SHEET 1 OF 2

* PREPARED FROM AUGUST 1969 BLACK AND WHITE VERTICAL AERIAL PHOTOGRAPHS, WITHOUT FIELD CHECKING.

PREPARED FOR

TECHNICAL SERVICES BRANCH
DEPARTMENT OF INDIAN AFFAIRS
AND NORTHERN DEVELOPMENT
OTTAWA

AIRPHOTO ANALYSIS ASSOCIATES
CONSULTANTS LIMITED
859 COLLEGE STREET, TORONTO, ONTARIO

DATE APRIL 1971 SCALE : AS SHOWN

CORAL HARBOUR
SOUTHAMPTON ISLAND
HUDSON BAY

SCALE

1000 0 1000 2000 3000 4000

1 INCH TO 1000 FEET
APPROXIMATE ONLY

CORAL HARBOUR
SOUTHAMPTON ISLAND
HUDSON BAY

SCALE

1000 0 1000 2000 3000 4000

I INCH TO 1000 FEET
APPROXIMATE ONLY

FACTOR MAP NO. I
ROUTE LOCATION *
(POWER LINE AND SERVICE ROAD)

LEGEND

EXISTING OIL PIPELINE

EXISTING UNDERGROUND OIL PIPELINE

EXISTING UNDERGROUND POWER LINE

STRAIGHT LINE DISTANCE

PROPOSED POWER LINE ROUTE

PROPOSED ROUTES FOR NEW ROAD

POSSIBLE SUBSURFACE ICE LENSING

LOWLAND AREAS SUBJECT TO FLOODING (POSSIBLE PERMAFOST)

DANGER FROM STORM WAVE EROSION

N POTENTIAL GRANULAR CONSTRUCTION MATERIAL
($N \times 10^3$ CU. YDS.)

EROSION HAZARD ON EXISTING ROADWAY

RIVER CROSSING, STRUCTURE REQUIRED

- LOWLAND AREAS SUBJECT TO FLOODING (POSSIBLE PERMAFOST)
- DANGER FROM STORM WAVE EROSION
- N — POTENTIAL GRANULAR CONSTRUCTION MATERIAL ($N \times 10^3$ CU. YDS.)
- EROSION HAZARD ON EXISTING ROADWAY
- RIVER CROSSING, STRUCTURE REQUIRED
- DIVERSION, USE ROADWAY EMBANKMENT AS DIVERSION
- POTENTIAL WASHOUT
- POSSIBLE CONTAMINATED WATER SUPPLY
- PERMANENT DRAINAGE CHANNEL
- FLOOD DRAINAGE CHANNEL

4. <u>Terrain</u>

The study shows that the drainage system is severely disorganized and it is conceivable that the principal channels may change from time to time either as a semi-permanent situation or as a temporary condition resulting from a particularly heavy run-off. A route must be selected where there is least likelihood of conditions changing and creating further problems of wash-out. The route should also be selected in areas where the ground is reasonably stable and near to a good source of granular material which can be used for road construction.

5. <u>The Social Interests of the Resident</u>

The availability of social facilities at the M.O.T. establishment make this the natural focal point for social activities. However when an improved community centre with pool tables, lounge, bar facilities, etc., is provided in the settlement, there will be less need for people to travel to the air base.

6. <u>Cost/Benefit Ratio</u>

If an entirely new route is selected, the investment in the existing road is largely lost and a major investment will have to be made in the new route. If this exceeds the capitalized value of the savings in transportation costs attributable to the new route, it may be better to improve the existing road. In assessing the relative benefits wear and tear on trucks, etc., must be considered.

7. <u>Snow Drifting</u>

The snow fall in Coral Harbour is relatively light but drifting can be a problem. The present route is on relatively high ground and drifting is not a serious problem. However, if a route is selected in a lower area, there may be a problem of drifting unless the road can be built on embankments which again entails high costs.

The M.O.T. has a good efficient, well-run power station containing three 250 KW generators. We understand that peak demand from the M.O.T. establishment is approximately 270 KW. It is M.O.T. policy to have at least 100% standby at all times. The settlement of Coral Harbour has a small power station containing four 60 KW diesel generators. The heat from the generators is used to heat the garage through heat exchangers. It will be seen that the M.O.T. power station is capable of generating all the power that is required by the settlement and the base. At peak period, this would entail the use of three generators and no standby would be available. There appear to be three possibilities of handling power generation in the area.

1. Continue with individual power stations operated by M.O.T. and the N.W.T. This is inefficient and leads to duplication of facilities.

2. Provide one major power station operated either by M.O.T. or by the Government of the N.W.T. providing power both for the base and the settlement. This would result in a more efficient operation with less duplication. A power transmission line would be required between the settlement and the M.O.T. establishment which could also carry telephone lines and, if it followed the road, street lights at intervals. If the line were routed via SNAFU Beach, it would also provide a power capability at this location.

3. The provision of a new power station owned and operated by Northern Canada Power Commission and providing power to both the community and the M.O.T. establishment. This would have the same advantages as the second alternative with the added advantage that if the suggested amendment to the N.P.C.P. Act receives approval, the method of accounting would be changed from individual plants to across the board accounting for the Commission as a whole. This could lead to a reduction in the rate paid for power in remote settlements such as Coral Harbour. If the power station were sited near to the settlement, it would offer the possibility of using waste heat from the generators to heat a utilidor for supplying domestic water to the community (either to individual homes or to water points distributed throughout the community). This settlement is, at present, relatively small but is increasing quite rapidly (population 1968 - 285, 1969 - 288, 1970 - 337). Many residents continue to hunt and trap very successfully both for subsistence and for trade. In addition, there is healthy business in carvings and other artifacts. Coral Harbour may well attract additional Eskimo families who wish to live on the land from areas which begin to feel the pressure of white civilization. Additional growth may therefore be anticipated and in the near future well organized and efficient municipal services will be required.

We would suggest that during the current summer the various proposed routes should be inspected, preferably by a member of your Regional staff and marked either with stakes or oil drums. Those points identified as being potential wash-outs or subject to erosion hazard from flood water or storm wave action should be examined. At the same time, the areas identified as sources of granular material should be checked for quality and quantity. This would provide a basis for a rough estimate of the cost of constructing the various routes, and the number and type of structures required. We would further suggest that when this information is available a service contract should be let for inspection of the site by Airphoto Analysis Associates at the period of maximum discharge of the Post River. This firm has a portable camera pack which can be mounted on any plane and is capable of taking good quality vertical air photos suitable for photo interpretation (but not photogrammetry). You might find it worthwhile to have this type of photo coverage of the various road alignments and the points at which they cross the Post River at the time of maximum flow. With this information, I believe you would be in a position to make a good decision on the selection of the road.

Copies of letters from Mr. C.M. Arkell and Mr. C.J. Crapper are appended for your information.

K.W. Stairs
for D.A. Davidson
A/Director
Territorial Affairs Branch.

APPENDIX C

WATER SUPPLY DESIGN BRIEF (THURBER 1979)

WATER SUPPLY SYSTEM
CORAL HARBOUR, N.W.T.
DESIGN BRIEF

A Report

to

DEPARTMENT OF PUBLIC WORKS
GOVERNMENT OF THE NORTHWEST TERRITORIES

THURBER CONSULTANTS LTD.
Edmonton, Alberta

D.A. Lindberg, P.Eng.
Review Principal

September 21, 1979
File: 15-22-3

W.G. Rogers, P.Eng.
Project Engineer

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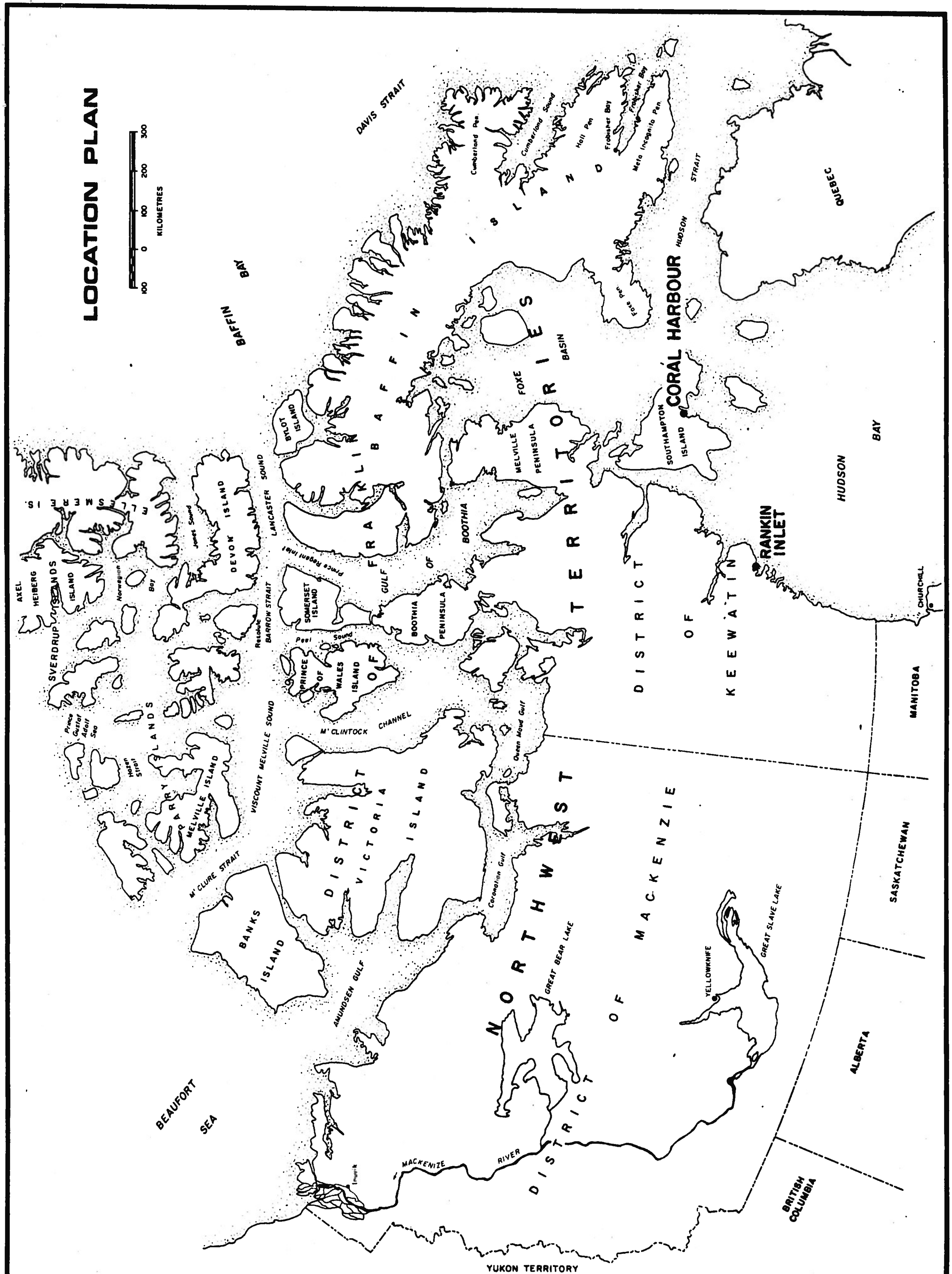
LOCATION PLAN
100 0 100 200 300
KILOMETRES
BAFFIN BAY
DAVIS STRAIT
Cumberland Pen.
Cumberland Sound
Hall Pen.
Frobisher Bay
Meta Incognita Pen.
HUDSON STRAIT
QUEBEC
B A F F I N I S L A N D
F R A N K L I N
BYLOT ISLAND
LANCASTER SOUND
Prince Regent Inlet
GULF OF BOOTHIA
FOXE BASIN
Foxe Pen.
CORAL HARBOUR
SOUTHAMPTON ISLAND
MELVILLE PENINSULA
HUDSON BAY
RANKIN INLET
CHURCHILL
ELLESMERE IS.
AXEL HEIBERG
SVERDRUP ISLANDS
Jones Sound
DEVON ISLAND
Norwegian Bay
ISLAND
SOMERSET ISLAND
BARROW STRAIT
Resolute
BOOTHIA PENINSULA
Peel Sound
PRINCE OF WALES ISLAND
Prince Gustaf Adolf Sea
Hazen Strait
PARRY ISLANDS
MELVILLE ISLAND
VISCOUNT MELVILLE SOUND
M'CLINTOCK CHANNEL
M'CLURE STRAIT
DISTRICT
VICTORIA ISLAND
Queen Maud Gulf
Coronation Gulf
BANKS ISLAND
AMUNDSEN GULF
BEAUFORT SEA
N O R T H W E S T T E R R I T O R I E S
DISTRICT OF KEEWATIN
DISTRICT OF MACKENZIE
GREAT BEAR LAKE
GREAT SLAVE LAKE
YELLOWKNIFE
MACKENZIE RIVER
Inuvik
MANITOBA
SASKATCHEWAN
ALBERTA
BRITISH COLUMBIA
YUKON TERRITORY

FIGURE I

4.2 Reservoir Siting

The feasibility of several possible reservoir sites would be investigated. These could consist of:

(a) dyking between natural topographic highs;

(b) blasting and excavating the entire reservoir capacity out of rock or frozen soil; or

(c) a combination of blasting, excavating and dyking.

4.3 Reservoir Capacity

4.3.1 Useable Water:

The agreed consumption which must be satisfied was the projected demand for 1998, approximately 600 cubic metres per week. It has been assumed that there may be only 12 weeks of suitable weather during the summer for filling the reservoir; therefore, a 40 week supply (24,000 cubic metres) of useable water was required as the design capacity.

4.3.2 Unuseable Water:

The depth of unuseable water in the bottom of the reservoir due to siltation and decrease in quality due to ice formation was assumed to be 1 metre. This may be somewhat conservative for a reservoir in rock, but the difference was considered too small to affect the preliminary study results.

4.3.3 Maximum Ice Thickness:

Residents of Coral Harbour indicated that the maximum ice thickness on the Post River is about 2 m. This figure was assumed for preliminary design purposes.

4.3.4 Freeboard:

A freeboard of 1 metre was assumed for bermed reservoirs. No freeboard was assumed for reservoirs in rock.

4.4 Mechanical Considerations

Originally, this study assumed that a permanent pipeline was required to provide service to the school (including fire protection) and the nursing station. Therefore, it was planned to have the pipeline run to a truck fill point in the hamlet for domestic distribution. However, we were informed on June 7, 1979, that the final design of the school included fire water storage within the school and therefore the truck fill point could be located at the pumphouse at the reservoir if this proved to be more cost effective than a pipeline to town.

Treatment of the water was assumed where required to meet the acceptable limits of the Canadian Drinking Water Standards and Objectives (1968).

The Department of Public Works has indicated that pre-heating of the water will not be required for the Coral Harbour supply system.

4.5 Water Sources Considered

The reservoir would be filled either directly or indirectly with water from the Post River or an acceptable existing lake in the vicinity of the reservoir site.

5. ENGINEERING EVALUATION AND RECOMMENDATIONS

5.1 Selection of Reservoir Type

Three types of reservoir were investigated, and a total of 10 sites were considered. The locations and size of these are shown on Drawing No. 15-22-3-1 in Appendix A.

Sites A and C were considered for dyking between topographic highs. Sites G, H, J and K were considered for blasting the complete storage capacity out of rock. Sites B_1, B_2, E and F were considered as a combination of the above.

The earth dyked reservoir sites (A and C) were considerably larger than the other sites studied, since the dimensions of these sites are controlled by the surrounding topography. Site A is the largest site with dimensions of approximately 500 m x 750 m, and a depth of useable water of only about 0.1 m. The other reservoir sites which included dyking (Sites B1, B2, E and F) were somewhat smaller, since less use was made of the surrounding topography. The blasted reservoir sites (G, H, J and K) were conveniently sized for minimum rock excavation, while maintaining a workable size and depth of reservoir for blasting and excavating purposes.

The geotechnical investigation revealed that the only readily available dyking material is a shallow depth of frozen clayey sandy silt. This occurs in the depressions between north-south trending rock ridges that dominate the area. This silt is poorly suited for dyking; it would be difficult to obtain from the lakes and to compact into dykes and is of very limited quantity.

Furthermore, any of the sites that are away from existing roads would require construction of a fairly high berm across existing lakes for an access road. The existing lakes are actually part of channels which experience heavy flooding during spring runoff.

The lack of acceptable berming material therefore restricts the choice of sites to the four rock blasting sites (G, H, J and K).

5.2 Site Selection

The four rock blasting sites are essentially the same size, of similar construction, and are therefore of about the same geotechnical merit. Cost estimates for the four sites are provided in Section 5.6.

Of these four sites, Site K is the most suitable from location and hydrological aspects, as well as having the smallest capital cost. The site is a bedrock topographic high, over which the "old" road passes. The relief above adjacent drainage channels is greater than for the other bedrock sites and therefore the site offers better protection against potential flooding than Sites G, H and J.

Access for construction and operation of the reservoir is readily available and only requires the upgrading the existing road and re-routing it around the reservoir. On the other hand, Sites G and H would require construction of an access road to the sites, and Site J would require major upgrading of the existing "new airport" road, which presently washes out at specific sections following spring thaw. Site K is also closer to the power supply at the north end of the hamlet.

If Site K is implemented, then the performance of the road during spring flooding should be monitored and culverts designed and constructed at appropriate locations. Rip-rap protection for these culverts will be readily available from the reservoir excavation. Waste rock could be stockpiled beside the road and reservoir for future uses at other locations (e.g. proposed POL tank farm site).

5.3 Reservoir Dimensions

The dimensions of a rock blasted reservoir would essentially be the same for the four rock sites studied. A reservoir 60 m long by 55 m wide by 10 m deep would approximately provide the required storage of about 24,000 cubic metres assuming the conservative depth of 3 m of unuseable water and ice. This shape fits conveniently onto all sites using the presently available topographic information. The actual shape and size could easily be modified depending on final design of the intake or if it was desired to construct the reservoir in stages depending on actual demand and amount of funds available.

5.4 Mechanical Considerations

The details of the mechanical systems analyzed are provided in Appendic C. As discussed previously, the requirements for fire protection to the school in the hamlet and thus a pipeline to the school are no longer valid.

The most cost-effective system therefore consists of a combined pumphouse and truck fill point situated at the reservoir. The distance to the edge of the hamlet for trucking purposes therefore ranges from 400 m for Site K to 800 m at Site H.

APPENDIX A

CAPITAL COST COMPARISON OF SELECTED RESERVOIR SITES

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TABLE A-1

CAPITAL COST COMPARISON OF SELECTED RESERVOIR SITES

Item No.	Item Description	Unit	Unit Price	SITE G Qty	SITE G Cost	SITE H Qty	SITE H Cost	SITE J Qty	SITE J Cost	SITE K Qty	SITE K Cost
1.	Rock Blasting and Excavation	m^3	20.00	34,000	680,000	34,000	680,000	34,000	680,000	34,000	680,000
2.	Pumphouse/truck fill (wetwell, structural, mechanical, & electrical)	lump sum	lump sum	1	150,000	1	150,000	1	150,000	1	150,000
3.	Temporary pump	lump sum	lump sum	1	20,000	1	20,000	1	20,000	1	20,000
4.	Temporary Pipeline	m	40.00	100	4,000	100	4,000	100	4,000	100	4,000
5.	Electrical power to pumphouse	m	45.00	250	11,250	500	22,500	910	41,000	245	11,000
6.	Access to pumphouse (road, culverts)	m	50.00	555	27,750	550	27,500	0	-	0	-
7.	Engineering				145,000		145,000		145,000		145,000
	TOTALS				1,038,000		1,049,000		1,040,000		1,010,000

Note: Contingencies and Operation and Maintenance costs will be applied after further input by DPW.

APPENDIX B

GEOTECHNICAL INVESTIGATION

TABLE OF CONTENTS

THURBER

1. INTRODUCTION

This report presents the results of a geotechnical investigation for a proposed water storage reservoir for the hamlet of Coral Harbour, Northwest Territories.

The purpose of the geotechnical investigation was to investigate possible alternative reservoir sites in the vicinity of the hamlet, both by surface reconnaissance, and sub-surface drilling. This would enable a detailed evaluation of the alternative sites, and thereby assist in the preparation of a preliminary design for a suitable water storage reservoir for the hamlet.

2. METHOD OF INVESTIGATION

2.1 Geological Investigation

The available geological information related to Southampton Island, N.W.T. was reviewed, and an airphoto interpretation of the hamlet and surrounding area was carried out prior to field investigation, to obtain preliminary information on the site topography, geology, drainage and other features of interest to this investigation.

Possible locations for a water storage reservoir were identified, based on the topographic and geological features, for preliminary engineering assessment, and these include all the sites noted on the site plan (Dwg. 15-22-3-2).

The reservoir considered for each of these sites, would consist of one of the following arrangements:

(a) dyking between natural topographic highs
(b) blasting and excavating the complete reservoir out of rock or frozen soil; or
(c) a combination of blasting, excavating and dyking.

2.2 Field Program

A preliminary site reconnaissance was made between February 13 and 15, 1979 as part of the meetings between the Government of the Northwest Territories, and the Settlement Council. At this time a possible granular borrow source located at Snafu, approximately 6 km west of the hamlet on the "new" airport road, was inspected. Three grab samples were obtained from the area for laboratory gradation analyses.

2.3 Laboratory Testing

Laboratory testing included visual classification and the determination of natural moisture content of all soil samples. Atterberg limits and gradation analyses were carried out on selected samples from the test holes. Gradation analyses were also performed on the granular samples from the Snafu pit.

The results of drilling and laboratory testing are summarized on the attached test hole logs. The results of the gradation analyses on the granular samples from the Snafu Pit are presented on Figure B-1.

3. SITE DESCRIPTION

3.1 Surface Conditions

The area shown on the site plan (Dwg. 15-22-3-2) is generally characterized by linear bedrock ridges traversing the site in an approximate north-south direction, and spaced about 400 m apart. The hamlet of Coral Harbour is located on the southernmost end of one such bedrock ridge. This bedrock is mainly Pre-Cambrian granite gneiss, which is fairly massive and not extensively weathered. The bedrock ridges have been rounded by glacial and marine activity. Bedrock was not only exposed along the linear rock ridges, but also frequent bedrock outcrops were observed within the troughs between ridges, and in fact isolated rock knobs were observed through the ice cover under all of the lakes, suggesting a limited and variable thickness of fine grained sediment cover over bedrock.

Where rock is not exposed at the surface in the troughs the ground is usually covered with a thin veneer of brown fibrous peat. This peat was encountered to depths of up to 0.46 m in test hole 79-9. The troughs between these ridges are generally in-filled with a thin mantle of fine grained sediments which range from fine silty sands, to silts and organic silty clays. The maximum relief between the tops of the ridges and the troughs within the study area is in the order of about 6 metres.

Aside from this system of gently rounded parallel ridges and troughs, the area slopes gently southwards towards the Coral Harbour inlet. Surface drainage is consequently also towards the south and is confined to these troughs. Local depressions in these troughs have shallow ponded lakes in them during most of the year. During spring thaw, however, when flows are greatest some of these channels between bedrock ridges are entirely covered with a slow moving sheet of water. Both roads (the "old" airport road and the "new" airport road) leading from the hamlet wash out at specific channel crossings during these flood periods, including the bridge crossing the Post River on the "new" airport road.

A limited granular borrow source is located in the vicinity of the present hamlet dump, about 2 km north of the village; however, a very extensive granular borrow source is located at the Snufu area, approximately 6 km west of the hamlet on the "new" airport road.

The test holes drilled through the lake ice within the study area encountered depths of ice ranging from 0.46 m in TH's 79-15 and 79-16 to 1.7 m in TH 79-1, although the maximum depth of ice measured was 1.98 m at King Lake.

3.2 Subsurface Conditions

The majority of soil cores obtained during drilling consisted of grey fine sandy silt (ML) and organic silt (OL) of low plasticity. In the northernmost lakes investigated (Sites E and F,) the soil graded coarser to silty sand (SM), with some gravel at Site F. Traces of clay were found in some of the soil cores.

The soil cores were frozen and the ground ice varied considerably in structure and content. The majority of silty cores examined exhibited thin ice lensing, with an ice content estimated to be about 20 to 30%. Reticulate ice and segregated ice structures were also noted frequently in the silt cores obtained, and pore ice was noted in the coarser grained cores.

A layer of unfrozen highly organic silt was noted from a depth of 1.9 m to 3.0 m in Test Hole 79-1, below which the auger drill met refusal in coarse sand and gravel.

The auger drill met refusal at depths ranging from 0.3 m in TH 79-2 to 3.2 m in TH 79-1. Refusal of the auger drill or core barrel was interpreted in the majority of test holes to be due to the presence of frozen gravelly sand, although bedrock was suspected in several shallow holes, as noted on the test hole logs. It is expected that the gravelly soil is quite thin and directly overlies the bedrock.

4. GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

The drilling program was carried out primarily to assess the quality and quantity of construction materials available for the construction of a reservoir by some combination of deepening an existing lake and/or berming up around a lake with suitable fine-grained material. The potential sites that would require rock blasting only were investigated by surface inspection.

The following evaluation and conclusions are based on the results of the drilling and laboratory testing:

(1) All the lakes examined were relatively shallow and were frozen to the bottom. The lake ice depths measured were generally less than 1 m, ranging from 0.5 m in TH's 79-15 and 79-16 (Site E) to 0.9 m in TH's 79-10 and 79-11 (Site C). The exceptions were in test holes 79-1 (1.7 m) and 79-8 (1.2 m).

(2) The depth of fine grained soil encountered in the test holes ranged from zero in several test holes to 2.0 m in TH 79-16 (Site E). The average depth of fine grained material in the test holes was approximately 0.6 metres.

Most of the lakes investigated by drilling have a thin cover of gravelly sandy soil immediately over the bedrock surface. The results of the drilling, coupled with the site inspections, indicate that the bedrock surface is highly irregular under these existing lakes.

(3) The fine grained material encountered during drilling is generally a frozen slightly clayey sandy silt of low plasticity which contains some black organic silt.

SYMBOLS AND TERMS USED ON TEST HOLE LOGS

I. VISUAL TEXTURAL CLASSIFICATION OF MINERAL SOILS

Classification	Apparent Particle Size
Boulders	Greater than 200 mm
Cobbles	75 to 200 mm
Gravel	5 to 75 mm
Sand	Not visible to 5 mm
Silt	Non-Plastic particles, not visible to the naked eye
Clay	Plastic particles, not visible to the naked eye

II. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

Descriptive Term	Approximate Undrained Shear Strength	
Very soft	Less than 10 kPa	
Soft	10 - 25 kPa	
Firm	25 - 50 kPa	
Stiff	50 - 100 kPa	
Very stiff	100 - 200 kPa	Modified from National Building Code
Hard	200 - 300 kPa	
Very hard	Greater than 300 kPa	

III. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

Descriptive Term	Number of Blows per 300 mm (STANDARD PENETRATION TEST)	
Very Loose	0 - 4	
Loose	4 - 10	
Compact	10 - 30	
Dense	30 - 50	Modified from National Building Code
Very Dense	Over 50	

IV. LEGEND FOR TEST HOLE LOGS

○ Water content (% by weight) as determined on disturbed samples.

● Water content (% by weight) as determined on undisturbed samples.

Disturbed bag or split spoon sample.

Undisturbed Shelby Tube sample or core from VTM core barrel.

No recovery.

Number of blows per 300 mm for Standard Penetration Test.

C_u Undrained shear strength determined by unconfined compression test.

C_{vane} Shear Strength determined by pocket vane.

C_{pen} Shear Strength determined by pocket penetrometer.

Water Level.

THURBER

TEST HOLE No 79-1

THURBER CONSULTANTS LTD.
Geotechnical Engineers

LOG OF TEST HOLE

LOCATION See Dwg. 15-22-3-2

CLIENT DPW - NWT

PROJECT Coral Harbour Water Supply

DATE April, 1979

METHOD Ranger Auger Drill

DRILLING Co Mobile Augers

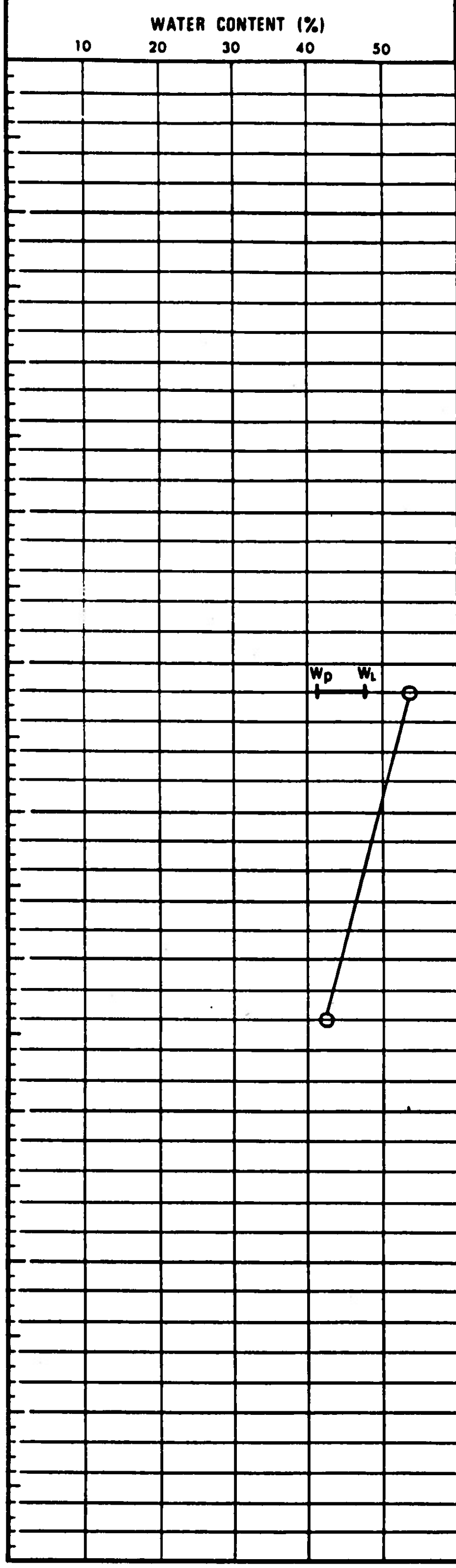
INSPECTOR S. Crossman

LEGEND:

SAMPLES: Disturbed; Undisturbed; No recovery

WATER CONTENT: Disturbed ○; Undisturbed ●

WATER LEVEL; W_L LIQUID LIMIT; W_P PLASTIC LIMIT



ELEVATION (METRES)

REMARKS

TOP OF HOLE ELEV 4.26 m

Depth of frost

Cpen=72 kPa

UNIFIED SOIL CLASSIFICATION SYSTEM

ML

SM-ML

LOG

DEPTH (METRES)

0

ICE

1

Black organic SILT frozen

2

Soft black, slightly sandy clayey organic SILT organic odor unfrozen

Dark grey to black gravelly silty SAND with pebbles and rocks

3

Bottom of test hole at 3.2 m, Auger refusal in gravel

4

SCALE: 1:25

FILE No 15-22-3

TEST HOLE No 79-2

THURBER CONSULTANTS LTD.
Geotechnical Engineers

LOG OF TEST HOLE

LOCATION See Dwg. 15-22-3-2

CLIENT DPW - NWT
PROJECT Coral Harbour Water Supply
DATE April, 1979
METHOD Ranger Auger Drill
DRILLING Co Mobile Augers
INSPECTOR S. Crossman

LEGEND:
SAMPLES: Disturbed, Undisturbed, No recovery
WATER CONTENT: ○ ●
WATER LEVEL
W_L LIQUID LIMIT
W_P PLASTIC LIMIT

WATER CONTENT (%) 10 20 30 40 50	ELEVATION (METRES)	REMARKS	UNIFIED SOIL CLASSIFICATION SYSTEM	LOG	DEPTH (METRES)
		TOP OF HOLE ELEV. 4.30 m			
				ICE	0
				Bottom of test hole at 0.9 m Auger met refusal in frozen gravel	1
					2
					3

SCALE: 1:25   FILE No 15-22-3

TEST HOLE Nº 79-3

THURBER CONSULTANTS LTD.
Geotechnical Engineers

LOG OF TEST HOLE

LOCATION: See Dwg. 15-22-3-2

CLIENT: DPW - NWT

PROJECT: Coral Harbour Water Supply

DATE: April, 9, 1979

METHOD: Ranger Auger Drill

DRILLING Cº: Mobile Augers

INSPECTOR: S. Crossman

LEGEND:
SAMPLES: ☐ Disturbed; ■ Undisturbed; ☒ No recovery
WATER CONTENT: ○ (Disturbed); ● (Undisturbed)
▼ WATER LEVEL
W_L LIQUID LIMIT
W_P PLASTIC LIMIT

WATER CONTENT (%): 10 20 30 40 50

ELEVATION (METRES)

REMARKS

TOP OF HOLE ELEV. 4.98 m

UNIFIED SOIL CLASSIFICATION SYSTEM

LOG

DEPTH (METRES)

Remarks	Unified Soil Classification	Log	Depth (m)
○ - 182%	Pt	Medium brown fibrous MUSKEG - frozen	0
		Bottom of test hole at 0.3 m	
		Auger met refusal in frozen granular material or possible bedrock	1
			2

SCALE: 1:25

FILE Nº 15-22-3