

INVESTIGATION METHODOLOGIES AND DESIGN FOR RUNWAY PAVEMENT REHABILITATION AT CHURCHILL FALLS AIRPORT

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ABSTRACT

Churchill Falls Airport is located in Churchill Falls, Labrador and is owned and operated by Nalcor Energy Company. The airside pavements at the airport include Runway 13-31, Taxiway Alpha and an apron. There is relatively limited aircraft traffic at the airport. The case study presented in this paper discusses the investigation and design methodologies that were used for the rehabilitation of the runway pavement. It includes the innovative testing tools and designs that can be utilized to investigate and address challenging geotechnical and climatic conditions in remote areas in the North.

The pavement at Churchill Falls airport was significantly distressed including numerous areas of frost heaving and extensive cracking. The field investigation included a detailed pavement distress inspection, test pit investigation and a geophysical survey using Ground Penetrating Radar (GPR). The field investigation program was developed based on an in depth review of findings from previous investigations. Due to the large mobilization costs for a drill rig it was decided to excavate a limited number of test pits using an on-site excavator in order to confirm subsurface soil information. Critical in ensuring the accuracy of the information obtained from the test pit investigation was the use of an experienced pavement specialist to both strategically locate the test pits in the field and prepare soils stratigraphy logs.

It was determined from previous investigation and our own limited geotechnical investigation that the bedrock at the airport site was undulating and very shallow at some locations. The frost heaving of the runway pavement was due to frost susceptible subgrade soils (glacial till) and shallow undulating bedrock trapping groundwater. In order to develop a suitable rehabilitation strategy to the severity of frost heaving it was necessary to obtain a detailed map of the depth to bedrock. This mapping and continuous profile was obtained by carrying out a GPR survey.

The results from the field investigations, in particular the GPR survey were used to develop pavement rehabilitation design alternatives and life cycle cost analysis to identify the most economically feasible alternative. One of the design alternatives developed included installation of polystyrene insulation to minimize frost penetration into the glacial till soils. The pavement rehabilitation was successfully completed in 2012.

INTRODUCTION

Churchill Falls Airport is located in Churchill Falls, Labrador approximately 280 km west of Goose Bay and 250 km east of Labrador City. The Airport is owned and operated by Nalcor Energy Company. Figure 1 shows the locations of Churchill Falls Airport in relation to the two nearest communities.

The airside pavements at the airport include Runway 13-31, Taxiway Alpha and an apron. There is relatively limited aircraft traffic at the airport with approximately three scheduled flights in a week. The aircrafts using the airfield pavements are limited to smaller aircrafts such as a Dehavill, Dash 8 or Beech King Air 250; however, occasionally larger aircrafts are required to use the facility. A limited geotechnical and pavement investigation were carried out to develop pavement rehabilitation alternatives for the pavement on Runway 13-31.

The challenges for runway design and construction in the north may include:

- Severe climatic conditions e.g. severe winters and deep frost penetration;
- Difficult subsurface soils and water conditions e.g. frost susceptible glacial till, undulating bedrock and shallow water table;
- Difficult construction conditions e.g. short construction season, long haul distances and very poor existing road conditions (mainly gravel roads);
- Few quality contractors are available;
- Cost of construction is high as compared to similar construction activities in more populated locations;
- There is limited experience and limited technologies are available to be utilized; and
- It is difficult to provide proper quality control and quality assurance during construction due to the high very high cost of providing the necessary equipment and facilities.

This paper details the investigation program that was developed and undertaken to provide suitable rehabilitation alternatives for the runway pavement at Churchill Falls Airport. It additionally details the challenges that were encountered during the design and construction phases and the steps that were taken to overcome the challenges to ensure a quality product was obtained.

BACKGROUND

The airport airfield was originally constructed to its current configuration in 1969. The initial pavement structure on all three airfield facilities consisted of 90 mm asphalt, 230 mm granular base, and 610 mm subbase with a surface drainage ditch on the north side and across the west end of the Runway. The pavement was first rehabilitated in 1981; the rehabilitation consisted of a 50 mm asphalt overlay on Runway 13-31 and Taxiway Alpha, and the Apron received a 90 mm asphalt overlay. The most recent pavement rehabilitation was completed in 2000. The rehabilitation treatment consisted of a 50 mm asphalt overlay of all airfield pavements, as well as the installation of 250 mm diameter sub-drains located approximately 6.0 m north side of the runway.

The airport site is generally covered by glacial till blanket. Bedrock that underlies the site is comprised predominantly of metamorphic and undivided crystalline orthogneiss.

The pavement profile on the runway has a consistent cross fall draining from north to south. The ditch drainage flows east and west from about the middle of the runway. Drainage flows to intercepting ditch at the west at the end of the runway and subsequently flows to the ditch on the north side of the Trans-Labrador Highway.

A review of the previous pavement condition surveys and geotechnical reports indicate that the existing pavements exhibited distress that included cracking and pavement roughness at the time of the respective remedial treatments. Currently the airport pavements have extensive deep cracks and there is obvious evidence of frost heaving at numerous locations (presence of surface marking, scraping and gouges from winter maintenance equipment).

FIELD INVESTIGATION

A pavement condition inspection was carried out by a senior pavement specialist for Golder Associates Ltd. between June 11 and 14, 2011. It included detailed visual condition evaluation of the airside pavements and advancing six test pits in the shoulders, taking digital photos of typical pavement distresses, and obtaining video recordings of Runway 13-31. A second site visit was carried out in September 2011, during which additional four (4) test pits were excavated through Runway 13-31. In addition to the pavement and geotechnical investigations, a geophysical survey using Ground Penetrating Radar (GPR) was also undertaken to delineate the shape of bedrock below the runway surface.

Pavement Condition Survey

A visual condition inspection of the pavements was carried out to determine the extent and severity of the primary distress manifestations (cracking, deformation, etc.). The specific objectives of the condition survey were:

- To document the present condition of the pavement surface;
- To determine the pavement distresses that require remedial treatment in 2011 in order to keep the airfield pavements serviceable until reconstruction or major rehabilitation is carried out in 2012; and
- Identify distresses and frost heave locations to assist in determining the appropriate strategy for construction/major rehabilitation of the runway.

To carry out the pavement condition inspection, the pavement was divided into branches and divided further into sections. Distresses can usually be categorized as load induced, material and environment induces, which provides some indication of structural integrity and surface operational conditions (i.e. localized roughness or safety concerns). As a minimum the pavement condition should be such as to preclude generation of debris that can cause foreign object damage (FOD) to aircraft.

The primary distresses that were identified on the runway pavement included the following:

- Extensive high severity block cracking;
- High severity longitudinal and transverse cracking;
- High severity frost heaving at numerous locations; and
- Depressions and soft spots in the granular shoulders.

The extent and severities of the distresses were generally similar in the keel and outside keel areas indicating that the majority of the distresses were environment, material and drainage related rather than due to aircraft loading of the pavement. Additionally, it was noted that the crack repairs that were carried out in 2010 were performing very poorly with significant raveling and cracking of the repaired asphalt.

Geotechnical Investigation

The geotechnical investigation consisted of a peer review of existing geotechnical reports, advancing six test pits on the north and south granular shoulders and additional four test pits through the Runway 13-31 pavement. During excavation of the test pits the thickness of each of the pavement layers (asphalt, granular base and subbase) were identified and samples of the granular base, subbase and subgrade materials were taken for laboratory testing. Table 1 shows a summary of the thickness of each of the pavement layers.

Bedrock was encountered beneath the pavement in Test Pits 1, 3 and 4. Test Pit 2 had to be terminated before encountering the bedrock due to localized shearing of the trench walls. For all the test pits in which the bedrock was encountered, pockets of subgrade material was found embedded within the bedrock. Subgrade material identified as silty sand with cobbles and boulders (glacial till) was found beneath the subbase material at Test Pit 1, 2, and 4. In Test Pit 3, the bedrock was encountered directly beneath the granular subbase layer. Water was encountered in Test Pits 3 and 4 and level of the bedrock was determined.

In the test pits advanced in the granular shoulder the soils stratigraphy was noted to consist of a shallow depth of granular material overlying a glacial till material (fill) with very large cobbles and boulders.

Geophysical Survey

The GPR system used consisted of two antennae (transmitter and receiver), a control console and a computer for real-time, graphic display and data recording. The antennae, separated by a fixed distance, were moved along a traverse and readings were taken at discrete intervals. At each step, pulses of radar frequency electromagnetic energy (megahertz range) were transmitted and reflections were received from subsurface horizons. The reflecting horizons occur where there is an abrupt change in the subsurface material dielectric permittivity such as at the interface between host rock and an underground void. The amplitude of received radar energy was recorded as a function of time, processed in real-time for display purposes, and the raw data recorded digitally for later processing and presentation.

The geophysical field work was carried out between June 23 and 24, 2011. Layout and location of the geophysical lines were determined on site. The GPR data was collected using the Smart Tow Noggin system (manufactured by Sensors and Software Ltd.) consisting of a transmitter and receiver pair operated at a 500 MHz frequency.

The survey grid consisted of three 1,680 m long GPR lines oriented parallel to the runway and spaced at distances approximately 22 m apart, and hundred and seventy 45 m long lines oriented perpendicular to the runway with lines nominally spaced 10 m apart. The GPR lines were collected on top of the asphalt runway area. Four supplementary GPR lines were collected on grass to the west and east of the runway.

Interpreted results for the GPR grid are shown on a bedrock depth contour map in Figure 2. The results from interpreting the GPR data indicate the presence of significant variations in the overburden thickness within the runway area with the depth to bedrock ranging from 0.5 to 2.85 meters.

LABORATORY TESTING

Grain size distribution analysis was carried out on select samples of base, subbase and subgrade materials. The grain size distributions for the base and subbase sample were compared to the requirements in the Public Works and Government Services Canada specification ASG-06, “Pavement Construction: Materials and Testing”, dated September 1996 (Public Works and Government Services Canada, 1996). The base and subbase material was generally found to be of good quality meeting the requirements of ASG-06.

The subgrade soil samples were variable with silt and clay contents ranging from 5 percent to 61 percent. Due to the high silt content of the subgrade soil samples, the soils were classified as being highly frost susceptible. The subgrade soils were also noted as having a significant proportion of cobble and boulders of varying sizes.

PAVEMENT DESIGN ANALYSIS

Based on review of previous geotechnical reports and results of the additional investigations, initially four rehabilitation options were developed for Runway 13-31 for the required design life of 15 years. At the request of the client a fifth rehabilitation option was also developed to provide a design life of 10-12 years. Each of the five (5) options below required the installation of a sub-drainage system in the runway. The first four (4) options were designed in accordance with the Public Works and Government Services Canada document entitled “Manual of Pavement Structural Design ASG 19” dated July 1992 (Public Works and Government Services Canada, 1992). The design for the fifth option was based on previous rehabilitation performed on Runway 13-31 which provided a design life of approximately 10 years prior to the next major rehabilitation being required.

Option 1: Reconstruction without Grade Raise – The following pavement design was developed for Option 1 with a total pavement structure thickness of 100 cm and an Equivalent Granular Thickness (EGT) of 110 cm.

- 10 cm of Hot-Mix Asphalt (HMA);
- 30 cm of granular base; and
- 60 cm of granular subbase.

Option 2: Reconstruction with Grade Raise – Due to very serious frost heave issues at the airport, it was recommended to raise the pavement elevation in this alternative by 0.5 m. The grade raise would reduce the proportion of frost susceptible soils that are frozen during the winter months and provide increased protection to the pavement structure against damage due to frost heaving. The following pavement design was developed for Option 2 with a total pavement structure thickness of 100 cm and an EGT of 110 cm

- 10 cm of HMA;
- 30 cm of granular base; and
- 60 cm of granular subbase.

Option 3: Major Pavement Rehabilitation – The following pavement design was developed for Option 3 with a total pavement structure thickness of 85 cm.

- Mill 5 cm of the existing pavement;
- Pulverize the remaining asphalt (15 cm) and mix 50/50 with the underlying granular material so that the total thickness of mixed materials is 30 cm;
- Place 40 cm of new granular base; and
- Place 10 cm of HMA.

As in Option 2 the pavement grade of would be raised by 0.5 m in Option 3 to provide increased protection to the pavement structure against frost related damage.

Option 4: Pavement Insulation and Rehabilitation – The pavement insulation design was carried out by the Golder Alaska Office which has extensive experience with this aspect of pavement design. The total pavement thickness for this design is 115 cm and the EGT is 110 cm. It includes:

- 10 cm of HMA;
- 30 cm of granular base (mixture of pulverized and new base material);
- 15 cm of gravel subbase;
- 15 cm of extruded polystyrene insulation; and
- 45 cm of subbase (pulverized asphalt mixed with granular materials).

This design was estimated to allow frost penetration into the subgrade soil of about 20 to 23 cm or 20 percent of the pavement structure thickness. This is greater than what is typical; however, this should provide a much improved design over existing conditions. The insulation recommended was a high density polystyrene board insulation.

Option 5: Pavement Rehabilitation without Grade Raise – As requested by the Client an additional rehabilitation option was considered to provide a design life between 10-12 years.

This design for this rehabilitation option was based on the previous rehabilitation performed on Runway 13-31 which has provided a design life of approximately 10 years. The pavement design for Option 5 was as follows.

- Mill entire thickness of existing asphalt (approximately 20 cm);
- Place 10 cm of new granular base; and
- Place 10 cm of HMA.

Option 5 should include the installation of a new sub-drainage system as well.

DRAINAGE IMPROVEMENTS

Drainage improvements were recommended to be included in all of the pavement rehabilitation options to ensure that the water drains from beneath the pavement structure. From previous investigations it was identified that the water table could be as close as 0.6 m below the ground surface; water that close to the pavement surface can lead to severe frost heaving. The review of previous documentation identified that at locations along the runway where there was high water table there was increased density and severity of frost related pavement distress.

Initially it was recommended that lateral subdrains be installed at the locations of significant frost heaving; however, this option was later eliminated from consideration due to the potential of differential frost movement. The final drainage improvement recommendations included the installation of subdrains along the north and south edge of pavement for the entire length of the runway as well as deepening the north interceptor ditch to improve its function. There was significant concern that the subdrains would be damaged by the moving cobbles and boulders within the glacial till, therefore it was recommended that the subdrains be installed a minimum of 0.5 m below the top of the bedrock.

LIFE CYCLE COST ANALYSIS

A life cycle cost analysis was performed for each of the five options. The difference between the life cycle cost for Option 3 and Option 5 was not significant; however, it is important to note that the main difference between the two options was the reliability in pavement performance. For Option 5, some deterioration of the pavement would likely occur relatively soon after construction. Conversely, the future performance of Option 3 would be significantly more reliable since the potential for frost damage would be significantly reduced. With Option 5 the expectation was that early and frequent maintenance will be required. Although Option 4 had the highest cost in terms of life cycle cost, this option would provide the most reliable pavement performance under the harsh conditions experienced at this site.

Based on LCCA analysis, Option 3 was recommended for the rehabilitation of the runway pavement; however, the Client selected Option 5 to be the preferred alternative.

PAVING SPECIFICATIONS

Following the pavement rehabilitation design, paving specifications were developed to be utilized during the construction. In order to ensure that the desired pavement design life is achieved for the pavement it was essential that the specifications used for construction were customized to meet local conditions including aircraft loading, desired surface texture, available materials and local experience. Customized specifications were developed for the granular base and asphalt paving. The specifications were modified version of the Transport Canada specifications and the Newfoundland and Labrador Department of Transportation and Works (NLDTW) specifications.

Granular Base

The property of concern for the granular base material on this project was its permeability and therefore the allowed gradation limits for the material were adjusted to reduce the proportion of dust in the material. Additionally, the aggregates for the granular base were required to be tested for its physical properties including, but not limited to, percent crushed material (minimum 90%), MicroDeval (maximum 25) and Los Angeles Abrasion (maximum 35). Prior to use of an aggregate source for granular base material, the source had to be checked for compliance with the specifications.

Asphalt Paving

For the asphalt paving specification the two main items that had to be addressed included obtaining a good quality durable mix and ensuring good construction practices were followed during paving. Taking into consideration the extreme temperatures that can be experienced in Churchill Falls the asphalt cement to be utilized was specified to be a PG 52-40 Polymer Modified Asphalt (PMA). Polymer modification of the asphalt cement was specified to improve the durability and elasticity of the asphalt cement and therefore the asphalt mix. Table 2 shows the aggregate properties that were required to be met for a source to be used for asphalt mixes.

Initially the limit for the LA abrasion was required to be maximum 35; however, after serious concerns were raised from bidding contractors that local aggregates in the area could not meet the LA abrasion requirement and following verification by NLDTW, the maximum allowable was increased to 37.

Table 3 shows the properties requirements that were specified for the asphalt mixes. The minimum allowable asphalt cement content was increased from the road specification used by NLDTW to compensate for the increased oxidation of runway pavement. The required stability for the Marshall mixes was also increased from what is required by NLDTW to provide for increased durability.

Acceptance of the asphalt mixes was to be based on Quality Control (QC) testing carried out by a third party which had to be verified by Quality Assurance (QA) testing carried out by the Owner's representative. Asphalt mix acceptance was based on AC content, gradation, air voids,

asphalt mat field compaction, longitudinal joint compaction and smoothness. Longitudinal joint compaction is typically not specified for most paving project; however, in order to ensure a good quality pavement where the joints will not open, it is essential to ensure adequate compaction is achieved at the joints. Further to achieve this paving in echelon was specified to minimize the number of cold joints and infrared heaters were also required to be used to heat up any cold joints. Finally, a Shuttle Buggy[®] Material Transfer Vehicle (MTV) was also specified to be used to minimize thermal and gradation segregation of the asphalt mix and to achieve better smoothness.

ISSUES DURING CONSTRUCTION

A number of issues and challenges were encountered during construction, the majority of which could be attributed to the remoteness of airport and difficult climatic conditions. Following is a summary of a few of the major challenges that were encountered and how they were overcome.

Following the award of the project the contractor raised concern that the local aggregates would be unable to meet the LA abrasion requirement included in the specification and therefore requested that it be increased or removed. Although some concessions were made and the requirement was slightly increased, the results were strictly reviewed for acceptance. Due to the insistence on the quality of aggregate, the contractor eventually located a local source of aggregate that was well below the abrasion requirement with LA value of approximately 20 percent. There was also a problem with proper supply of the specified PG 52-40 asphalt cement which had to be shipped from a very distant manufacturer.

Another issue that was encountered was localized segregation of the aggregate base due to overworking in order to meet grade of the material with low dust content. The issue was solved by the experienced full time on site inspector and the segregated areas were scarified, reggraded and recompactd.

There was a significant hauling distance and difficult road conditions (gravel roads) from the asphalt plant to the airport which resulted in delays in asphalt delivery and had a somewhat negative impact on asphalt lay down temperatures. In order to minimize the temperature loss due to the long haul distances there were stringent requirements for the haul trucks to be adequately insulated and covered with tarpaulin. Additionally the temperatures of the asphalt arriving on site were routinely checked by the on-site inspector.

Another significant concern that was initially encountered at the start of the project was high air voids and low stabilities for the asphalt mix samples from QC testing. The high air voids and low stabilities were a cause for concern regarding the quality of the asphalt mix being delivered to site. The first step taken was sending the QA samples to be tested by a certified laboratory in southern Ontario to verify the QC results. Following the QA testing an experienced pavement specialist was sent to the QC laboratory to inspect the facilities. It was identified from the QA testing and the inspection that the high air voids and low stabilities were due to problems with the testing equipment rather than the asphalt quality.

Other minor issues were also encountered and solved during construction including issues with the asphalt plant, compaction testing and construction schedules. Of critical importance for the timely resolution of all the issues was good cooperation between the contractor, contract administrator, owner and geotechnical consultant.

CONCLUSIONS

Construction in the north and remote areas poses unique challenges as highlighted by the case study described in this paper. Some of the main difficulties to be overcome include harsh climates, difficult soil and water conditions, lack of availability of materials and technologies, lack of availability of contractors and high construction costs. In order to resolve issues and challenges while still getting a good quality product it is essential to not compromise on performance for immediate cost reductions. Timely resolution of issues is critical due to the short construction seasons and limited resources and this can only be achieved by full cooperation between all parties involved.

REFERENCES

Public Works and Government Services, September 1996. *Pavement Construction: Materials and Testing*, Canadian Standards and Recommended Practices Airport Engineering, ASG-06.

Public Works and Government Services, July 1992. *Manual of Pavement Structural Design*, ASG-19 (AK-68-12).

Table 1: Summary of pavement layer thicknesses from test pit investigation

Test Pit ID	Layer Thickness (mm)		
	Asphalt	Granular Base	Granular Subbase
TP1	170	190	505
TP2	185	245	560
TP3	155	245	490
TP4	195	165	520
Average	176	211	519

Table 2: Aggregate property requirements for asphalt mixes

Physical Test	Test Method	Requirement	
		Surface Course	Base Course
Los Angeles Abrasion - % Maximum (1)	ASTM C131	37	37
Absorption - % Maximum	ASTM C127	1.75	2
Magnesium Sulphate Soundness - 5 Cycles - % Maximum (2)	ASTM C88	12	12
Petrographic Number - Maximum	CSA A23.2-15A	135	135
Freeze-Thaw Test - 5 Cycles - Percent Maximum	CSA A23.2-24A	8	10
Crushed Particles - % Minimum (3)	ASTM D5821	90	90
Flat & Elongated Particles - % Maximum (4)	ASTM D4791	20	20
Loss by Washing - % Maximum Passing (5)	ASTM C117	1.75	1.75
Micro Deval - % Maximum	ASTM D6928	18	18
Clay Lumps - % Maximum	CSA A23.2-3A	1	1
Low Density Particles - % Maximum	CSA A23.2-4A	1	1
Friable or Slaty Siltstone - % Maximum	CSA A23.2-15A	1	1

Table 3 – Asphalt mix physical property requirements

Physical Property	Surface Course		Binder Course	
	Minimum	Maximum	Minimum	Maximum
Asphalt Cement Content Percent	5.3	-	5.0	-
Marshall Stability N. at 60C	10,000	-	9,000	-
Marshall Flow Index (mm)	2.5	4.25	2.5	4.25
Percent Air Voids (1)	2.5	4.5	3.0	5.0
Target Percent Air Voids (1)	3.5		4.0	
Percent Voids in Compacted Mineral Aggregate	15	-	14	-
Modified Lottman AASHTO T283 - Tensile Strength Ratio	0.8	-	0.8	-
Moisture Content of Hot-Mix Asphalt by Oven Method, AASHTO T329 as Percent of HMA	-	0.3	-	0.3

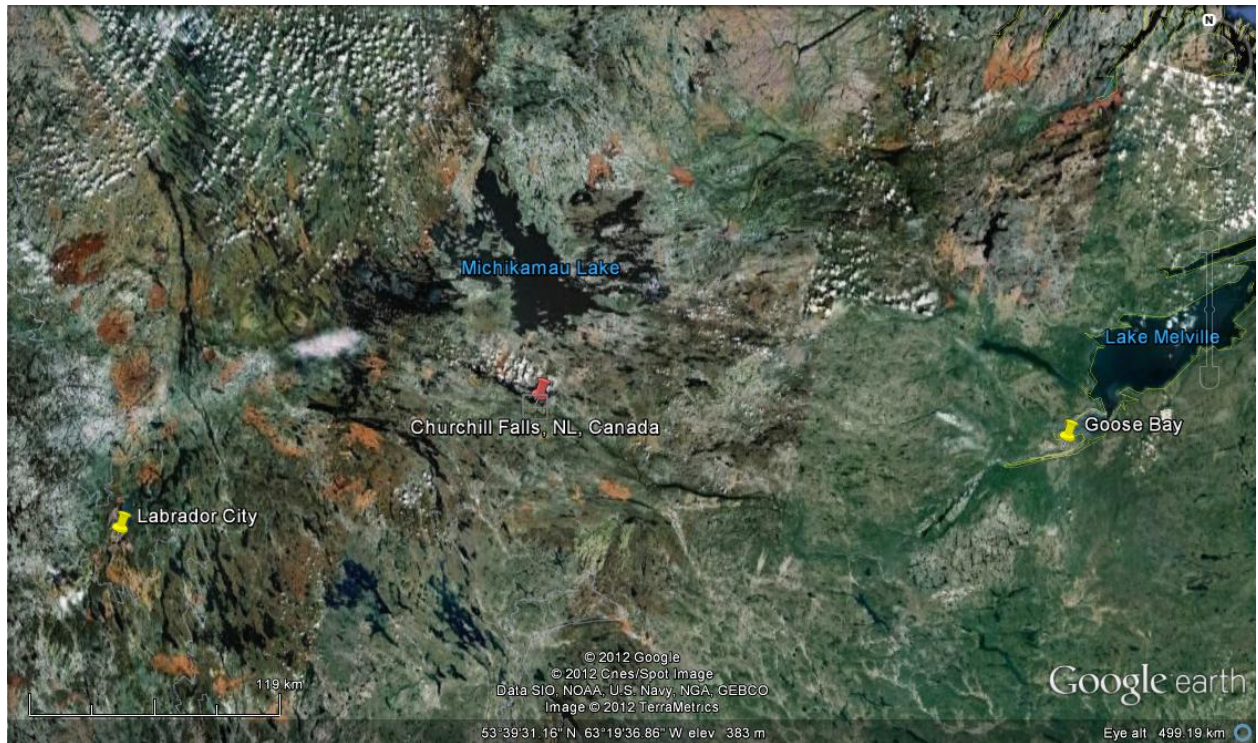


Figure 1: Map showing location of Churchill Falls and two nearest communities

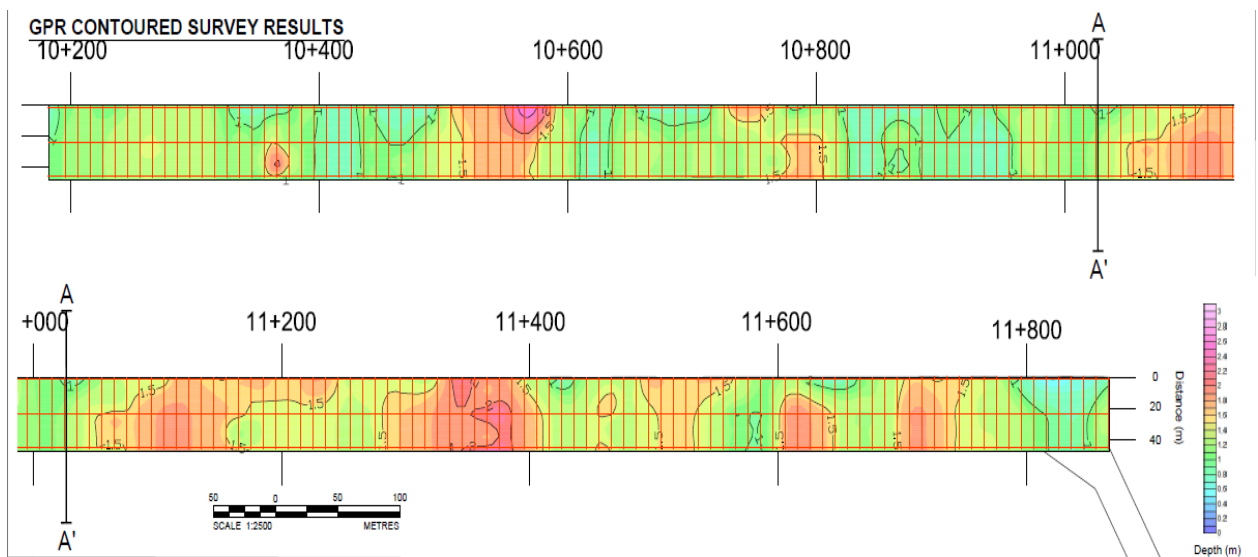


Figure 2: Bedrock contour map produced from GPR survey of runway pavement.