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CONCEPTUAL DESIGN OF ROAD AND BRIDGE SUBSTRUCTURE IN NORTHERN CANADA WITH CONSIDERATIONS FOR CONSTRUCTABILITY AND CLIMATE CHANGE

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Abstract: This paper presents the conceptual design process for a bridge substructure and the road surface of a permafrost-rich section along an undisclosed all-season road in Northern Canada. Vehicular access to and from numerous communities in the North relies on winter roads that traverse bodies of water subjected to deep freeze in winter months. Climate change has however caused a decrease in the reliability of many winter roads, leading to the development of all-season roads. As most all-season roads traverse regions of permafrost, the design process often requires the investigation of special technologies to mitigate permafrost disturbance. This requirement is especially important in light of the increasingly prominent impacts of climate change in the North. In addition, the remote nature of most all-season roads presents the need to consider constructability in the design of both roads and bridge structures. Hairpin thermosyphons and geotextiles were examined as potential solutions for weak and ice-rich subgrades in the road design, and socketed piles were discussed for the bridge substructure design. With the growing need for all-season roads, more papers and case studies are needed to advise practitioners on the unique challenges that are often beyond the scope of typical design guidelines. As such, this paper aims to be part of a bigger investigation that is required to ensure northern transportation networks are able to grow in the face of climate change and meet the needs of future generations.

Keywords: Conceptual Design, Cold Regions, Permafrost, All-Season Road, Bridge

1 INTRODUCTION

Global warming has led to limitations in the movement of people, goods, and natural resources via winter roads in various parts of Northern Canada. Due to the ongoing challenges associated with the maintenance and operation of winter roads, various all-season roads are being developed in the Canadian territories. All-season roads provide year-round access to directly connected communities and increase the days of service for connecting winter roads. The alignment of most roads in Northern Canada inevitably encounters permafrost, an element that poses problems to infrastructure in regions of high latitude and altitude around the world. When permafrost-rich soil is disturbed or thawed, the originally solid ground loses the bearing capacity required by the road or bridge structure above. Existing guidelines in Canada and the U.S. do not provide considerations for the design of Northern roads, and studies dealing with these roads rarely discuss the design procedure and reasoning process used when unusual scenarios are encountered. Steps taken in the design of the road surface and bridge substructure of an existing project are presented in this paper, and justifications are provided throughout for any decisions and assumptions made. General advice is also provided that may be applicable to the design of other Northern roads in similar conditions.

2 SCOPE

Discussions of road design in this paper are limited to the design of surface layers, and discussions of bridge design are limited to the substructure, or foundation. Location of the road in question and field data sources are omitted to maintain project confidentiality.

3 METHODOLOGY

As no gravel road design guidelines are available from the Transportation Association of Canada, road surface design followed the Gravel Road Thickness Design Methods presented in the Gravel Roads Maintenance and Design Manual (Federal Highway Authority, 2000). Surface design results were imported into AutoCAD Civil 3D, and typical sections were output. Substructure design followed the Canadian Highway Bridge Design Code, or CHBDC (CSA, 2014). The Handbook of Steel Construction (CISC, 2017) and the Concrete Design Handbook (CAS, 2016) were consulted. Three-dimensional renderings were generated with Autodesk 3ds Max.

4 ROAD SURFACE DESIGN

4.1 Design Procedure and Assumptions

Like most roads in Northern Canada, the surface of the all-season road in question was to be designed as entirely gravel. The granular material would be obtained from various borrow sources along the predetermined alignment. The road surface design followed the Gravel Road Thickness Design Methods presented in the Gravel Roads Maintenance and Design Manual (Federal Highway Authority, 2000). The design used the suggested roadbed resilient moduli for the roadbed material quality of “fair”, with the winter (frozen) value being 20,000 psi, spring/thaw (saturated) being 2,000 psi, spring/fall (wet) being 4,500 psi, and summer (dry) being 6,500 psi.

Based on geotechnical results from an unnamed contractor, one specific prospect would be the primary borrow source for base material despite the resulting haulage distance. Laboratory testing showed 46% to 71% gravel, 26% to 52% sand, and 1% to 6% silt at this source. Soil with a high percentage of gravel typically takes on an elastic modulus of 25,000 psi. This is also the typical value used by the U.S. Department of Transportation in all examples presented in the Gravel Road Thickness Design Methods (Federal Highway Authority, 2000). The geotechnical study also reported that general fill material for the subbase was available in various areas along the alignment and is primarily made up of sand and silt. Soil of this nature typically takes on a modulus of 10,000 psi.

Traffic loading is typically projected using existing volumes of the road itself when designing the surface for a rehabilitation project. However, traffic loading for new roads cannot be as easily estimated. In the case of this all-season road, it was expected to take on almost all of the traffic from an existing winter road nearby. The two-directional average annual daily traffic (AADT) values of said winter road between 2007 and 2016 were known but fluctuated greatly. The mean AADT over those years was 159, with the median being 110 and the mode being 456. To ensure the design would meet the traffic demand, the highest observed AADT value of 456 was used. An annual growth rate was calculated linearly using the AADT of 107 in 2007 and the AADT of 456 in 2016, with the growth period being 9 years. The growth rate was found to be 17.5%. Since no traffic data was available by vehicle classification for the winter road, conservative estimates were made from available data of a nearby bridge. In 2015, around 32% of traffic on the bridge were small trucks, and around 23% were combination-unit trucks, totaling to 54.7% of heavy vehicles as defined by Federal Highway Administration (FHWA) classes 4 to 13. The heavy vehicle distribution in the design lane was assumed to be 50% (i.e. half in each direction). Using the standard equation by the American Association of State Highway and Transportation Officials (AASHTO), equivalent single axle load (ESAL) was calculated to be 17,381. Using annual growth of 17.5% and design life of 20 years, cumulative ESAL was computed to be 2.39 million.

The serviceability index immediately after the construction (p_0) was taken as 4.2, and the terminal serviceability index (p_t) as 2, resulting in a ΔPSI of 2.2. The rutting criteria was set at the typically assumed value of 2 inches. The allowable loss of surface aggregate was assumed to be 1 inch as the road will be covered in packed snow and ice for at least four months each year, reducing surface deterioration.

Using the Gravel Road Thickness Design Methods (Federal Highway Authority, 2000), the surface thickness of 230 mm was computed. However, since the geotechnical report indicated that quality gravel is scarce in most areas along the alignment, it was deemed worthwhile to convert part of the surface layer thickness to equivalent subbase thickness. Replacing part of the surface layer gravel with inferior material was expected to reduce hauling cost since inferior material was more abundant along the alignment. The resulting thickness of surface gravel course was found to be 130 mm with a subbase of 230 mm, as depicted in Figure 1.

4.2 Mitigation Strategies for Permafrost Disturbance and Weak Subgrades

More than 50% of Canada is underlain with permafrost (Government of Canada, 1995). The alignment of all-season road in question was located on soil of which 50% to 90% contains permafrost. The changing climate has led to increased warming and thawing of permafrost around the world. Not only is the preservation and protection of permafrost important for the integrity of infrastructure, but also for its role in northern ecosystems and hydrological systems.

Disturbed permafrost typically results in thawing and potentially contributes to subgrade saturation, notably in spring and fall. Weak subgrades were found to be present at various locations along the all-season road alignment, often due to the presence of high water tables. Weak subgrade is especially undesirable beneath gravel roads as the unbound surface layers rely entirely on the subgrade for structural support. The Gravel Roads Maintenance and Design Manual (Federal Highway Authority, 2000) recommends the stabilization of weak subgrades with geotextile. This would be done by placing a layer of fabric, or geotextile, over the subgrade soil before any selected fill material is placed, as seen in Figure 1. The tightly woven fabric would prevent wet silts from travelling upward into the selected material while still allowing the top layers to achieve proper drainage. Without the separator, the subgrade material would mix into the new material due to deflection of the road surface under vehicle loading, which would create an adverse effect known as “pumping”. With regular and proper maintenance, the initial higher cost of placing geotextile was expected to be the more cost-effective option in the long term.

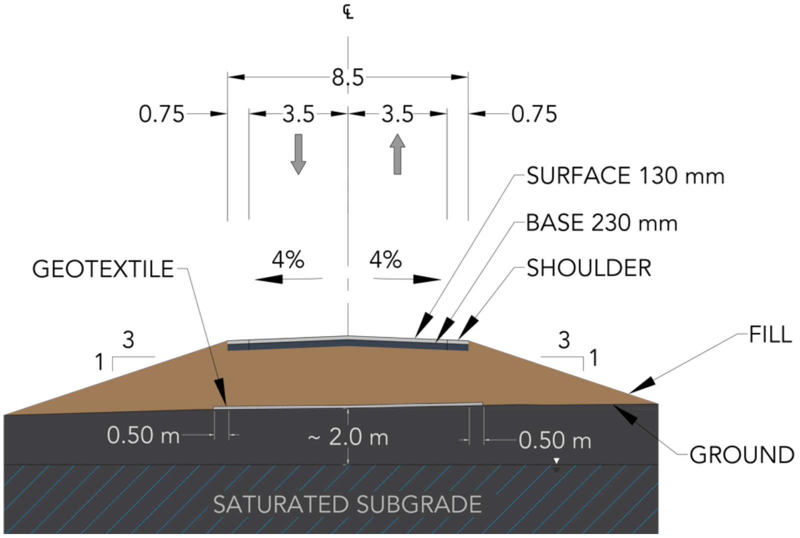

Figure 1: Result of road surface design showing 130 mm of gravel surface and 230 mm of subbase material, fill, along with geotextile for subgrade stabilization.

In 2017, Xu and Goering experimentally validated two permafrost cooling technologies, one of which being hairpin thermosyphons. For this project, the hairpin thermosyphons of interest consisted of three components: evaporator tubes, insulation, and condenser tubes. In the winter, when the road surface becomes colder than the ground, the refrigerant fluid would evaporate in the lower portion of the tube and travel upward as a gas, then it would condense inside the upper, colder pipe and drop back down. As this process happens, it would absorb ground heat from the surrounding earth and permafrost layer, which would in turn supercool the permafrost throughout the winter and allow it to stay frozen through the warmer months (Zhi et al., 2005). For the all-season road in question, the thermosyphons were designed to be offset along the alignment at 2.5 m o/c in a left-right alternating pattern, and the insulation sheets to be 10 mm thick, with dimensions of 2.4 m x 0.6 m, as seen in Figure 2.

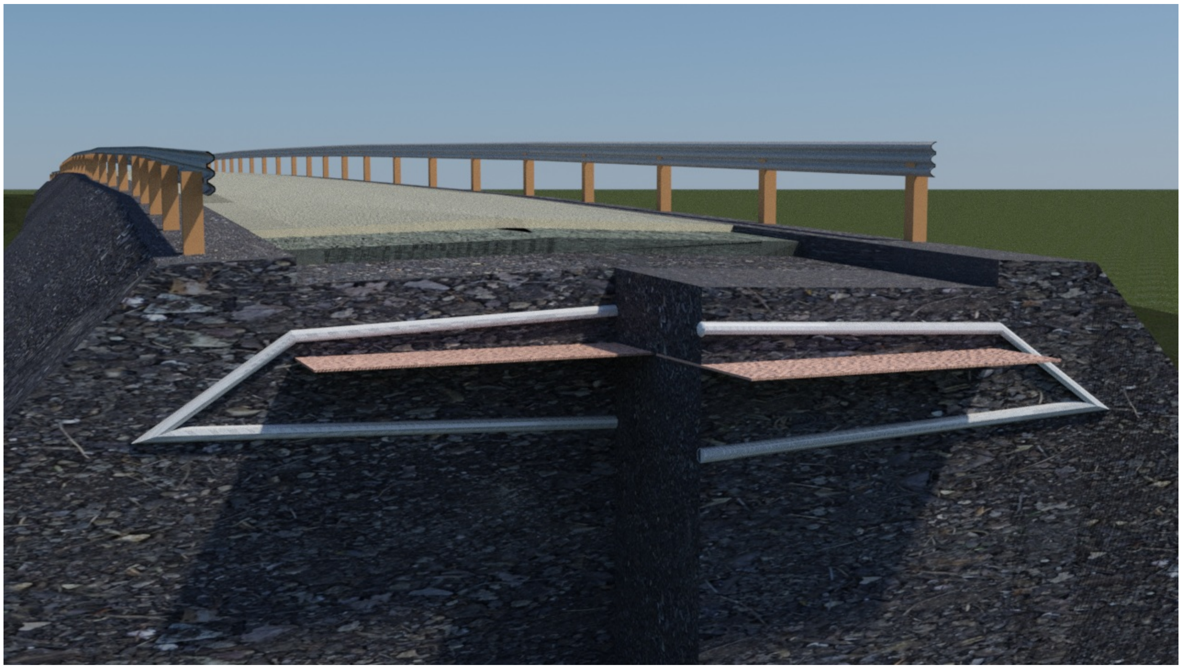

Figure 2: Three-dimension rendering of placement of one set of hairpin thermosyphon system.

5 BRIDGE SUBSTRUCTURE DESIGN

5.1 Design Procedure and Assumptions for Piles

A 100-m bridge crossing was required along the all-season road being designed, with three spans of 30 m, 40 m, and 30 m in length, respectively. As such, two piers are required, along with two abutments.

An unnamed consultant was retained in 2017 to complete a detailed geological investigation of the proposed bridge crossing location. Four borehole samples aligning with the future locations of abutments and pier foundations were analyzed. Thermistor strings were also installed for the collection of in-situ temperature readings. Borehole samples indicated that strong dolostone bedrock runs continuously throughout the site at depths of 7 to 10 meters. A layer of well graded, gravelly sand (GW) overlays this bedrock. Detailed properties of the soil could not be obtained due to the presence of fluvial cobble and boulders. Thermistor string readings recorded above-freezing temperatures in all boreholes below the 5-meter mark, indicating that permafrost and ice-lenses are not present in significant quantities at the foundation sites. The summary of these findings is presented in the table below.

Preliminary design options had been focused around skin friction piles, which would have relied on the strength of the surrounding soil. This was the case because it was initially assumed that bedrock would be prohibitively deep to be relied upon for foundational support. From the findings of continuous shallow bedrock, however, and with the lack of information regarding soil properties, end-bearing piles were selected instead. These piles were assumed to take full advantage of the strength from the rock strata and can be designed neglecting the support from the cohesionless soil layer.

Table 1: Soil properties at bridge crossing.

Borehole	South Abutment	South Pier	North Pier	North Abutment
Ground elevation	262	261.3	261.53	262.1
Layer 1	GW	ML	ML	OL
Layer 2	-	GW	GW	GW
Bedrock depth (m)	255.3	255.1	255.3	254.2
Bedrock strength (MPa)	189	114	92	127
Bedrock type	Dolostone	Dolostone	Dolostone	Dolostone

GW: Gravel ML: Silt OL: Organic Silt

Steel HP piles with temporary casings were selected for the design. The required steel sections would be relatively lightweight and not longer than 12 m due to the shallow bedrock. The presence of large aggregate within the soil layer could disrupt the driving of piles, and the risk of damage or refusal was deemed high. As such, boring was determined to be necessary despite the high cost of mobilization and required expertise. Due to the neutral soil pH and its moderate resistivity, temporary pile casings were deemed sufficient. A detailed corrosion analysis could be merited in some cases.

Strength of individual piles was ascertained through an evaluation of the construction materials and the bedrock strata. As the piles were expected to be continuously supported by the surrounding soil, the compressive strength is estimated at half of the yielding strength. The bond strength was calculated for the area of contact between the steel, grout, and bedrock. Bearing resistance of the bedrock (q_t) was calculated with Equation 1, developed by Rowe and Armitage in 1987, using the unconfined compressive strength (q_u) in MPa given by the geotechnical report.

$$[1] \qquad q_t = 2.5q_u$$

The bond strength between bedrock and grout was found to be the limiting factor in both tension and compression. The calculated results for a typical HP pile can be seen in Table 2.

Table 2. Strength of 360x174 HP Piles.

	Interface	Strength (MPa)	Resistance (kN)
Grout Bond Strength	Bedrock/Grout	0.7	2012
	Grout/Steel	0.55	2108
Compressive Strength	Bedrock (Min. F_{cr})	92	5106
	Steel (Fs)	10	2331

Piles were designed using the moment of inertia method as outlined in the Retaining Wall Report (Ontario Ministry of Transportation, 2012). In this method, total pile axial load is found by adding the axial contributions of lateral forces, vertical forces, and moments. In this case, piles were assumed to have a pinned connection with the pile cap, therefore transmitting zero moment. Resistances are factored by the angle of inclination of the piles, also called the battering slope. The horizontal component of battered piles counteracts lateral loading on the system, including dynamic ice loads. The applied moments were assumed to pivot the base of the pile cap, imparting axial forces which are resisted by the piles. The arrangement of piles based on ultimate limit state (ULS) loadings are presented in Figure 3. The piling system is situated within the abutment base and resists the vertical and horizontal forces imparted to the system. The pile cap rests three metres below grade past the frost layer.

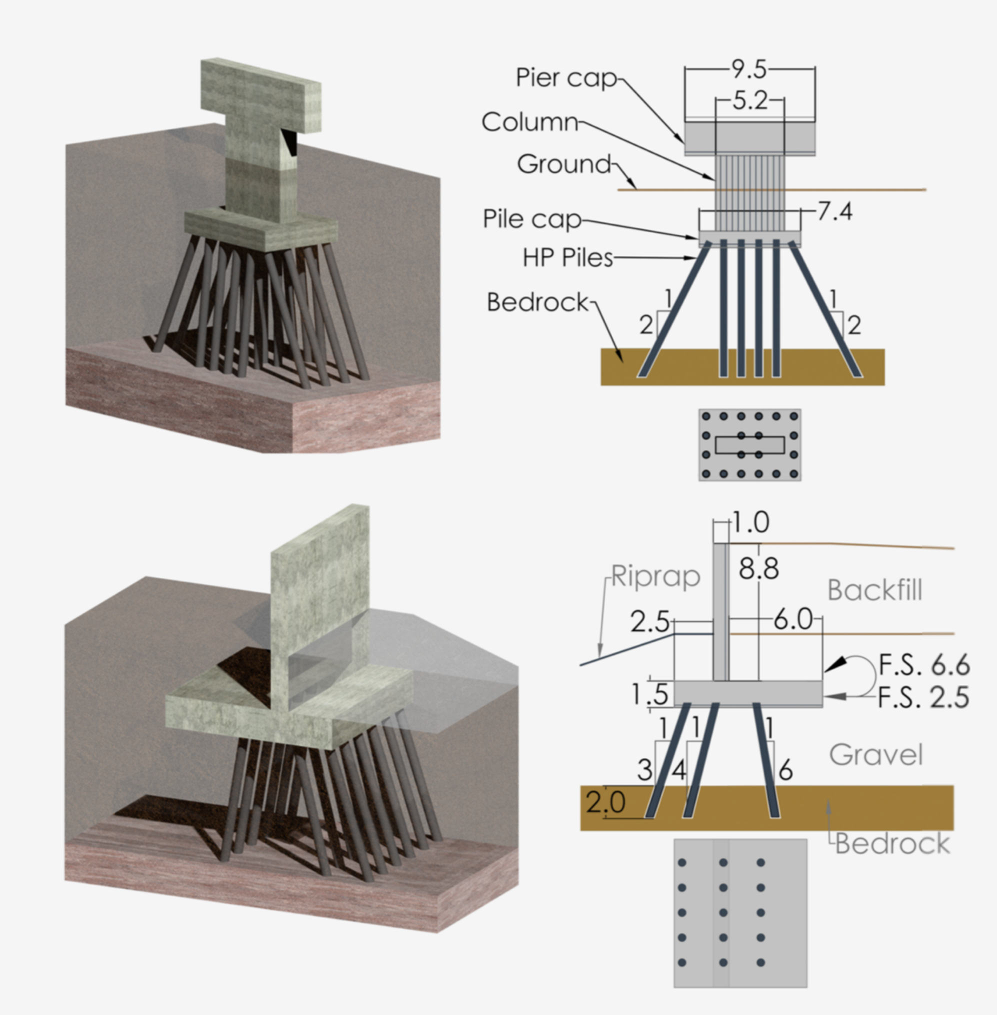

Figure 3: Result of pile design for abutments (left) and piers (right) based on ULS loadings.

5.2 Piers and Abutments

The typical pier and abutment designs can be seen in Figure 3. The piers and pier caps were comprised of rectangular slabs of reinforced concrete. Rebar reinforcement was designed to withstand ULS loading scenarios, omitting forces from earthquake loading. The abutments were dimensioned to resist overturning and sliding and reinforced to the same ULS loads. The retaining wall extends from the soffit of the bridge to the base at 3 meters below grade, preventing frost upheave. 3 meters of riprap rests above the toe of the retaining wall, and 8.3 meters of soil sits above the heel.

6 DISCUSSIONS

6.1 Climate Change

Discontinuous permafrost is present at various locations along the alignment, posing potential problems that could reduce the roadway's resiliency in the face of the changing climate. As such, a design with hairpin thermosyphon was proposed in order to keep any permafrost encountered along the alignment frozen year-round. Geocells could also be used as a light-weight and economical way to stabilize steep embankments and protect against erosion. Both of these solutions serve to increase roadway resiliency.

In areas with high water table, geotextile is proposed to act as a separator and stabilizer between the saturated weak subgrade and fill material.

As foundation piles are supported by bedrock, changing soil conditions have negligible effects on the axial bearing capacity. Additionally, high tensile capacity resulting from socket friction can withstand considerable uplift forces from frost action. Finally, the pile assembly is protected from initial corrosion through its temporary casing. Pile footings are situated deep below grade in order to prevent frost upheave. Dynamic and static ice loads from increasingly variable spring thaw conditions have been accounted for in the horizontal loading of the piles.

One of the common environmental concerns when constructing new roads in Northern Canada is the presence of sensitive habitats. In addition, migration patterns are becoming more unpredictable due to climate change. Constructing a road near or through known habitats can lead to population segregation for the species in question, as well as an increase in vehicle-wildlife collisions. This could be mitigated through crossing zones that present as more favourable crossing locations for the animals. Crossing zones should have low embankments along with warning signs, reduced speed limits, and minimal roadside vegetation to increase visibility of animals approaching the roadway.

Environmental impacts arising from the emissions by cubic meter of construction material were also considered in order to minimize contribution to climate change. The quantity of high-quality material was minimized in the road design to reduce carbon emissions associated with transportation. Material quantity for the bridge substructure was minimized through providing resistance only to the forces that are present. In the design of the substructure, piles are sized based on the individual axial loading, leading to multiple types of piles in the same arrangement. This prevented over-reinforcement, which would lead to wasted materials.

6.2 Constructability

Since the majority of quality gravel is located at the north end of the all-season road corridor, the surface thickness requiring such gravel should be minimized. Using the quality gravel conservatively should reduce hauling cost and ensure that there is enough material for the entire roadway, thus increasing the overall constructability of the design. The original design required 230 mm in thickness of quality gravel as the top course, which would rest directly on fill and subgrade. However, the additional step of converting part of the top course to an equivalent base or sub-base course was taken, and the thickness of quality gravel was reduced by nearly 50%.

The remote nature of the bridge crossing renders the constructability of the bridge structure especially important. The substructure piles are pre-fabricated and are each less than 15 m in length. As such, they can be easily transported to site and minimal assembly would be required, further minimizing the footprint of the laydown area during construction.

7 CONCLUSION

Design procedures for the road surface and bridge substructure of an unnamed all-season road in Northern Canada were presented. Gravel Road Thickness Design Methods were used to calculate the thickness of granular layers for the road surface. Optimization was conducted in order to minimize the amount of quality material needed, which in turn minimized the cost and carbon emissions associated with transportation. Steel piles were selected for their minimal complexity in design, ease of transportation, and fairly cost-effective installation procedures aside from the boring that was deemed to be required. Dynamic ice loads from the river and frost upheave were also both considered in the design of the substructure, although wind and earthquake loadings were neglected. Assumptions were discussed to provide potential application to the design of other roads in similar conditions.

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