APPENDIX D

ARENA (THURBER 1985)

CORAL HARBOUR ARENA
GEOTECHNICAL EVALUATION

Submitted

to

GOVERNMENT OF THE NORTHWEST TERRITORIES
DEPARTMENT OF PUBLIC WORKS AND HIGHWAYS

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SECTION 1

INTRODUCTION

1.1 General

This report presents the results of a geotechnical evaluation undertaken in connection with the design of an ice arena to be constructed in Coral Harbour, N.W.T.

The scope of work for this investigation was determined in discussions with Mr. Rick Martin, P.Eng. of the Government of the Northwest Territories. Authorization for the work was received by Service Contract SC236769 dated August 19, 1985.

1.2 Proposed Development

The proposed recreational facility will be a steel frame structure, which will be about 40 metres wide and 80 metres long. It is understood that the structure will consist of steel beams which will provide a clear span of about 30 metres inside the building. The structure will be clad with sheet metal. A major portion of the structure will be unheated and will contain an ice arena as well as several sheets of ice for curling.

The ice areas will be kept below freezing through the winter, but will be allowed to melt during the summer months. At the present time, there are no plans to install an artificial ice making system in the arena.

Initially, it is expected that the ice will be placed directly on the ground surface. However, in the future, a concrete slab on grade may be poured in the arena area, so that the facility can be used for tennis and similar sports during the summer months.

A heated lounge area will be constructed at one end of the building. In the future, a second floor may be added above the lounge area to provide an area for community meetings.

It is understood that construction of the facility is scheduled to commence in 1986. The construction will be phased, and the first year's construction may only involve the foundations for the structure.

1.3 Method of Investigation

The geotechnical evaluation and recommendations presented in this report are based on a review of available information in our files, a terrain interpretation using aerial photographs at a scale of 1:5000, and a site inspection on August 21 and 22, 1985 by Mr. Rick Martin, P.Eng of the Government of the Northwest Territories, Mr. Ron Duffy of the Hamlet of Coral Harbour and Mr. Nick Hernadi of Thurber Consultants Ltd.

The site inspection involved visual examination of surficial soil and bedrock conditions at five potential sites. Test holes were not drilled, nor were test pits excavated.

SECTION 2

SITE DESCRIPTION

2.1 Geological Setting

The Hamlet of Coral Harbour is located at the head of South Bay, on the southern coast of Southampton Island, N.W.T. The community is located approximately 1600 km by air northeast of Yellowknife.

The terrain in the vicinity of Coral Harbour is comprised of low, rounded bedrock ridges which trend in a north-south direction. Relief is very low and the ground surface varies from level to gently undulating. The bedrock is composed of granitic gneiss, which is usually quite massive and not excessively fractured and weathered.

The area was inundated by the sea as the glacial ice retreated from the area about 7000 years before the present. The marine waters reworked and modified the surficial sediments. Fine grained marine sediments, ranging in texture from sand to clay, were deposited in the troughs or swales between the bedrock ridges. A lag deposit of cobbles and boulders may be expected between the fine marine sediments and the bedrock surface in some locations.

Coral Harbour is located within the zone of continuous permafrost. The thickness of the active layer is dependent upon several variables, however it is estimated that the maximum thickness of the active layer is likely in the order of 1 to 2 metres.

2.2 Site Conditions

2.1.1 Site 1

Site 1 is on a north-south trending bedrock ridge located to the northeast of the community as shown in Appendix A, Drawing A1 and Appendix B, Photograph No. 1. Approximately 300 m of utility lines will be required to connect this site to existing systems. Short extensions of existing roads either from the south or the north would be required to provide access.

The site is well drained, gently undulating and has exposed bedrock within the entire proposed building site. Some boulders up to 0.5 m in diameter are present on the surface, and portions of the bedrock are covered with moss and other vegetation. A minor amount of organic and inorganic soil overburden may be present in depressions in the bedrock topography. The relief within the building site is in the order of 2 metres.

Site preparation would require removal of boulders, the sparse vegetation cover and the minor amount of soil from bedrock depressions. Approximately 2 m of fill would be required in some portions of this site to achieve a design grade such as to avoid having to excavate bedrock.

The types of foundations considered suitable for this site include rock-socketed end bearing piles, footings founded on bedrock or footings on compacted granular fill. Detailed recommendations for the design and construction of foundation alternatives are given in Section 3.

2.2.2 Site 2

Site 2 is on a north-south trending bedrock ridge located to the southwest of the community as shown in Appendix A, Drawing A2 and Appendix B, Photograph No. 2. Approximately 300 m of utility lines would be required to connect this site to existing utility systems. Access to this site is currently available from the existing airport road.

The site is well drained, essentially level and has exposed bedrock over all, except at the extreme northeast corner of the proposed building site. Some boulders up to 0.5 m in diameter are present on the surface and portions of the bedrock are covered with moss and other vegetation. A minor amount of organic and inorganic overburden may be present in depressions in the bedrock topography.

The depth to bedrock in the extreme northeast corner of the proposed building site is not known, and the existing ground surface is approximately 1 metre lower in this area relative to the remainder of the proposed building site.

The overall relief within the building site is in the order of 1.5 m.

Site preparation would require removal of boulders, the sparse vegetation cover and the minor amount of soil from bedrock depressions, as well as the unknown thickness of soil overburden from the extreme northeast corner of the building site.

Except for the extreme northeast corner of the building site, less than 1.5 m of fill would be required on this site to achieve a design grade such as to avoid having to excavate bedrock. Due to the unknown depth to bedrock in the extreme northeast corner of the proposed building site, the depth of fill required in this corner is unknown.

The types of foundations considered suitable for this site include rock-socketed end bearing piles, footings founded on bedrock or footings on compacted granular fill. Detailed recommendations for the design and construction of foundation alternatives are given in Section 3.

2.2.3 Site 3

Site 3 is located immediately north of the community and east of the water reservoir, on a large area of relatively level and well drained rock fill, as shown in Appendix A, Drawing A1 and Appendix B, Photograph No. 3. Utilities and access are in close proximity to this site.

The rock fill at this site consists of the material blasted for the reservoir and was likely placed by end dumping or dozing onto the site over the existing ground surface. The thickness of this fill ranges from 3 to 4 m at the south end, to 1 to 2 m at the north end, based on examinations around the perimeter of the fill area only. Some bedrock exposures are evident at the east and north perimeter of the fill area, however, it appears that this area was poorly drained prior to filling and may have a significant thickness of overburden above bedrock.

Boulders up to 2 m in diameter are evident within this fill material, and the grain size distribution of the fill ranges from very coarse open and gap graded in some areas, to poorly graded with significant fines content.

We understand that this fill was dumped or pushed into place during summer 1981.

Due to the variable nature of the rock fill at this site and the uncertainty in subgrade conditions below the fill, the likely performance of building foundations at this site cannot be predicted with any level of confidence. Accordingly, this site is not recommended for the proposed structure with respect to geotechnical considerations.

2.2.4 Site 4

Site 4 is located to the north of the community and immediately adjacent to the Northern Canada Power Commission power plant on an area of exposed bedrock, as shown in Appendix A, Drawing A1 and Appendix B , Photograph No. 4. Utilities and access are in close proximity to this site. It is understood that this location is being considered in order to possibly utilize waste heat from the NCPC power plant.

The site is well drained, has exposed bedrock on all of the proposed building site, but slopes at 3 to 4° down towards the northwest. A large number of boulders to 2 m in diameter cover this site. Moss and other vegetation occurs on the bedrock to some extent, and a minor amount of organic and inorganic overburden is present on the bedrock.

The overall relief within the building site may be as much as 3 to 4 m, depending on the location and orientation of the proposed building.

Site preparation would require removal of the large number of boulders, as well as removal of the vegetation cover and minor amount of organic and inorganic soil overburden.

A large volume of fill would be required on this site to achieve a design grade which would avoid having to excavate bedrock.

The types of foundations considered suitable for this site include rock-socketed end bearing piles, footings founded on bedrock, or footings on compacted granular fill. Detailed recommendations for the design and construction of foundation alternatives are given in Section 3.

2.2.5 Site 5

Site 5 is located to the east of the Northern Canada Power Commission power plant in a low lying area that is poorly to moderately well drained, as shown in Appendix A, Drawing A1 and Appendix B, Photograph No. 5. Utilities and access are in close proximity to this site. It is understood that this location is being considered in order to possibly utilize waste heat from the NCPC power plant.

The site is mostly vegetated with grass. It slopes gently down towards the northwest and may be wet over some portions. No exposures of bedrock are evident. The thickness of overburden above bedrock is not know at this site.

The overall relief within the building site is in the order of 1 to 2 m, depending on the location and orientation of the building.

Site preparation at this location would require placement of a minimum of 1 m of clean gravel over the existing surface in order to achieve a satisfactory bearing surface. The fill would be required both inside and outside the building. It may be necessary to excavate soft materials in some locations and backfill to design grade.

Deep spread footings could be used at this site, however, footing excavations may be wet. The side slopes of the excavations can be expected to slough and will have to be cut back. If footings are selected at this site, it would be desirable to make footing excavations as early as possible in the summer, before the active layer thaws to its full depth.

Steel pipe piles designed as adfreeze piles would be the preferred choice for foundations at this site in view of the likely construction difficulties with deep spread footings. The steel pipe piles may encounter bedrock within the required depth of embedment. Detailed recommendations for the design and construction of foundation alternatives are given in Section 3.

If this site is selected, it is recommended that test holes be drilled prior to finalizing the design of the building foundation. The test holes are recommended in order to identify any unforseen ground conditions which may affect the design and construction of the facility at this location.

SECTION 3

GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

3.1 General

This section of the report presents geotechnical evaluations and recommendations which are applicable to all sites. The applicable foundations types considered suitable for each of the sites are as follows:

Site 1, 2 and 4
Rock-socketed pipe piles, footings on bedrock, or shallow footings on compacted gravel.

Site 5
Adfreeze pipe piles or deep spread footings.

Site 3
Not recommended for development.

General recommendations concerning the foregoing foundations are given in the following sections. The optimum choice of foundations will depend on ecomonic considerations, together with considerations of the level if risk accepted with respect to foundation movements.

On all sites, fill must be placed to obtain a level surface for the facility. The recommendations in this report assume that the fill and foundations for the structure will be placed during the same construction season, namely summer 1986.

3.2 Rock-Socketed Pipe Piles

This type of foundation system could be considered for Sites 1, 2, and 4. A sketch showing a typical installation is presented in Appendix A, Drawing A3.

Rock-socketed pipe piles must extend at least 1 metre into unweathered, sound bedrock. The inside and outside of the pile must be filled with permafrost grout in order to provide uplift resistance against frost heaving forces in the overburden soils, as well as to provide lateral resistance.

The use of rock-socketed pipe piles is not recommended where overburden depths exceed about 1 metre. Past experience indicates that where the thickness of overburden soils is excessive, it becomes very difficult to clean out the rock socket and ensure it is completely filled with grout.

The pipe pile hole should be drilled to the required depth. The rock socket should be thoroughly cleaned and the grout should then be placed down the hole. The pile should be placed in the hole and then driven to ensure it is properly seated on bedrock.

The allowable vertical load for piles seated on sound bedrock, using the procedure outlined above, will be a function of the structural capacity of the pile rather than the bearing capacity of the rock. It is recommended that the allowable vertical capacity of the piles be taken as two-thirds of the allowable short column capacity of the piles. Where the piles have an unsupported length extending above ground surface, then the allowable pile capacity should be reduced accordingly.

It is recommended that the pile shaft in the fill above bedrock, or to a maximum depth of 1.5 metres, be coated with heavy grease to reduce uplift forces in the active layer. Care should be taken to ensure the portions of the pile within the concrete pile cap or within the bedrock socket are not coated with grease.

Once the piles have been installed, fill can be placed to achieve the required finished grade. It should be noted that on some of the proposed sites, the piles will have to extend through a significant depth of fill. The piles may have to be braced in order to ensure the pile tops will remain in the required location as fill is placed around them. It may be very difficult to maintain the tops of the piles in the correct position and consideration should be given to installing a suitable steel or concrete pile cap at the top of these piles, to transfer loads from the steel columns to the pile cap in the event that the top of the pile is not directly below the column.

3.3 Footings on Bedrock

This type of a foundation system could be considered for Sites 1, 2 and 4.

Structural loads may be carried on concrete spread footings or piers placed directly on the bedrock surface. An allowable bearing pressure of 4800 kPa may be used for the design of footings placed on intact rock. Where the rock is weathered, an allowable bearing value of 1000 kPa should be used. Footings should be at least 0.5 m wide.

If the rock surface slopes at more than 10 horizontal to 1 vertical, the rock should be chipped to form a level surface prior to placing the footings. Alternatively, steel dowels can be grouted into the rock to prevent the footing from slipping on the sloping bedrock. The number of steel dowels and their diameters will depend on the shear forces calculated to act along the sloping surface of the bedrock. Steel dowels should be installed in those cases where the bedrock surface is flatter than 10 horizontal to 1 vertical, in order to provide uplift resistance against frost heaving forces. All dowels should be installed to a minimum depth of 0.5 m below the surface of the bedrock. Greater penetration may be required in order to achieve uplift capacity due to design loads. A bond value between the rock and the grout, of 0.033 times the 28 day strength of the grout should be used for the design of the dowels for uplift. The computed bond value should not exceed 1400 kPa.

The bedrock surface should be hosed and broomed clean prior to pouring the footings so that the concrete will bond to the rock. The surface of the rock should be inspected for signs of weathering or for the presence of major fissures or faults. If major defects are encountered, the bearing capacity at that location should be reviewed by a qualified geotechnical engineer.

Footing columns should be greased to reduce uplift forces due to frost heaving in the backfill soils. The joint between the footing column and the base must be designed to resist uplift due to frost heave.

3.4 Shallow Footings on Compacted Gravel Fill

This type of foundation system could be considered for Sites 1, 2 and 4. At these sites, compacted gravel fill would be placed on bedrock to provide a level surface and a common elevation for all footings.

This foundation system is considered less desireable with respect to geotechnical considerations than rock-socketed piles or footings on bedrock, because the footings on gravel fill may initially be subject to settlement due to settlement of the fill, and to seasonal heave and settlement due to frost effects. In addition, it is essential that all fill placed below the footings be thoroughly compacted. Past experience in other communities in the Arctic has demonstrated that adequate fill compaction is not always achieved, for a variety of reasons. Unless assurance can be obtained that the fill will be compacted, footings placed on fill should not be used.

Following removal of any exisiting overburden, including vegetation and organic soil from the bedrock surface, the bedrock surface should be carefully examined to determine whether significant low areas are present in the bedrock topography in the areas where footings will be placed. Such low areas in the bedrock surface could trap water, possibly creating frost heaving problems for footings. Should such low areas in the bedrock topography be identified, we recommend that they be filled with clean, compacted gravel backfill.

As mentioned earlier, it is essential that the full depth of fill below spread footings be thoroughly compacted in order to minimize settlement and to prevent tipping of the footings under structural loads. It would be highly desirable to compact all the fill as it is placed, in order to provide a satisfactory foundation for the proposed concrete slab on grade which may eventually be poured inside the arena.

The fill should be placed and compacted in lifts which do not exceed 150 mm in compacted thickness. The fill should be compacted to a uniform density of not less than 100 percent Standard Proctor Maximum Density. This density can usually be achieved by 4 to 6 passes of a vibrating compactor. Granular fill can be most effectively compacted with a vibratory compactor and we recommend that such equipment be used for this work.

The granular material used for fill should meet the following gradation:

Sieve Size	% Passing by Weight
75 mm	100
38 mm	80 - 100
20 mm	60 - 80
10 mm	45 - 65
No. 40	10 - 30
No. 200	0 - 2

The fill should not contain ice, snow, frozen lumps, or organic material. Fill should not be placed when air temperatures are less than about 0°C.

Fill sideslopes should be trimmed to 3 horizontal to 1 vertical in order to prevent surface sloughing and reduce erosion.

A sufficient width should be provided between the building and the top of the fill to provide access for construction and maintenance equipment around all sides of the building.

For footings placed on compacted granular fill, we recommend that strip footings be designed for the perimeter of the structure. The use of continuous reinforced concrete strip footings would tend to reduce differential movements between column loads.

We recommend that a minimum 1.0 m cover, measured to the bottom of the footing be provided, and that the minimum footing width be 1.0 m, to provide uplift resistance.

A maximum allowable bearing pressure of 250 kPa for total dead and normal live loads may be used for footings placed on compacted gravel fill. This value may be increased by one-third for transient loads such as wind loading.

3.5 Adfreeze Piles

Adfreeze piles could be considered for foundations at Site 5. Bedrock exposures do not occur at this site, and the depth to bedrock is not known.

With adfreeze piles, structural loads are carried on open ended steel pipe piles frozen into permafrost. A sketch showing a typical pile installation is presented in Appendix A, Drawing A4. The adfreeze pipe piles should be installed before any fill is placed on site and before the active layer begins to thaw in the spring. Difficulties will be encountered in installing the piles if loose fill is on the site or if the active layer thaws, because the pile holes will slough.

The vertical load carrying capacity of adfreeze pipe piles can be calculated from the following formula:

$$P = \pi DLf$$

where:

P denotes the allowable vertical capacity of the pile (kN).

D denotes the outside diameter of the pile (m).

L denotes the depth of embedment of the pile below the active layer (m). For design calculations, an active layer depth of 2 metres should be assumed.

f denotes the allowable adfreeze bond acting on the pile shaft below the permafrost table (kPa).

The following values for adfreeze bond (f) are recommended for the design of the piles at all sites:

Depth Below Existing Grade (m)	Maximum Allowable Adfreeze Strength (kPa)	
	Maximum Live & Dead Loads	Long Term Dead Loads
0 to 2.0	0	0
Below 2.0	50	30
Unfrozen fill	0	0

Lower values for long term dead loads in the preceeding table are recommended to minimize creep of the piles under continuous loading. The pile length under both design conditions should be calculated and the maximum length of pile calculated should be used. It should be noted, however, that it is not practical to install pile lengths which are greater than 7 metres, with the equipment available to local contractors.

The structural capacity of the piles may govern the maximum allowable loads. The upper portions of the piles should be designed as a column subjected to maximum design vertical and horizontal loads. In computing lateral resistance, the pile can be assumed to be fixed at a depth of 2.0 m below existing grade. The soil above a depth of 2.0 m and any overlying fill should be assumed to be loose.

The pile tips should be placed at least 6 m below the existing ground surface in order to provide adequate resistance against uplift due to frost heave.

Steel piles may be installed at any time, however April or May are optimum months for installation because ground temperatures are low and piles freeze back faster than in summer. If piles are installed during the warmer months, difficulties may be encountered due to surface runoff and sloughing of the active layer. Temporary casing through the active layer may be required under these conditions. In order to ensure that the piles are completely refrozen, it is recommended that they be installed at least 30 days prior to the commencement of steel erection for the structure.

The piles should be placed in pre-bored holes, drilled to the required depth. The annulus between the pile and walls of the drill hole must be completely filled with water. The centre of each pile should be filled to ground surface with water or sand slurry to prevent thawing of the permafrost adjacent to the pile.

It is recommended that the pile shaft, from finished grade to a depth of 1.5 metres, be coated with heavy grease to reduce uplift forces in the active layer. Care should be taken to ensure that the portion of the pile within the concrete pile cap is not coated with grease.

It is expected that the freeze-back time for piles will range from 7 to 21 days, depending on the pile spacing. It is recommended that the centre-to-centre pile spacing be not less than 1.5 m, in order to ensure adequate heat loss.

The comments given earlier concerning fill placement around rock-socketed piles also apply for adfreeze piles.

3.6 Deep Spread Footings

Alternatively for Site 5, structural loads could be carried on spread footings placed at a depth of at least 1.8 metres below finished grade. A sketch showing a typical design for deep spread footings is presented in Appendix A, Drawing A5. The spread footings can be constructed of concrete or treated timber.

Spread footings founded directly on permafrost or on compacted granular fill placed on permafrost should be proportioned to achieve a maximum bearing capacity of 150 kPa for normal live and dead loads. This value may be increased by one-third for transient loads such as wind loads.

Styrofoam insulation at least 75 mm in thickness should be placed above the base of the footing as shown on the drawing. This insulation should extend at least 600 mm out from the edge of the footing.

Frost heaving of spread footings is a concern. It is recommended that the footing column be coated with heavy grease in order to reduce adfreeze forces acting upwards on the column. In addition, it is essential that the joint between the column and the base of the footing be designed to resist tensile forces due to uplift.

Difficulties may be encountered with footing excavation at this site. If footing excavations are carried out in early summer, it will be difficult to excavate through the frozen ground in order to achieve the required depth of excavation. On the other hand, if the excavation is delayed until late summer or early fall, some difficulties are expected due to groundwater flowing into the footing excavations. It is expected that small excavations can be successfully dewatered by pumping from sumps located at the base of the excavation, however, some instability of the excavation side slopes is expected and the sideslopes may have to be cut back to achieve a stable slope.

3.7 Fill Placement

Details concerning fill placement have been given previously in Section 3.4, for shallow footings on compacted granular fill. These comments also apply to fill placed in conjuction with other foundation systems.

To minimize seepage into the clean gravel fill as the ice surface thaws from the granular fill surfaced ice area, we recommend that the final 150 mm lift of fill in the ice arena contain 15 to 20 percent by weight of fines passing a No. 200 seive. In addition, consideration could be given to placing an impervious synthetic liner below the final lift of fill in the ice arena.

3.8 Heated Lounge

At Site 5, it is recommended that an air space of at least 0.5 metres be provided below the heated portion of the structure, in order to prevent degradation of the permafrost below this area. Degradation of the permafrost could lead to settlement of slabs on grade and loss of vertical carrying capacity of foundation piles or spread footings bearing on permafrost.

At Sites 1, 2 and 4, where the building foundations would be on bedrock, or on compacted gravel fill on bedrock, it would not be necessary to maintain an airspace below the heated lounge area. At those sites, the floor for the heated lounge area could be designed as a grade supported slab.

3.9 Grade Beams

For the arena portion of the structure, where grade beams may be placed at or below ground surface, provision must be made to prevent frost heaving forces in the active layer from acting upwards on the bottom of the grade beams. Grade beams may not be required if continuous strip footings are used for shallow footings on compacted gravel fill.

The total amount of upwards heave for grade beams is difficult to predict but is expected on most of the sites to range from 0 to 100 mm, with a probable maximum heave of 150 mm.

The uplift forces can be reduced by placing a treated timber form below the grade beam, such that the timber can move upwards, without contacting the grade beams. Care must be taken with this design to ensure that water and ice do not fill the void created by the timber form. In order to prevent ice building up, and to provide support for the concrete during pouring, a compressible filler, such as Styrofoam or Plastispan, can be placed below the grade beam. With this method, the grade beam must be designed to be capable of resisting an upwards pressure equal to the crushing strength of the compressible filler material.

One suggested configuration for a suitable void form below the grade beam is presented in Appendix A, Drawing A6.

The potential for frost heaving can be reduced significantly by using non-frost susceptible material for fill and backfill whenever possible. Non-frost susceptible soils consist of sand or gravel which contains no silt or clay size particles.

3.10 Concrete

Although ample sources of granular materials occur in the Coral Harbour area, a source for concrete aggregate for use in construction of the arena has not as yet been established. It would be desirable to locate a suitable source and prepare several trial batches of concrete which should be tested in order to establish a suitable mix design for construction.

The fill material which will be placed adjacent to concrete footings or grade beams should be tested to ensure it does not contain water soluable sulphates or other materials which may cause deterioration of the concrete.

3.11 Site Drainage

The ground surface adjacent to the structure should be graded so as to ensure runoff water does not pond adjacent to the structure.

Surface gradients of at least 1 percent are desirable in order to ensure efficient runoff. Gradients should not exceed 2 percent, wherever possible, in order to prevent surface erosion.

3.12 Geotechnical Review and Inspection

It is recommended that once the design of the structure has proceeded to a more advanced stage, that the foundation design be reviewed by a qualified geotechnical engineer. This is particularly important for this structure, because small details in the design of the foundations for the building, if overlooked, could lead to unsatisfactory performance.

It is recommended that the building foundations be inspected during construction by a qualified engineer or technician, in order to ensure that the foundations are installed as specified by the design, as well as to identify unforseen ground conditions which may require modifications to the proposed design. Unforseen ground conditions should be reported to Thurber Consultants so that the design recommendations can be re-evaluated.

It is also recommended that grade preparation prior to fill placement, as well as fill placement and compaction procedures, be inspected and tested by qualified personnel to ensure that specification requirements are met.

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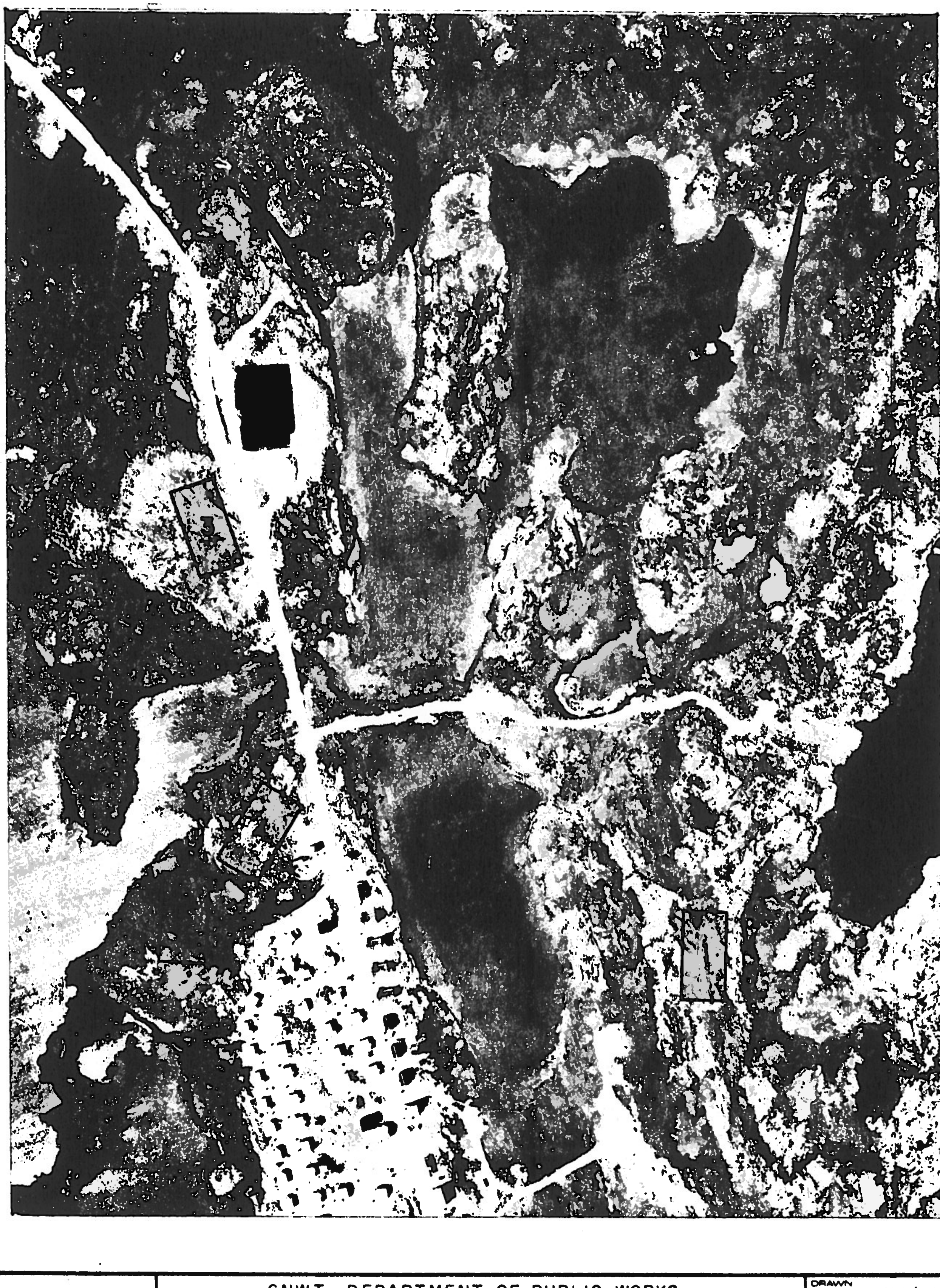
APPENDIX A

DRAWINGS



GNWT - DEPARTMENT OF PUBLIC WORKS

DRAWN	NH / JAB
FILE NO.	15 - 22 - 60
DATE	SEPT. 1985
APPROVED	N. Harris
SCALE	1 : 5000
DRAWING Nº	A1

PROPOSED ARENA SITES

1, 3, 4 AND 5

CORAL HARBOUR ARENA

THURBER CONSULTANTS LTD., Geotechnical Engineers