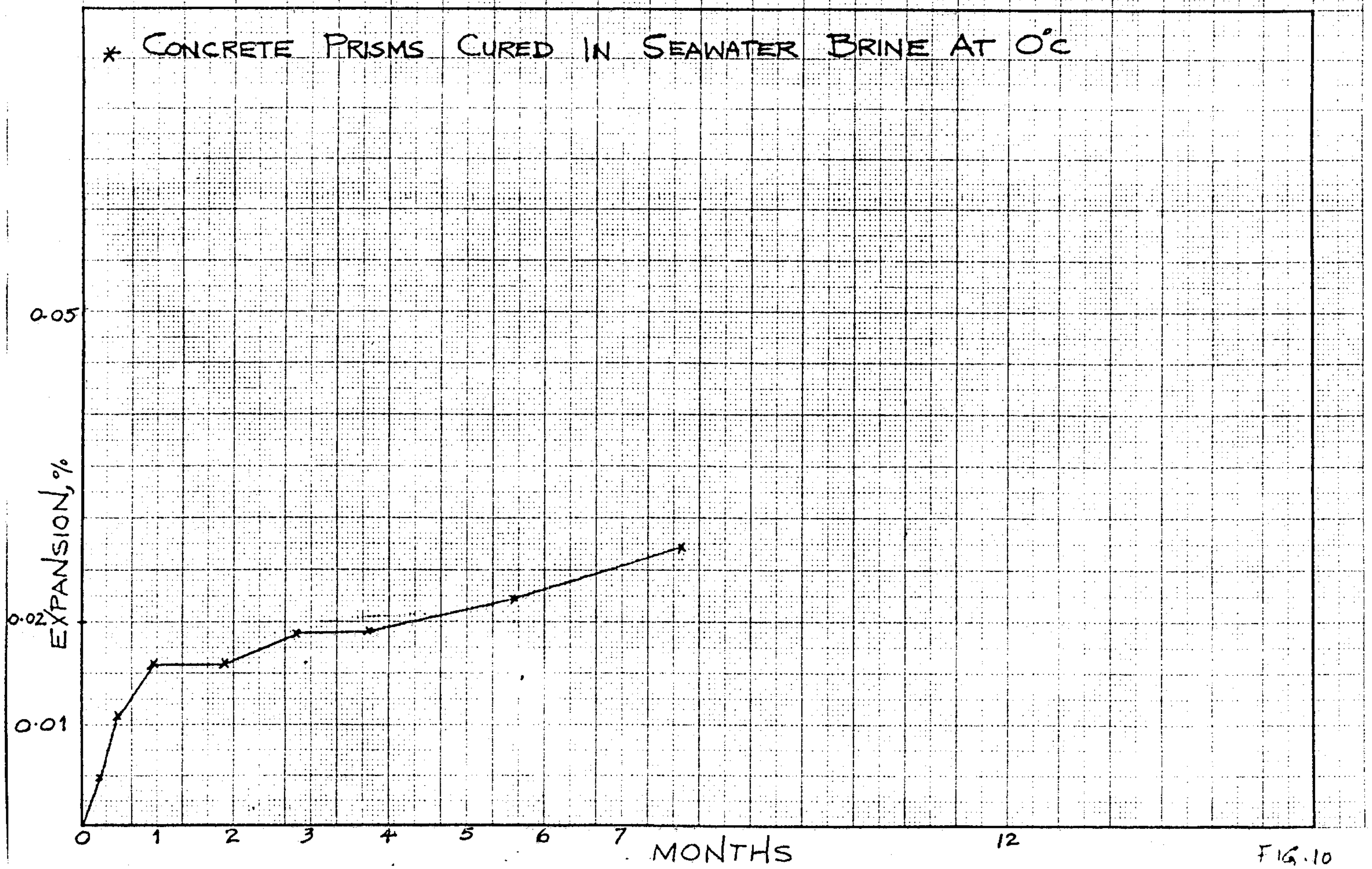


FIG. 10

SAMPLE NO. 5 – COAL MINE LAKE – MOOSE CREEK

EXPANSION CURVE – NON STANDARD* ALKALI REACTIVITY TESTING

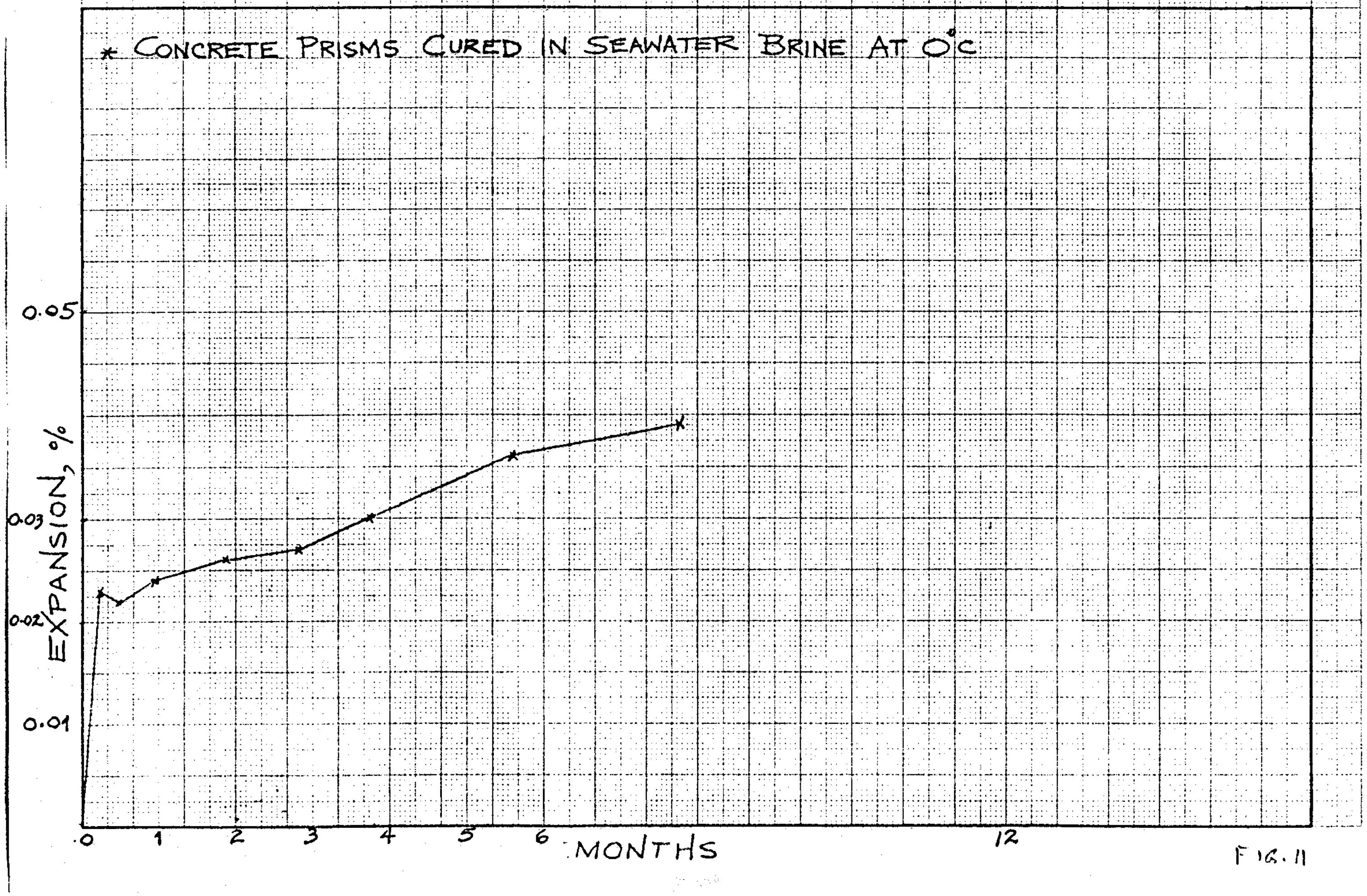
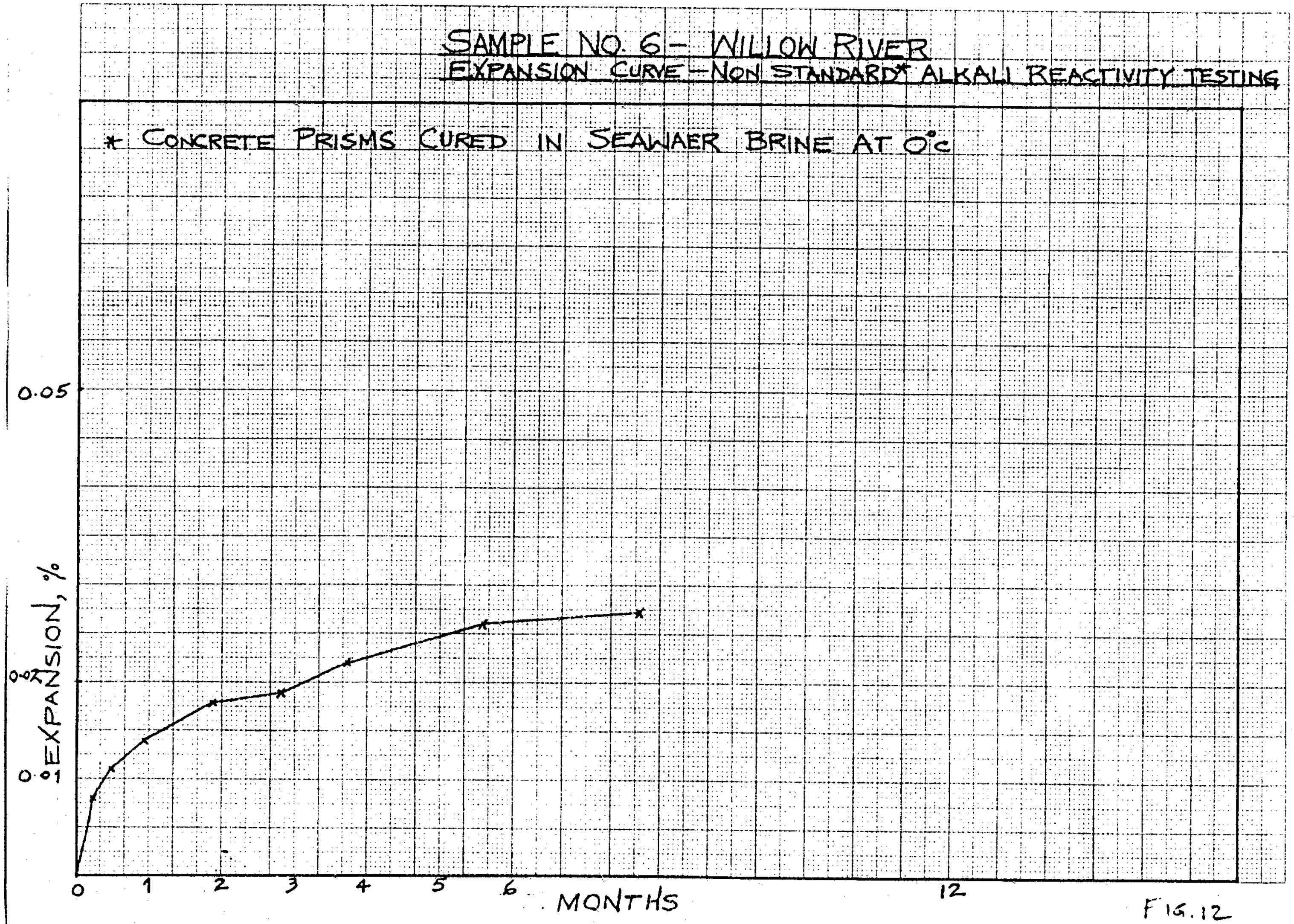


FIG. 11



SAMPLE NO. 6 – WILLOW RIVER
EXPANSION CURVE – NON STANDARD* ALKALI REACTIVITY TESTING

FIG. 12

9.0 DISCUSSION OF RESULTS

9.1 General

Firstly, a review of Table 1, being an overview of all results, shows that not one of the aggregates from the six separate sources passed all the tests.

However, this Table suggests that the Shingle Point, Running River and King Point are the three best sources of aggregate while the Willow River source is the single worst.

9.2 Soundness

This test consists of a number of cycles of immersion of the aggregate in a standard solution of magnesium sulphate followed each time by drying. The concept behind its common use as a predictor of aggregate durability is that the sulphate solution crystallizes during the drying cycle and the internal pressure exended by those crystal on the pores of the aggregate is considered analogous to pressures exerted by ice crystals. Obviously a highly absorptive aggregate would tend to disintegrate more - and hence show a greater weight loss- than a less absorptive aggregate, if this test is considered reliable. In fact it is tending to be relied upon less now than formerly. However, high weight losses can at least raise a flag that further investigation is required.

In the case of the six samples tested, all were within the standard specification in terms of weight loss although the Willow River sample which had the highest absorption had twice the weight loss of any of the others. The test is considered reasonably relevant in the case of possible freeze-thaw conditions which would occur in concrete immersed in freezing sea water which would likely occur if these aggregates were used for concrete in marine structures in the Beaufort Sea.

9.3 Los Angeles Abrasion

The standard test allows a maximum weight loss of 50% after exposure to a fixed number of rotations of hard steel balls cascading repeatedly into the aggregate under test.

The test is therefore a standard measure of toughness or hardness of aggregate and as such is considered one of the necessary primary attributes which must be evaluated for concrete aggregate. That is to say soft materials such as certain sandstones for example, are unsuitable as they may have crushing strengths less than that of the design strengths of the concrete into which they are to be incorporated.

All the samples easily met this criterion although again the Willow River sample performed more poorly relative to all the others.

9.4 Petrographic Results

From Table 1, we note that the petrographic tests conducted according to CSA A23.2 - Methods of Test for Concrete, Appendix B, resulted in excessively high Petrographic Numbers with the single exception of the Moose Creek source, with the result that the samples from the other sources are all considered to have failed this test.

The reason for this becomes apparent from a study of the detailed petrographic results summarized in Appendix X. All the failed samples contained high values of chert which without further study is normally classed as unstable. This factor translates into high Petrographic Numbers when interpreted according to the qualitative rating listed below which has been used for many years by various petrographers.

Petrographic Number Range	Qualitative Rating
100-110	Excellent
110-125	Good
126-140	Fair
141-155	Poor

However, this method of petrographic analysis and rating should be considered as only a first step and further tests should be conducted before rejecting aggregate on the sole basis of this one rather primitive method. At the same time a 'flag' has been raised and the actual reactivity of the chert which is directly responsible for the high number, should be investigated further.

This point is brought out, even in our own records of Arctic aggregates, inasmuch as these results are considered high in general terms even in relation to the results noted in the Klohn Leonoff Ltd. report prepared for the Department of Indian and Northern Affairs during 1972 and 1973.

Nevertheless at this point the end result is that only one of the six samples of aggregate is classified as petrographically acceptable for use in concrete pending further testing. In fact, much of our recommendations concerning further testing centre around this matter of chert and the exact determination of the effects of this particular type of chert when used in concrete.

9.5 Durability

The next failure apparent from Table 1 is the predicted durability of the Willow River source. That sample did not meet the minimum required durability index number of 52. In fact from Table 4 we note a value of only 32 was produced compared for example with the highest value of 80 reached by the Jacobs Ridge sample.

The results from that test are correlated closely with the results of the soundness tests as well, which are also commonly used as a measure of aggregate durability. That is from Table 4 again, it may be seen that the Willow River sample performed by far the worst in the soundness test whereas the Jacobs Ridge sample was an order of magnitude better than most of the others in that parameter.

9.6 Organic Impurities

As noted in Table 1, there are three failures in the organic impurities test on fine aggregate. These are the samples from King Point, Jacobs Ridge and Willow River. This basic test is directed at detecting unacceptable amounts of tannic acid associated with fine aggregate, usually as the result of the presence of decayed organic matter. The effect of this acid is to react adversely with the hydration products of portland cement such that compressive strength is reduced. In extreme cases final set is precluded all together.

Once again though, this test functions initially to raise the alert in the case of failure, that something is amiss and further investigation should be conducted. One reason the test is not conclusive in itself is that particles of coal and lignite for example, are materials that may be present in fine aggregate which may also in themselves produce a high, unacceptable colorimetric number and yet which may not, within reasonable limits, adversely affect subsequent concrete strengths and setting properties.

For this reason the lightweight particles, which include coal and lignite, are removed from the sand sample by a flotation process involving a low density fluid medium. The test is then rerun on the remaining sample. In the case of these three specific samples - King Point, Jacobs Ridge and Willow River the colorimetric number remained unacceptably high even after the removal of the lightweight particles. Therefore we know the high number is not due to coal or lignite.

The case against the acceptability of the three samples for use as high quality concrete aggregate is therefore more damaging. Before rejection though, comparative compression tests are normally conducted using those aggregates and also with a known satisfactory fine aggregate - usually standard Ottawa silica sand. A comparison of the strength results from the two sets of compressive tests in which all other factors are equal - principally cement and water content - forms the basis for acceptance or rejection of the suspect source.

It has been noted that studies in these areas have previously been conducted by others with somewhat differing results. One specific example of this is a report by Hardy and Associates conducted during 1976. As it happens the writer of this Klohn Leonoff Ltd. report was a senior partner with R.M. Hardy and Associates Ltd. at the time of their 1976 study and hence has some personal knowledge of that particular report.

Still more specifically, and as an example, the Hardy report tested a Willow River sample for organic impurities and produced a value of 5+ as opposed to Klohn Leonoff's 5 as reported herein. The discrepancy in this instance occurs when the light weight material was removed in each case, resulting in a color rating of 2+ in the Hardy case while it remained at 5 in the Klohn Leonoff case.

The explanation is not precisely known but undoubtedly relates to sampling theory inasmuch as we are dealing with sites each with areas of several square kilometers and are providing evaluation of the entire area in each case on the basis of only one or two finite samples and from different locations within the large area. The statistical probability in this situation, of two different samples being identical and representative of the entire non homogenous deposit, is very low indeed.

This comment of course, in our opinion, is applicable to all the results presented in this limited study as well as to tests from those other studies of which we have no knowledge.

9.7 Cleanness - Coarse Aggregate

Table 1 lists two failures amongst the six coarse aggregate samples in the matter of cleanness, the Moose Creek sample and the Willow River sample. This particular test measures the accumulation of silt which settles out of a solution in which the coarse aggregate is immersed after agitation for a fixed period. These two samples each produced an inordinate amount of silt after this procedure.

In fact this is simply visual confirmation of the very high silt contents of these two samples relative to the others, which is indicated in the grading curves of Appendix IX. However in the case of those curves the actual extent of siltiness (materials passing the No. 200 mesh sieve) is masked to a degree because the entire pit run sample up to 75 mm (3 in.) size is considered as the base, including all the fine aggregate as well.

9.8 Cleanness - Fine Aggregate

Three samples of fine aggregate failed to meet cleanness requirements for use as is, in first class concrete. These are the same two samples which produced dirty coarse aggregate - Moose Creek and Willow River with the addition now of the Jacobs Ridge sample. More specifically, standard specifications allow up to 5% silt in the fine aggregate whereas our Table 3 indicates values of 16.5, 12.2 and 8.2% silt respectively for the fine aggregate from those three sources.

9.9 Aggregate Reactivity Tests

9.9.1 General

Finally from our Table 1 overall summary are the results to be considered of both the standard and non-standard aggregate reactivity tests.

The writer of this report, as a long-time committee member of CSA A23.1 - "Concrete Materials and Methods of Concrete Construction", is part of the process of bringing out an updated revised version of the standard. One of the new portions of that standard which will appear in the next edition is of direct relevance to this report. That section concerns the alkali-aggregate reaction and particularly the current state-of-the-art in that regard.

Accordingly, in the interest of accessibility while reading this report, we include as Appendix XI the as yet unpublished section of the future standard dealing with this matter.

In view of the on-going nature of these tests and the fact that relatively little has been published on the possibility of alkali reactivity in Arctic aggregates, perhaps still further background on the phenomenon is warranted beyond that included as Appendix XI of this report.

Alkali-aggregate reactions which have an adverse affect on the durability of portland cement concrete have been known since work in North America in the 1930's and 40's. The most important factors which affect the reaction are the nature of the expansive rock or mineral and the concentration of alkali in the pore water; this may differ in different parts of a concrete element because of local environmental effects. Water is required and the reaction is affected by temperature, pozzolans and the proportion and sometimes size of the expansive component in the aggregate. Recent discoveries and the strong probability that the amount of alkalies in cement will increase in the future have re-awakened interest in the subject.

Expansive reactions appear to result from more than one mechanism and it has beem proposed that alkali-aggregate reactions be classified into three types:

> Type I - the alkali-silica reaction is characterized by the poorly ordered forms of silica such as opal, chert and chalcedony; occasionally quartzites are expansive.
>
> Type II - the alkali-carbonate-rock reaction involves certain varieties of argillaceous dolomitic limestone.
>
> Type III - alkali-silicate reactions - involves phyllites, greywackes, argillites and silicate glasses found in volcanic rocks.

Type I - Expansion in the alkali-silica reaction was accounted for by moisture uptake by gels formed by reaction between the poorly crystalline forms of silica and NaOH and KOH from the hydrating portland cement. The mechanism is believed to be related to osmosis but without the need for a semi-permeable membrane.

Type II - The alkali-expansion of the argillaceous dolomitic limestone from Kingston, Ontario and elsewhere evidently results from a different mechanism, since these rocks do not normally contain the disordered forms of silica perviously thought essential. It is now thought that expansion resulted from moisture uptake by dry clay-grade minerals in the rock. The mechanism therefore involves two steps:

a) attack by cement alkalies on the dolomite (dedolomitization) which opens micro-cracks allowing water and solutions to penetrate into the rock and;

b) moisture uptake by the dry clay with development of expansive forces due to surface hydration and build-up of hydrous double layer surrounding the clay minerals.

Type III - Alkali-silicate reactions involve rocks of the type described in one of the earliest reports of poor durability of concrete attributed to expansive alkali-aggregate reactions.

Scanning electron microscopy and X-ray diffraction showed that some of the layer structure silicates in these rocks expand or exfoliate on treatment with alkali.

Alkalies in portland cement are derived from the feldspars, micas and clay minerals contained in the argillaceous rocks and limestones used as raw materials. Sodium and potassium are appreciably volatile at sintering temperatures and in former times these elements were often permitted to escape from the kilns into the atmosphere, thereby decreasing the amount present in the cement. Nowadays requirements for environmental protection have led to the introduction of more efficient dust precipitators. Disposal of kiln dust as a waste product is a problem and its incorporation in the cement is economically attractive, so higher contents of alkalies are to be expected from this cause. Sharp increases of fuel prices have accelerated the replacement of wet process kilns by more energy efficient dry process kilns which however produce cements containing more alkalies. The cement is by far the most important source of alkalies but alkali compounds are also sometimes present in the aggregate, the mixing water or in ground water.

Attempts to recognize alkali-expansive rocks and minerals on the basis of composition, fabric and physical properties such as crystallinity, porosity and specific surface area are still only partly successful. This is because mechanisms of expansion are not always fully understood and because analytical procedures have certain limitations.

A partial bibliography on this subject, chosen from among the very extensive number of papers which have been published, is included following the conclusions of this report.

In any event, the standard tests to investigate this phenomenon for this program were conducted according to CSA A23.2-14A-M77 - "Alkali-Aggregate Reaction", a supplement to which, just received, is entitled Supplement No. 2 - 1986.

This test requires that the portland cement used in preparing the mortar and concrete bars that are to be prepared and subsequently measured for possible length change over time, shall be one with an alkali content of 0.9 ± 0.1 percent. A further caveat in the form of a note states: "Where cements with alkali content higher than 0.9 percent, expressed as equivalent Na_2O, are encountered, the test should be carried out using the cement with the highest expected alkali content."

The alkali content of local cements generally is in the order of 0.7% or less. This is just outside the lower limit of the specification. Accordingly and in compliance with the note we obtained two samples of cement from out of province with higher alkali contents.

The concrete prism tests for coarse aggregate were therefore conducted with cement with an alkali content of 1.06%. The mortar bar tests for fine aggregate as per ASTM C-227 "Standard Test Method for Potential Alkali Reactivity of Cement - Aggregate Combinations (Mortar Bar Method)", were conducted with cement having an alkali content of 1.07%.

9.9.2 Alkali-Aggregate Reactivity

9.9.2.1 Fine Aggregate

Considering firstly the fine aggregates, it may seem from the graphs of Figures 1 to 6 inclusive and as noted in Table 1, that the Moose Creek sample has reached the maximum expansion when incorporated into Mortar bars, allowed under standard conditions in three months.

The implications and reasons of this for this investigation are stated in the following quote from ASTM C 227.

"When expansions in excess of 0.10% at 6 months (or in excess of 0.05 at 3 months) are shown in results of tests performed using this method, it is strongly recommended that supplementary information be developed to confirm that the expansion is actually due to alkali reactivity. Sources of such supplementary information include: (1) petrographic examination of the aggregate to determine if known reactive constituents are present; (2) examination of the specimens after tests to identify the products of alkali reactivity; and (3) tests of the aggregate for potential reactivity by chemical methods (Test Method C289)."

Our recommended further testing is again focused on detailed, more sophisticated investigation of precisely these points.

At it happens, although the 3 month expansion of the Moose Creek sample was excessive, the six month result was just within the allowable limit. Nevertheless as noted in the previously quoted ASTM commentary, a flag has been raised by the 3 month results and further investigation is warranted and is recommended.

9.9.2.2 Standard Expansion Test - Coarse Aggregate

In terms of the coarse aggregate, examined by means of concrete prisms, it is apparent from the same Figures 1 to 6 inclusive, that three samples have reached the maximum expansion allowed under standard conditions in three months. These are the samples from Shingle Point, Running River and Jacobs Ridge. These three samples are among those with the highest chert contents so the chert is at least suspected of being the expansive source in this case.

A standard limit for allowable expansion of coarse aggregate after six months testing, is not part of the CSA specification, although one year limits are available. Our comments will therefore be reserved in this regard until the appropriate time has elapsed.

9.9.2.3 Non Standard Test - Coarse Aggregate

Lastly from our Table 1 summary, we have noted the three months expansions of the non-standard brine immersion tests of concrete prisms.

By definition this test was non standard. The concept in general was that ultimately the concrete to be made from aggregate from one or more of the sites we sampled would be immersed in the cold Arctic sea water. We therefore attempted to approximate such environmental conditions on concrete prisms made largely from each of the six aggregates.

More specifically, the concrete prisms were cast using in each case procedures identical to the standard test method of CSA A23.2-14A for alkali-aggregate reaction. That is the concrete prisms were cast at laboratory ambient air temperature and then cured for 20±4 hours under standard curing conditions of 23±2° and 100 percent relative humidity.

They were then removed and placed in a brine solution. This solution was prepared using a commercial product labelled Forty Fathoms Marinemix Bio-crystals, manufactured by Marine Enterprises of Baltimore, Maryland, U.S.A., 21204. The correct sea water salinity is produced according to the manufacturer's instructions, by adding enough water to the crystals to bring hydrometer readings to between 1.020 and 1.022 at 28°C.

The resulting solution was put in our laboratory freezer - one of several previously used for other Arctic assignments - kept at just below 0°C such that ice just formed. These conditions were maintained by automatic control systems.

When length measurements of the concrete prisms were required, they were taken out of the submerged freezing condition and immersed in the same brine solution, a portion of which has been maintained at ambient laboratory air temperature. After 24 hours they were removed, measured and then returned again to the brine at freezing point as before.

In addition to our comments contained previously in section 9.9.1 we would point out that these non standard alkali-reactivity tests were conducted on concrete and not mortar. This is to say concrete is comprised largely of coarse rather than fine aggregate both on a weight and a volume basis. Partially for this reason amongst others, the allowable expansion of coarse aggregate is only about 20 percent of that for fine aggregate. This is illustrated graphically on pages 24 to 30 where the limits for the standard test have been plotted for both fine and coarse aggregate.

In view of the proportionaly greater importance of coarse aggregate relative to fine in concrete we therefore tested all six of the coarse aggregates which we sampled from the various sites. This was opposed to testing only three coarse and three fine as specified since we had no means of knowing in advance which three of the six would be most critical.

In that test a study of the graphs comprising Figures No. 7 to 12 inclusive shows that three of the samples have expanded, after three months exposure at 0° temperature, to a much greater extent than the remaining three. The suspect sources, from this point of view at least, are Shingle Point, King Point and Moose Creek.

The reasons why Running River and Jacobs Ridge sample did not also greatly expand, as in the standard test are not known and also need to be studied further.

To quote the CSA comments from our Appendix XI, referred to earlier, relative to Arctic Canada "....little is known about the quality of northern aggregates." Therefore, for the same reasons as noted earlier in the quote from ASTM C 227, we feel further tests are required at least on the three most expansive aggregates identified above as those from Shingle Point, King Point and Moose Creek.

The non standard expansion tests were, as noted earlier, conducted in an environment intended to simulate Beaufort Sea marine conditions. That is the specimens were continuously immersed in salt water, maintained at 0°C by means of ice. This means that the comments of Appendix XI related to Arctic conditions would apply; specifically - "Expansion due to alkali-aggregate reactivity is slowed by low temperatures, but low temperature should not be relied upon to give protection to the concrete if highly reactive aggregates are used."

We agree with this comment and therefore feel it especially important in that case, to carry on the readings for periods of time well beyond the relatively short duration of this contract.

10.0 SOURCE EVALUATION MATRIX

The following matrix is based on the results of the limited field and laboratory test data that has been obtained or completed for this project. The matrix has been prepared in accordance with Task A.4C of your Statement of Work included in your Request for Proposal.

The matrix includes such components as access, site conditions, and deposit characteristics. Each of the six sources has been rated on a numerical scale of I to VI relative to the others with the highest number, VI, being best. Our basis for the ratings are as follows:

A. FIELD EVALUATION

a) Access

This component considers distance from the Beaufort Sea coast.

b) Site Conditions

This component considers terrain conditions (flat or sloping), presence of water, space, environmental concerns, location of processing equipment, curing areas, stockpiles, etc. Most of these items would depend on the size of the proposed future operations. Site conditions would have to be confirmed prior to development.

c) Deposit Characteristics

This component considers development of the source with respect to thickness of overburden, permafrost, thickness of the deposit, variability of the deposit. Most of the items in this component are based on visual inspections and would be subject to confirmation by a detailed drilling investigation.

B. LABORATORY EVALUATION

Our basis for relatively rating the material properties of samples from the six sources on this scale is simply transposition of the two point pass-fail evaluation of our Table 1 to the more finely graded six point evaluation used in this matrix. At the same time it must be kept in mind that a material either passes or fails most of these tests so our Table 1 governs in this respect.

TABLE 5

Combined Field and Laboratory Relative Evaluation

Component	Shingle Point	Running River	King Point	Jacobs Ridge	Moose Creek	Willow River
A. Field Evaluation						
1. Access	VI	III	V	IV	II	I
2. Site Conditions	I	III	IV	VI	II	V
3. Deposit Characteristics	V	II	IV	VI	I	III
B. Laboratory Evaluation						
1. Soundness	III	II	IV	VI	V	I
2. L.A. Abrasion	III	II	IV	VI	V	I
3. Petrographic	IV	V	II	III	VI	I
4. Density	III	IV	V	VI	II	I
5. Abosrption/Course	V	IV	III	II	VI	I
6. Absorption/Fine	IV	III	V	VI	II	I
7. Durability Absorption Ratio	IV	V	II	VI	III	I
8. Durability Index	III	V	IV	VI	II	I
9. Organic	II	III	IV	V	VI	I
10. Cleanness Coarse	V	IV	III	VI	II	I
11. Cleanness Fine	VI	IV	V	III	I	II
12. 3 mos. Expansion Fine	II	IV	V	VI	I	III
13. 3 mos. Expansion Coarse	V	I	VI	II	IV	III
14. Expansion Brine	I	VI	III	V	II	IV

Note: I indicates relatively poorest of the six
VI indicates relatively best of the six

11.0 STATE-OF-THE-ART DESIGN FOR ARCTIC MARINE STRUCTURES

Firstly, in terms of design the use of concrete has a basic technical design advantage over steel for reasons related to much higher localized ice pressures relative to global pressures. Similarly corrosion considerations favor concrete over steel - although we feel there is a need for further work on this aspect. One item we would consider in this regard would be the use of an integral corrosion inhibitor in the concrete at the time of mixing. The effect on corrosion of different cement types would be another.

On the other hand economics in 1987 currently favor steel. However, future design improvements resulting from expected changes in Code requirements are expected to favor concrete inasmuch as significantly reduced amounts of reinforcing steel will likely be the result.

Finally in design matters it would seem prudent to use a higher prestress force than is normally used - say 500 p.s.i. rather than 100 p.s.i.. We would also recommend prestressing to this higher level in two directions rather than just one as is often the case. That is, a horizontal prestress load would be aimed at minimizing the vertical concrete cracks resulting from normal drying shrinkage as well as volume reductions due to thermal factors.

A series of vertical prestress cables should be incorporated so that loads induced by this means would be aimed at minimizing horizontal concrete cracking. Cracks of this nature could easily occur due to high structural stresses induced by waracking action which occurs with settlement.

The use of fly ash and/or silica fume as additives to arctic marine concrete has the advantage of increasing impermeability. This, in turn, leads to minimizing of corrosion of any reinforcing steel or prestress strand incorporated in the concrete. It also helps to minimize the ingress of water so that destructive freeze-thaw forces, particularly in the vicinity of the water line, are minimized.

Concurrently, however, the use of additives brings with it the disadvantage of requiring additional separate storage and dispensing systems, at isolated remote locations.

We also include a reference to this general question of additives in our bibliography.

12.0 RECOMMENDATIONS FOR FUTURE TESTS

12.1 Petrographic Results

Due to the unduly high petrographic numbers, we recommend further investigation to determine reactivity of the cherts.

It is proposed to approach this by means of thin sections to study the rim material as such as well as the rate of growth.

X-ray diffraction will be used to identify precisely the reaction products. It is required to determine if the cherts contain silica in the form of ultra fine microcrystalline quartz (opal-like) or chalcedony.

12.2 Arctic Weather

It is recommended critical dilation tests be conducted on the aggregate in accordance with ASTM C 682. This is particularly important in the event it is envisaged that concrete made from these aggregates may be utilized for structures which might be submerged or partially submerged in the Beaufort Sea.

12.3 Alkali-Reactivity

These test readings should be continued for a minimum of one year as required by specifications.

Periodic thin sections should be taken of the extra prisms and mortar bars which we cast and which are presently undergoing standard tests, in order to quantify the rate of growth of rim material. This program should continue for a minimum of one year.

The same examination by thin section of the non standard prisms should be periodically undertaken to determine the rate of growth of rim material. This program too, should also continue for a minimum of one year.

12.4 Organic Material

Compression cubes should be cast using fine aggregate from the three suspect sources. These should be compared with strengths at various ages, from cubes cast using standard Ottawa silica sand.

That program should continue monthly for six months and then bimonthly until one year has elapsed.

12.5 Deleterious Material

The quick chemical test ASTM C 289 should be conducted to determine if the aggregate is innocuous, potentially deleterious or deleterious. This is evaluated by the CaO:MgO ratio and the Al_2O_3 content.

13.0 CONCLUSIONS

From a field point of view the most accessible site as well as those more nearly adjacent to the area most likely for future development are King Point, Shingle Point and Running River.

From the point of view of the laboratory test results to date, these three sources are also the most promising for use as concrete aggregate, with the caveat that amongst those three, further tests must be conducted on the sample from Shingle Point as to its performance in brine, since that sample has expanded the most up to six months, at least under that brine curing regimen.

The aggregate from the Willow River source should be rejected for use in high quality concrete, based on results available to date.

Aggregates from Shingle Point, Running River and King Point would require a screening operation only, prior to use whereas Jacobs Ridge, Moose Creek and also Willow River if it should be used, would also require a washing operation in addition to screening.

An extended more sophisticated testing program on the aggregates should be implemented as described in section 11.0 of this report.

It is estimated the cost of that additional one year long program would be in the order of $14,000.

PERMIT TO PRACTICE
KLOHN LEONOFF LTD.
Signature
Date 25/04/88
PERMIT NUMBER: P 433
The Association of Professional Engineers,
Geologists and Geophysicists of Alberta

APPENDIX I

LOCATION PLAN AND MISCELLANEOUS DRAWINGS

TO BE READ WITH KLOHN LEONOFF REPORT DATED ____________

SCALE 1:250,000

KLOHN LEONOFF LTD.
CONSULTING ENGINEERS

PROJECT WESTERN BEAUFORT REGION CONCRETE AGGREGATE STUDY

TITLE SOURCE LOCATION PLAN

CLIENT: SUPPLY AND SERVICES CANADA

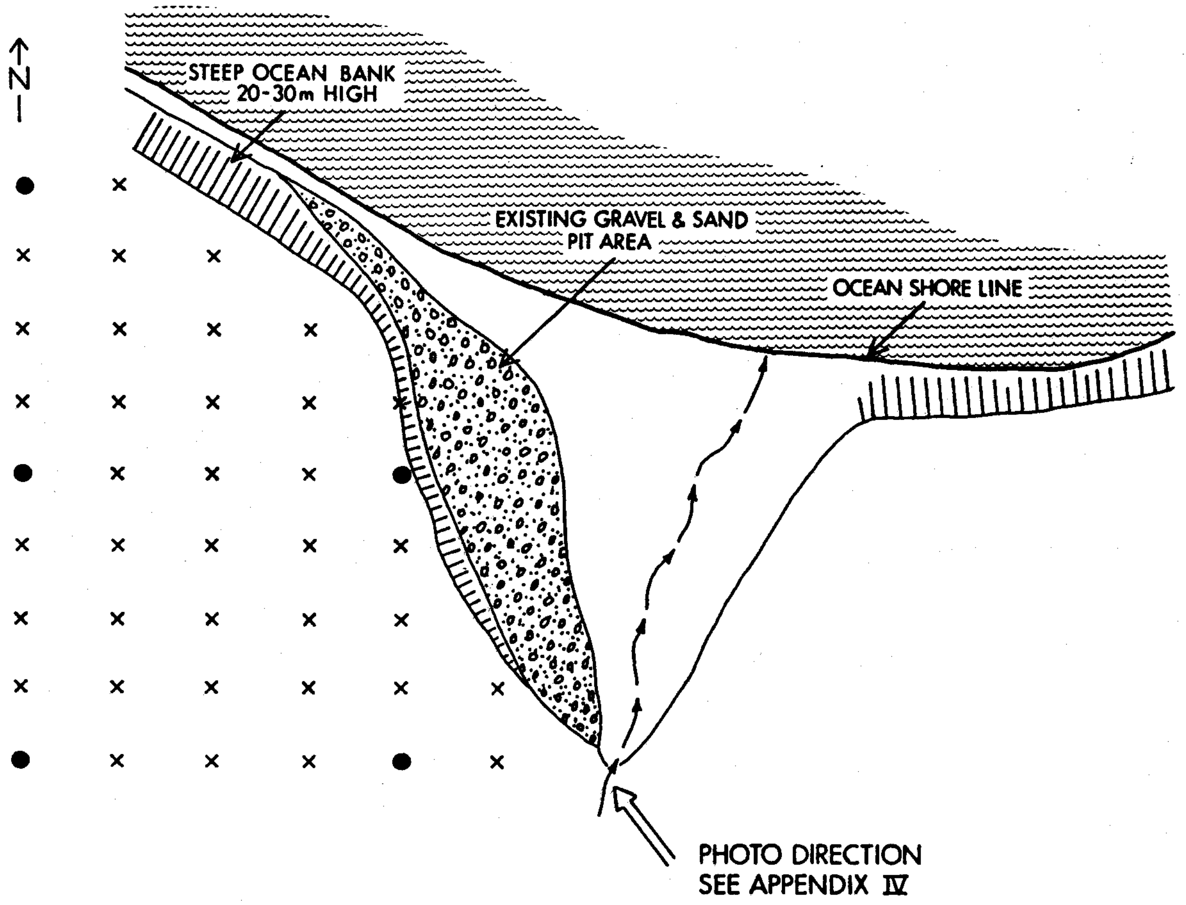
DATE OF ISSUE NOV. 1986

APPROVED KBH

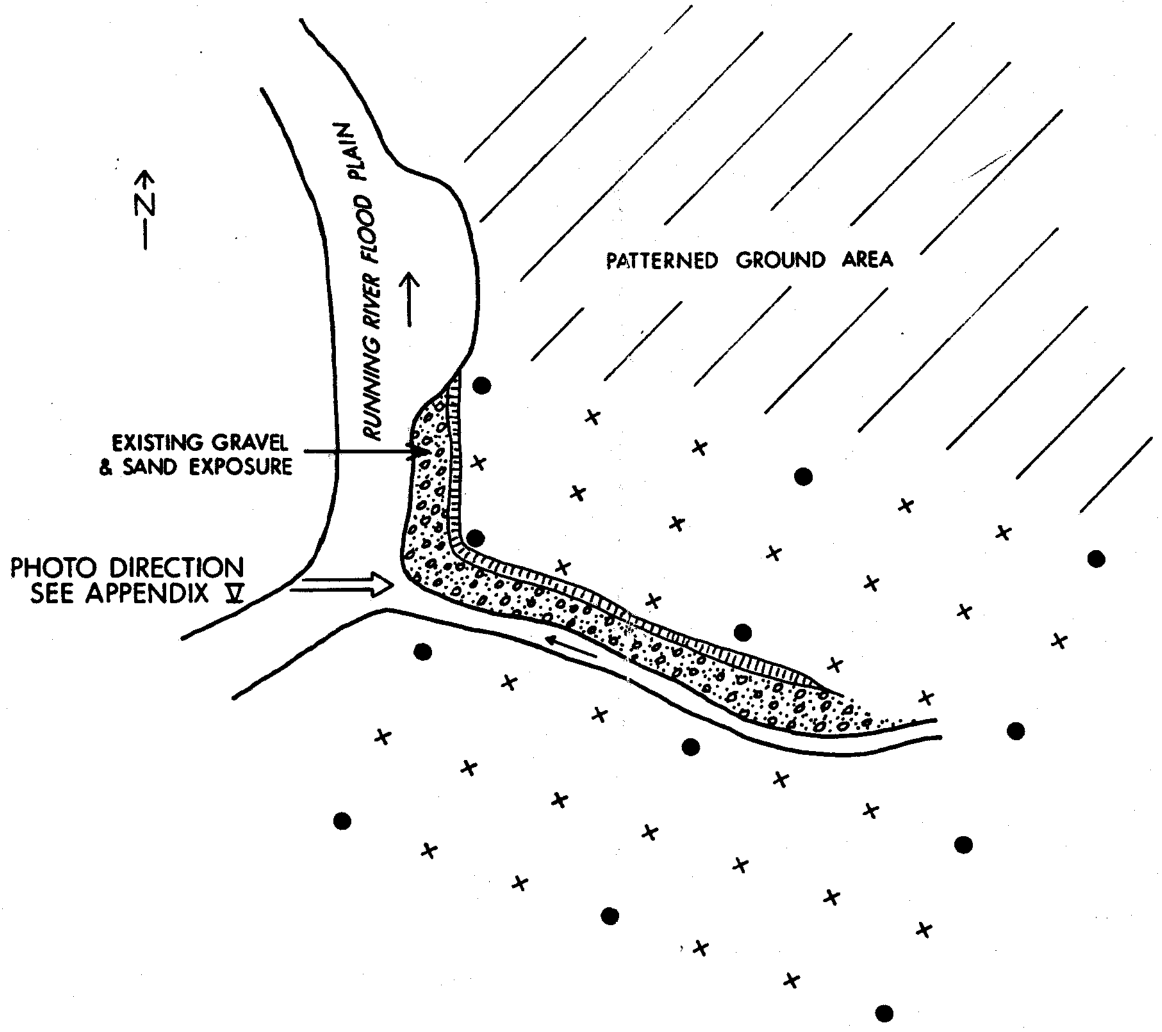
PROJECT No. PA2291

DWG. No. B-2291-1

REV.



EXPOSURE NO. 4
SHINGLE POINT



EXPOSURE NO. 5
RUNNING RIVER

LEGEND:
- ● FIRST STAGE PROGRAM RANDOM SPACING (150-200m)
- × SECOND STAGE PROGRAM GRID SPACING (50m)

TO BE READ WITH KLOHN LEONOFF REPORT DATED ____________

SCALE NOT TO SCALE

KLOHN LEONOFF LTD.
CONSULTING ENGINEERS

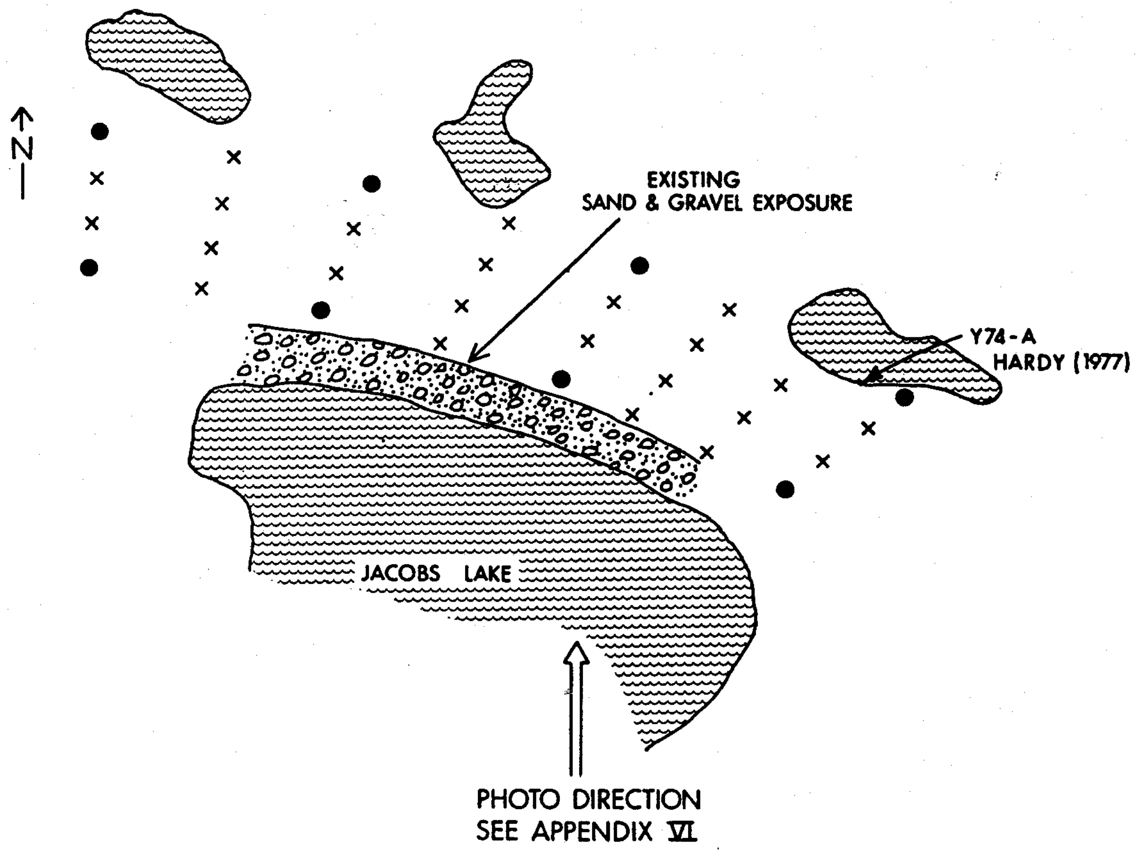
PROJECT WESTERN BEAUFORT REGION
CONCRETE AGGREGATE STUDY

TITLE SUGGESTED INVESTIGATION PROGRAM
SHINGLE POINT & RUNNING RIVER

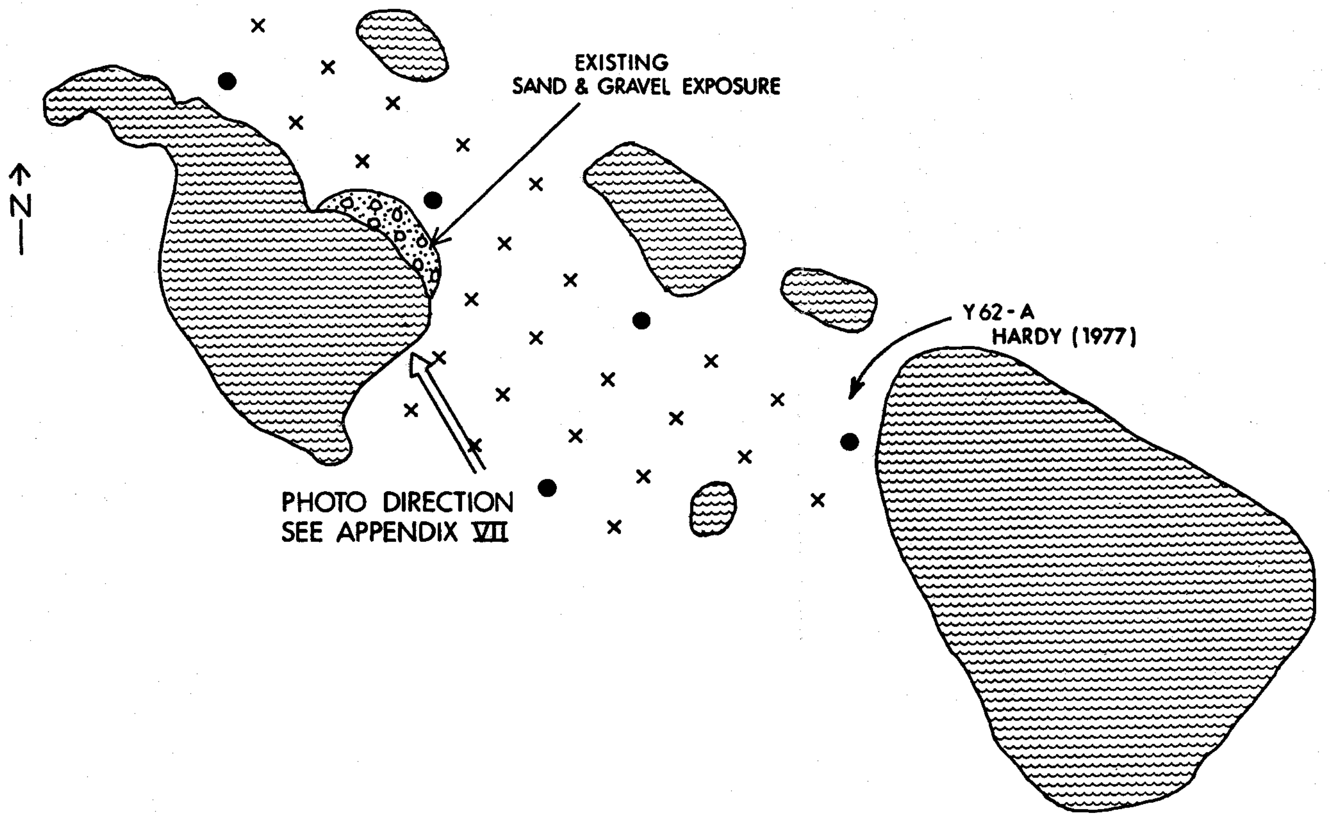
CLIENT: SUPPLY AND SERVICES CANADA

DATE OF ISSUE OCTOBER 1987
APPROVED
PROJECT No. PA 2291
PLATE I-2
REV.

NCI-456-K.L.



EXPOSURE NO. 6
JACOBS RIDGE



EXPOSURE NO. 7
KING POINT

LEGEND :
● FIRST STAGE PROGRAM
RANDOM SPACING (150-200m)
× SECOND STAGE PROGRAM
GRID SPACING (50m)

TO BE READ WITH KLOHN LEONOFF REPORT DATED

SCALE NOT TO SCALE

KLOHN LEONOFF LTD.
CONSULTING ENGINEERS

PROJECT WESTERN BEAUFORT REGION
CONCRETE AGGREGATE STUDY

TITLE SUGGESTED INVESTIGATION PROGRAM
JACOBS RIDGE & KING POINT

CLIENT:
SUPPLY AND SERVICES CANADA

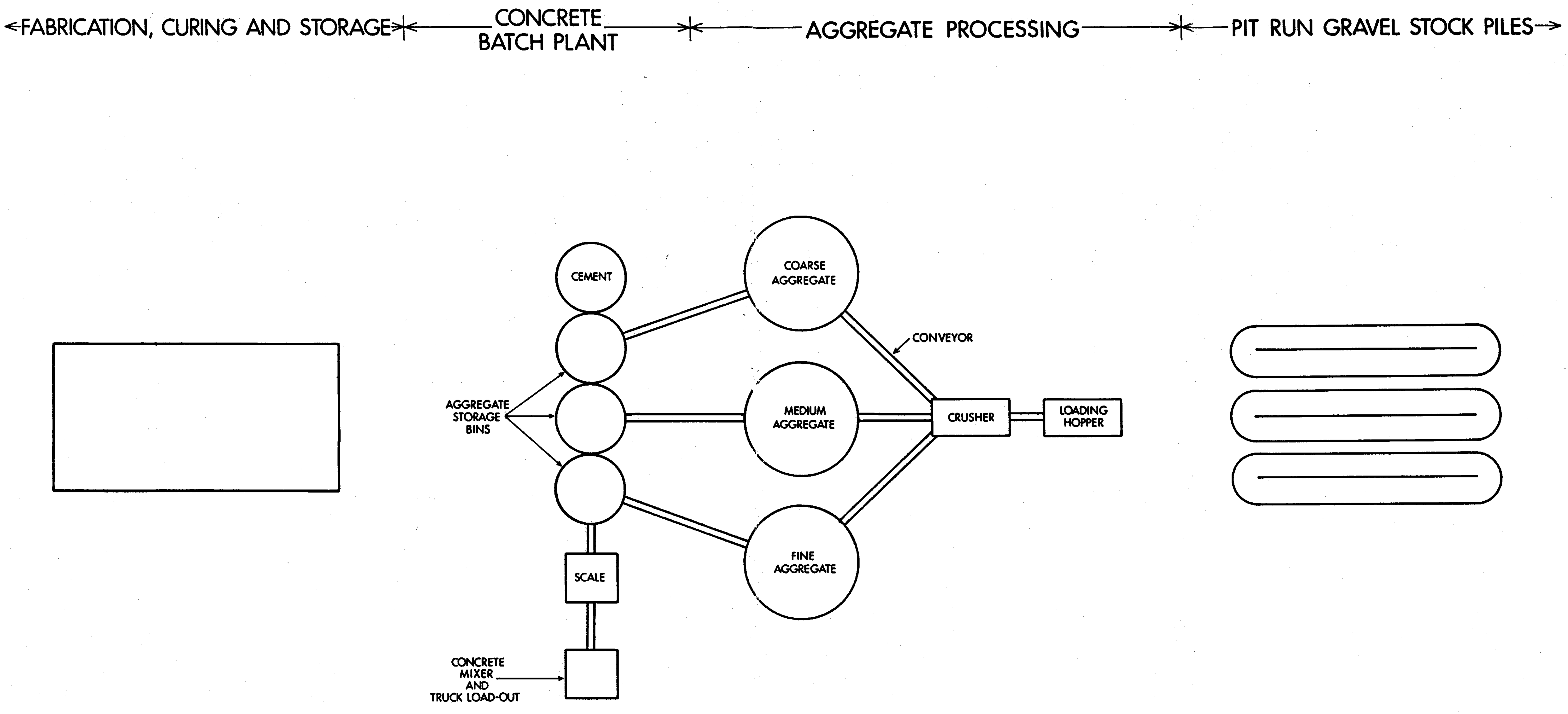
DATE OF ISSUE
OCTOBER 1987
APPROVED

PROJECT No.
PA 2291

PLATE I-3

REV.

NCI-456-K.L.



AS A MUTUAL PROTECTION TO OUR CLIENT, THE PUBLIC AND OURSELVES, ALL REPORTS AND DRAWINGS ARE SUBMITTED FOR THE CONFIDENTIAL INFORMATION OF OUR CLIENT FOR A SPECIFIC PROJECT AND AUTHORIZATION FOR USE AND/OR PUBLICATION OF DATA, STATEMENTS, CONCLUSIONS OR ABSTRACTS FROM OR REGARDING OUR REPORTS AND DRAWINGS IS RESERVED PENDING OUR WRITTEN APPROVAL.

TO BE READ WITH KLOHN LEONOFF REPORT DATED ____________

KLOHN LEONOFF LTD.
CONSULTING ENGINEERS

CLIENT:
SUPPLY AND SERVICES CANADA

SCALE NOT TO SCALE

PROJECT WESTERN BEAUFORT REGION
CONCRETE AGGREGATE STUDY

TITLE
CONCEPTUAL LAYOUT
PROCESSING FACILITY

DATE OF ISSUE
OCTOBER 1987

APPROVED

PROJECT No.
PA 2291

DWG. No.
B PLATE I - 4

REV.

NCI-456-K.L.

APPENDIX II

WILLOW RIVER

WILLOW RIVER

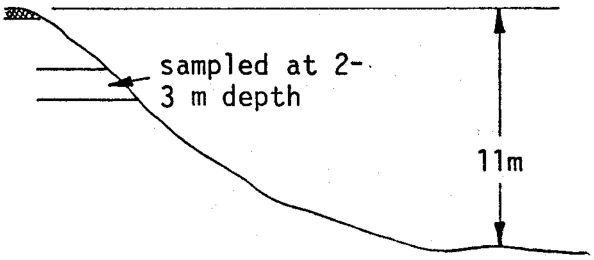
Airphoto No. A 23816-47

Approx. Scale: 1:68 000

Deposit Outline – R.M. Hardy 1977	▬▬▬▬	Test Pit ○	Hardy 1977
		Exposures: △	Hardy 1977
Deposit Outline – Klohn Leonoff 1986	▬ ▬ ▬	☒	Klohn Leonoff 1986

TEST HOLE LOG

SAMPLE DATA				SYMBOL		UNCONFINED COMPRESSION kPa
WEIGHT HAMMER 63.5 Kg					ELEV COLLAR	100 200 300 400
HEIGHT DROP 0.76 m					ELEV GROUND	● FIELD VANE △ LAB VANE ■ UNCONF
					CO-ORD LOCATION	PLASTIC LIMIT WATER CONTENT LIQUID LIMIT
DEPTH ELEV	O.D I.D	BLOWS .15m	NO.		DESCRIPTION OF MATERIAL	x ------ o ------ x 10 30 50 70 90 %
					0.3m Peat and Organics -silty	
1						
2					Gravel and Sand - well graded to 20cm - thin silt layers - sub angular to sub rounded - brown, dry	
3			☒		NOTE: Sample taken between 2 and 3m depth.	
4						
10						
11					11.0m	



Klohn Leonoff Consultants Ltd.

CIVIL & GEOTECHNICAL ENGINEERS

JOB No PA 2291.01.01
PROJECT CONCRETE AGGREGATE STUDY
LOCATION WILLOW RIVER
HOLE No. EXPOSURE No. 1
DATE Sept. 27/86 PLATE

K L C METRIC

TEST HOLE LOG

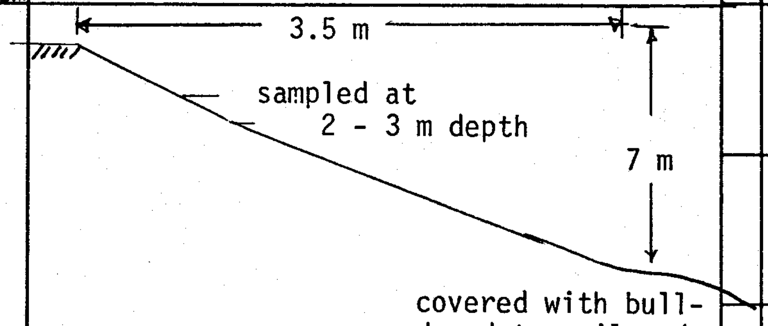
SAMPLE DATA
WEIGHT HAMMER 63.5 Kg
HEIGHT DROP 0.76 m

ELEV. COLLAR
ELEV. GROUND
CO-ORD. LOCATION

UNCONFINED COMPRESSION kPa
100 200 300 400
● FIELD VANE △ LAB VANE ■ UNCONF.

PLASTIC LIMIT x — WATER CONTENT o — LIQUID LIMIT x
10 30 50 70 90%

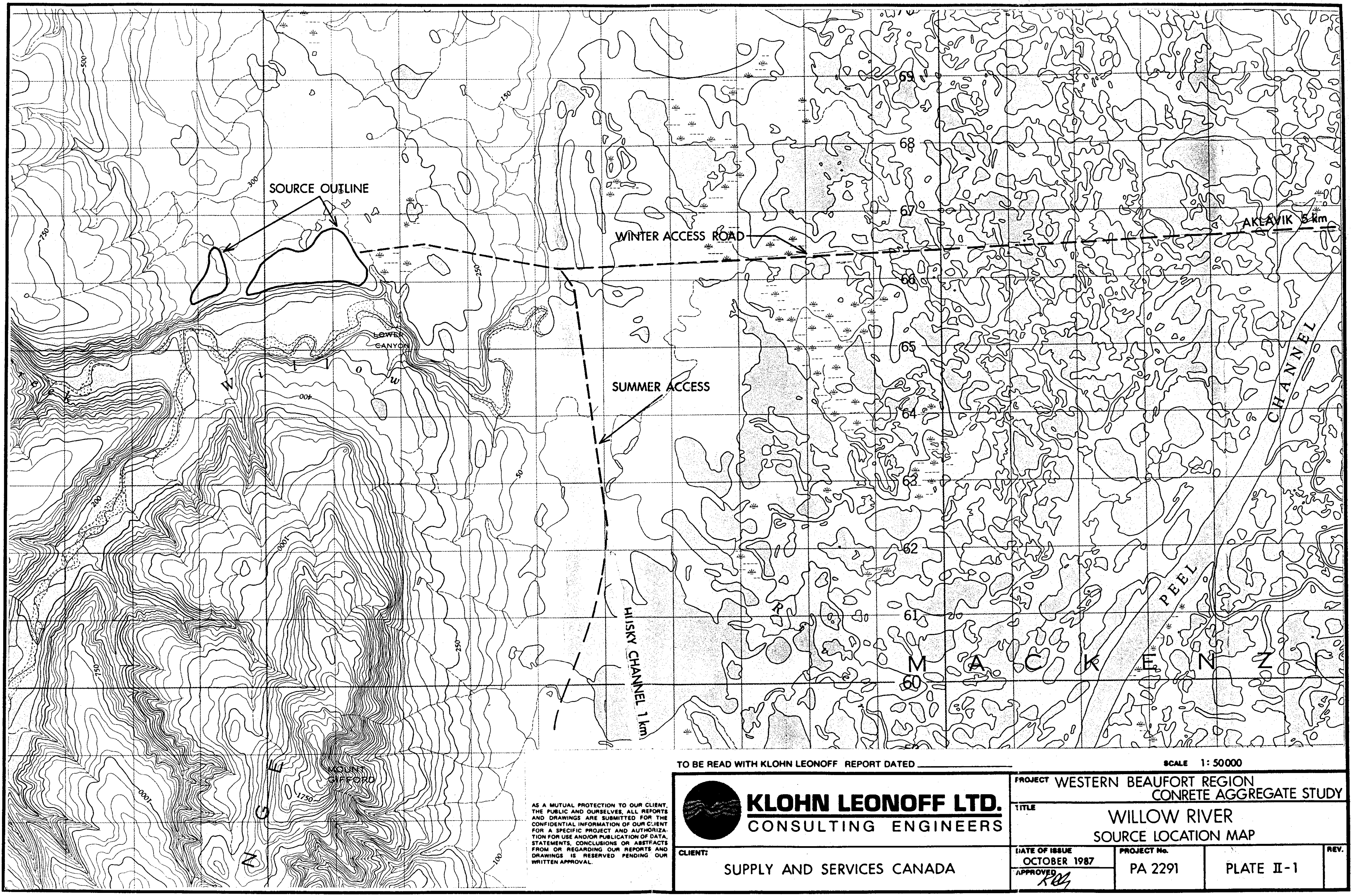
DEPTH ELEV.	O.D. I.D.	BLOWS .15m	NO.	SYMBOL	DESCRIPTION OF MATERIAL
					0.3 PEAT AND ORGANICS
1					
2					
					GRAVEL AND SAND - well graded to 15 cm - trace of silt - angular to sub rounded - dry
3					
4					
5					
6					
7					7.0
8					



covered with bull-dozed topsoil and stripping

KLOHN LEONOFF
CONSULTING ENGINEERS

JOB No. PA 2291.01.01
PROJECT CONCRETE AGGREGATE STUDY
LOCATION WILLOW RIVER
HOLE No. EXPOSURE NO. 1b
DATE Sept. 27/86 PLATE



TO BE READ WITH KLOHN LEONOFF REPORT DATED ____________

SCALE 1: 50000

KLOHN LEONOFF LTD.
CONSULTING ENGINEERS

PROJECT WESTERN BEAUFORT REGION
CONRETE AGGREGATE STUDY

TITLE WILLOW RIVER
SOURCE LOCATION MAP

CLIENT: SUPPLY AND SERVICES CANADA

DATE OF ISSUE OCTOBER 1987
APPROVED
PROJECT No. PA 2291
PLATE II-1
REV.

NCI-456-K.L.

WILLOW RIVER SOURCE AREA

Scarp of flow slide towards east end of source. Note thin silt seams in exposure. Scarp 8 to 10 m high. (Exposure No. 1)

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
	Pt	0.2	PEAT - organic cover.		UF		
1	CL	1.6	CLAY - little gravel, silty, brown, damp.				
2 3 4 5 6 7	GW	7.5	GRAVEL - well graded to 8" cobbles, little fine to coarse sand, trace fines, occasional boulder to 20" size, subrounded to rounded, loose, brown, damp.				
8			Bottom of Pit				Sample from 1.6'-2.5'

DATE: Oct. 10, 1976

LOGGED BY: CPM

DRWN BY: MB/vh

CHKD BY: GCD/TJF

GOVERNMENT OF CANADA

DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO. 467-A(e)

SHEET 1 OF 1

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
	Pt	0.3	PEAT		UF		Organic Color: #5+ Coal Removed: #2+
1	ML	1.3	SILT - little gravel, little sand, trace organics, brown, damp.				Lightweight Pieces - Fine Aggregate: 0.05%
2 3	GW		GRAVEL - well graded to 13" boulders, some fine to coarse sand, subangular to rounded, brown, damp.				
4 5 6		4.2 6.2	— — becoming cleaner.				
7			Bottom of Pit				Sample from 1.3'-6.2'

ATE: Oct. 11, 1976	LOGGED BY: CPM	DRWN BY: MB/vh	CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
467-B
SHEET 1 OF 1

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
	Pt		PEAT - fibrous, silty, brown, moist.		UF		Lightweight Pieces - Fine Aggregate: 0.20%
5		5.0					
	GW		GRAVEL - well graded to 7" cobbles, little fine to coarse sand, trace silty fines, subangular to subrounded, brown moist.				
10							
15							
							Exposure not logged in detail below 14' because of steepness of bank.
55							
60							
65		65.0	increasing in cobble and boulder sizes.				
70							
75							
80							
85		85.0	Bottom of Pit				Sample from 5.0'-13.0'
90							

DATE: Oct. 10, 1976

LOGGED BY: SA

DRWN BY: MB/vh

CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS AND NORTHERN DEVELOPMENT

R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO. 467-C(e)

SHEET 1 OF 1

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
1	Pt	1.2	PEAT - fibrous, silty, brown, moist.		UF		Lightweight Pieces - Fine Aggregate: 11.23%
2 3 4 5 6 7 3 9 0 1 12	GW	12.5	GRAVEL - well graded, some fine to coarse sand, frequent cobbles to 6" size, subrounded, medium dense, brown, moist, rootlets to 3.5' depth.				Gravel continues to estimated depth of 65'.
			Bottom of Pit				Sample from 2.0'-12.0'

ATE: Oct. 11, 1976	LOGGED BY: SA	DRWN BY: MB/vh	CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

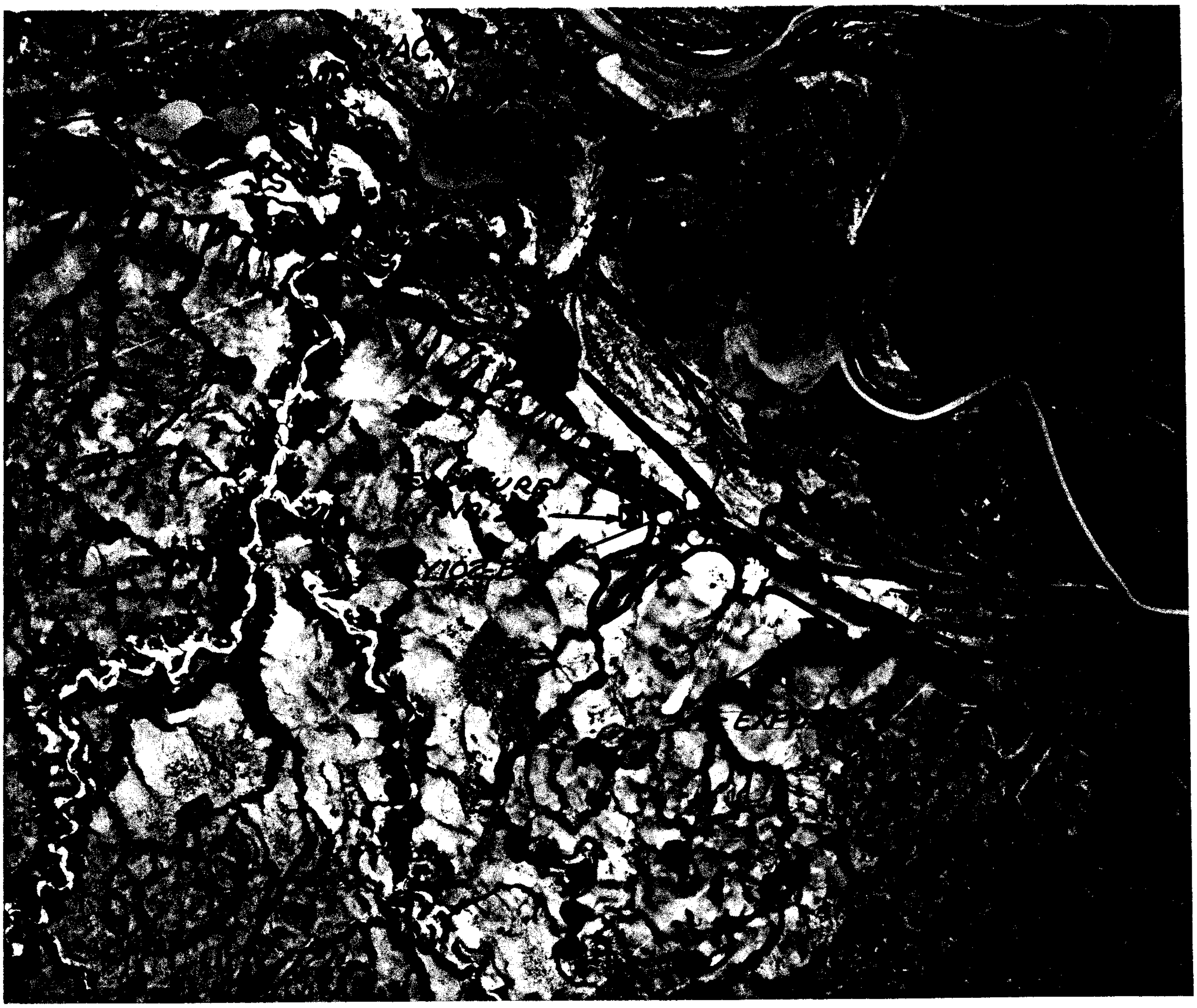
R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
467-D(e)

SHEET 1 OF 1

APPENDIX III
MOOSE CREEK

MOOSE CREEK



Airphoto No. *A23816-174*		Approx. Scale: *1: 60 000*	
Deposit Outline – R.M. Hardy 1977	▬▬▬▬	Test Pit ○	Hardy 1977
		Exposures: △	Hardy 1977
Deposit Outline – Klohn Leonoff 1986	▬ ▬ ▬	☒	Klohn Leonoff 1986

TEST HOLE LOG

SAMPLE DATA

WEIGHT HAMMER 63.5 Kg

HEIGHT DROP 0.76 m

SYMBOL

ELEV COLLAR

ELEV GROUND

CO-ORD. LOCATION

UNCONFINED COMPRESSION kPa
100 200 300 400
● FIELD VANE △ LAB VANE ■ UNCONF.

PLASTIC LIMIT x — WATER CONTENT 0 — LIQUID LIMIT x
10 30 50 70 90 %

DEPTH / ELEV.	O.D / I.D.	BLOWS / .15m	NO.	DESCRIPTION OF MATERIAL
				0.2 PEAT AND ORGANICS
				0.6 CLAY - silty, medium plastic - gravel sizes to 10 cm - brown
1				
2				

TEST PIT stopped at 0.6 m
because of frozen ground.

KLOHN LEONOFF
CONSULTING ENGINEERS

JOB No. PA 2291.01.01
PROJECT CONCRETE AGGREGATE STUDY
LOCATION MOOSE CREEK, YUKON
HOLE No. EXPOSURE NO. 2a
DATE Sept. 28/86 PLATE

K.L.C. - METRIC.

TEST HOLE LOG

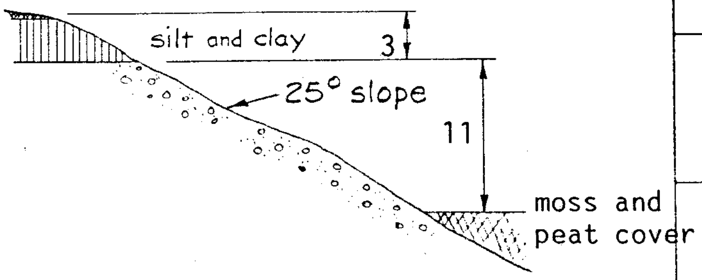
SAMPLE DATA
WEIGHT HAMMER 63.5 Kg
HEIGHT DROP 0.76 m

ELEV COLLAR
ELEV GROUND
CO-ORD LOCATION

UNCONFINED COMPRESSION kPa
100 200 300 400
● FIELD VANE △ LAB VANE ■ UNCONF

PLASTIC LIMIT x — WATER CONTENT o — LIQUID LIMIT x
10 30 50 70 90 %

DEPTH / ELEV	O.D / I.D	BLOWS / .15m	NO.	SYMBOL	DESCRIPTION OF MATERIAL
m					Moss overlying silt and clay till
2					
					3.0 m
4					GRAVEL AND SAND - well graded to 20 cm - trace of silt - rounded to sub rounded - very dense, dry
6			☒		
8					
10					NOTE: Sample taken between 6 and 8 m depth
12					
14					14.0
					- grading into Moss and peat cover



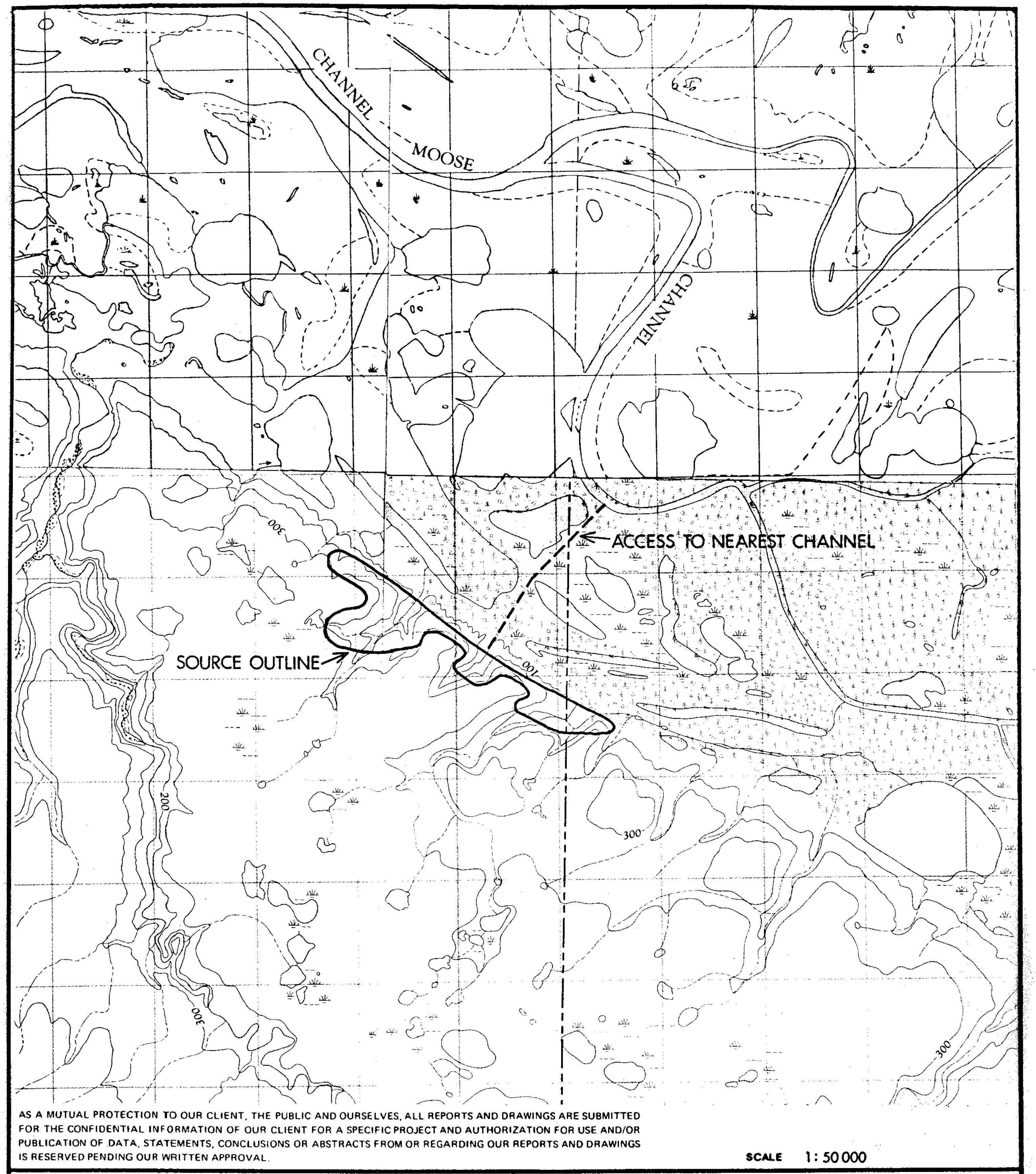
Klohn Leonoff Consultants Ltd.
CIVIL & GEOTECHNICAL ENGINEERS

JOB No. PA 2291.01.01
PROJECT CONCRETE AGGREGATE STUDY
LOCATION MOOSE CREEK, YUKON
HOLE No. EXPOSURE NO. 2b
DATE SEPT. 28/86 PLATE

K.L.C. - METRIC

MOOSE CREEK SOURCE AREA

Exposure No. 2, 1 mile southeast of Y-102 source.



SCALE 1: 50000

KLOHN LEONOFF LTD.
CONSULTING ENGINEERS

PROJECT WESTERN BEAUFORT REGION CONCRETE AGGREGATE STUDY

TITLE MOOSE CREEK
SOURCE LOCATION MAP

CLIENT:	DATE OF ISSUE	PROJECT No.		REV.
SUPPLY AND SERVICES CANADA	OCTOBER 1987 APPROVED KR	PA 2291	PLATE III - 1	

NCI-455-KL

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
	Pt	0.2	PEAT - fine, fibrous.		UF		Lightweight Pieces - Fine Aggregate: 0.11%
1 2 3 4	CI	4.2	CLAY - medium plastic, some gravel, few cobbles and boulders to 10" size, brown, damp to wet.		Vc- Vx 20%	2.7	
5	GW	5.5	GRAVEL - well graded to 6" cobbles, some fine to coarse sand, subangular to subrounded, well bonded.				
6			Bottom of Pit				Sample from 4.2'-5.5'

DATE: Oct. 9, 1976	LOGGED BY: CPM	DRWN BY: MB/vh	CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
Y102-A
SHEET 1 OF 1

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
	Pt	0.5	PEAT - fine, fibrous.		UF		
1	SM	1.7	SAND - some gravel, little silty fines, occasional cobbles.				
2	GW		GRAVEL - well graded to 13" boulders, little fine to coarse sand, trace fines, subangular to subrounded.				
3							
4							
5		5.5					
6			Bottom of Pit				Sample from 1.7'-5.5'

DATE: Oct. 12, 1976

LOGGED BY: CPM

DRWN BY: MB/vh

CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

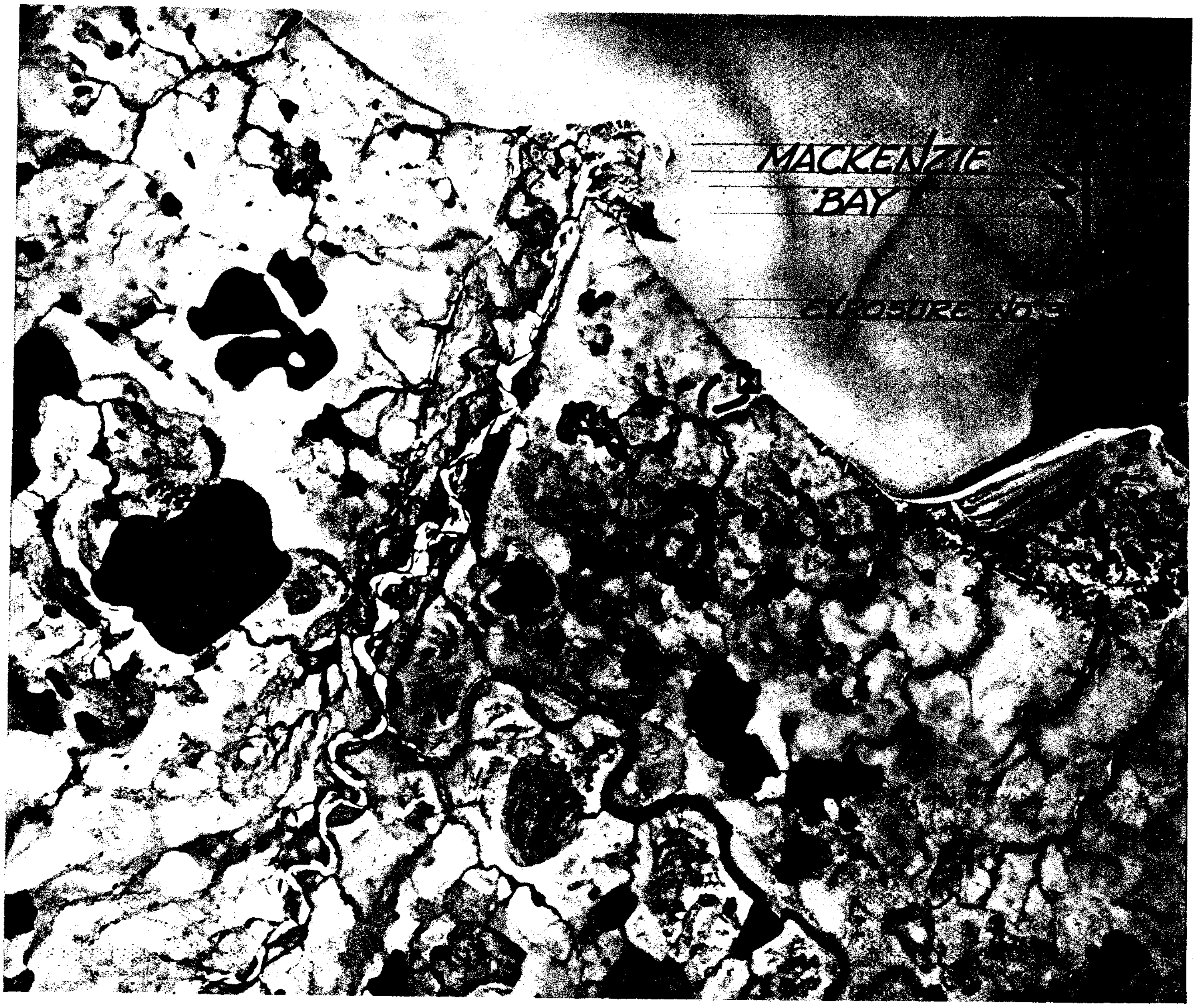
R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
Y102-B

SHEET 1 OF 1

APPENDIX IV
SHINGLE POINT

SHINGLE POINT



Airphoto No. A13751-32

Approx. Scale: 1:68 000

Deposit Outline – R.M. Hardy 1977 ▬▬▬

Deposit Outline – Klohn Leonoff 1986 ▬ ▬ ▬

Test Pit ○ } Hardy 1977

Exposures: △ } Hardy 1977

☒ Klohn Leonoff 1986

TEST HOLE LOG

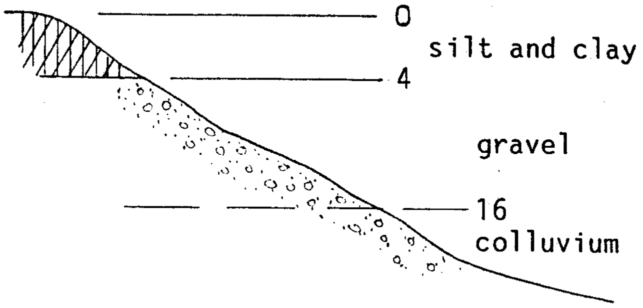
SAMPLE DATA
WEIGHT HAMMER 63.5 Kg
HEIGHT DROP 0.76 m

ELEV COLLAR
ELEV GROUND
CO-ORD. LOCATION

UNCONFINED COMPRESSION kPa
100 200 300 400
● FIELD VANE △ LAB VANE ■ UNCONF

PLASTIC LIMIT WATER CONTENT LIQUID LIMIT
x - - - - - o - - - - - x
10 30 50 70 90 %

DEPTH ELEV	O.D I.D	BLOWS .15m	NO.	SYMBOL	DESCRIPTION OF MATERIAL
m					MOSS AND PEAT COVER
2				2.0m	SILT AND CLAY - medium plastic - trace of gravel - brown - black
4				4.0 m	GRAVEL AND SAND - well graded to max size of 20cm - slightly stratified with thin fine sand and silt. - some cross bedding - angular to sub angular - dry
6					
8					
12			☒		
14					NOTE: Sample taken between 12 and 16 m
16				16.0	- grading into colluvium at bottom of slope



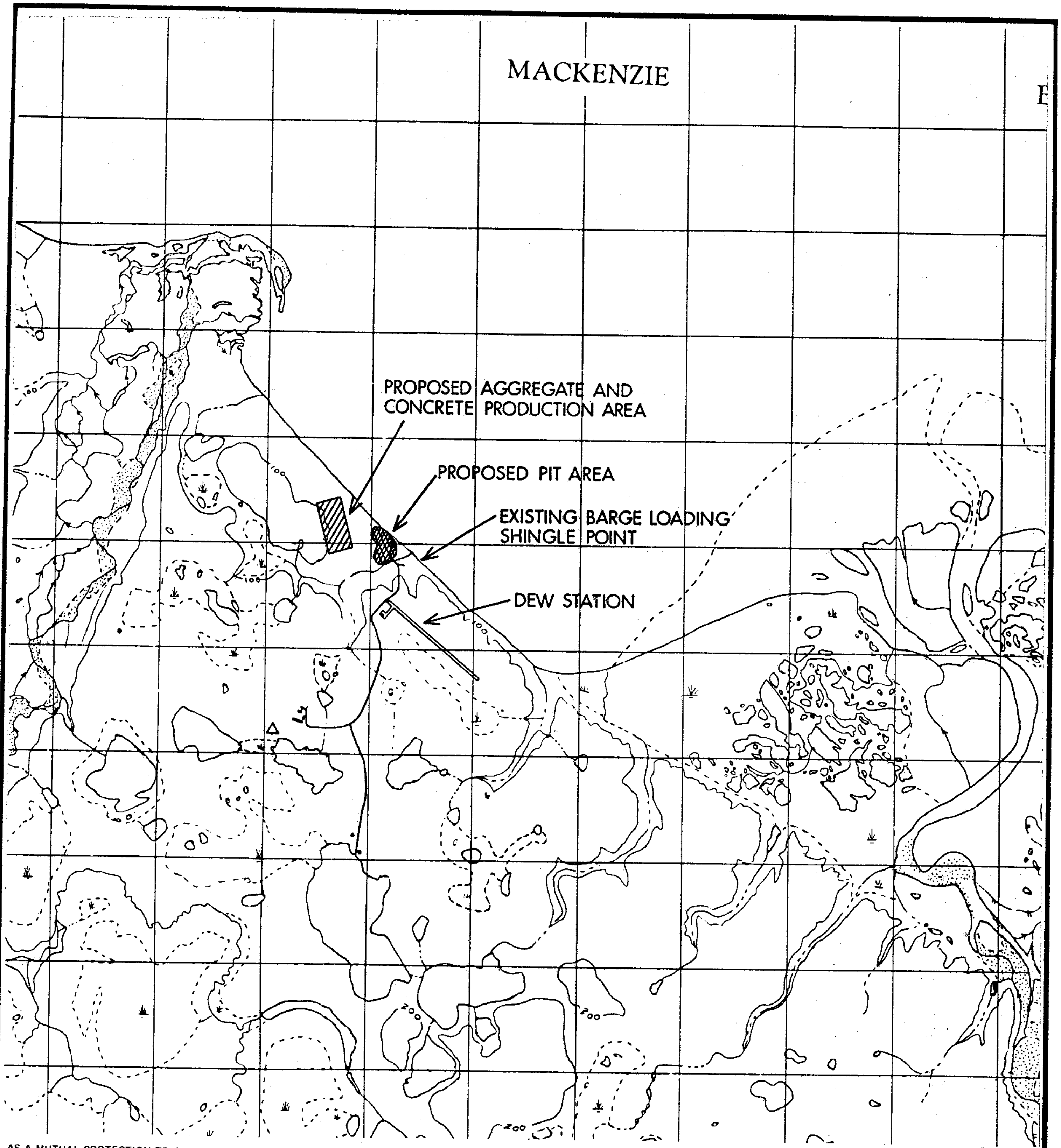
Klohn Leonoff Consultants Ltd.
CIVIL & GEOTECHNICAL ENGINEERS

JOB No PA 2291.01.01
PROJECT CONCRETE AGGREGATE STUDY
LOCATION SHINGLE POINT
HOLE No EXPOSURE NO. 3
DATE Sept. 26 /87 PLATE

K.L.C. - METRIC

SHINGLE POINT SOURCE AREA

Exposure No. 3 at sample area.



SCALE 1: 50 000

KLOHN LEONOFF LTD.
CONSULTING ENGINEERS

PROJECT WESTERN BEAUFORT REGION
CONCRETE AGGREGATE STUDY

TITLE
SHINGLE POINT
SOURCE LOCATION MAP

CLIENT:
SUPPLY AND SERVICES CANADA

DATE OF ISSUE
OCTOBER 1987

APPROVED

PROJECT No.
PA 2291

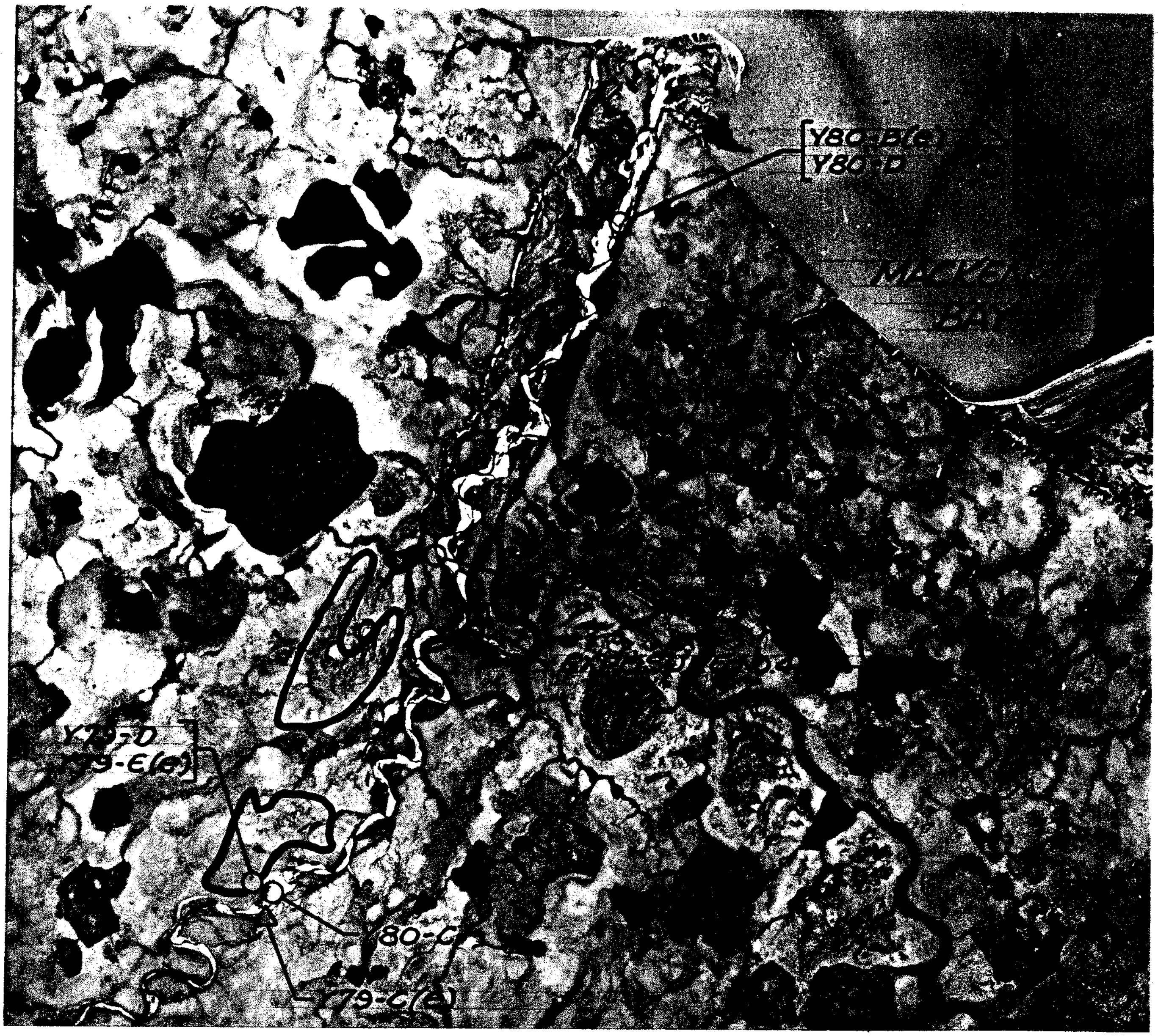
PLATE IV-1

REV.

APPENDIX V

RUNNING RIVER

RUNNING RIVER



Airphoto No. *A-13751-32*

Approx. Scale: *1:68 000*

Deposit Outline – R.M. Hardy 1977 ━━━━━━

Deposit Outline – Klohn Leonoff 1986 ━ ━ ━

Test Pit ○ } Hardy 1977
Exposures: △ } Hardy 1977

☒ Klohn Leonoff 1986

TEST HOLE LOG

SAMPLE DATA

WEIGHT HAMMER 63.5 Kg

HEIGHT DROP 0.76 m

DEPTH ELEV.	O.D. I.D.	BLOWS .15m	NO.	SYMBOL

ELEV. COLLAR

ELEV. GROUND

CO-ORD. LOCATION

UNCONFINED COMPRESSION kPa 100 200 300 400

● FIELD VANE △ LAB VANE ■ UNCONF.

PLASTIC LIMIT **WATER CONTENT** **LIQUID LIMIT**

X — — — — — — O — — — — — — X

10 30 50 70 90 %

DESCRIPTION OF MATERIAL

- moss and peat 0.3 m thick
- clay and silt

1.0

SAND
- trace of silt
- thin silt and clay layers
- fine to medium
- few pebbles
- brown

5.0

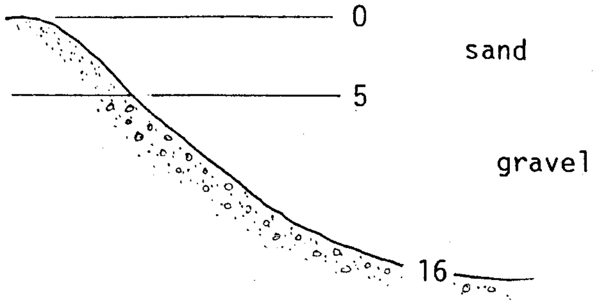
GRAVEL AND SAND

- trace of silt
- slightly stratified
- well graded
- max size to 15 cm
- angular to sub rounded
- dry

NOTE: Sample taken between 14 and 16 m

18.0

Depth scale: 2, 4, 6, 8, 14, 16, 18



Klohn Leonoff Consultants Ltd.

CIVIL & GEOTECHNICAL ENGINEERS

JOB No. PA 2291.01.01

PROJECT CONCRETE AGGREGATE STUDY

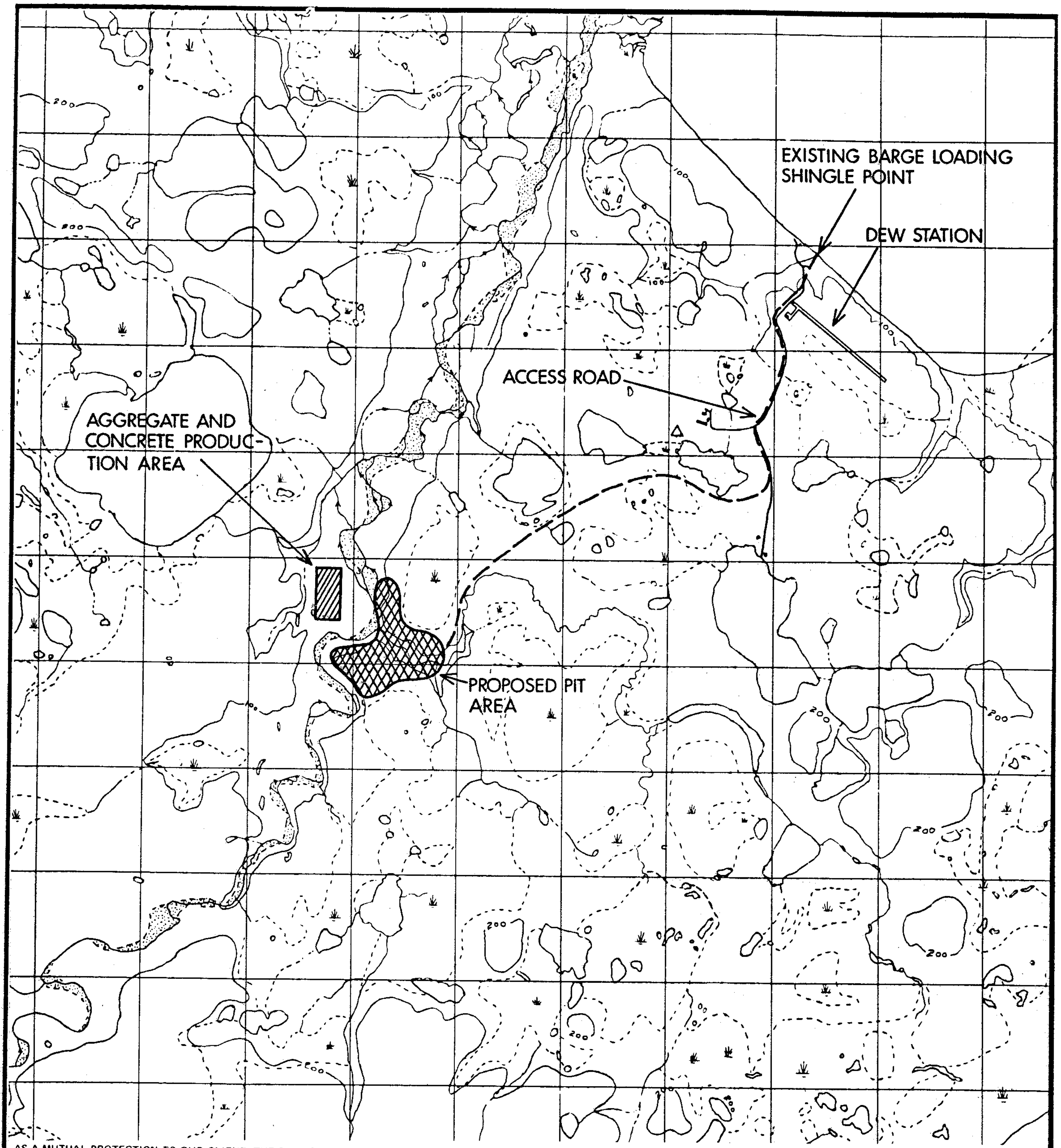
LOCATION RUNNING RIVER

HOLE No. EXPOSURE NO. 4

DATE Sept. 27 /86 PLATE

RUNNING RIVER SOURCE AREA

Exposure No. 4 looking upstream adjacent to area sampled.



SCALE 1 : 50000

KLOHN LEONOFF LTD.
CONSULTING ENGINEERS

PROJECT WESTERN BEAUFORT REGION
CONCRETE AGGREGATE STUDY

TITLE
RUNNING RIVER
SOURCE LOCATION MAP

CLIENT:
SUPPLY AND SERVICES CANADA

DATE OF ISSUE OCTOBER 1987

APPROVED

PROJECT No. PA 2291

PLATE V - 1

REV.

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
1	GW	2.0	GRAVEL - well graded to 3" size, and fine to coarse sand, in 2" layers, subangular to subrounded.		UF		Lightweight Pieces - Fine Aggregate: 0.05%
2	SP	2.4	SAND - fine to medium, 1" layer of fine gravel to ½" size.				
3 4	GW	4.5	GRAVEL - 1½" maximum size, some fine to coarse sand, subangular to subrounded, sandstones.				
5 6 7		7.4	increasing in size to 7" cobbles.				
8 9 10 11		11.0	decreasing to 3" maximum size.				river level 2' below bottom of pit.
12			Bottom of Pit				Sample from 0'-11.0'

DATE: Sept. 27, 1976

LOGGED BY: GCD

DRWN BY: MB/vh

CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
Y79-C(e)

SHEET 1 OF 1

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
	Pt		PEAT - fibrous, brown.		UF		
1		10					
	ML		SILT - firm, brown, wet.				
2			- frozen at 2.4'.				
		24	Bottom of Pit			24	No sample taken.
3							

DATE: Oct. 1, 1976

LOGGED BY: SA

DRWN BY: MB/vh

CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
Y79-D
SHEET 1 OF 1

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
10	GW	16.5	GRAVEL - well graded to 3" size, some fine to coarse sand, trace silty fines, subrounded to rounded, medium dense, grey-brown, wet.		UF		Lightweight Pieces - Fine Aggregate: 0.69%
	ML	19.5	SILT - trace sand and gravel, clayey firm, grey-brown, wet.				
20 30	GM- SM	38.0	GRAVEL AND SAND - well graded to 3" size, little silty fines, subrounded to rounded, medium dense, wet, predominantly shale particles.				
40	GW	47.0	GRAVEL - well graded to 3" size, little sand, subrounded to rounded, medium dense, wet, some platy rock.				
50			Bottom of Pit				Sample from 10.0'-15.0'

DATE: Oct. 1, 1976

LOGGED BY: SA

DRWN BY: MB/vh

CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
Y79-E(e)
SHEET 1 OF 1

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
	GW		GRAVEL - well graded to 4" cobbles, some fine to coarse sand, medium brown, damp, clean.		UF		Lightweight Pieces - Fine Aggregate: 0.05%
1							
2							
3			3.0 3.2 - fine sand layer.				
4			4.0 wet.				
			4.5 ▽ free water.				
5			Bottom of Pit				Sample from 0'-4.5'

DATE: Sept. 27, 1976

LOGGED BY: JDF

DRWN BY: MB/vh

CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
Y80-A

SHEET 1 OF 1

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
	GW		GRAVEL - well graded, some fine to coarse sand, subrounded to rounded, platy, dense, brown, moist, clean.		UF		
1							
2							
3							
		4.0	▽ - river level at 4.0'				
4			Bottom of Pit				Sample from 0'-4.0'
5							

DATE: Oct. 1, 1976

LOGGED BY: SA

DRWN BY: MB/vh

CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
Y80-B(e)

SHEET 1 OF 1

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
1 2 3 4 5	GW	░ 5.7	GRAVEL - well graded to 12" boulders, little fine to coarse sand, subangular to subrounded, igneous rocks, sandstones, fine gravel cementations, clean.		UF		Moisture Content: 2.2% Lightweight Pieces - Fine Aggregate: 0.06% hole sloughing in.
6			Bottom of Pit				Sample from 0'-5.7'

DATE: Sept. 27, 1976
LOGGED BY: GCD
DRWN BY: MB/vh
CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS AND NORTHERN DEVELOPMENT

R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO. Y80-C
SHEET 1 OF 1

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
	SM	0.5	SAND - fine, silty, organic, rootlets, grey-brown, wet.		UF		Organic Color: #5+
1 2 3 4	GW	4.0	GRAVEL - well graded to 2½" size, some fine to coarse sand, layered, subrounded to rounded, grey-brown, wet.				Lightweight Pieces - Fine Aggregate: 0.07%
5			Bottom of Pit				Sample from 0.5'-3.5'

DATE: Oct. 1, 1976

LOGGED BY: SA

DRWN BY: MB/vh

CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

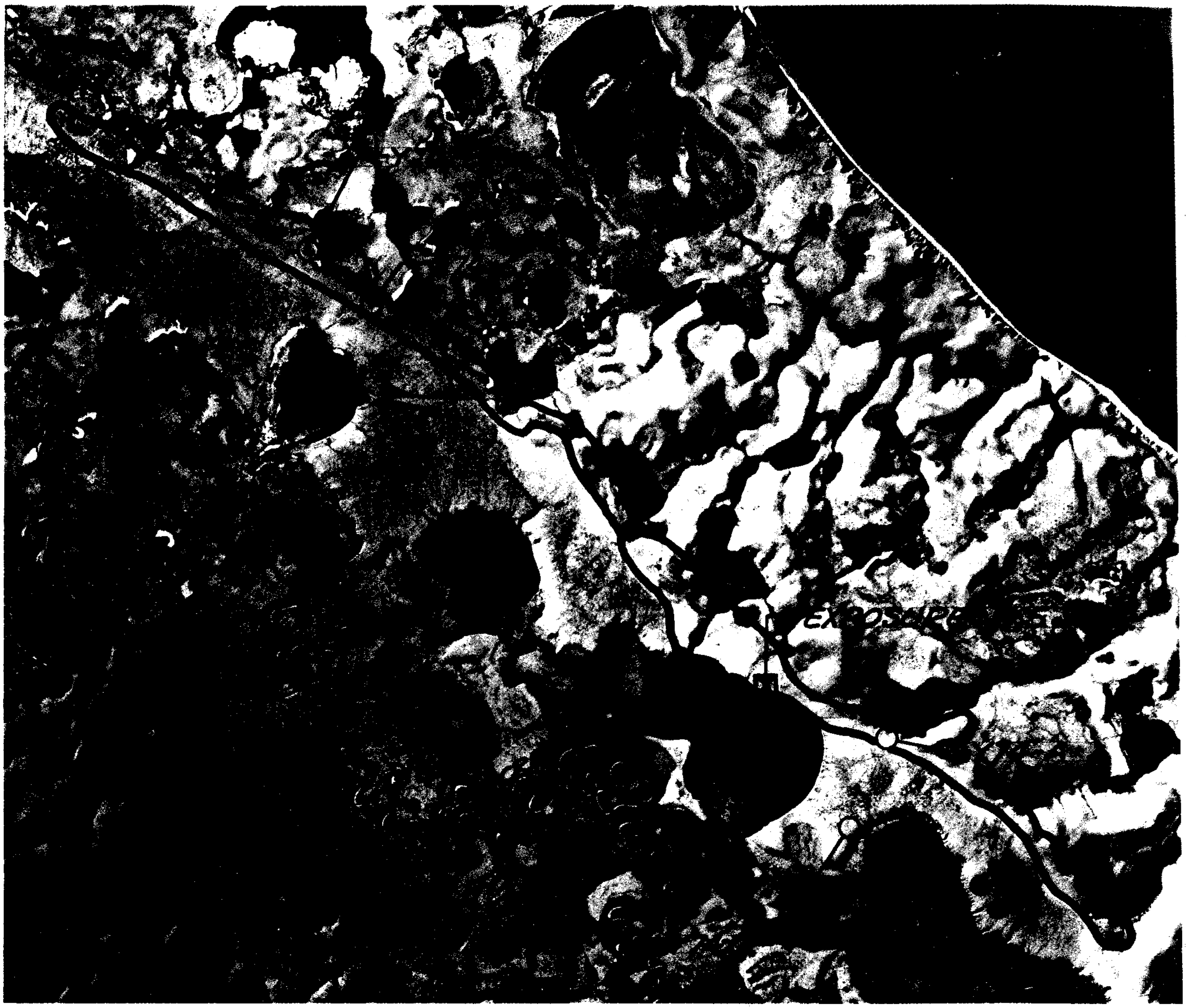
R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
Y80-D
SHEET 1 OF 1

APPENDIX VI

JACOBS RIDGE

JACOBS RIDGE



Airphoto No. A22975-153/A22882-24

Approx. Scale: 1:60 000

Deposit Outline – R.M. Hardy 1977 ▬▬▬▬▬

Deposit Outline – Klohn Leonoff 1986 ▬ ▬ ▬

Test Pit ○ } Hardy 1977
Exposures: △ } Hardy 1977

☒ Klohn Leonoff 1986
● Klohn Leonoff 1975

TEST HOLE LOG

SAMPLE DATA

WEIGHT HAMMER 63.5 Kg

HEIGHT DROP 0.76 m

SYMBOL

ELEV COLLAR

ELEV GROUND

CO-ORD. LOCATION

UNCONFINED COMPRESSION kPa

100 200 300 400

● FIELD VANE △ LAB VANE ■ UNCONF

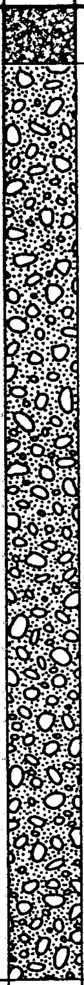
PLASTIC LIMIT WATER CONTENT LIQUID LIMIT

x – – – – – o – – – – – x

10 30 50 70 90 %

DEPTH ELEV	O.D / I.D	BLOWS .15m	NO.
2			
4			
6			
12			
14			
16			
18			

DESCRIPTION OF MATERIAL



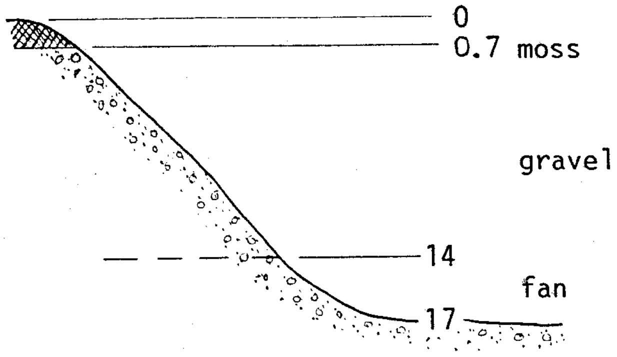
0.7 – moss and peat
– silt and sand

GRAVEL AND SAND

- clean, well graded
- cobbles to 30 cm
- angular to sub rounded
- contains thin silt and sand layers
- dry
- sandier below 12 m

-fan and slope wash

17.0



Klohn Leonoff Consultants Ltd.

CIVIL & GEOTECHNICAL ENGINEERS

JOB No PA 2291.01.01

PROJECT CONCRETE AGGREGATE STUDY

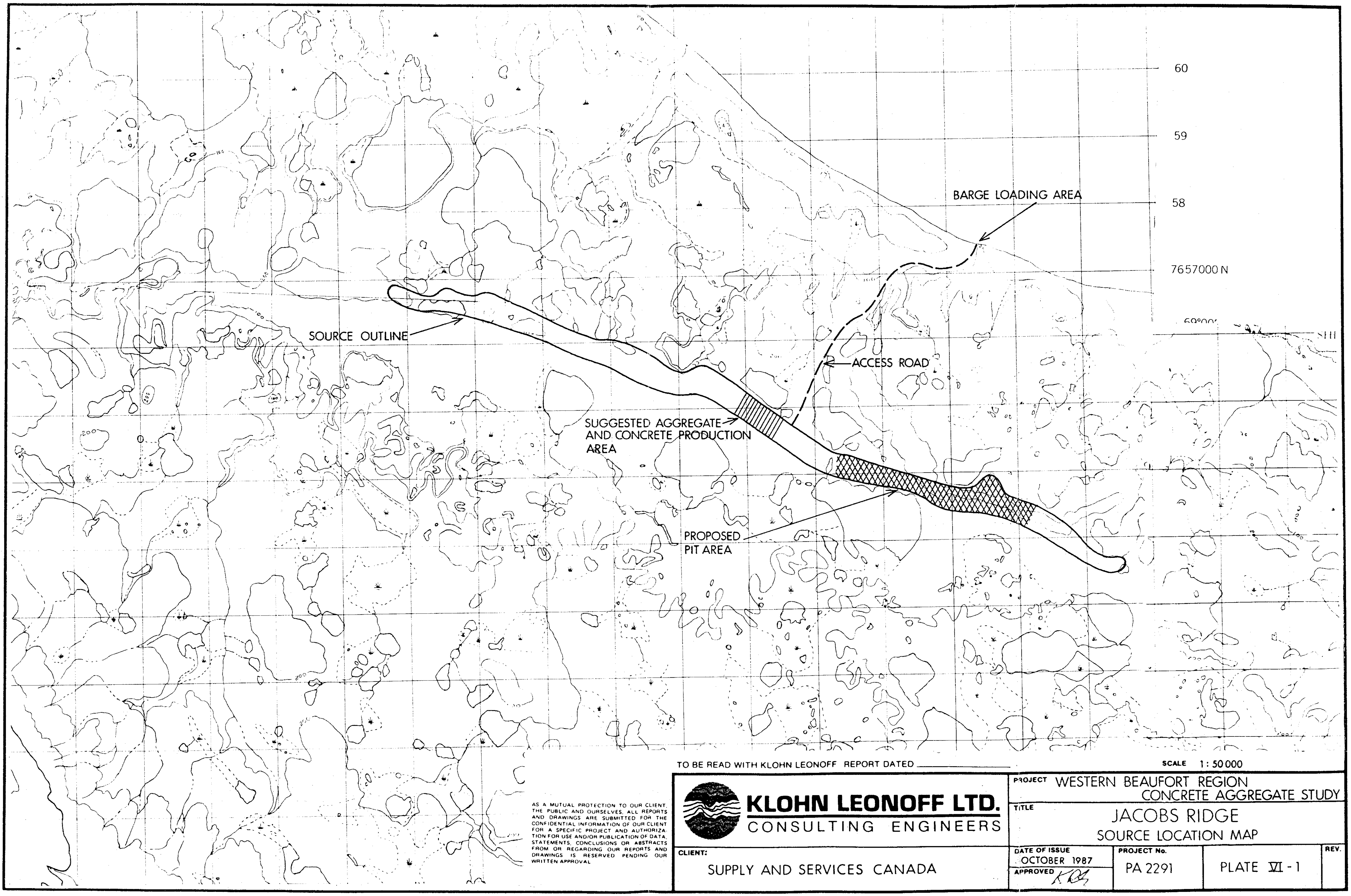
LOCATION JACOBS LAKE RIDGE

HOLE No. EXPOSURE NO. 5

DATE Sept. 27 /87 PLATE

JACOBS RIDGE SOURCE AREA

Exposures in Jacobs Lake



TO BE READ WITH KLOHN LEONOFF REPORT DATED ____________

SCALE 1 : 50 000

KLOHN LEONOFF LTD.
CONSULTING ENGINEERS

CLIENT:
SUPPLY AND SERVICES CANADA

PROJECT WESTERN BEAUFORT REGION
CONCRETE AGGREGATE STUDY

TITLE
JACOBS RIDGE
SOURCE LOCATION MAP

DATE OF ISSUE
OCTOBER 1987

APPROVED K.B.

PROJECT No.
PA 2291

PLATE VI - 1

REV.

NCI-456-KL

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
	Pt	0.3	PEAT		UF		Moisture Content: 4.1%
	ML	0.9	SILT - low plastic, little coarse gravel, trace sand, rootlets, light brown, wet.				Lightweight Pieces - Fine Aggregate: 0.29%
1	GM	1.2	GRAVEL - some fine to coarse sand, little silty fines, occasional cobble, subangular, wet.				
2	GW		GRAVEL - well graded, some fine to coarse sand, trace silty fines.				
3							
4							
5							
6		6.0	Bottom of Pit				Sample from 1.2'-6.0'
7							

DATE: Sept. 17, 1976
LOGGED BY: JDF
DRWN BY: MB/vh
CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
Y74-A

SHEET 1 OF 1

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
	SM	0.4	SAND - fine to medium, little silty fines, trace gravel, organic, rootlets, brown, wet.		UF		
1	GW		GRAVEL - well graded to 4" cobbles, and fine to coarse sand, trace fines, subangular, damp.				
2							
3		3.0 3.2	- layer of clean subangular gravel.				
4							
5		5.0	wet.				depth of active layer estimated at 6.5' by probe.
		5.6	▽ saturated at 5.6'.				
6			Bottom of Pit				Sample from 0.4'-5.6'

DATE: Sept. 17, 1976	LOGGED BY: JDF	DRWN BY: MB/vh	CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
Y74-B

SHEET 1 OF 1

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
1	GW	1.6	GRAVEL - well graded to 4" cobbles, some fine to coarse sand, subrounded to rounded, brown to black, damp, sandstones.		UF		Organic Color: #4 Coal Removed: #2+ Moisture Content: 4.2%
2	GP	2.0	GRAVEL - fine to ½" size, some coarse sand.				Lightweight Pieces - Fine Aggregate: 0.07%
3	GW	3.9	GRAVEL - well graded to 6" cobbles, some fine to coarse sand, subrounded to rounded, moist from 3.5', sandstones, cherts, coal to 1" size.				
4	GP	4.1	GRAVEL - fine, and coarse sand.				
5 6	GW	6.5 ▽ 7.0	GRAVEL - well graded to 3" size, some fine to coarse sand, subrounded to rounded, sandstones, cherts.				
7	GP	7.5	GRAVEL - fine to ½" size, some fine to coarse sand.	+ +	Vx 5%	7.0	
8			Bottom of Pit				Sample from 0'-7.5'

DATE: Sept. 17, 1976

LOGGED BY: GCD

DRWN BY: MB/vh

CHKD BY: GCD/TJF

GOVERNMENT OF CANADA

DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
Y74-C

SHEET 1 OF 1

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
	Pt	0.5	PEAT - fine, fibrous.		UF		Organic Color: #5+
1	CL	1.5	CLAY - silty, trace gravel, low plastic, brown, moist.				Lightweight Pieces - Fine Aggregate: 0.25%
2	GW		GRAVEL - well graded to 3" size, some fine to coarse sand, occasional cobbles to 5" size, trace silty fines, subangular to rounded, brown, moist.				
3					Vc Vx 7%	2.7	
4							
5							
6		6.0					
			Bottom of Pit				Sample from 1.5'-6.0'
7							

DATE: Oct. 6, 1976

LOGGED BY: CPM

DRWN BY: MB/vh

CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

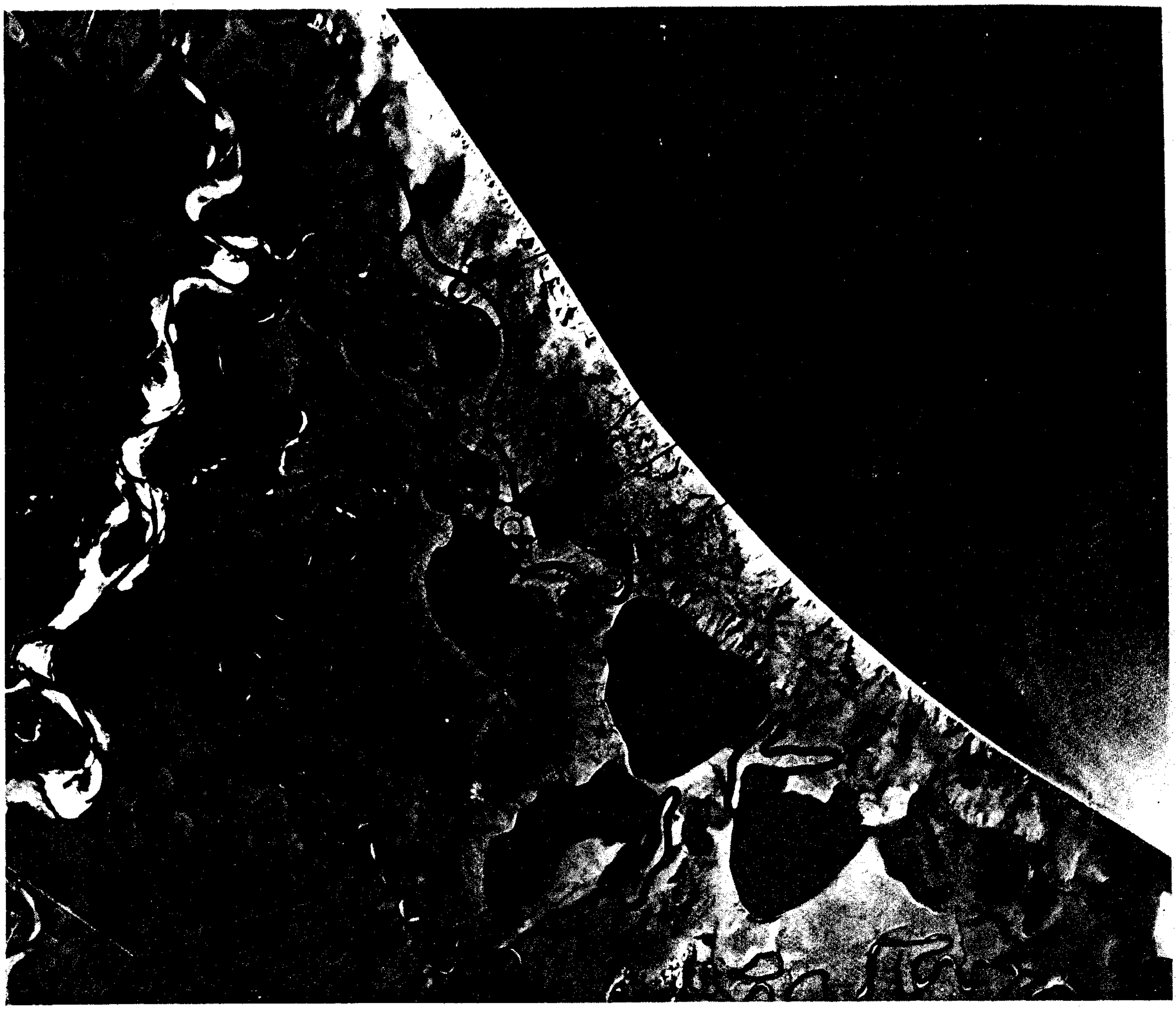
R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
Y74-D

SHEET 1 OF 1

APPENDIX VII
KING POINT

KING POINT



Airphoto No. A14406-48

Deposit Outline – R.M. Hardy 1977 ━━━━━━

Deposit Outline – Klohn Leonoff 1986 ▬ ▬ ▬

Approx. Scale: 1:68 000

Test Pit ○ } Hardy 1977
Exposures: △ } Hardy 1977

☒ Klohn Leonoff 1986

TEST HOLE LOG

SAMPLE DATA

WEIGHT HAMMER 63.5 Kg

HEIGHT DROP 0.76 m

DEPTH ELEV	O.D I.D	BLOWS .15m	NO.
2			
4			
6			
8			
10			

SYMBOL

ELEV COLLAR

ELEV GROUND

CO-ORD. LOCATION

DESCRIPTION OF MATERIAL

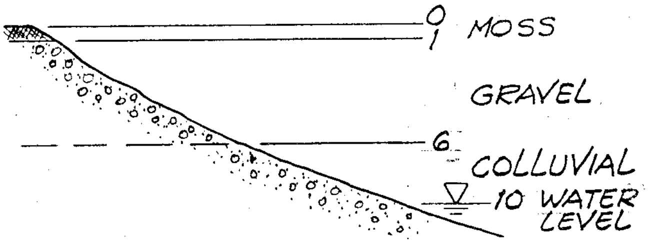
1.0
- moss cover
- silt and sand

GRAVEL AND SAND

- clean, well graded
- rounded to sub rounded
- some coarse gravel layers to 10 cm size
- dry

- lower 4 m covered with colluvial material

10.0



UNCONFINED COMPRESSION kPa

100 200 300 400

● FIELD VANE △ LAB VANE ■ UNCONF.

PLASTIC LIMIT x — WATER CONTENT o — LIQUID LIMIT x

10 30 50 70 90 %

Klohn Leonoff Consultants Ltd.

CIVIL & GEOTECHNICAL ENGINEERS

JOB No PA 2291.01.01

PROJECT CONCRETE AGGREGATE STUDY

LOCATION KING POINT

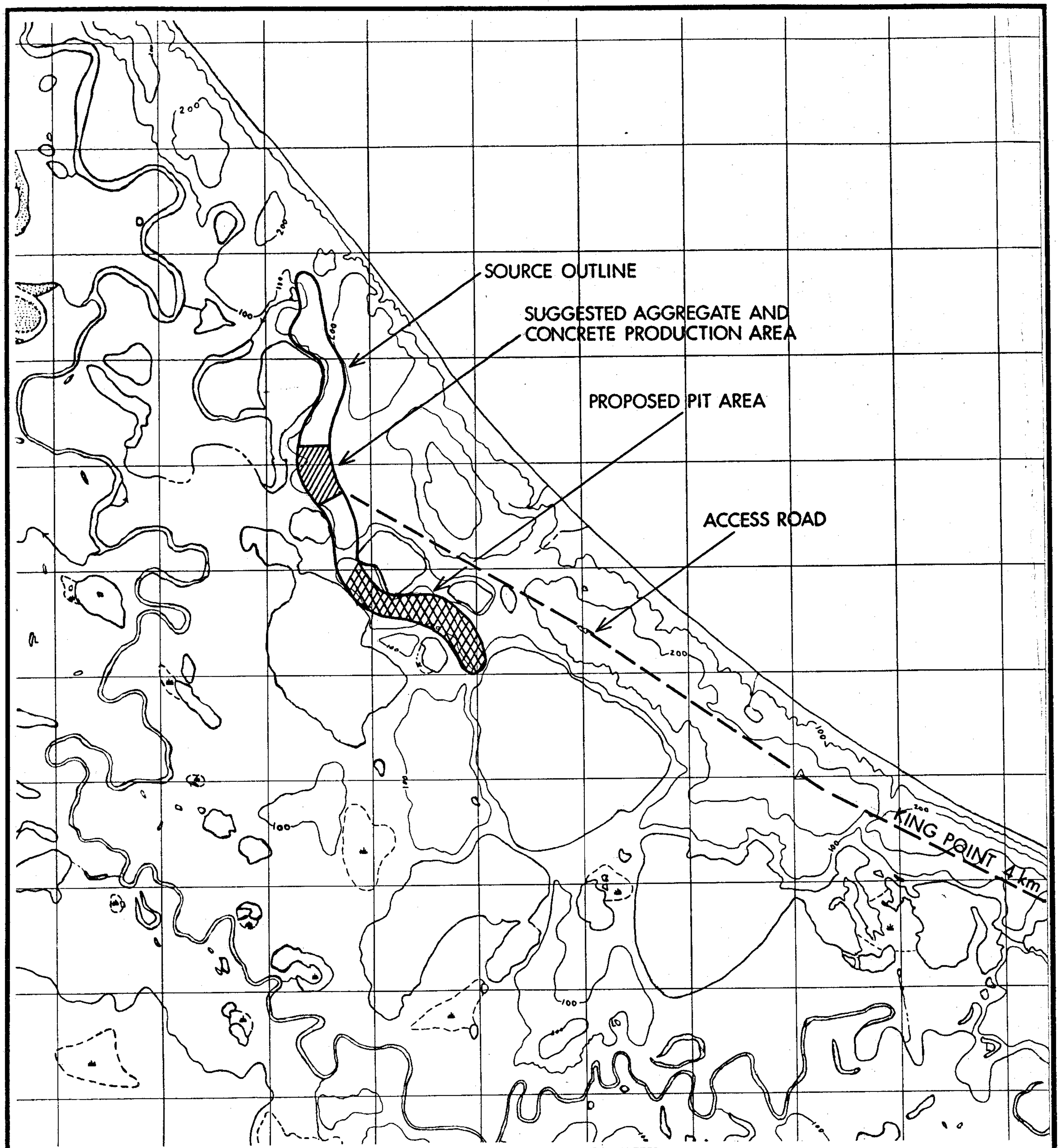
HOLE No EXPOSURE NO. 6

DATE Feb. 27 /86 PLATE

K.L.C. - METRIC

KING POINT SOURCE AREA

Exposure along gully at north end of source (HARDY Y-62D[e])



SCALE 1:50000

KLOHN LEONOFF LTD.
CONSULTING ENGINEERS

PROJECT WESTERN BEAUFORT REGION
CONCRETE AGGREGATE STUDY

TITLE
KING POINT
SOURCE LOCATION MAP

CLIENT:
SUPPLY AND SERVICES CANADA

DATE OF ISSUE
OCTOBER 1987

APPROVED

PROJECT No.
PA 2291

PLATE VII-1

REV.

NCI-455-KL

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
1 2 3	GW	3.0 ▽ 3.5	GRAVEL - well graded to 3" size, and fine to coarse sand, subrounded to rounded, light to dark brown, wet below 2'.		UF		Lightweight Pieces - Fine Aggregate: 0.07%
4			Bottom of Pit				Sample from 0'-3.0'

DATE: Sept. 19, 1976	LOGGED BY: GCD	DRWN BY: MB/vh	CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
Y62-A
SHEET 1 OF 1

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
	Pt	0.5	PEAT		UF		Organic Color: #5+
1	ML	1.8	SILT - low plastic, trace fine to medium sand, brown.		Vx 30%	1.3 1.7	Coal Removed: #2 Lightweight Pieces - Fine Aggregate: 0.23%
2 3 4 5	GP	5.9	GRAVEL - fine, and fine to coarse sand, subrounded to rounded, becoming rounded and coarser to 2" size at 2.5', cherts, sandstones, trace friable gravels.		Vc 20%		
6 7			Bottom of Pit				Sample from 1.8'-5.9'

DATE: Sept. 20, 1976

LOGGED BY: GCD

DRWN BY: MB/vh

CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
Y62-B

SHEET 1 OF 1

TEST HOLE LOG

DEPTH (FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
	Pt	0.5	PEAT - organic cover.		UF		
1	OL		SILT - organic, low plastic, black.		Vr Vs 10%	1.0	ice lenses to ½".
2							
3							
4		4.0	Bottom of Pit				No sample taken.
5							

DATE: Sept. 19, 1976

LOGGED BY: JDF

DRWN BY: MB/vh

CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
Y62-C

SHEET 1 OF 1

TEST HOLE LOG

(FT)	SOIL GROUP SYMBOL	SOIL GRAPHIC LOG	MATERIAL DESCRIPTION	ICE GRAPHIC LOG	NCR ICE TYPE VISUAL ICE %	DEPTH (FT)	OTHER INFORMATION
1 2	SM	2.5	SAND - fine to medium, little silty fines, organic, light brown.		UF		Top of exposure at Y62-D(e) 10' in elevation below ground surface at Y62-C on nearby crest of out-wash fan. Lightweight Pieces - Fine Aggregate: 0.19%
3 4	GW	10.0	GRAVEL - well graded to 3" size, and fine to coarse sand, trace silty fines, rootlets to 5.0' depth, subangular sandstones, angular shale fragments, dark brown, wet.				
			Bottom of Pit				Sample from 2.5'-10.0'

TE: Sept. 19, 1976	LOGGED BY: JDF	DRWN BY: MB/vh	CHKD BY: GCD/TJF

GOVERNMENT OF CANADA
DEPARTMENT OF INDIAN AFFAIRS
AND
NORTHERN DEVELOPMENT

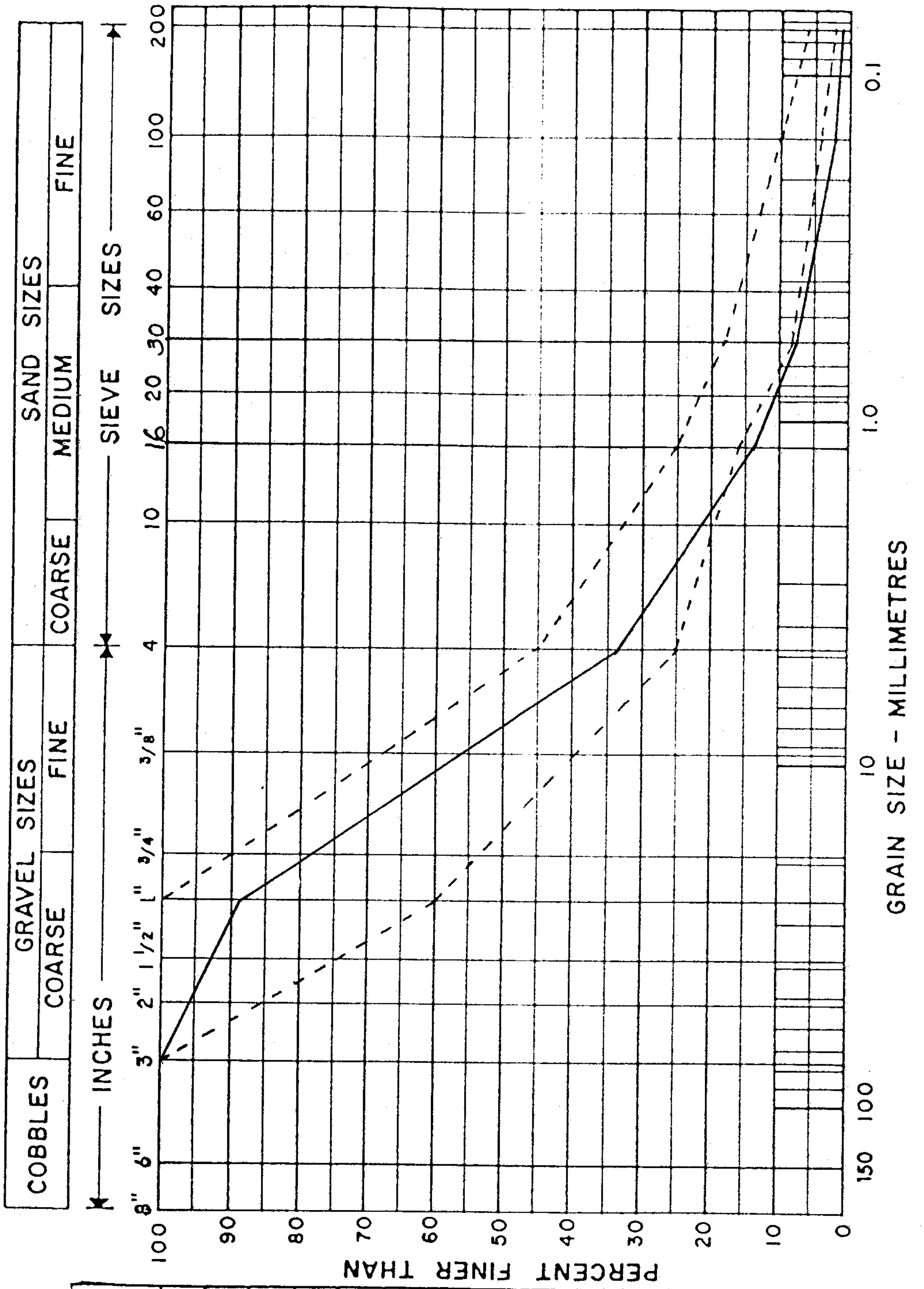
R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERS & PROFESSIONAL SERVICES
• GEOTECHNICAL DIVISION

TEST PIT NO.
Y62-D(e)
SHEET 1 OF 1

APPENDIX VIII

PIT RUN GRADING ANALYSES

GRAIN SIZE ANALYSIS
GRANULAR SOILS



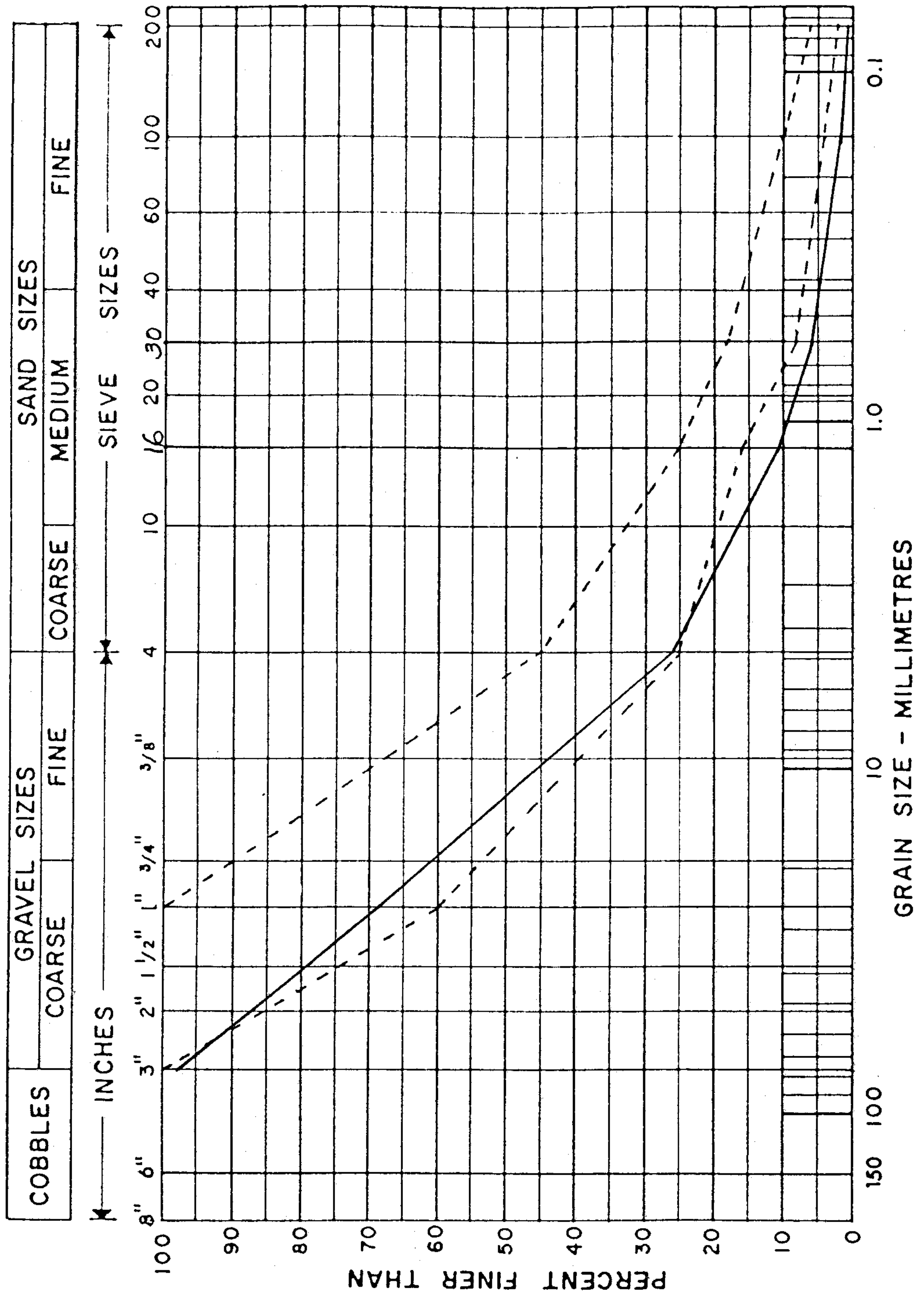
SIEVE SIZES U.S.	METRIC	% PASSING
6"	150,000	
3"	80 000	
2"	50 000	
1.5"	40 000	
1"	25 000	
.75"	20 000	
.5"	12 500	
.375"	10 000	
No. 4	5 000	
No. 8	2 500	
16	1 250	
30	630	
40	400	
50	315	
100	160	
200	80	

REMARKS:

KLOHN LEONOFF
CONSULTING ENGINEERS

JOB No.
PROJECT
LOCATION
SAMPLE TYPE No. 1 Shingle Point
DATE

GRAIN SIZE ANALYSIS
GRANULAR SOILS



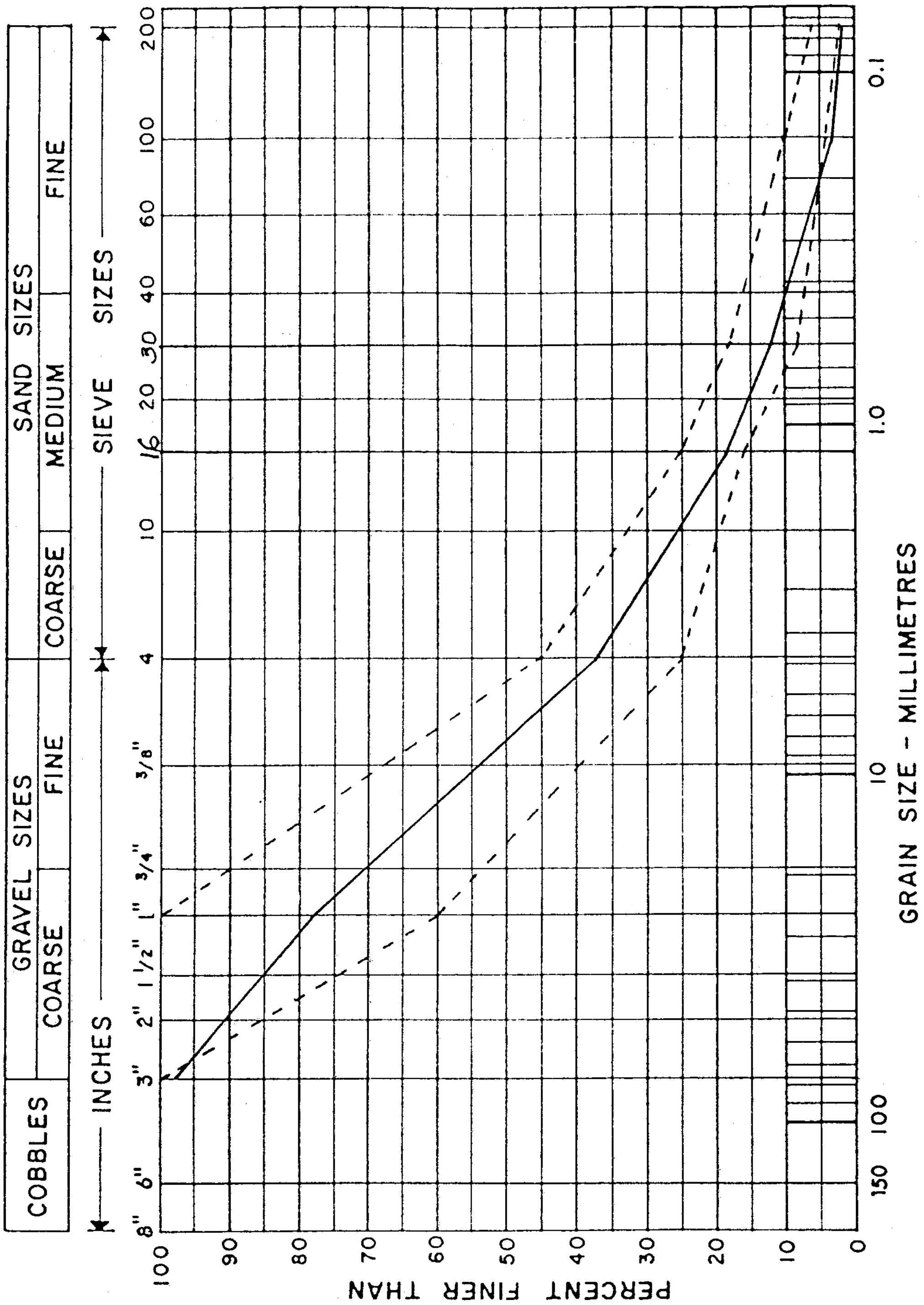
SIEVE SIZES		% PASSING
U.S.	METRIC	
6"	150 000	
3"	80 000	
2"	50 000	
1.5"	40 000	
1"	25 000	
.75"	20 000	
.5"	12 500	
.375"	10 000	
No. 4	5 000	
No. 8	2 500	
16	1 250	
30	630	
40	400	
50	315	
100	160	
200	80	

REMARKS:

KLOHN LEONOFF
CONSULTING ENGINEERS

JOB No.
PROJECT
LOCATION
SAMPLE TYPE No. 2 Running River
DATE

GRAIN SIZE ANALYSIS
GRANULAR SOILS



SIEVE SIZES U.S.	SIEVE SIZES METRIC	% PASSING
6"	150 000	
3"	80 000	
2"	50 000	
1.5"	40 000	
1"	25 000	
.75"	20 000	
.5"	12 500	
.375"	10 000	
No. 4	5 000	
No. 8	2 500	
16	1 250	
30	630	
40	400	
50	315	
100	160	
200	80	

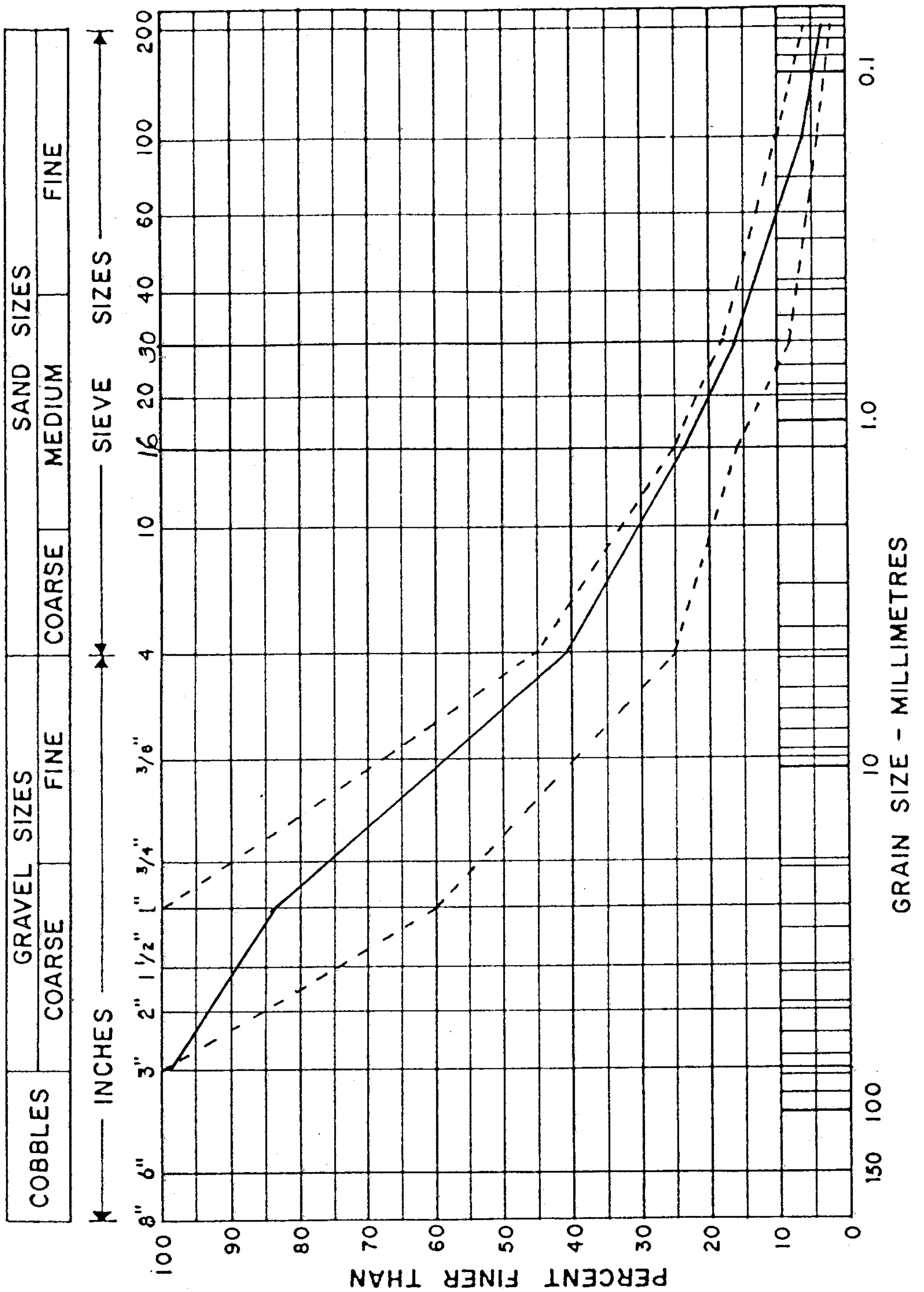
REMARKS:

KLOHN LEONOFF
CONSULTING ENGINEERS

JOB No.
PROJECT
LOCATION
SAMPLE TYPE No. 3 King Point
DATE

GRAIN SIZE ANALYSIS

GRANULAR SOILS



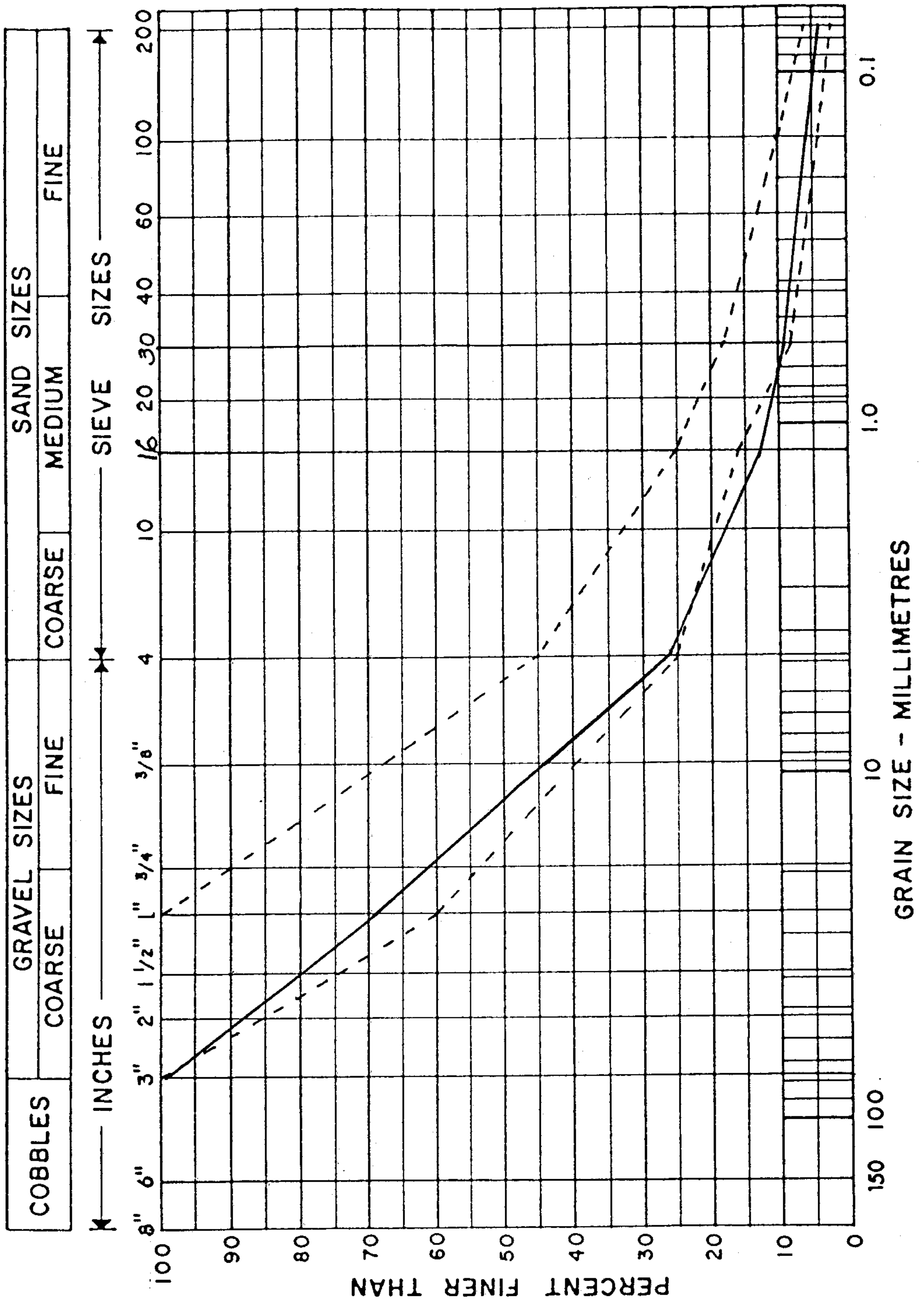
SIEVE SIZES U.S.	SIEVE SIZES METRIC	% PASSING
6"	150 000	
3"	80 000	
2"	50 000	
1.5"	40 000	
1"	25 000	
.75"	20 000	
.5"	12 500	
.375"	10 000	
No. 4	5 000	
No. 8	2 500	
16	1 250	
30	630	
40	400	
50	315	
100	160	
200	80	

REMARKS:

KLOHN LEONOFF
CONSULTING ENGINEERS

JOB No.
PROJECT
LOCATION
SAMPLE TYPE No. 4 Jacobs Ridge
DATE

GRAIN SIZE ANALYSIS
GRANULAR SOILS



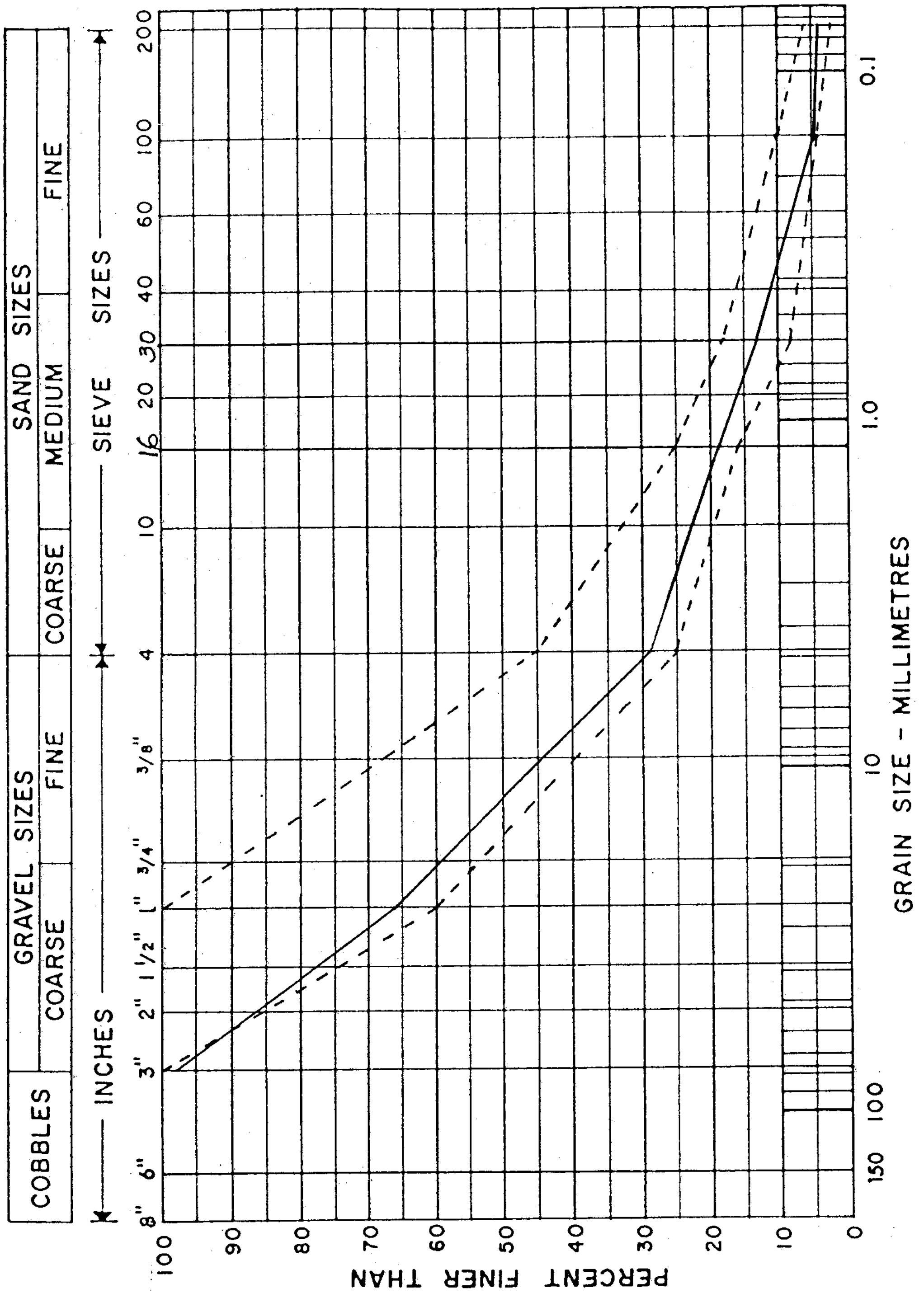
SIEVE SIZES		% PASSING
U.S.	METRIC	
6"	150 000	
3"	80 000	
2"	50 000	
1.5"	40 000	
1"	25 000	
.75"	20 000	
.5"	12 500	
.375"	10 000	
No. 4	5 000	
No. 8	2 500	
16	1 250	
30	630	
40	400	
50	315	
100	160	
200	80	

REMARKS:

KLOHN LEONOFF
CONSULTING ENGINEERS

JOB No.
PROJECT
LOCATION
SAMPLE TYPE No. 5 Moose Creek, Yukon
DATE

GRAIN SIZE ANALYSIS
GRANULAR SOILS



SIEVE SIZES U.S.	SIEVE SIZES METRIC	% PASSING
6"	150 000	
3"	80 000	
2"	50 000	
1.5"	40 000	
1"	25 000	
.75"	20 000	
.5"	12 500	
.375"	10 000	
No. 4	5 000	
No. 8	2 500	
16	1 250	
30	630	
40	400	
50	315	
100	160	
200	80	

REMARKS:

KLOHN LEONOFF
CONSULTING ENGINEERS

JOB No.
PROJECT
LOCATION
SAMPLE TYPE No. 6 Willow River
DATE

APPENDIX IX

PLOT OF CONFORMANCE TO CONCRETE AGGREGATE GRADING LIMITS

SIEVE ANALYSIS

DATE SAMPLED: ______________________________ DATE TESTED: ______________________

COARSE AGGREGATE

SIEVE SIZE	PERCENT PASSING
100mm (4")	
75mm (3")	
50mm (2")	
38.1mm (1-1/2")	
25.0mm (1")	
19.0mm (3/4")	
12.5mm (1/2")	
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	

% FINER THAN 75um (#200) SIEVE =

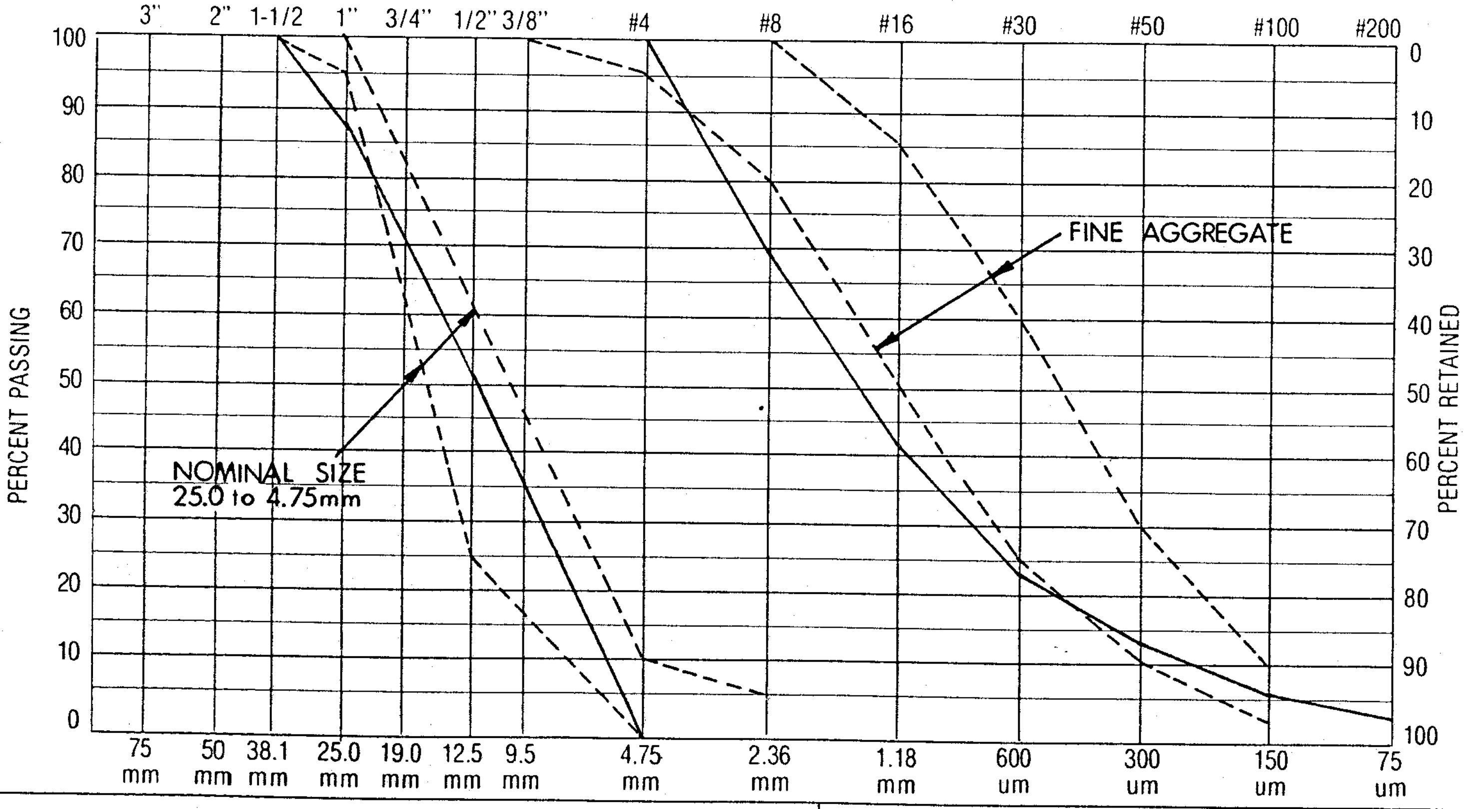
FINE AGGREGATE

SIEVE SIZE	PERCENT PASSING
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	
600um (#30)	
300um (#50)	
150um (#100)	
75um (#200)	

FINENESS MODULUS =
% FINER THAN 75um (#200) SIEVE =

ASTM C33 GRADING LIMITS

SIEVE SIZE



KLOHN LEONOFF CONSULTANTS LTD.
CIVIL • GEOTECHNICAL • HYDRAULIC

JOB NO.
PROJECT
LOCATION
TYPE OF SAMPLE No. 1 Shingle Point
DATE PLATE NO. A-

SIEVE ANALYSIS

DATE SAMPLED: ____________________

DATE TESTED: ____________________

COARSE AGGREGATE

SIEVE SIZE	PERCENT PASSING
100mm (4")	
75mm (3")	
50mm (2")	
38.1mm (1-1/2")	
25.0mm (1")	
19.0mm (3/4")	
12.5mm (1/2")	
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	

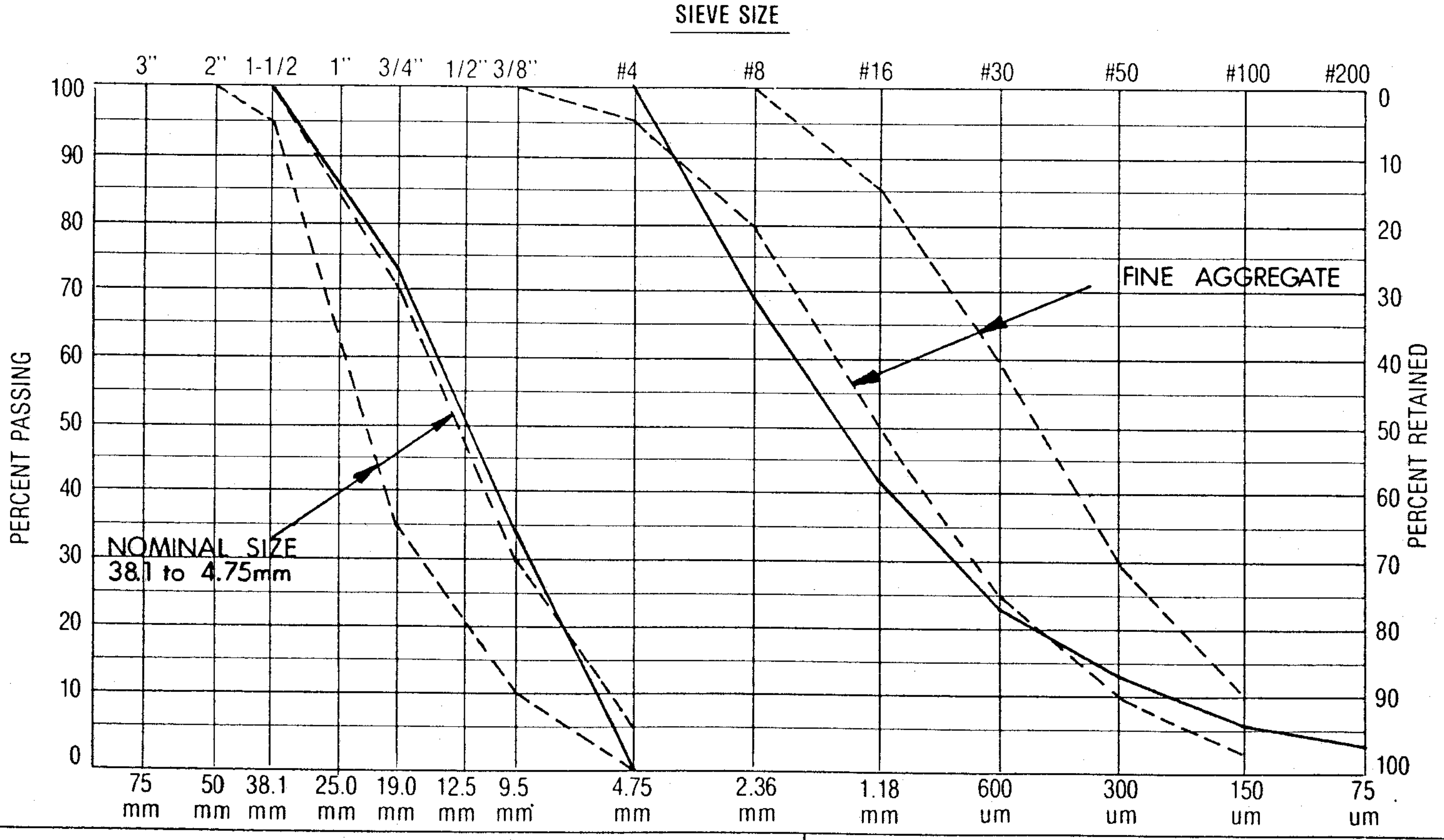
% FINER THAN 75um (#200) SIEVE =

FINE AGGREGATE

SIEVE SIZE	PERCENT PASSING
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	
600um (#30)	
300um (#50)	
150um (#100)	
75um (#200)	

FINENESS MODULUS =
% FINER THAN 75um (#200) SIEVE =

ASTM C33 GRADING LIMITS



KLOHN LEONOFF CONSULTANTS LTD.
CIVIL • GEOTECHNICAL • HYDRAULIC

JOB NO.
PROJECT
LOCATION
TYPE OF SAMPLE No. 1 Shingle Point
DATE
PLATE NO. A-

SIEVE ANALYSIS

DATE SAMPLED: ____________________ DATE TESTED: ____________________

COARSE AGGREGATE

SIEVE SIZE	PERCENT PASSING
100mm (4")	
75mm (3")	
50mm (2")	
38.1mm (1-1/2")	
25.0mm (1")	
19.0mm (3/4")	
12.5mm (1/2")	
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	

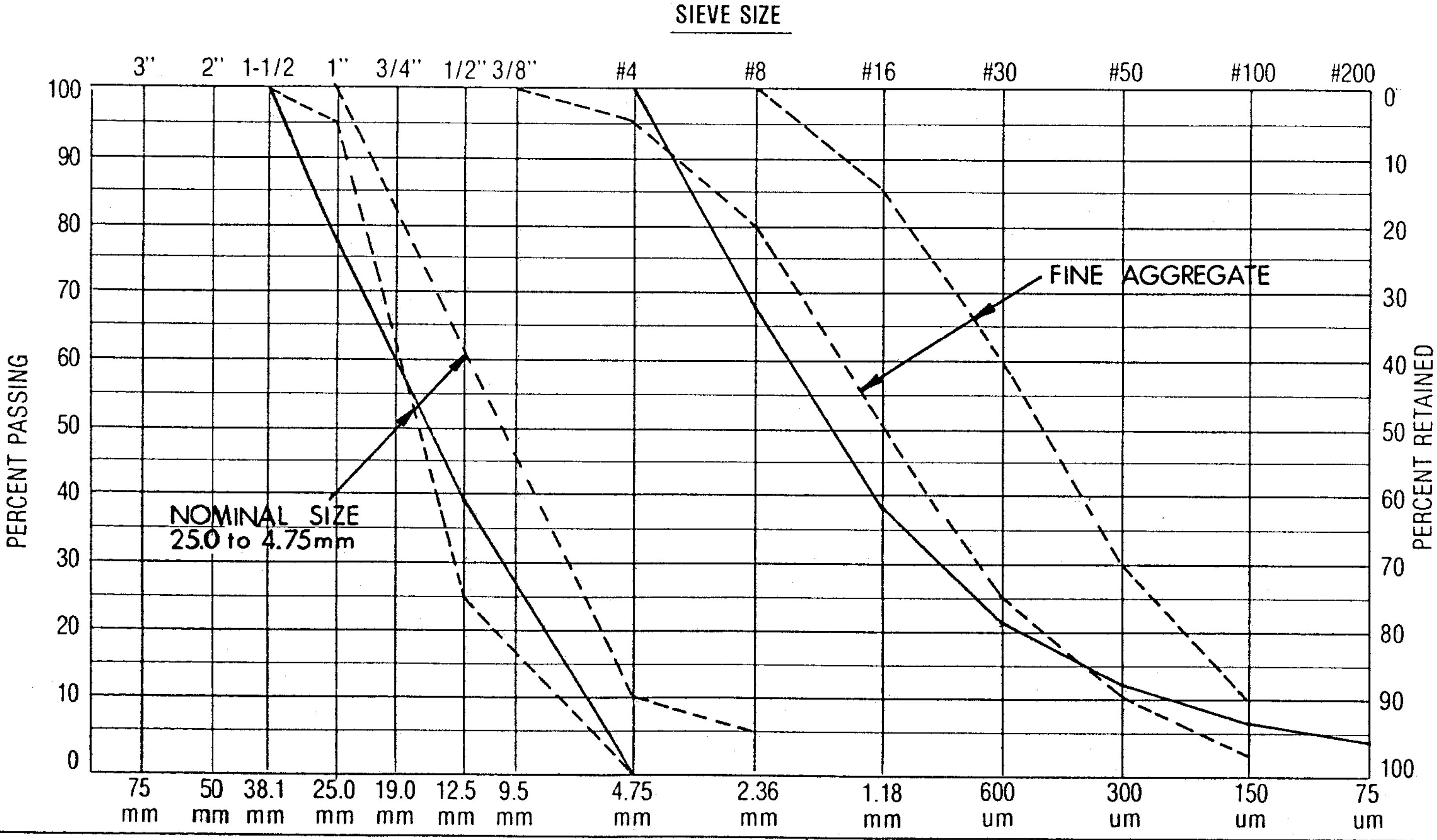
% FINER THAN 75um (#200) SIEVE =

FINE AGGREGATE

SIEVE SIZE	PERCENT PASSING
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	
600um (#30)	
300um (#50)	
150um (#100)	
75um (#200)	

FINENESS MODULUS =
% FINER THAN 75um (#200) SIEVE =

ASTM C33 GRADING LIMITS



KLOHN LEONOFF CONSULTANTS LTD.
CIVIL • GEOTECHNICAL • HYDRAULIC

JOB NO.
PROJECT
LOCATION
TYPE OF SAMPLE No. 2 Running River
DATE PLATE NO. A-

SIEVE ANALYSIS

DATE SAMPLED: ______________________ DATE TESTED: ________________

COARSE AGGREGATE

SIEVE SIZE	PERCENT PASSING
100mm (4")	
75mm (3")	
50mm (2")	
38.1mm (1-1/2")	
25.0mm (1")	
19.0mm (3/4")	
12.5mm (1/2")	
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	

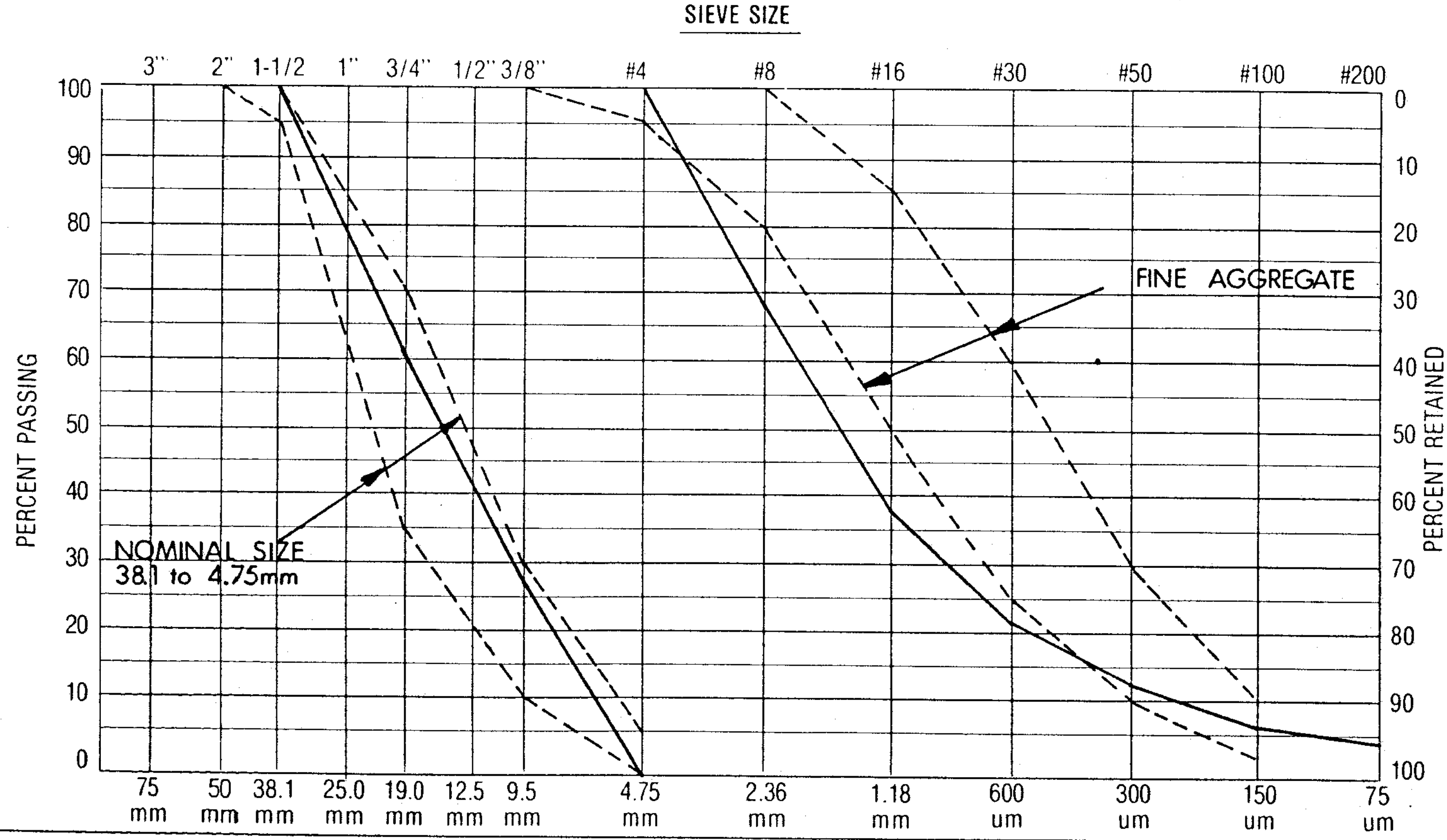
% FINER THAN 75um (#200) SIEVE =

FINE AGGREGATE

SIEVE SIZE	PERCENT PASSING
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	
600um (#30)	
300um (#50)	
150um (#100)	
75um (#200)	

FINENESS MODULUS =
% FINER THAN 75um (#200) SIEVE =

ASTM C33 GRADING LIMITS



KLOHN LEONOFF CONSULTANTS LTD.
CIVIL • GEOTECHNICAL • HYDRAULIC

JOB NO.
PROJECT
LOCATION
TYPE OF SAMPLE No. 2 Running River
DATE PLATE NO. A-

SIEVE ANALYSIS

DATE SAMPLED: ____________________

DATE TESTED: ________________

COARSE AGGREGATE

SIEVE SIZE	PERCENT PASSING
100mm (4")	
75mm (3")	
50mm (2")	
38.1mm (1-1/2")	
25.0mm (1")	
19.0mm (3/4")	
12.5mm (1/2")	
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	

% FINER THAN 75um (#200) SIEVE =

FINE AGGREGATE

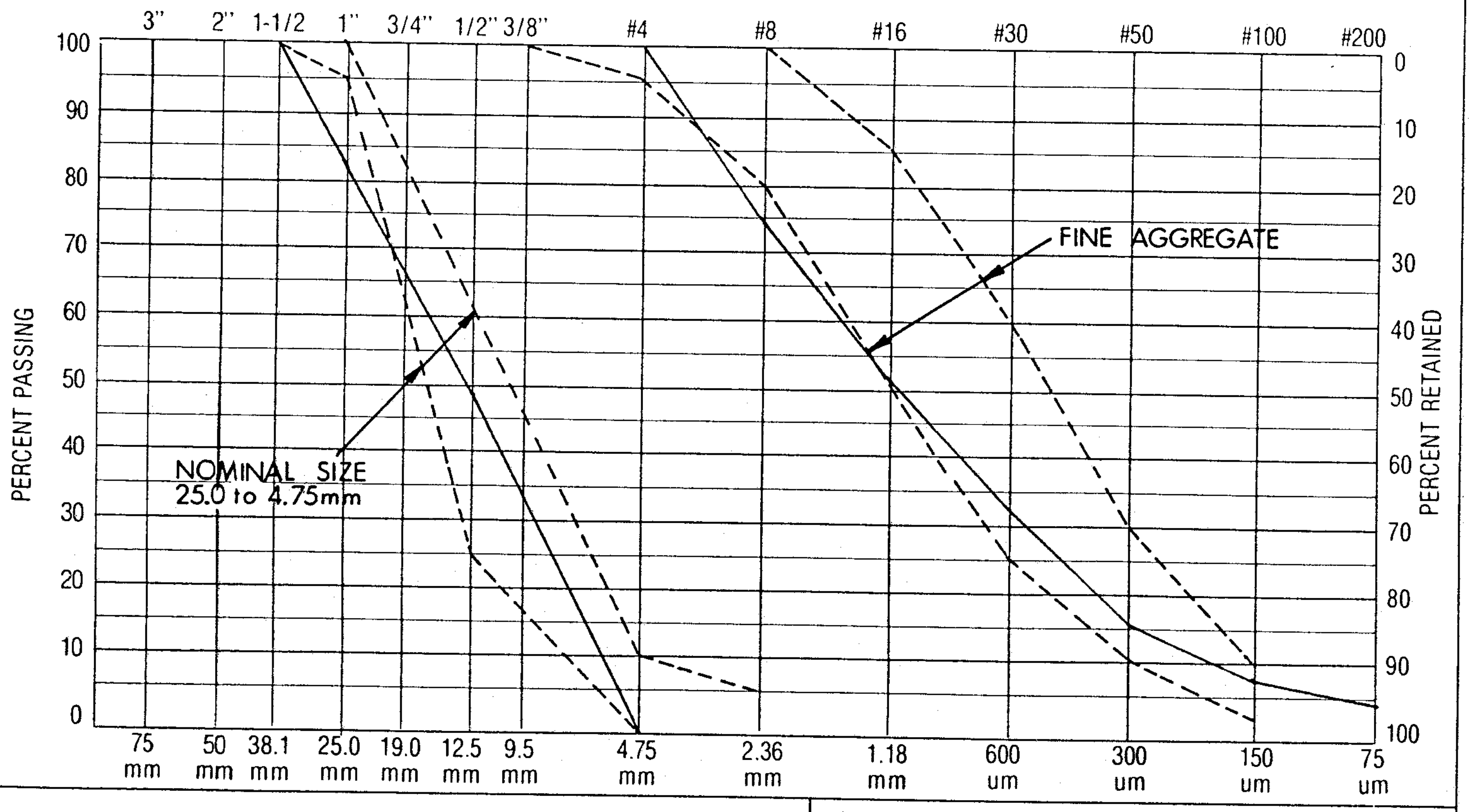
SIEVE SIZE	PERCENT PASSING
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	
600um (#30)	
300um (#50)	
150um (#100)	
75um (#200)	

FINENESS MODULUS =

% FINER THAN 75um (#200) SIEVE =

ASTM C33 GRADING LIMITS

SIEVE SIZE



KLOHN LEONOFF CONSULTANTS LTD.

CIVIL • GEOTECHNICAL • HYDRAULIC

JOB NO.

PROJECT

LOCATION

TYPE OF SAMPLE No. 3 King Point

DATE

PLATE NO. A-

SIEVE ANALYSIS

DATE SAMPLED: ______________________ DATE TESTED: ____________________

COARSE AGGREGATE

SIEVE SIZE	PERCENT PASSING
100mm (4")	
75mm (3")	
50mm (2")	
38.1mm (1-1/2")	
25.0mm (1")	
19.0mm (3/4")	
12.5mm (1/2")	
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	

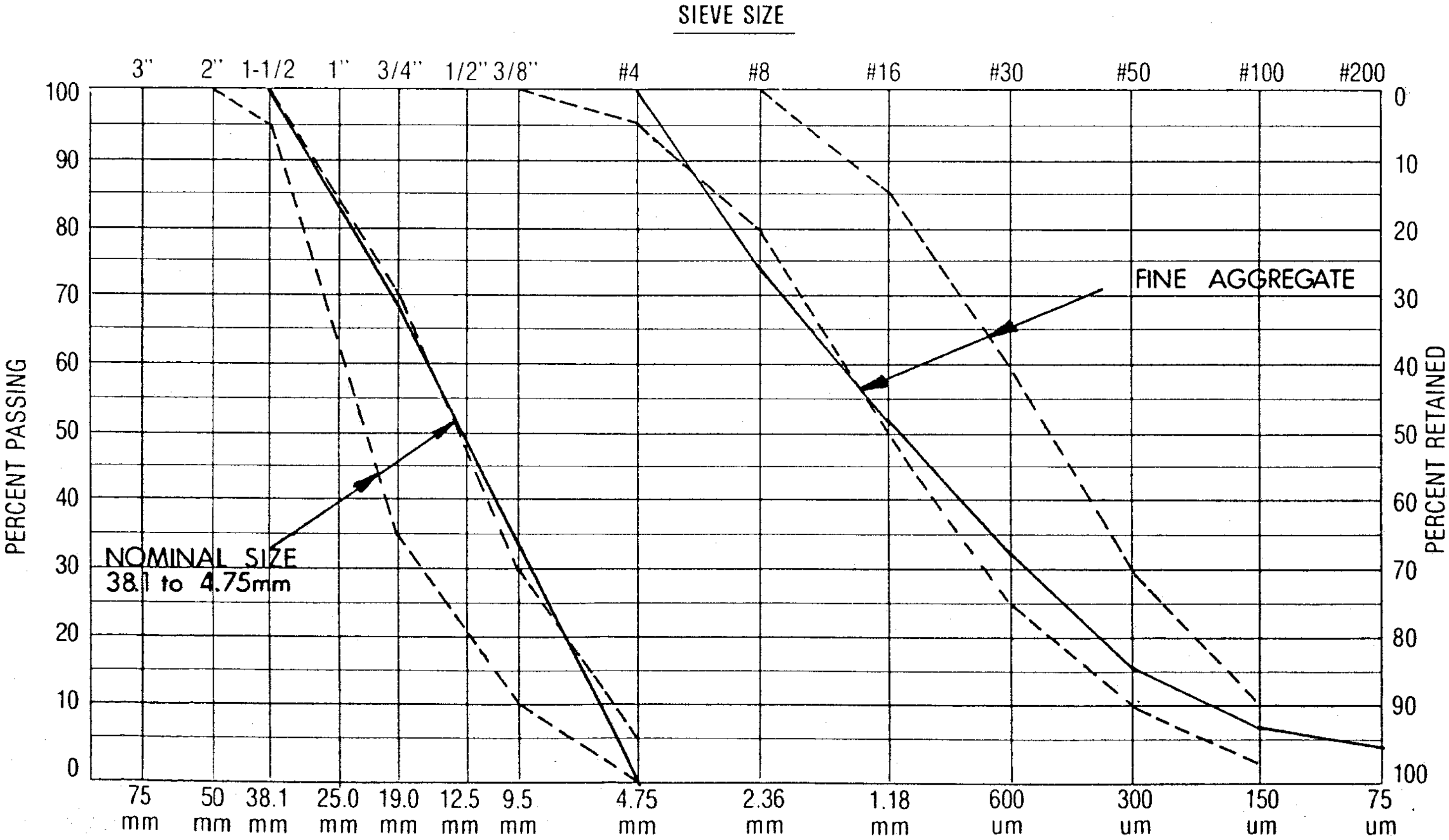
% FINER THAN 75um (#200) SIEVE =

FINE AGGREGATE

SIEVE SIZE	PERCENT PASSING
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	
600um (#30)	
300um (#50)	
150um (#100)	
75um (#200)	

FINENESS MODULUS =
% FINER THAN 75um (#200) SIEVE =

ASTM C33 GRADING LIMITS



KLOHN LEONOFF CONSULTANTS LTD.
CIVIL • GEOTECHNICAL • HYDRAULIC

JOB NO.
PROJECT
LOCATION
TYPE OF SAMPLE No. 3 King Point
DATE PLATE NO. A-

SIEVE ANALYSIS

DATE SAMPLED: ____________

DATE TESTED: ____________

COARSE AGGREGATE

SIEVE SIZE	PERCENT PASSING
100mm (4")	
75mm (3")	
50mm (2")	
38.1mm (1-1/2")	
25.0mm (1")	
19.0mm (3/4")	
12.5mm (1/2")	
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	

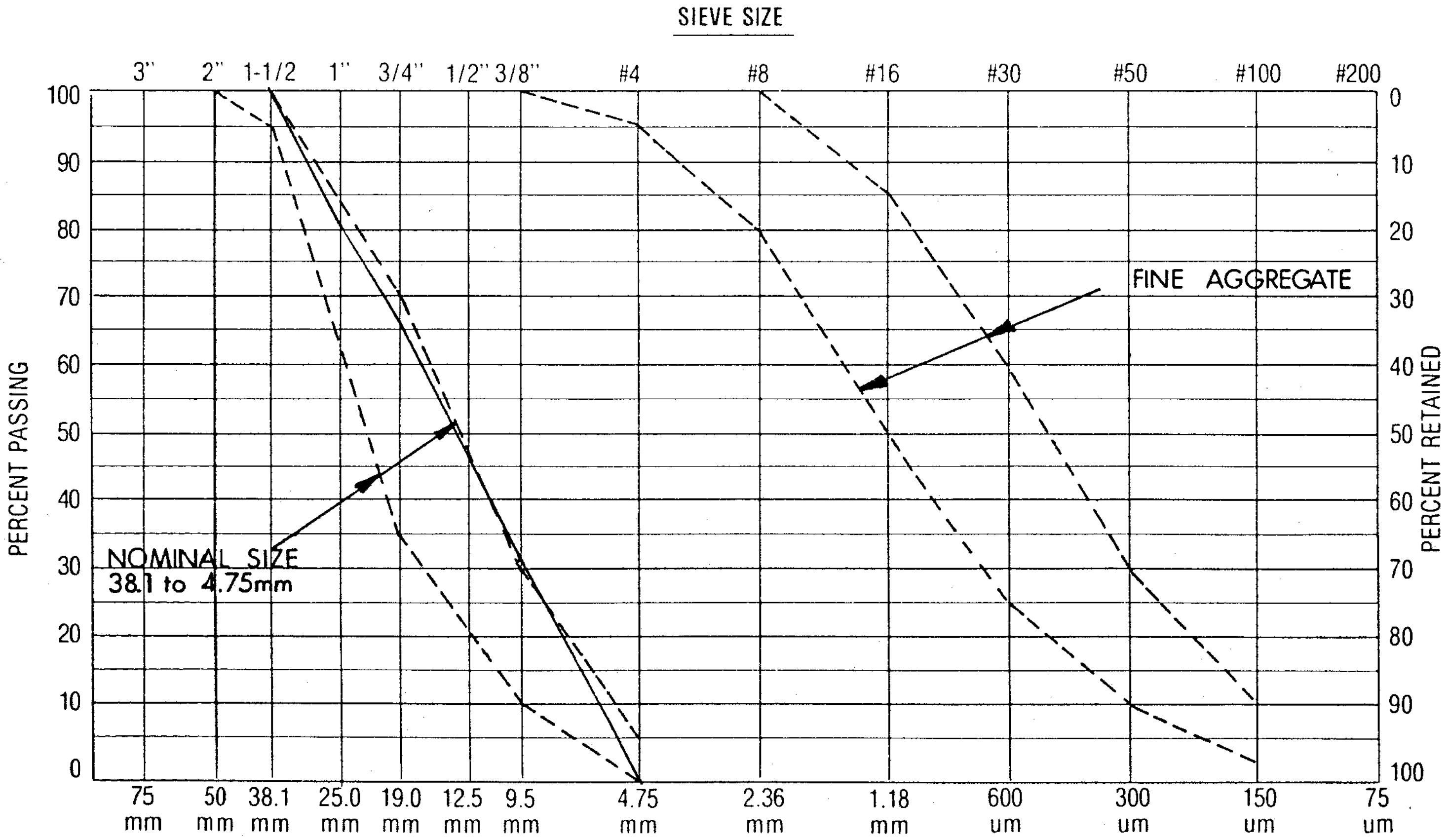
% FINER THAN 75um (#200) SIEVE =

FINE AGGREGATE

SIEVE SIZE	PERCENT PASSING
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	
600um (#30)	
300um (#50)	
150um (#100)	
75um (#200)	

FINENESS MODULUS =
% FINER THAN 75um (#200) SIEVE =

ASTM C33 GRADING LIMITS



KLOHN LEONOFF CONSULTANTS LTD.
CIVIL • GEOTECHNICAL • HYDRAULIC

JOB NO.
PROJECT
LOCATION
TYPE OF SAMPLE No. 4 Jacobs Ridge
DATE
PLATE NO. A-

SIEVE ANALYSIS

DATE SAMPLED: ____________________

DATE TESTED: ____________________

COARSE AGGREGATE

SIEVE SIZE	PERCENT PASSING
100mm (4")	
75mm (3")	
50mm (2")	
38.1mm (1-1/2")	
25.0mm (1")	
19.0mm (3/4")	
12.5mm (1/2")	
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	

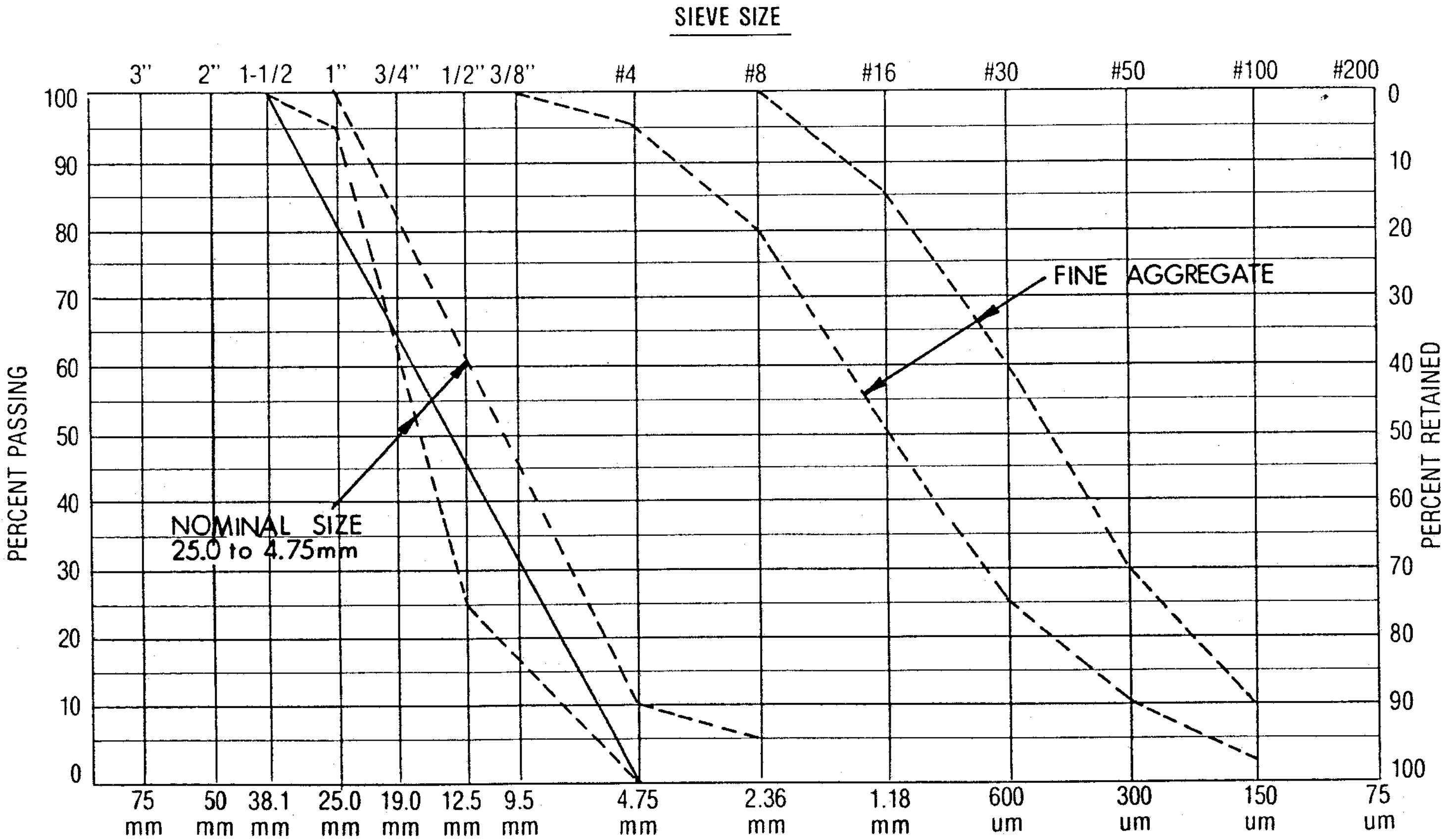
% FINER THAN 75um (#200) SIEVE =

FINE AGGREGATE

SIEVE SIZE	PERCENT PASSING
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	
600um (#30)	
300um (#50)	
150um (#100)	
75um (#200)	

FINENESS MODULUS =
% FINER THAN 75um (#200) SIEVE =

ASTM C33 GRADING LIMITS



KLOHN LEONOFF CONSULTANTS LTD.
CIVIL • GEOTECHNICAL • HYDRAULIC

JOB NO.
PROJECT
LOCATION
TYPE OF SAMPLE No. 4 Jacobs Ridge
DATE
PLATE NO. A-

SIEVE ANALYSIS

DATE SAMPLED: ______________________ DATE TESTED: ____________________

COARSE AGGREGATE

SIEVE SIZE	PERCENT PASSING
100mm (4")	
75mm (3")	
50mm (2")	
38.1mm (1-1/2")	
25.0mm (1")	
19.0mm (3/4")	
12.5mm (1/2")	
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	

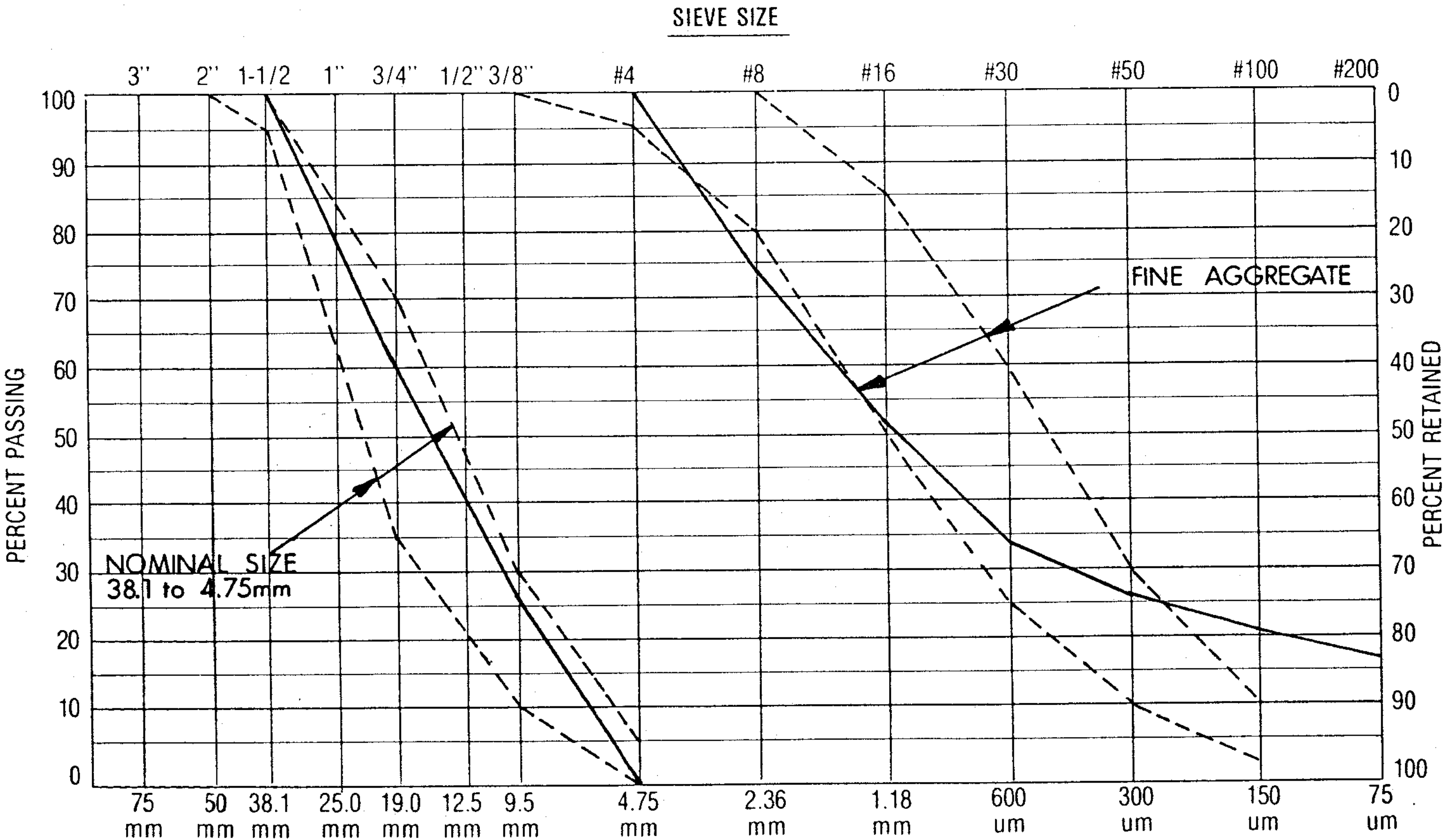
% FINER THAN 75um (#200) SIEVE =

FINE AGGREGATE

SIEVE SIZE	PERCENT PASSING
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	
600um (#30)	
300um (#50)	
150um (#100)	
75um (#200)	

FINENESS MODULUS =
% FINER THAN 75um (#200) SIEVE =

ASTM C33 GRADING LIMITS



KLOHN LEONOFF CONSULTANTS LTD.
CIVIL • GEOTECHNICAL • HYDRAULIC

JOB NO.
PROJECT
LOCATION
TYPE OF SAMPLE No. 5 Moose Creek
DATE PLATE NO. A-

SIEVE ANALYSIS

DATE SAMPLED: ____________________ DATE TESTED: ____________________

COARSE AGGREGATE

SIEVE SIZE	PERCENT PASSING
100mm (4")	
75mm (3")	
50mm (2")	
38.1mm (1-1/2")	
25.0mm (1")	
19.0mm (3/4")	
12.5mm (1/2")	
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	

% FINER THAN 75um (#200) SIEVE =

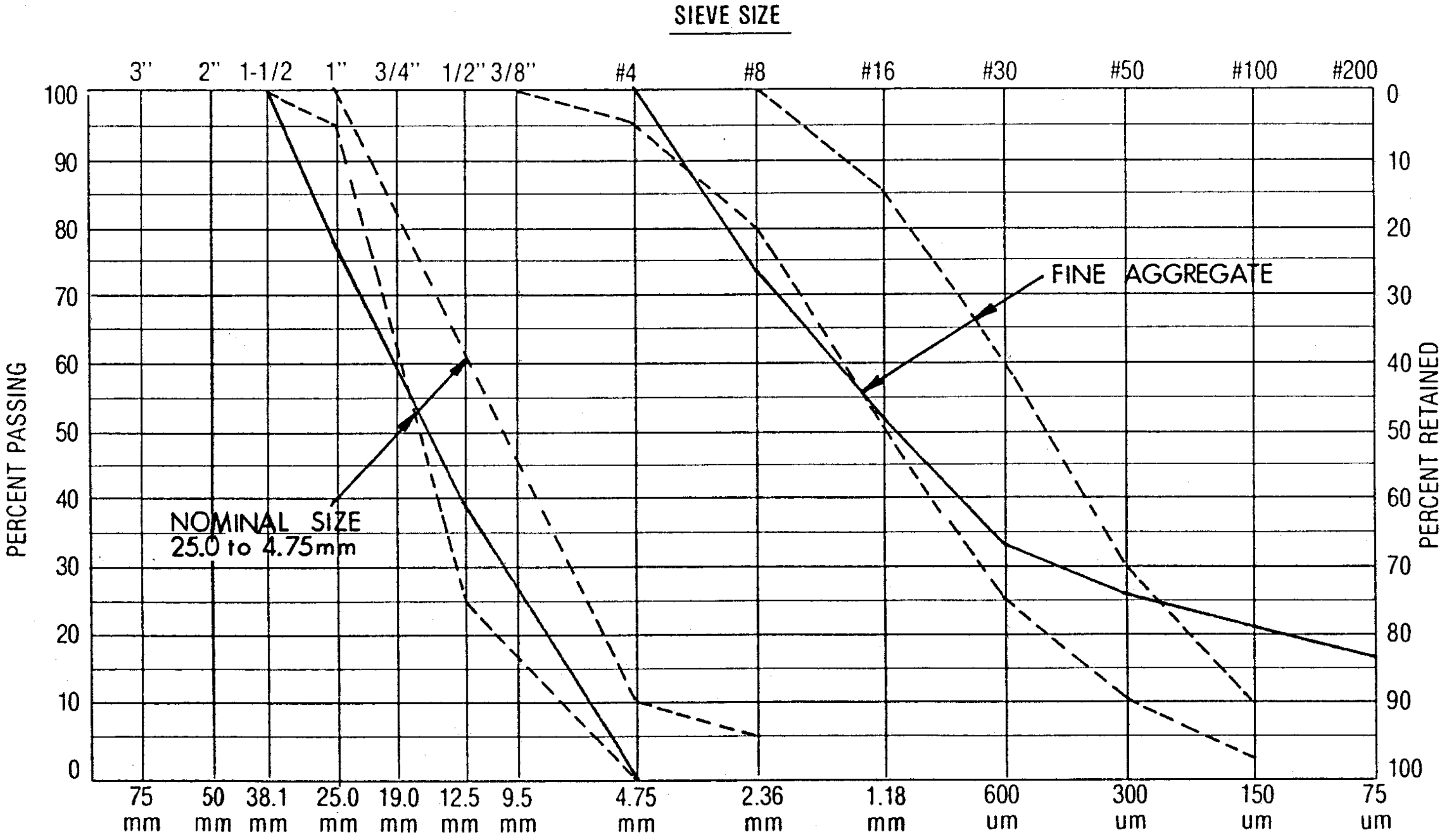
FINE AGGREGATE

SIEVE SIZE	PERCENT PASSING
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	
600um (#30)	
300um (#50)	
150um (#100)	
75um (#200)	

FINENESS MODULUS =

% FINER THAN 75um (#200) SIEVE =

ASTM C33 GRADING LIMITS



KLOHN LEONOFF CONSULTANTS LTD.
CIVIL • GEOTECHNICAL • HYDRAULIC

JOB NO.
PROJECT
LOCATION
TYPE OF SAMPLE No. 5 Moose Creek
DATE PLATE NO. A-

SIEVE ANALYSIS

DATE SAMPLED: ______________________ DATE TESTED: ________________

COARSE AGGREGATE

SIEVE SIZE	PERCENT PASSING
100mm (4")	
75mm (3")	
50mm (2")	
38.1mm (1-1/2")	
25.0mm (1")	
19.0mm (3/4")	
12.5mm (1/2")	
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	

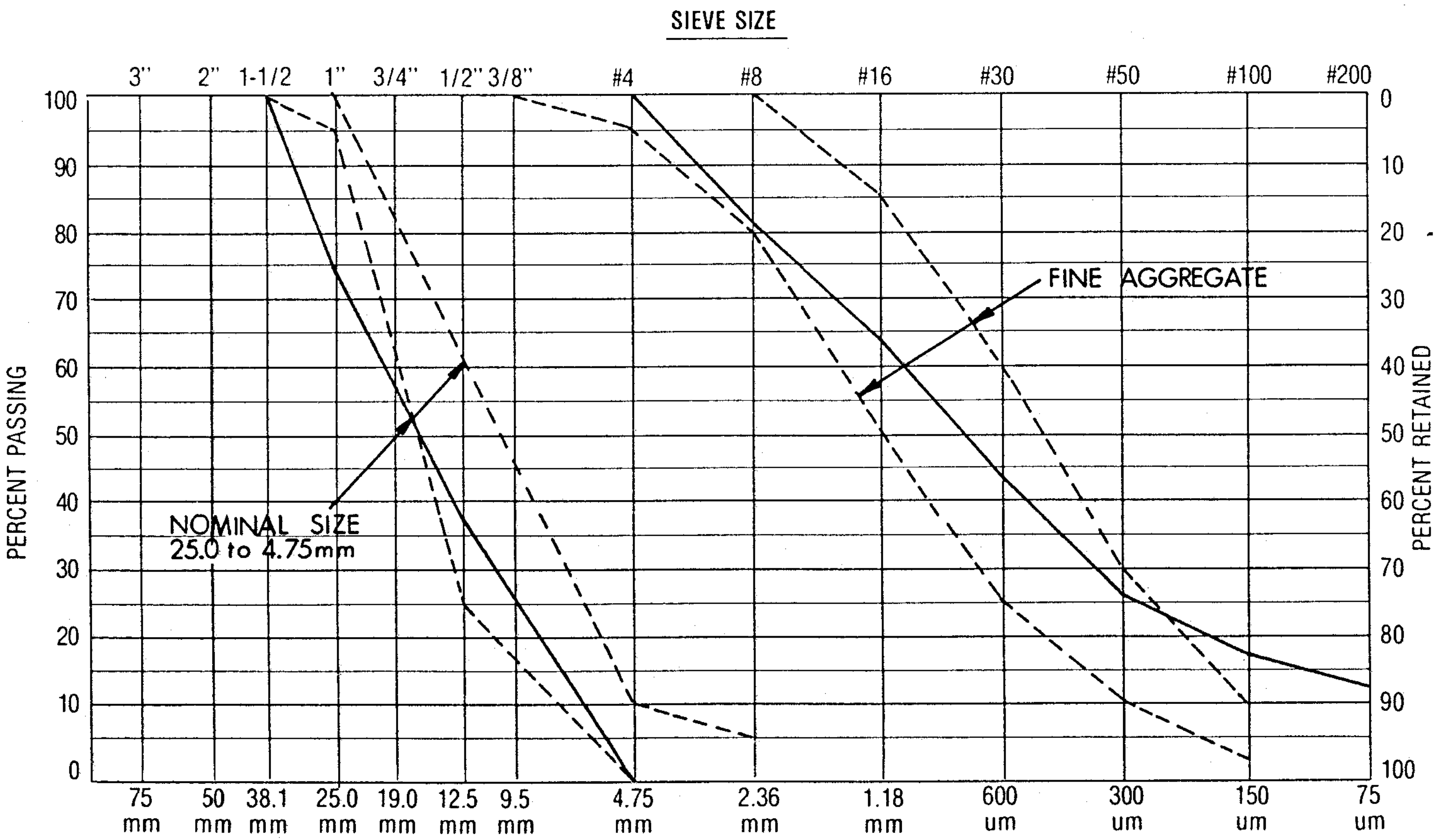
% FINER THAN 75um (#200) SIEVE =

FINE AGGREGATE

SIEVE SIZE	PERCENT PASSING
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	
600um (#30)	
300um (#50)	
150um (#100)	
75um (#200)	

FINENESS MODULUS =
% FINER THAN 75um (#200) SIEVE =

ASTM C33 GRADING LIMITS



KLOHN LEONOFF CONSULTANTS LTD.
CIVIL • GEOTECHNICAL • HYDRAULIC

JOB NO.
PROJECT
LOCATION
TYPE OF SAMPLE No. 6 Willow River
DATE PLATE NO. A-

SIEVE ANALYSIS

DATE SAMPLED: ______________________ DATE TESTED: ______________________

COARSE AGGREGATE

SIEVE SIZE	PERCENT PASSING
100mm (4")	
75mm (3")	
50mm (2")	
38.1mm (1-1/2")	
25.0mm (1")	
19.0mm (3/4")	
12.5mm (1/2")	
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	

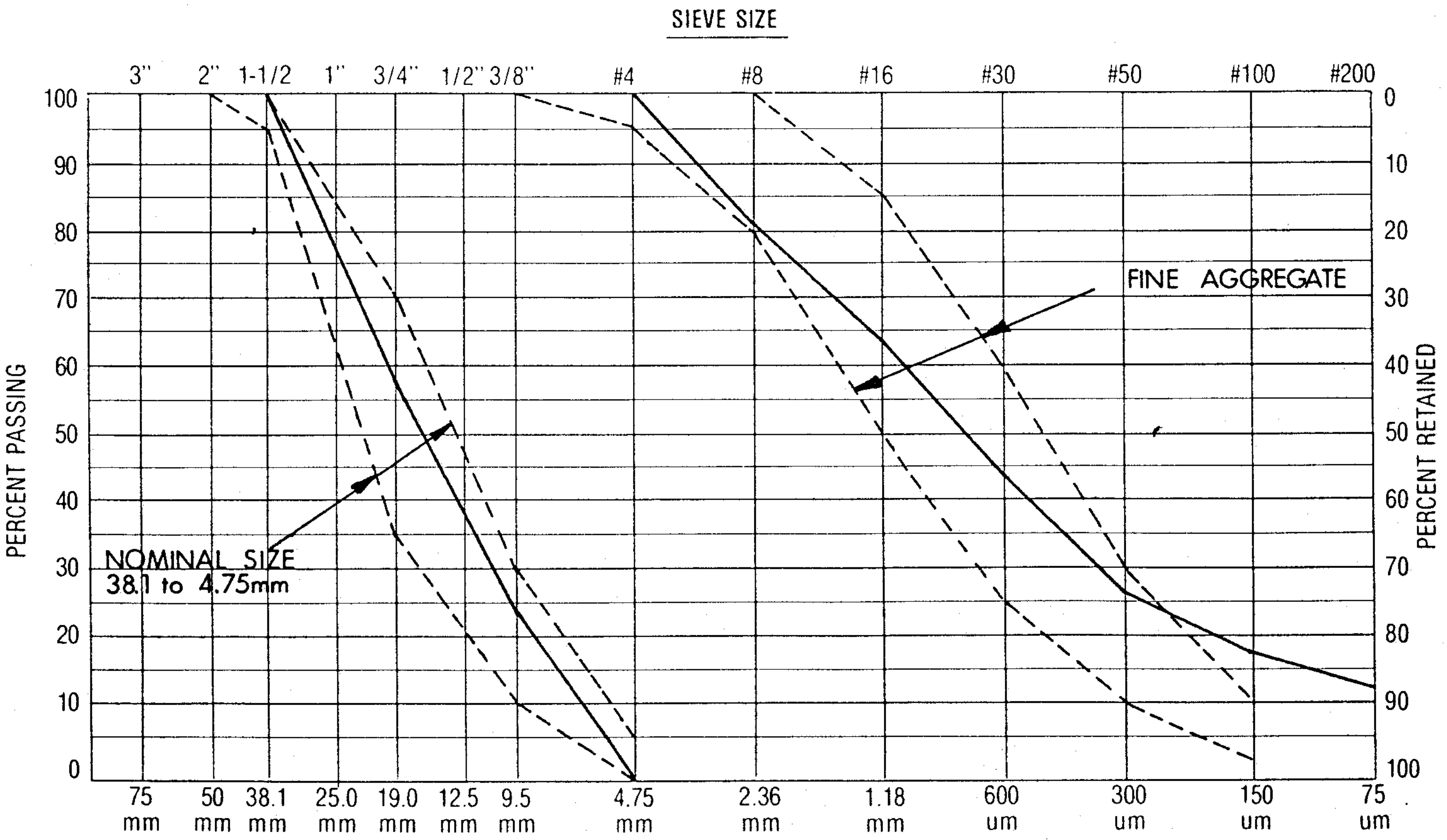
% FINER THAN 75um (#200) SIEVE =

FINE AGGREGATE

SIEVE SIZE	PERCENT PASSING
9.5mm (3/8")	
4.75mm (#4)	
2.36mm (#8)	
1.18mm (#16)	
600um (#30)	
300um (#50)	
150um (#100)	
75um (#200)	

FINENESS MODULUS =
% FINER THAN 75um (#200) SIEVE =

ASTM C33 GRADING LIMITS



KLOHN LEONOFF CONSULTANTS LTD.
CIVIL • GEOTECHNICAL • HYDRAULIC

JOB NO.
PROJECT
LOCATION
TYPE OF SAMPLE No. 6 Willow River
DATE
PLATE NO. A-

APPENDIX X

SUMMARY OF PETROGRAPHIC RESULTS

Conducted according to
CSA A23.2 - "Methods of Test for Concrete
Appendix B, Rock Type and Factors"

PETROGRAPHIC ANALYSIS OF AGGREGATES

PROJECT NO.: PA 2291.01.03

SOURCE: No.1, SHINGLE POINT

CONSTITUENTS	REMARKS	COMPOSITION OF FRACTIONS RETAINED ON SIEVES (%)					PHYSICAL QUALITY	WEIGHTED COMPOSITION OF SAMPLE % BY WEIGHT	PETRO. FACTOR	PN
		1"	3/4"	½"	3/8"	#4				
QUARTZITE	*			17.6	37.2	55.7	Good	39.9	1	39.9
	**			5.3	8.5	26.1				
CHERT	*			36.1	39.6	33.0	Fair	35.4	3	106.2
	**			10.9	9.1	15.4				
SANDSTONE	*			42.0	21.9	9.8	Good	22.3	1	22.3
	**			12.7	5.0	4.6				
ARKOSE	*			3.0	1.1	0.8	Good	1.6	1	1.6
	**			0.9	0.3	0.4				
SILTSTONE	*			1.3	0.2	0.7	Fair	0.8	3	2.4
	**			0.4	0.1	0.3				

NOTE: Approximately 20-25% particals can be classified as flat or flat and elongated.

* on each sieve size   ** weighted composition of sample

BASIC PETROGRAPHIC NUMBER = 172.4
CORRECTION FOR CONCRETE USE = 1.6
PN = 170.8
≃ 171

PETROGRAPHIC ANALYSIS OF AGGREGATES

PROJECT NO.: PA 2291.01.03

SOURCE: No. 2, RUNNING RIVER

CONSTITUENTS	REMARKS	COMPOSITION OF FRACTIONS RETAINED ON SIEVES (%) 1"	3/4"	½"	3/8"	#4	PHYSICAL QUALITY	WEIGHTED COMPOSITION OF SAMPLE % BY WEIGHT	PETRO. FACTOR	PN
QUARTZITE	*			45.0	33.1	32.5	Good	36.8	1	36.8
	**			15.2	7.0	14.6				
CHERT	*			28.8	28.7	36.9	Fair	32.4	3	97.2
	**			9.8	6.0	16.6				
SANDSTONE	*			23.1	30.3	20.9	Good	23.6	1	23.6
	**			7.8	6.4	9.4				
ARKOSE	*			2.2	3.1	1.2	Good	2.1	1	2.1
	**			0.7	0.8	0.6				
SILTSTONE	*			0.9	4.8	8.5	Fair	5.1	3	15.3
	**			0.3	1.0	3.8				
NOTE: Approximately 15-20% of particals can be classified as flat or flat and elongated.										

* on each sieve size   ** weighted composition of sample

BASIC PETROGRAPHIC NUMBER = 175.0
CORRECTION FOR CONCRETE USE = 10.2
PN = 164.8
≃ 165

PETROGRAPHIC ANALYSIS OF AGGREGATES

PROJECT NO.: PA 2291.01.03

SOURCE: No. 3, King Point

CONSTITUENTS	REMARKS	COMPOSITION OF FRACTIONS RETAINED ON SIEVES (%)					PHYSICAL QUALITY	WEIGHTED COMPOSITION OF SAMPLE % BY WEIGHT	PETRO. FACTOR	PN
		1"	3/4"	½"	3/8"	#4				
QUARTZITE	*			52.9	47.2	39.8	Good	45.2	1	45.2
	**			15.0	10.2	20.0				
CHERT	*			31.1	33.5	45.2	Fair	38.7	3	116.1
	**			8.8	7.3	22.6				
SANDSTONE	*			13.1	12.6	9.7	Fair	11.2	3	33.6
	**			3.7	2.7	4.8				
ARKOSE	*			0.0	0.9	1.0	Good	0.8	1	0.8
	**			0.0	0.3	0.5				
SILTSTONE	*			0.7	2.9	2.5	Poor	2.0	6	12.0
	**			0.2	0.6	1.2				
CLAY IRONSTONE	*			2.2	2.9	1.8	Deleterions	2.1	10	21.0
	**			0.6	0.6	0.9				

* on each sieve size   ** weighted composition of sample

NOTE: Approximately 2-3% of particals can be classified as flat or flat and elongated.

BASIC PETROGRAPHIC NUMBER = 228.7
CORRECTION FOR CONCRETE USE = 26.4
PN = 202.3
≃ 202

PETROGRAPHIC ANALYSIS OF AGGREGATES

PROJECT NO.: PA 2291.01.03

SOURCE: No. 4, JACOBS RIDGE

CONSTITUENTS	REMARKS	COMPOSITION OF FRACTIONS RETAINED ON SIEVES (%)					PHYSICAL QUALITY	WEIGHTED COMPOSITION OF SAMPLE % BY WEIGHT	PETRO. FACTOR	PN
		1"	3/4"	½"	3/8"	#4				
QUARTZITE	*			36.9	43.1	27.8	Good	34.0	1	34.0
	**			11.3	9.7	13.0				
CHERT	*			29.7	36.2	49.9	Fair	40.6	3	121.8
	**			9.1	8.1	23.4				
SANDSTONE	*			28.9	16.6	12.3	Fair	18.3	3	54.9
	**			8.8	3.7	5.8				
ARKOSE	*			3.4	1.2	2.3	Good	2.4	1	2.4
	**			1.0	0.3	1.1				
SILTSTONE	*			1.1	2.9	7.7	Fair	4.7	3	14.1
	**			0.5	0.6	3.6				
NOTE: Approximately 5% of particals can be classified as flat or flat and elongated										

* on each sieve size   ** weighted composition of sample

BASIC PETROGRAPHIC NUMBER = 227.2
CORRECTION FOR CONCRETE USE = 46.0
PN = 181.2
≃ 181

PETROGRAPHIC ANALYSIS OF AGGREGATES

PROJECT NO.: PA 2291.01.03

SOURCE: No. 5, MOOSE CREEK

CONSTITUENTS	REMARKS	COMPOSITION OF FRACTIONS RETAINED ON SIEVES (%)					PHYSICAL QUALITY	WEIGHTED COMPOSITION OF SAMPLE % BY WEIGHT	PETRO. FACTOR	PN
		1"	3/4"	½"	3/8"	#4				
QUARTZITE	*			96.9	89.8	87.4	Good	91.1	1	91.1
	**			33.8	19.3	38.0				
CHERT	*			1.7	3.8	5.5	Fair	3.8	3	11.4
	**			0.6	0.8	2.4				
SANDSTONE	*			0.2	0.6	0.7	Fair	0.6	3	1.8
	**			0.1	0.1	0.3				
SILTSTONE	*			1.5	5.8	6.4	Fair	4.5	3	13.5
	**			0.5	1.2	2.8				
	*									
	**									
NOTE: Approximately 2-3% of particals can be classified as flat or flat and elongated										

* on each sieve size   ** weighted composition of sample

BASIC PETROGRAPHIC NUMBER = 117.8
CORRECTION FOR CONCRETE USE = 9.0
PN = 108.8
≃ 109

PETROGRAPHIC ANALYSIS OF AGGREGATES

PROJECT NO.: PA 2291.01.03

SOURCE: No. 6, WILLOW RIVER

CONSTITUENTS	REMARKS	COMPOSITION OF FRACTIONS RETAINED ON SIEVES (%)					PHYSICAL QUALITY	WEIGHTED COMPOSITION OF SAMPLE % BY WEIGHT	PETRO. FACTOR	PN
		1"	3/4"	½"	3/8"	#4				
QUARTZITE	*			79.3	66.8	47.6	Good	63.4	1	63.4
	**			28.0	15.4	19.8				
CHERT	*			6.0	14.5	25.1	Fair	15.8	3	47.4
	**			2.1	3.3	10.4				
SANDSTONE	*			8.6	4.2	7.4	Fair	7.3	3	21.9
	**			3.0	1.0	3.1				
SILTSTONE	*			1.0	1.1	1.5	Poor	1.1	6	6.6
	**			0.3	0.2	0.6				
CLAY IRONSTONE	*			5.1	13.4	18.4	Deleterions	12.5	10	125.0
	**			1.8	3.1	7.6				
NOTE: Approximately 1-2% of particals can be classified as flat or flan and elongated										

* on each sieve size ** weighted composition of sample

BASIC PETROGRAPHIC NUMBER = 264.3

CORRECTION FOR CONCRETE USE = 14.6

PN = 249.7

≃ 250

APPENDIX XI

FUTURE CSA A23.1 COMMENTARY

ALKALI-AGGREGATE REACTION

Appendix B
Alkali-Aggregate Reaction

Note: *This Appendix is not a mandatory part of this Standard.*

B1. General

B1.1
Research and testing in the past 30 years has shown that, in several regions of Canada, concrete deterioration occurs due to a reaction between some minerals in certain rock types and the soluble alkaline components in concrete, which are usually derived from the cement. All natural rocks react to some extent with the alkaline pore solution in concrete, but in certain cases the reactions produce deleterious expansion and cracking.

B1.2
Deleterious expansion and cracking of concrete due to alkali-aggregate reaction may, in some circumstances, be minimized or prevented by use of corrective measures. These include selective extraction of the aggregate to reduce or eliminate the reactive material, reduction in the cement content of the concrete and/or the use of a cement with a low alkali content and the use of supplementary cementing materials. Silica fume, pulverized fly ash, pulverized blast furnace slag and natural pozzolans may be effective in preventing or reducing expansion due to alkali-silica or slow/late expanding alkali-silicate/silica reactions, when used in appropriate amounts. Such supplementary cementing materials are not effective with alkali-carbonate reactive aggregates.

B1.3
Corrective measures, other than selective aggregate extraction, may not prove effective with some types of aggregate, and for this reason should only be accepted when either laboratory testing or field experience under exposure conditions, similar to those of the proposed structure, demonstrate the effectiveness of the selected method in preventing or minimizing deterioration of concrete due to alkali-aggregate reaction.

B1.4
In some cases where the concrete is exposed to a continuously moist environment or external sources of alkali, eg, NaCl, and when a small expansion of the concrete is unacceptable, limiting the alkali content of the cement or the replacement of part of the cement by a supplementary cementing material or reducing the amount of cement used, may not provide adequate protection against long-term expansion.

B1.5
A summary of the general procedures to be followed in assessing the quality of concrete aggregate is shown in the flow diagrams Figures B1, B2, and B3.

B2. Types of Alkali-Aggregate Reaction
Three types of alkali-aggregate reaction are encountered in Canada:

(a) alkali-silica reaction;
(b) slow/late-expanding alkali silicate/silica reaction;

(c) alkali-carbonate reaction.

Note: *The mechanisms of these expansive reactions is not clearly understood. The alkali-silica reaction is associated with the formation of expansive alkali-silica gel in concrete. The slow/late expanding alkali silicate/silica reaction is associated with the expansion of coarse aggregate particles in addition to gel formation. Alkali-carbonate reaction is caused by expansion of coarse aggregate particles.*

B2.1 Alkali-silica Reaction

Aggregates exhibiting this type of reactivity contain various forms of reactive silica: opal, chert, flint, chalcedony, tridymite, cristobalite, volcanic glasses and manfactured glass. Aggregate containing such materials, eg, some cherty gravels may cause deterioration of concrete when present in amounts of 1 to 5 per cent. Time expansion graphs of concrete prisms or mortar bars made with these aggregates are characterized by the early onset of expansion and also usually by a high rate of expansion (Figure B4(a)). Cracking of concrete structures is usually observed within ten years of construction.

B2.2 Slow/Late Expanding Alkali Silicate/Silica Reaction

This type of reactivity is distinguished from alkali-silica reactivity by the delayed onset of expansion of concrete test prisms (Figure B4 (b)) and the length of time before cracking becomes evident in concrete structures, which may be up to 20 years. Strained quartz is thought to be one reactive component in many of these rocks. A wide variety of quartz-bearing rocks such as: greywackes, argillites, quartzwackes, quartz-arenites, quartzites, hornfels, quartz biotite gneiss, granite, phyllite, arkose and sandstone have been found to be reactive.

B2.3 Alkali-carbonate Reaction

Alkali-carbonate reaction occurs between certain argillaceous dolomitic limestones and the alkaline pore solution in the concrete. It causes expansion and extensive cracking of concrete. The reaction under laboratory conditions is usually characterized by the rapid onset of expansion of concrete test prisms (Figure B4(c)). Expansive dolomitic limestones are characterized by a matrix of fine calcite and clay minerals with scattered dolomite rhombohedra. The characteristic texture may be observed in thin sections with a petrographic microscope or in the scanning electron microscope (Reference B6.1). Structures undergoing this reaction usually show cracking within five years of construction.

B3. Methods of Evaluating Potential Reactivity of Aggregates

B3.1 General

A field investigation of concrete structures containing the aggregate and having about the same alkali level and environment as the proposed concrete structure is possibly the best method of evaluating acceptability of concrete aggregates. In many instances, such a field investigation is not possible, because the aggregate has not been used previously in concrete, for instance, when it comes from a new lift or horizon in a quarry. In other cases, variations in cement content of the concrete or curing history may result in field performance investigation giving misleading results. Under these circumstances, some type of laboratory investigation must be undertaken. Laboratory test methods fall into two categories. Those in which the composition of an aggregate under investigation is determined by petrographic or chemical analysis and compared to the composition of known reactive and non-reactive aggregates. Secondly, those in which the expansivity of an aggregate is determined, either in rock prisms in an alkaline solution, in mortar bars or in concrete prisms. The first type of test is rapid and convenient but of limited use due to the inherent uncertainty in the correlation between the determined composition and the reactivity of the aggregate. Even with the second method, caution must be exercised due to variability in the tests and difficulties in interpreting the results as there are few studies in which the correlation between laboratory expansion and field deterioration of concrete has been reliably established. Engineering judgement is needed when interpreting test data and performance of concrete structures, when aggregates are found to be marginally or deleteriously reactive.

B3.2
CSA A23.2-15; Petrographic Examination of Aggregates for Concrete
Petrographic examination is the first step when evaluating the potential reactivity of an aggregate. It is carried out to determine the types of rock comprising the aggregate so that appropriate laboratory tests can be made (see Figure B1). In certain cases, where specific rocks or minerals are known from experience to cause deterioration of concrete, identification of these constituents in an aggregate by petrographic examination may be sufficient evidence to reject the aggregate. Care is needed in making petrographic examinations of aggregates containing reactive silica, eg, Ordovician limestones, in which as little as one per cent of chert, an amount easily overlooked, can cause deleterious expansion of concrete. Petrographic examination may also be used to indicate potential reactivity of late-expansive quartz-bearing rocks by determining the presence or absence of undulatory extinction of quartz grains. The acceptability of potentially reactive alkali-carbonate aggregates may also be determined by petrographic examination. However, it must be stressed that a careful, detailed examination is necessary to differentiate between some expansive and non-expansive aggregates which appear similar upon superficial examination.

B3.3 ASTM Standard Test Method C289; for Potential Reactivity of Aggregates (Chemical Method)
Test results of aggregates represented by points lying to the right of the solid line in Figure 2 of ASTM Standard C289 should usually be considered potentially alkali-silica reactive. Experience outside Canada has, however, shown the need to develop separate limits for the permissible amount of dissolved silica (S_c) for certain rock types on a regional basis (Reference B6.2). When test results indicate an aggregate to be innocuous, engineering judgement should be exercised before it is accepted as test results may be affected by a number of factors:
(a) variability in the aggregate source;
(b) problems of obtaining a representative sample;
(c) interlaboratory variability in the test results.

It is advisable to run occasional duplicate samples as a check on the analytical procedure. If the results obtained using this

Appendix B
Alkali-Aggregate Reaction

Note: *This Appendix is not a mandatory part of this Standard.*

B1. General

B1.1

Research and testing in the past 30 years has shown that, in several regions of Canada, concrete deterioration occurs due to a reaction between some minerals in certain rock types and the soluble alkaline components in concrete, which are usually derived from the cement. All natural rocks react to some extent with the alkaline pore solution in concrete, but in certain cases the reactions produce deleterious expansion and cracking.

B1.2

Deleterious expansion and cracking of concrete due to alkali-aggregate reaction may, in some circumstances, be minimized or prevented by use of corrective measures. These include selective extraction of the aggregate to reduce or eliminate the reactive material, reduction in the cement content of the concrete and/or the use of a cement with a low alkali content and the use of supplementary cementing materials. Silica fume, pulverized fly ash, pulverized blast furnace slag and natural pozzolans may be effective in preventing or reducing expansion due to alkali-silica or slow/late expanding alkali-silicate/silica reactions, when used in appropriate amounts. Such supplementary cementing materials are not effective with alkali-carbonate reactive aggregates.

B1.3

Corrective measures, other than selective aggregate extraction, may not prove effective with some types of aggregate, and for this reason should only be accepted when either laboratory testing or field experience under exposure conditions, similar to those of the proposed structure, demonstrate the effectiveness of the selected method in preventing or minimizing deterioration of concrete due to alkali-aggregate reaction.

B1.4

In some cases where the concrete is exposed to a continuously moist environment or external sources of alkali, eg, NaCl, and when a small expansion of the concrete is unacceptable, limiting the alkali content of the cement or the replacement of part of the cement by a supplementary cementing material or reducing the amount of cement used, may not provide adequate protection against long-term expansion.

BI.5

A summary of the general procedures to be followed in assessing the quality of concrete aggregate is shown in the flow diagrams Figures B1, B2, and B3.

B2. Types of Alkali-Aggregate Reaction

Three types of alkali-aggregate reaction are encountered in Canada:

(a) alkali-silica reaction;
(b) slow/late-expanding alkali silicate/silica reactic

(c) alkali-carbonate reaction.
Note: *The mechanisms of these expansive reactions is not clearly understood. The alkali-silica reaction is associated with the formation of expansive alkali-silica gel in concrete. The slow/late expanding alkali silicate/silica reaction is associated with the expansion of coarse aggregate particles in addition to gel formation. Alkali-carbonate reaction is caused by expansion of coarse aggregate particles.*

B2.1 Alkali-silica Reaction

Aggregates exhibiting this type of reactivity contain various forms of reactive silica: opal, chert, flint, chalcedony, tridymite, cristobalite, volcanic glasses and manfactured glass. Aggregate containing such materials, eg, some cherty gravels may cause deterioration of concrete when present in amounts of 1 to 5 per cent. Time expansion graphs of concrete prisms or mortar bars made with these aggregates are characterized by the early onset of expansion and also usually by a high rate of expansion (Figure B4(a)). Cracking of concrete structures is usually observed within ten years of construction.

B2.2 Slow/Late Expanding Alkali Silicate/Silica Reaction

This type of reactivity is distinguished from alkali-silica reactivity by the delayed onset of expansion of concrete test prisms (Figure B4 (b)) and the length of time before cracking becomes evident in concrete structures, which may be up to 20 years. Strained quartz is thought to be one reactive component in many of these rocks. A wide variety of quartz-bearing rocks such as: greywackes, argillites, quartzwackes, quartz-arenites, quartzites, hornfels, quartz biotite gneiss, granite, phyllite, arkose and sandstone have been found to be reactive.

B2.3 Alkali-carbonate Reaction

Alkali-carbonate reaction occurs between certain argillaceous dolomitic limestones and the alkaline pore solution in the concrete. It causes expansion and extensive cracking of concrete. The reaction under laboratory conditions is usually characterized by the rapid onset of expansion of concrete test prisms (Figure B4(c)). Expansive dolomitic limestones are characterized by a matrix of fine calcite and clay minerals with scattered dolomite rhombohedra. The characteristic texture may be observed in thin sections with a petrographic microscope or in the scanning electron microscope (Reference B6.1). Structures undergoing this reaction usually show cracking within five years of construction.

B3. Methods of Evaluating Potential Reactivity of Aggregates

B3.1 General

A field investigation of concrete structures containing the aggregate and having about the same alkali level and environment as the proposed concrete structure is possibly the best method of evaluating acceptability of concrete aggregates. In many instances, such a field investigation is not possible, because the aggregate has not been used previously in concrete, for instance, when it comes from a new lift or horizon in a quarry. In other cases, variations in cement content of the concrete or curing history may result in field performance investigation giving misleading results. Under these circumstances, some type of laboratory investigation must be undertaken. Laboratory test methods fall into two categories. Those in which the composition of an aggregate under investigation is determined by petrographic or chemical analysis and compared to the composition of known reactive and non-reactive aggregates. Secondly, those in which the expansivity of an aggregate is determined, either in rock prisms in an alkaline solution, in mortar bars or in concrete prisms. The first type of test is rapid and convenient but of limited use due to the inherent uncertainty in the correlation between the determined composition and the reactivity of the aggregate. Even with the second method, caution must be exercised due to variability in the tests and difficulties in interpreting the results as there are few studies in which the correlation between laboratory expansion and field deterioration of concrete has been reliably established. Engineering judgement is needed when interpreting test data and performance of concrete structures, when aggregates are found to be marginally or deleteriously reactive.

B3.2
CSA A23.2-15; Petrographic Examination of Aggregates for Concrete

Petrographic examination is the first step when evaluating the potential reactivity of an aggregate. It is carried out to determine the types of rock comprising the aggregate so that appropriate laboratory tests can be made (see Figure B1). In certain cases, where specific rocks or minerals are known from experience to cause deterioration of concrete, identification of these constituents in an aggregate by petrographic examination may be sufficient evidence to reject the aggregate. Care is needed in making petrographic examinations of aggregates containing reactive silica, eg, Ordovician limestones, in which as little as one per cent of chert, an amount easily overlooked, can cause deleterious expansion of concrete. Petrographic examination may also be used to indicate potential reactivity of late-expansive quartz-bearing rocks by determining the presence or absence of undulatory extinction of quartz grains. The acceptability of potentially reactive alkali-carbonate aggregates may also be determined by petrographic examination. However, it must be stressed that a careful, detailed examination is necessary to differentiate between some expansive and non-expansive aggregates which appear similar upon superficial examination.

B3.3 ASTM Standard Test Method C289; for Potential Reactivity of Aggregates (Chemical Method)

Test results of aggregates represented by points lying to the right of the solid line in Figure 2 of ASTM Standard C289 should usually be considered potentially alkali-silica reactive. Experience outside Canada has, however, shown the need to develop separate limits for the permissible amount of dissolved silica (S_c) for certain rock types on a regional basis (Reference B6.2). When test results indicate an aggregate to be innocuous, engineering judgement should be exercised before it is accepted as test results may be affected by a number of factors:

(a) variability in the aggregate source;
(b) problems of obtaining a representative sample;
(c) interlaboratory variability in the test results.

It is advisable to run occasional duplicate samples as a check on the analytical procedure. If the results obtained using this test indicate deleterious reactivity, other confirmatory tests should be conducted.

B3.4 ASTM Standard C227, Potential Alkali Reactivity of Cement Aggregate Combinations (Mortar Bar Method)

B3.4.1 General

This test can be used with some confidence to identify potentially reactive aggregates of the alkali-silica reactive type. The test may also be used to evaluate the effectiveness of partial replacement of the cement with a supplementary cementing material in preventing or minimizing deleterious expansion and cracking of concrete made with alkali-silica reactive aggregates. Some carbonate rocks containing chert exhibit alkali-silica reactivity. When the mortar bar test is used to evaluate such aggregates, it is recommended that the alkali content of the cement, used in the fabrication of the mortar bars, be increased to 1.25% by the addition of NaOH to the mix water as specified in the accelerated concrete prism test A23.2-14A, Clause 5.1. The mortar bar test is not recommended for evaluating aggregates which may cause concrete deterioration due to the alkali-carbonate reaction.

B3.4.2

Maintaining adequate humidity and temperature in the storage containers for the mortar bars is of critical importance for expansion measurements. For this reason it is a good practice to include in each test series, as a check on the storage conditions, some mortar bars made with pyrex and a non-reactive aggregate with known expansion characteristics.

B3.4.3

When certain reactive minerals, eg, chert and opal occur in relatively small amounts, in some cases as low as 3% in an aggregate, maximum expansion of mortar bars is observed. This percentage is known as the pessimum proportion. Smaller expansions are observed with either higher or lower amounts of the reactive mineral (Reference B6.3).

B3.4.4

Cracking of mortar bars is usually observed when expansion exceeds 0.05%. Hence, expansions greater than this may be considered to be deleterious (see Table B1 for suggested expansion limits). Expansion limits at three months should only be considered when six-month results are not available. The rate of expansion should be taken into account when evaluating an aggregate for even if the expansion is below the specified limit, and the rate is still high, excessive expansion may occur later than six months.

B3.4.5

The percentage of alkali-silica reactive minerals, such as opal and chert, may vary in different parts of an aggregate deposit. For this reason it is important to test the aggregates for the pessimum proportion (B3.4.3).

B3.5 CSA Test Method A23.2-14A; Concrete Prism Expansion Test for Alkali-Aggregate Reaction

B3.5.1

This test method is recommended for the determination for potential expansion of alkali-carbonate reactive aggregates. An accelerated test in which the concrete prisms are stored at 38°C is the current method for evaluating the potential expansivity of slow/late expanding alkali silicate/silica reactive aggregates. The concrete prism expansion test can also be used to evaluate the effect of supplementary cementing materials, lower alkali cement, or alternate mix designs, including job mixes on expansion of concrete containing reactive aggregates.

B3.5.2 Alkali-carbonate reaction

Cracking of concrete test prisms is usually observed before expansion reaches 0.05%. The exposure conditions of the concrete will affect the extent of damage occurring due to alkali-carbonate reaction. For this reason, separate expansion limits are suggested for different classes of exposure in Table B1. Experience has shown that when an aggregate causes expansion of over 0.025% at one year in concrete prisms made with a cement having its alkali content increased to 1.25% by the addition of NaOH to the mixing water, cracking is also observed in Class A exposure conditions such as highway structures (Reference B6.4).

Concrete test prisms continue to expand for a year or more before the rate of expansion decreases. Therefore, when possible, tests should be conducted for at least one year. The three-month expansion limits should be used with caution since some delayed expansive aggregates pass the three-month requirement but fail at one year and cause cracking of concrete in the field.

B3.5.3 Slow/Late-Expanding Alkali Silicate/Silica Reaction

The accelerated concrete prism test in which the prisms are stored at 38°C and 100% humidity is satisfactory for the evaluation of the potential reactivity of slow/late expanding alkali-silicate/silica reactive aggregates.

Note: *This test may be used with alkali-silica reactive aggregates, but no expansion limits have been developed for these aggregates.*

Expansion of concrete structures made with these types of aggregates is dependent on the exposure conditions and for this reason, in Table B1, suggested expansion limits are given for various classes of exposure.

The average time elapsing before the commencement of significant expansion of concrete test prisms is about three months (see Figure B4). For this reason, no expansion limits can be specified at early ages. Expansion limits for one year are specified. An estimate of the ultimate expansion can be obtained after about six months by extrapolation to one year (Reference B6.5).

B3.5.4 Evaluation of Preventative Measures or Job Mix on Expansion of Concrete Test Prisms

When the concrete prism test A23.2-14A is used to evaluate the effect of the "job mix", use of a cement with a lower alkali content or the use of supplementary cementing materials on expansion due to alkali-aggregate reaction, two tests must be made; one with the standard mix specified in A23.2-14A, the second with the job mix or experimental mixes containing a cement with a lower alkali content, lower or higher cement contents or supplementary cementing materials. Due to the variables inherent in this second test, it is not possible to establish firm criteria for the evaluation of the test results. However, as a general rule, samples showing expansion in excess of 0.04% at one year should be considered excessive and indicative of potentially deleteriously expansive concrete. The trend of the graph of expansion at the end of the test period should also be taken into account when evaluating the results and, above all, engineering judgement must be exercised.

B3.6 ASTM Standard Test Method C586, for Potential Alkali Reactivity of Carbonate Rocks for Concrete Aggregates (Rock Cylinder Method)

B3.6.1

The test is useful for determining the relative expansion characteristics of rock cylinders cored from different layers or horizons in a quarry and as an indicator of the need for additional testing of the aggregate in concrete prisms. The test is particularly useful for the preliminary evaluation of potential aggregate sources. The frequency of sampling is important if the results obtained from this test are to be applicable to the bulk aggregate. The sampling interval is, to some extent, determined by the character of the rock, generally, samples should be taken at not more than 0.3 m intervals.

Experience has shown that expansions of rock cylinders, in 1M NaOH solution, of 0.1% or more at four weeks or 0.2% at sixteen weeks, are usually indicative of deleteriously expansive aggregates (Reference B6.4).

B3.6.2 Alkali-silica Reaction

The rock cylinder test does not give an indication of the potential expansion of alkali-silica reactive aggregates. However, cylinders of cherty limestone immersed in 1M NaOH may develop patches of alkali-silica gel. Observation of gel may be taken as a qualitative indicator that the rock is potentially reactive and should be evaluated further.

B3.6.3 Slow/Late-expanding Alkali Silicate/Silica Reaction

The rock cylinder test has been used with some success to evaluate the potential reactivity of greywacke, argillite, phyllite, quartz arenites and quartzwacke aggregates. For this purpose the test is modified by storing the samples at 38°C, rather than at room temperature. Good correlations have been obtained between the expansion of rock cylinders in alkaline solution and the expansion of concrete prisms made with very reactive argillites, quartz-arenites and greywackes (References B6.5, B6.6, B6.8). Due to the difficulty of obtaining representative samples of the bulk aggregate for rock cylinders and to the uncertain correlation between the expansion of rock cylinders in alkaline solution and concrete test prisms, this method does not lend itself to the development of acceptance or rejection criteria for concrete aggregates. However, cylinders showing expansions greater than 0.1 per cent at one year are usually indicative of deleteriously expansive aggregate.

B4. Distribution of Potentially Reactive Aggregates

B4.1

Potentially reactive aggregates occur in most regions of Canada (Reference B6.9). However, deleterious expansion of concrete containing such aggregates has generally not been obvserved in Western Canada where the normal type 10 cement had an alkali content of about 0.7% Na_2O equivalent. In compiling the occurrence of potentially reactive aggregates the published and unpublished experience of a number of agencies has been drawn upon. However, this catalogue of known reactive aggregate sources is probably not all inclusive and it may be expected that new occurrences of such aggregates will be found.

B4.2 Atlantic Canada

Slow/late-expanding alkali silicate/silica aggregates have caused deterioration of concrete structures in Nova Scotia and New Brunswick. Similar aggregates are found in Newfoundland. Laboratory testing has shown some quartzites, argillites, greywackes and phyllites to be reactive. Rhyolites exhibiting alkali-silica reactivity also occur in Nova Scotia (Reference B6.8).

B4.3 Quebec

Alkali-silica reactive aggregates:
Limestones of Trenton age which occur in an area stretching from Quebec City through Three Rivers to Montreal have been found to be alkali-silica reactive (Reference B6.10). Potsdam sandstone, occurring near Montreal, is also deleteriously expansive (Refence B6.11).

Slow/late expanding alkali silicate/silica aggregates:
Some marginally expansive aggregates have been identified in the carbonate sediments and metavolcanics of the Ascot formation in the Eastern townships. Expansion is thought to be due to strained quartz. A reactive quartz biotite gneiss has been identified from the James Bay region. The reactivity is thought to be due to strained quartz in the rock. Quarried granite aggregate of Grenville age should be considered as potentially alkali-silica reactive.

B4.4 Ontario

Alkali-silica Reactive Aggregates:
Some limestones of Black River and Trenton age, in the Ottawa and Peterborough regions, which contain a small percentage of chert, have been shown to be deleteriously expansive in concrete. Potentially reactive palaeozoic cherts from the James Bay Lowlands, occur in gravels over much of Northern Ontario. Potentially reactive Precambrian cherts occur in Northern Ontario. Results of the chemical test ASTM C289 showed some cherts from Southwestern Ontario to be potentially reactive (Reference B6.12) but this was not confirmed by subsequent laboratory studies and damage was usually not observed in concrete structures when the chert content of the aggregate was less than 5%. Recent laboratory and field studies, however, have shown that at least some aggregates containing chert are reactive.

Slow/late-expanding alkali silicate/silica reactive aggregates:
Some Precambrian sandstones, argillites, quartz arenites, quartzites and greywackes of the Huronian Supergroup from North of Lake Huron, Sudbury and New Liskeard regions have been found to be slowly reactive (References B6.5, B6.6). Damage to concrete

containing these aggregates generally does not show up for at least ten years. However, recently some bridges in the Sudbury area were found to be cracked after only four years.

Quarried granite aggregate of Grenville age has been found to be alkali-silica reactive and cause deterioration of concrete. Granites of Grenville age occur in the Westport area of Southeastern Ontario.

Alkali-carbonate reactive aggregates:
Rock exhibiting alkali-carbonate reactivity is found in the Lower Gull River Formation which extends along the southern margin of the Canadian Shield from Midland to Kingston (References B6.1 and B6.13). The same reactive rock also occurs in the Ottawa-St. Lawrence Lowlands near Cornwall and in the Ottawa Area (Reference B6.4).

B6.5 Hudson Bay Lowlands

Potentially alkali-carbonate reactive rocks of Ordovician age may also be found in the Hudson Bay Lowlands of Northern Manitoba and Northern Ontario. Dolomitic limestone of the Bad Cache Rapids Group near the Nelson River causes expansion in concrete prisms; cylinders of this rock immersed in NaOH also expand excessively.

B6.6 Manitoba and Prairies

Alkali-silica Reactive Aggregates:
A "siliceous" shalestone containing varying amounts of opal has been found in many granular deposits in southwestern and southcentral Manitoba and in much of the grainbelt area of Saskatchewan. This material was first identified during the pre-construction stages for Gardiner Dam (Reference B6.14). Although the siliceous shalestone is very reactive, it is generally found only in small percentages in the surficial sands and gravels, and does not usually produce harmful expansions in the concrete. The material does, however, cause problems relating to pop-outs on flat work and other exposed concrete surfaces.

Cherts are found in the gravels near the western margin of the Prairies, eg, in the Calgary region. Some of these cherts may be potentially reactive if used in concrete having a high alkali content.

B6.7 British Columbia

Alkali-aggregate reactivity has not been a significant problem in British Columbia to date, due largely to the use of relatively low alkali cement. However, this could change if alkali levels in the cements increase. Potentially reactive siliceous, glassy volcanic rocks occur in British Columbia. It is reported that in the Cache Creek Basin glassy volcanic rocks (rhyolites) have caused problems in concrete made with a low alkal cement when the concrete was exposed to deicing salts. Similar aggregates in the Western USA have been shown to cause deterioration when used as concrete aggregate (Reference B6.15). Chert occurs in fluvial-glacial gravel deposits. A sample of such a fluvial-glacial gravel exhibited marginal reactivity when tested in mortar bars (Reference B6.16).

B6.8 Arctic Canada

Because of the low volume of construction, little is known about the quality of Northern aggregates. When the demand of concrete aggregate increases due to pipeline construction or other large-scale projects, there will be a need to evaluate potential sources of aggregate. A reactive greywacke has been identified at Alert in Ellesmere Island (Reference B6.17). Expansion due to alkali-aggregate reactivity is slowed by low temperature, but low temperatures should not be relied upon to give protection to the concrete if highly reactive aggregates are used.

B5. General Assessment of Reactive Aggregate

B5.1

Expansion and cracking of concrete made with reactive aggregate is affected by a number of factors: the expansion characteristics of the aggregate, the alkali content of the cement and the microclimate to which the structure will be exposed. Because interaction of these factors complicates the decision-making process, the authority or other contractural party having responsibility for assessing if an aggregate is acceptable is well advised to seek advice from experienced people. Due to the long period of time required to obtain results, testing of aggregate should be undertaken at least one year before concrete will be placed.

B5.2

To ensure that the non-reactive nature of an aggregate has not changed, periodic testing of the source is required. The frequency of testing will vary depending on the nature of the source of the aggregate and the type of construction. In some cases inspection and or testing on a daily basis may be necessary, in other cases testing once a year may be sufficient provided that there has been no obvious change in the aggregate deposit.

B5.3

A modification to standard test procedures, sometimes found useful, is to employ a mix design and curing procedure similar to that to be employed in practice, particularly when a rich mix design or curing under elevated temperature conditions (eg, accelerated steam-curing) is to be used. Unusually high cement contents may be expected to increase the rate and/or degree of alkali-aggregate reactivity because of the increase in alkali content per unit mass of concrete. Curing at elevated temperatures may increase the severity of the problem because the rate of chemical reactions increases with temperature. Furthermore, because of the more rapid rate of hydration of the cement minerals, in which some alkali ions may be held in solid solution, a more rapid release of alkalies to solution may occur with consequent higher than normal alkali concentrations in pore solutions in the concrete at early ages. Increase in concentration is commonly accompanied by increase in rate of a chemical reaction. Modification to the structure of the hydration products formed during accelerated curing may also be signficant.

B6. References

1. Swenson, E.G. and Gillott, J.E., 1964. Alkali-Carbonate Rock Reaction. Highway Research Record No. 45, pp 21-40.

2. Grattan-Bellew, P.E., 1983. Evaluation of Test Methods for Alkali-Aggregate Reactivity. Danish Concrete Association, 6th International Conference "Alkalis in Concrete: Research and

16. by ?

Practice". Technical University of Denmark, Copenhagen, June 1983. Proceedings pp 303-314.

3. Hobbs, D.W., 1980. Influence of Mix Proportions and Cement Alkali Content upon Expansion due to the Alkali-Silica Reaction. Cement and Concrete Association (U.K.) Technical Report 534.

4. Rogers, C.A., 1985. Evaluation for Potential for Expansion and Cracking due to the Alkali-Carbonate Reaction. Ontario Ministry of Transportation and Communications, Engineering Materials Office Report EM75, Feb 1985, 38 p.

5. Grattan-Bellew, P.E., 1981. A Review of Test Methods for Alkali-Expansivity of Concrete Aggregates, Proc. 5th International Conf. on Alkali-Aggregate Reaction in Concrete, S252/9. Cape Town, South Africa. National Building Research Inst. of CSIR Pretoria, S.A.

6. Dolar-Mantuani, L., 1969. Alkali-Silica-Reactive Rocks in the Canadian Shield. Highway Res. Record 268, 99-117.

7. Duncan, M.A.G., Swenson, E.G., Gillott, J.E. and Foran, M.R., 1973.

8. Alkali-Aggregate Reaction in Nova Scotia, parts I, II, III and IV. Cement and Concrete Research V3, 1973, pp 55-70, 119-128, 233-245 and 521-535.

9. Rogers, C.A. and Worton, S., 1984. Alkali-Aggregate Reactions in Canada, a Bibliography. Ontario Ministry of Transportation and Communications, Engineering Materials Office Downsview, Ontario. Report EM-73.

10. Fournier, B. and Berube, M.A. 1985. Charactérisation des granulats calcaires exploités dans la région de Québec et évaluation de leur potentiel de reactivité alcaline. Dépt. de Géologie, Faculté des Sciences et de génie Ciré universitaire Québec (Université Laval).

11. Bérard, J. and Lapierre, N., 1977. Réactivités aux alkalis du grés de Potsdam dans les bétons. Can. J. of Civil Engineering 4, pp. 332-344.

12. Ingham, K.W., and Koniuszy, Z.D., 1966. Distribution, Character and Basic Properties of Chert in Southwestern Ontario. Hwy. Res. Rec. No. 124, pp 50-78.

13. Ryell, J., Chojnacki, B., Woda, G. and Koniuszy, Z.D., 1974. The Uhthoff Quarry Alkali-Carbonate Rock Reaction: A Laboratory and Field Performance Study. Transportation Research Record 525, pp 43-54.

14. Price, G.E., 1961. Investigation of Concrete Materials for South Saskatchewan River Dam. Proc. ASTM 61, pp 1155-1179.

15. Stark, D.C., 1980. Alkali-Silica Reactivity: Some Reconsiderations, Cement, Concrete and Aggregates CCAGDP, Vol 2, No. 2, Winter 1980.

16. Mindess, S. and Gilley, J.C., 1973. The Staining of Concrete by an Alkali-Aggregate Reaction. Cem. Concr. Res. 3, pp 821-828.

17. Gillott, J.E. and Swenson, E.G., 1973. Some Unusual Alkali-Expansive Aggregates, Eng. Geol. 7, 181-195.