Strategic Drainage Reconfiguration to Remediate a Landslide Affecting Saskatchewan's Highway 914

Liam McCann
Geotechnical Engineer-In-Training
AtkinsRéalis
Saskatoon, Saskatchewan
Liam.McCann@AtkinsRéalis.com

Dwayne Rowlett, M.Sc., P.Eng.
Senior Project Manager – Environment
Government of Saskatchewan
Saskatoon, Saskatchewan
Dwayne.rowlett@gov.sk.ca

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Abstract

Saskatchewan's river valleys contain numerous unstable slopes subject to landslides, across which roads, railways, pipelines and other infrastructure are aligned. An erosional failure at a river bend along the Haultain River valley slope was posing a hazard to the highway integrity. Through the Saskatchewan Ministry of Highway's Geohazard Risk Management Program (GRMP), the site was evaluated and assigned a risk level of urgent due to the rate of erosion and potential impact on the highway. Mitigation of the site instability included technical design, construction supervision, and quality assurance and quality control. Finally, recommendations for maintaining optimal operational conditions of the drainage system and the highway were made. This case study demonstrates an example of a strategic drainage system reconfiguration resulting in an effective mitigation that reduced site risk and restored the geotechnical integrity of this segment of Highway 914.

Site Location

The project is located in the Saskatchewan Northern Administration District along Highway 914, at CS 914-03, km 40 (the Site). Highway 914 is an unpaved provincial highway, starting at Highway 165 and ending at the Key Lake mine. This highway is the primary access road to the McArthur River and Key Lake Uranium Mines, in addition to the Village of Pinehouse, which is the only community along the highway. The highway does not intersect with any provincially owned roads, leaving this route without any alternative access to serve the uranium mines. The primary features at the Site are the Haultain River, which flows from the northwest to southeast, and a lake approximately 250.0 m wide, which is located approximately 500 m east of the highway. The Site is located on the east side of the Haultain River valley slope. The Site location can be seen in Figure 1, and the location site plan can be seen in Figure 2.

Figure 1: CS 914-03 km 40 site location

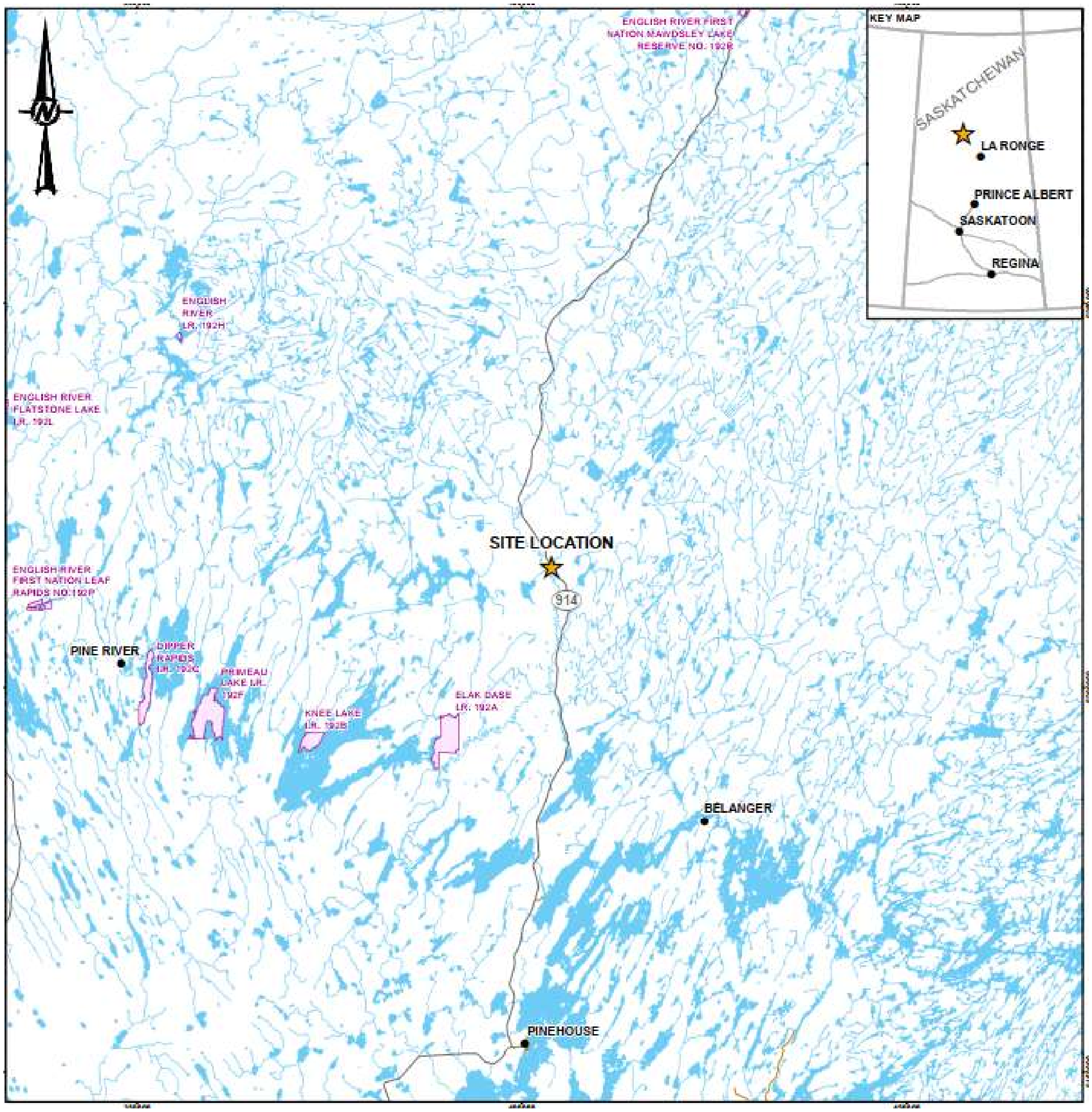

Figure 2: Site Plan - CS 914-03

Introduction to Site and Geohazard Risk Management Program

Highway 914 is owned and operated by the Saskatchewan Minsitry of Highways (the Ministry). The Ministry employs a geohazard risk management framework to evaluate the condition of its geohazard assets, under the Geohazard Risk Management Program (GRMP). The GRMP incorporates a risk-based approach, which considers both capital and efforts in proportion to the level of hazard and potential consequence of failure for its assets (Saskatchewan Ministry of Highways, 2018).

The Site was not previously inspected through the GRMP and had only ever been inspected briefly by local Ministry operations employees before 2019. The first note of concern at the site was in 2017, when the operations staff noted erosion of the river bend occurring. In 2019, the site was inspected for the

Saskatchewan Ministry of Highways. “Foundation Investigation Manual”. (2018)

first time by a geotechnical consulting firm and employees from operations staff (Golder Associates Ltd, 2020).

Site conditions as noted in the 2019 site inspections include:

- 20 September 2019 to 27 September 2019 – riverbank eroded 8 m in a week (over 1 m per day) and the riverbank top of slope was 50.0 m from the highway. This can be seen in Photograph 1.
- 27 September 2019 to 1 October 2019 - riverbank eroded less than 1 m per day and the riverbank top of slope was 46.5 m from the highway, and;
- 1 October 2019 to 15 October 2019 – riverbank eroded 3 m in two weeks (0.2 m per day) and the riverbank top of slope was 43.5 m from the highway. 5.0 m offset stakes were installed on 18 October 2019 along the crest of the riverbank to monitor movement and can be seen in Photograph 2.

Photograph 1: Haultain riverbank - Sept 27, 2019

Photograph 2: Haultain riverbank – Oct 18, 2019

The primary cause of the riverbank failure at the time had been suspected to be related to high groundwater east of the highway, which exists on the silt and sand river embankment slope causing erosional failure and piping within the riverbank. An example of this failure can be seen in Photograph .

Photograph 3: Piping within Haultain riverbank

The Site received a landslide risk assessment rating of 78, with a probability of failure of 13, and a consequence of failure of 6. The Site received an erosion risk assessment rating of 130, with a

Golder Associates Ltd. “CS914-03 KM. Pinehouse Failure – Geotechnical Factual Summary”. Technical Memorandum. Project No. 19131994. (2020)

probability of failure of 13, and a consequence of failure of 10. This can be seen in Figure 3, which displays the erosion risk rating as defined within the Foundation Investigation Manual (Saskatchewan Ministry of Highways, 2018).

Figure 3: GRMP rating's

Table 14 – Probability Factors

PF	Natural Slope	Engineered Slopes
1	Geologically Stable. Very low probability of landslide occurrence.	$F_s > 1.5$ on basis of effective stress analysis with calibrated data and model*. Historically stable. Very low probability of landslide.
3	Inactive, apparently stable slope. Low probability of landslide occurrence or remobilization.	$1.5 > F_s > 1.3$ on basis of effective stress analysis with calibrated data and model. Historically stable. Low probability of landslide.
5	Inactive landslide with moderate probability of remobilization. Moderate uncertainty level; or, active slope with very slow constant rate of movement; or, indeterminate movement pattern.	$1.3 > F_s > 1.2$ on basis of effective stress analysis with calibrated data and model. Minor signs of visible movement. Moderate probability of landslide
7	Inactive landslide with high probability of remobilization, or additional hazards present. Uncertainty level high. Perceptible movement rate with defined zones of movement.	$1.2 > F_s > 1.1$. on basis of effective stress analysis with calibrated data and model. Perceptible signs of movement, or additional hazards present. High probability of landslide.
9	Active landslide with moderate, steady or decreasing rate of movement in defined shear zone.	$F_s < 1.1$ on basis of effective stress analysis with calibrated data and model. Obvious signs of ongoing slow to moderate movement.
11	Active landslide with moderate, increasing rate of movement.	Active landslide with moderate, increasing rate of movement.
13	Active landslide with high rate of movement at steady or increasing rate.	Active landslide with high rate of movement at steady or increasing rate.
15	Active landslide with high rate of movement with additional hazards**.	Active landslide with high rate of movement with additional hazards.
20	Catastrophic landslide is occurring.	Catastrophic landslide is occurring.

Table 15 – Consequence factors

CF	Typical Consequences
1	Shallow cut slopes where slide may spill into ditches or fills where slide does not impact pavement to driver safety, maintenance issue.
2	Moderate fills and cuts, not including bridge approach fill or head slopes, loss of portion of the roadway or slide onto road possible, small volume. Shallow fills where private land, waterbodies or structures may be impacted. Slides affecting use of roadways and safety of motorists, but not requiring closure of the roadway. Potential rock fall hazard sites.
4	Fills and cuts associated with bridges, intersectional treatments, culverts and other structures, high fills, deep cuts, historic rock fall hazard areas. Sites where partial closure of the road or significant detours would be a direct and avoidable result of a slide occurrence.
6	Sites where closure of the road would be a direct and unavoidable result of a slide occurrence.
10	Sites where the safety of public and significant loss of infrastructure facilities (such as a bridge abutment) or privately owned structures will occur if a slide occurs. Sites where rapid mobilization of a large-scale slide is possible.

Table 16 – Response levels and management approach

Risk Level (RL)	Response Level	Management Approach
>125	Urgent	Inspect at least once per year. Monitor instrumentation at least twice per year in the spring and fall. Investigate and evaluate mitigation measures.
>75 to 125	Priority	Inspect once per year. Monitor instrumentation at least once per year.
27.5 to 75	Routine	Inspect every 3 years. Monitor instrumentation at least every 3 years with an increased frequency for selected sites as required.
< 27.5	Inactive	No set instrumentation monitoring or inspection schedule. Monitored and inspected as required in response to maintenance requests

Objectives and Significance

This paper presents a case study on an effective drainage reconfiguration of a geohazard in an isolated location in northern Saskatchewan. The paper provides insight into the history of the site, its geohazard rating throughout time, design options selection, construction, maintenance, and lessons learned. Influence can be drawn from its options selection and construction implementation and can be used a resource for sites with similar geometrics, geotechnical and hydrotechnical features, and site constraints

Mitigation Implementation

<u>Site Investigation</u>

AtkinsRéalis completed a site investigation in the summer of 2020 to characterize the groundwater flow and soil conditions at the Site. Lab results from the tests pits can be found in Table 1.

Table 1: Test Pit Lab Results

Test Pit	Sample No.	Depth (m)	MC (%)	Wash Sieve			Grain Size Hydrometer			
				Gravel (%)	Sand (%)	Fines (%)	Gravel (%)	Sand (%)	Silt Size (%) <75µ>2µ	Clay (%) <2µ
TP 01	SAO-01	0.3	6.0							
TP 01	SAO-02	2.5	9.8							
TP 01	SAO-03	3.3	15.9	0.5	98.9	0.6				
TP 01	SAO-04	4	20.4	0	98.8	1.1				
TP 02	SAO-05	0.2	7.1							
TP 02	SAO-06	0.75	3.8							
TP 02	SAO-07	1.5	2.2							
TP 02	SAO-08	2.5	2.8							
TP 02	SAO-09	5.2	26.7	0	67.7	32.3	0.1	44.7	53.6	1.6
TP 03	SAO-10	0.2	5.2							
TP 03	SAO-11	0.8	2.1							
TP 03	SAO-12	2.6	1.9							
TP 03	SAO-13	4.3	27.8	0	11.7	88.3	0	9.6	86.4	4
TP 04	SAO-14	0.3	7.3							
TP 04	SAO-15	0.6	7.1							
TP 04	SAO-16	1.8	10.3							
TP 04	SAO-17	2.8	17.7							
TP 04	SAO-18	4	20.8							
TP 04	SAO-19	4.6	22.7	0.1	99.1	0.9				
TP 05	SAO-20	0.3	17.7							
TP 05	SAO-21	1.2	16.3							
TP 05	SAO-22	2.3	18.3							

All nine of the standpipe piezometers that were previously installed on site were measured numerous times, and five test pits were dug in order to obtain soil samples for lab testing. Within the standpipe piezometers, groundwater levels varied greatly, from a high of 1.33 mbgs in October 2019 to a low of 8.81 mbgs in September 2021, and can be found summarized in Table 2

Table 2: Summary of groundwater monitoring data:

Piezometer Name	Borehole No.	Ground Elevation (masl)	WL Below Ground Surface (Oct 2019) (m)	WL Below Ground Surface (June 2020) (m)	WL Below Ground Surface (July 2020) (m)	WL Below Ground Surface (September 2021) (m)
SP1	PH-19-01	424.71	1.33	1.57	2.03	-
SP2	PH-19-02	424.70	2.32	4.18	3.55	5.27
SP3	PH-19-03	424.51	3.22	5.34	4.77	-
SP4	PH-19-04	424.84	1.34	3.04	2.29	-
SP5	PH-19-05	424.93	2.22	4.49	3.38	-
SP6	PH-19-06	424.49	4.11	4.63	3.82	6.77
SP7	PH-19-07	425.05	5.01	7.61	6.58	8.81
SP8	PH-19-08	424.43	4.13	5.81	5.19	5.91
SP9	PH-19-09	425.55	1.72	3.37	2.82	-

Ground water levels from the five tests pits conducted can be found in Table 3.

Table 3: Summary of test pits conducted:

SNC-Lavalin Test Pits (July 16, 2020)	Pit Depth (m)	Seepage Level Below Ground (m)
TP 01 – East of Hwy	4.1	3.3
TP 02 – West of Hwy	5.2	-
TP 03 – East of Hwy	4.7	1.4
TP 04 – East of Hwy	4.7	1.8
TP 05 – East of Hwy	3.8	1

Desktop Study

AtkinsRéalis completed significant efforts within the desktop study portion of the project, including geotechnical, hydrological and hydrotechnical, environmental, and historic air photo interpretation. Efforts within the desktop study portion of the work formed the hypothesis for the design options, but will not be discussed at length within this report.

Options Review

During the preliminary mitigation options review, AtkinsRéalis initially considered 6 options for mitigation, as shown below.

- Option 1: Embankment Protection;
- Option 2: Groundwater and Surface Water Interception;
- Option 3: Seepage Cut-off Wall;
- Option 4: Road Realignment;
- Option 5: River Realignment, and;
- Option 6: Do-Nothing (Observational Approach)

Three options, road realignment, river realignment, and do-nothing, were removed from the options considered as they did not meet the objective of the study, which was to provide a geohazard mitigation for the site at CS 914-03, km 40. The follow five options were considered within the predesign portion of the work, and were ranked in an evaluation matrix, as described in Table 4.

Option 1: Embankment Protection with Drainage

- Option 1 consisted of riverbank protection as a mitigation for the slope failure. Constructability is the biggest challenge for this option. The depth of the embankment is approximately 10 m high, and the embankment is weak due to high groundwater.

Option 2a: Groundwater Interception – Deep Option

- Option 2 provides slope protection by lowering the groundwater and resulting seepage face at the river embankment. The stability of the riverbank can be improved by lowering the groundwater level in the slope to increase the effective stress and hence shear resistance. The depressed groundwater level also reduces the hydraulic gradient which will minimize piping and surficial slumping. This is achieved by intercepting the groundwater through a perforated pipe, running parallel and east of Highway 914. Installation of the groundwater interception pipe will require local dewatering.
- Option 2 does not address erosion of the riverbank caused by river flow. The lowered groundwater levels are anticipated to address the erosion caused by seepage. For this reason, the construction requirements in terms of placement will be less critical such that placement of material may accomplished by working from the top of the embankment using a long reach excavator. This would eliminate the need for an access road and working platform within the river; thereby, minimizing the environmental impact, as well as the associated cost implication. Option 2 requires some long-term maintenance and monitoring of the drains performance with additional piezometers to monitor groundwater drawdown.

Option 2b: Speciality Contractor

- Option 2b would consist of speciality contractor providing one-pass trenching for installation of the interception pipe. This method of construction would provide the highest probability of success, lowest risk, and have relatively low environmental impact compared to open cut excavation; however, with the high cost it is not recommended for implementation.

Option 3: Seepage Cutoff Wall

- As in Option 2, Option 3 provides slope failure mitigation by lowering the groundwater at the river embankment face. This is achieved by installing a cut-off wall east of Highway 914. The cut-off wall is designed as a seepage barrier structure to reduce the flow of groundwater to the river embankment. Option 3 does not address erosion of the riverbank caused by river flow, similar to Option 2.
- If further riverbank erosion is observed, through a monitoring program, and is deemed to be the result of river flow, riverbank protection can be installed for this option (as Phase 2) just as discussed above for Option 2. This option also requires some long-term maintenance and monitoring of the performance with additional piezometers to monitor groundwater drawdown.

Option 4: Groundwater Interception – Shallow Option

- Option 4 is a combination of Option 1 and 2 which provides mitigation measures from both erosion against river flow and embankment failure due to seepage of groundwater. A phased approach is proposed for this option. The Phase 1 mitigation includes groundwater interception and surface water management east of Highway 914. Unlike Option 2, the groundwater interception will be shallower to bring down the seepage face at the river embankment to the 100-Year flow event level.

Preliminary Hydrotechnical and Hydrogeological Analysis

Hydrotechnical

To assess the river embankment protection option, a hydraulic model was developed. A steady state HEC-RAS 1-dimensional model was developed using seven surveyed river cross-sections along Haultain River at the Site. The model was used to simulate the water surface elevation and mean velocities at the Site.

A flow frequency analysis of the maximum instantaneous peaks in Haultain River was completed using 46 years of flow data from a nearby Water Survey of Canada gauging station and transposed to the Site using a watershed scaling method. The HEC-RAS model was used to simulate three flood events; the 100 Year (144.5 m^3/s), 10 Year (95.6 m^3/s), and 2 Year (54.6 m^3/s) flow scenarios.

Figure 4 display the simulated water surface profiles of the Haultain River at the Site, while Figure 5 and Table 4 displays the simulation result at the river bend cross-section closest to the highway.

The maximum mean velocity obtained at the river bend from the simulation, was 1.0 m/s, which typically does not warrant protection. However, due to the severe river bend, the fine sand and silty sand soil, and the saturation of the embankment, the river bend is eroding and riprap could be used for mitigation. A nominal thickness of 300 mm thick riprap (D_{50} = 175 mm) was proposed with a 200 mm bedding material to control seepage and minimize piping of the underlying fine silt and sand through the voids.

Figure 4: HEC-RAS simulation plot profile:

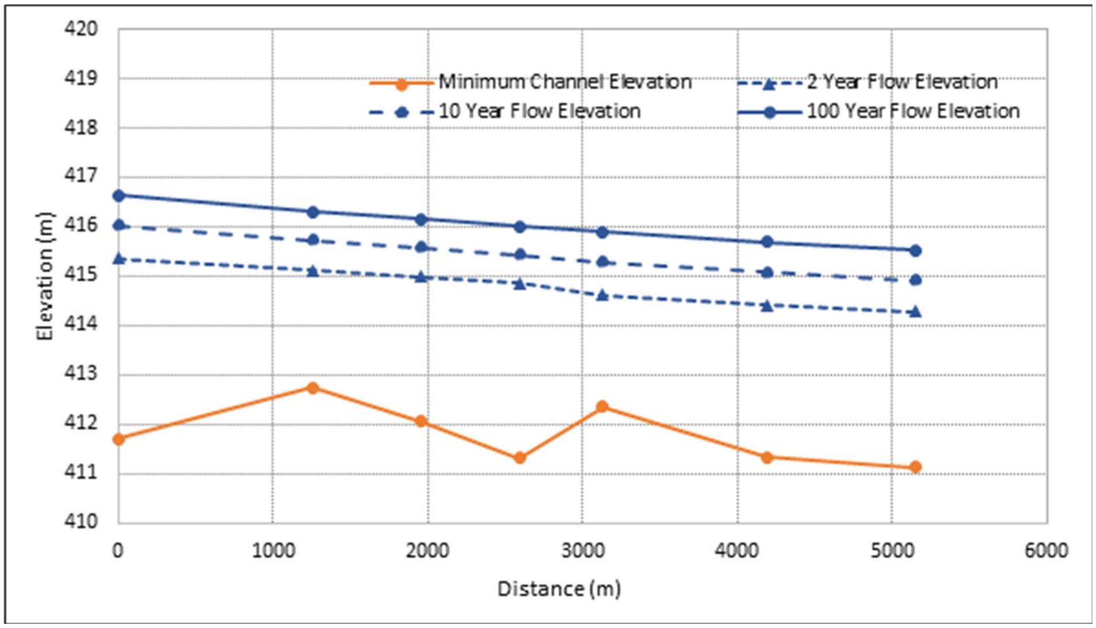

Figure 5: HEC-RAS simulation result at river bend cross-section:

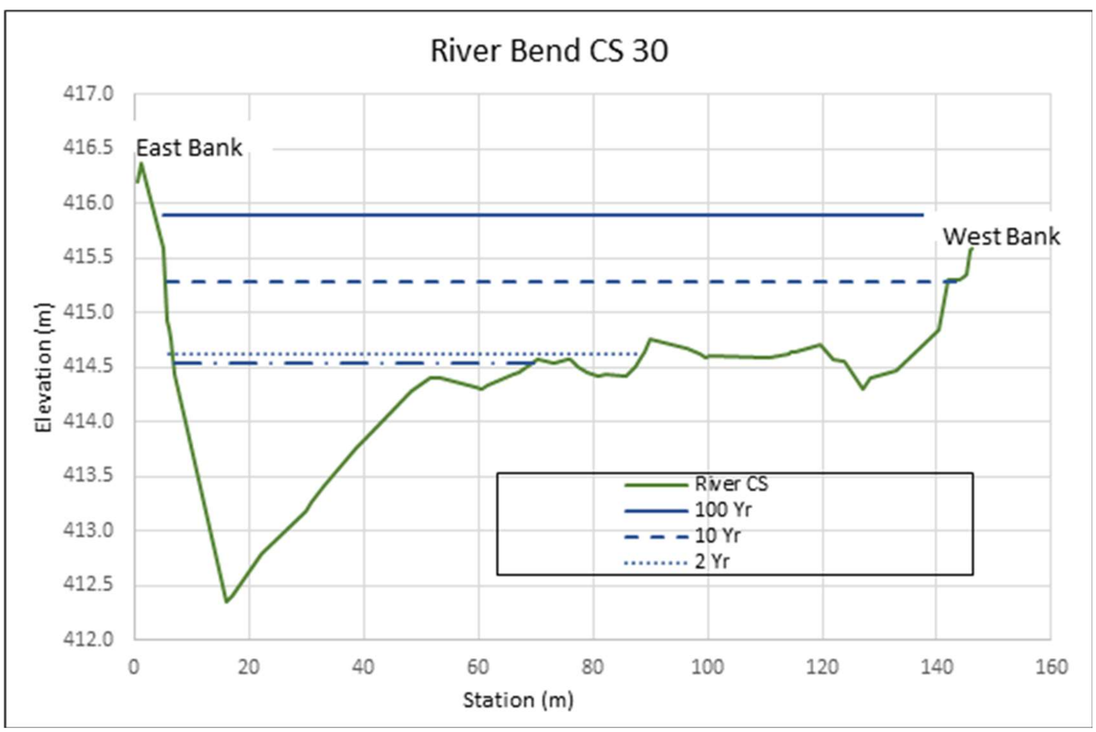

Table 4: Simulation result at river bend

Parameter	2-Year Event	10-Year Event	100-Year Event
Water Surface Elevation (m)	414.62	415.28	415.9
Velocity (m/s)	1.0	0.91	0.89

Hydrogeological

In order to access the groundwater and surface water interception option, which includes interception of groundwater east of the highway to draw down the groundwater at or below the riverbed and determine design parameters, three analytical models were developed. Two of the analytical models represented the site as a cross-sectional prism of sand between the inferred recharge area (the lake and marsh area to the east) and the Haultain River to the west. These two models were based on models presented in Bear (2013) and Kresic (1997) and were used to estimate potential flow rates into the groundwater interception pipe. The third model represented the site as a quasi-three-dimensional unit between the lake/marsh area and Haultain River. This model was based on the Theis model (1935) with superposition of 100 wells to represent the recharge from the lake/marsh area, discharge to Haultain River, and intercepted groundwater into the pipe (see Fitts, 2013). For all models the following input values were assumed based on the background information from Golder, as well as past experience in similar environments:

- A bulk hydraulic conductivity of 5 x 10-5 m/s;
- A uniform sand unit thickness of 12.0 m;
- An aquifer storativity of 0.2;
- A constant recharge groundwater level of 426.0 masl at the lake/marsh area; and,
- An initial groundwater seepage face of 418.0 masl at the Haultain River.

Based on the model results, constructing a groundwater interception pipe in the vicinity of the existing highway, at an invert depth of 419.0 masl at the north end and 417.0 masl at the south end, would decrease the groundwater seepage face at the Haultain River to approximately 414.0 masl (which is approximately at the 2 Year flood event). This means, seepage would exit the river embankment significantly lower than the current state which would increase the stability of the embankment. This means the river bank erosion, caused by groundwater seepage, would most likely not be an issue at the site if this design option was constructed. Typical flow rates to the drain are simulated to be approximately 1.0 m3/d per metre of drain. However, the drain should be sized to accommodate at least 5.0 m3/d per metre of drain to accommodate periods of higher flow. The results of this assessment will be used for the design of the groundwater and surface water interception option.

Kresic, N. "Quantitative Solutions in Hydrogeology and Groundwater Modelling". CRC Press. FL, USA. (1997)

Bear, J. "Hydraulics of Groundwater". Dover books on Engineering. NY, USA. (2013)

Fitts, C. "Groundwater Science". Academic Press. Oxford, UK. (2013)

Option Selection

Each of the potential mitigation options were ranked (1 – low score to 10 – high score) based on the evaluation criteria. The comparison in Table 5 indicates that Option 2b, Groundwater interception with a speciality contractor, was the preferred design alternative; however, this option was eliminated due to very high anticipated costs. Considering this, Option 2a, Groundwater Interception – Deep Option was recommended as it had the second highest score.

Table 5: Design option evaluation matrix

Parameter	Option 1	Option 2a	Option 2b	Option 3	Option 4
Cost	10	6	4	3	7
Performance	10	10	10	6	2
Ease of Construction	1	4	6	4	5
Safety	1	4	6	6	4
Heritage/Environment Impact	1	4	7	6	5
Third Party Impact	5	5	5	5	5
Total Score	28	33	38	30	28

Construction of a 600 m perforated subsurface drainage pipe was selected as the mitigation option to be implemented during the 2021 field season. AtkinsRéalis completed a tender package for the design option, and a contract was awarded to a prime contractor, with a contract value of just over $2,500,000.00.

Construction – Phase II

The high groundwater table throughout the Site required significant dewatering to be undertaken before full depth excavation and pipe installation could be achieved. AtkinsRéalis initially installed 26 dewatering wells along the eastern perimeter of the proposed main excavation alignment. In April 2021, it was determined that the dewatering system implemented would not sufficiently achieve groundwater drawdown to the required elevation. The original dewatering schematic did not perform well due to the presence of more permeable sands than what was designed for, which did not allow for sufficient pumping capacity to achieve required drawdown. The Contractor submitted a formal design proposal to utilize a well point system and the design change requested was approved by the Ministry.

The formal well point system was implemented to facilitate construction. The well point system consisted of 2-inch PVC dewatering wells connected through a main line header pipe to Thompson rotary wet prime pumps. Dewatering well drilling and removal throughout the project was completed in segments as the subsurface drainage pipe installation progressed. Before the wells were installed an initial excavation was completed to the approximate groundwater table elevation in order to maximize the pumping production efforts. Throughout the majority of the subsurface pipe installation, two rows of wells points were installed on either side of the main excavation at an approximate 1.0 m spacing. The dewatering well scheme can be seen in Figure 6.

Figure 6: Multiple dewatering well rows

Dewatering well discharge was disposed of west of the Site as to not interfere with construction, with geotextile filter bags utilized to reduce sediment deposition. AtkinsRéalis completed turbidity testing at all dewatering discharge locations and at various locations throughout the Haultain River. The Haultain River turbidity readings remained consistent throughout the project and dewatering activities did not impact the river. No environment permit exceedances were recorded throughout the project.

The drainage system installed at the Site consisted of a 300.0 mm diameter HDPE SDR 11 perforated HDPE pipe wrapped in non-woven geotextile, which ran 585.0 m in length, and is comprised of forty 8 mm circular perforations per meter. A granular filter was incorporated into the design and was isolated from the in-situ deposits by a non-woven geosynthetic fabric. A manhole access location was constructed to connect the north to south running perforated drainage pipe to the east to west running solid drainage pipe. The perforated subsurface drainage pipe was installed with a grade to flow southeast towards the manhole access location at the south end of the Site. Horizontal drilling was conducted to install the solid subsurface outlet pipe from the manhole location and through the highway to a riprap swale, which diverts water to a natural low-lying location away from the eroding bend along the Haultain River. Installation of the solid subsurface drain pipe can be seen in Figure 7.

Figure 7: Solid subsurface drainage pipe installation

Flush ports were installed for future use in cleaning out sediment that accumulated within the perforated subsurface drainage pipe. A total of six flush ports were installed at increments of 100.0 meters. The HDPE flush ports installed were 100.0 mm in diameter and were cut to an approximate 1.0 m stick up above ground surface.

Backfilling of the excavation was completed using the in-situ sand and silt materials excavated during construction. The east ditch was reconstructed once backfilling and final grades were achieved. As part of the project scope an existing culvert at the Site which had previously been damaged was replaced with a 600.0 mm diameter corrugated steel culvert. The outlet of the culvert was tied into the existing ditch on the west side ditch and riprap was placed to prevent erosion and scour. Hydroseeding of the east ditch and broadcast seeding of the main excavation area was completed in early October 2021.

Maintenance and Monitoring

To ensure integrity, performance, and safety along Highway 914, continued monitoring of the Haultain riverbank is crucial. The monitoring stakes should continue to be used as a reference for erosion progression. If riverbank erosion progresses further, the Ministry may need to consider further mitigation measures.

At the Site, maintenance and monitoring of the designed structure is essential to continued performance. Recommendations for maintenance and monitoring at the Site can be found below.

A visual inspection of the Site should be conducted on a semi-annual basis to evaluate the performance of the subsurface pipe, ditches, drainage system, and Haultain riverbank adjacent to the highway. Items to be included in the inspections:

- Visual inspection down the manhole and measurement from the manhole bottom to the lip of the manhole cover;
- Complete an estimation of the flow rate discharging from the pipe outlet by measuring the time it takes to fill a known volume, and;
- Visual inspection of the Haultain river bend adjacent to the site. Note any erosion of the river bend, referencing 6 offsets stakes that were established in October 2023 at a 10.0 m and 15.0 m offset from the river bank.

Observe the condition of the drainage swale west of the roadway and the culvert inlet and outlet twice per year for indication of erosion on the side slopes along the alignment of the watercourse and the culvert.

- Visual inspection of the flow from downstream pipe outlet discharging into the swale. Note any erosion or lifting of the erosion control matting on the swale side slopes, and;
- Visual Inspection of the overland seepage and drainage system throughout the ditches/culvert. Note the amount of flow through the ditches and culvert.

Download the Vibrating Wire Piezometer data at the Site twice per year.

- Create separate graphs for each piezometer to identify seasonal variance and monitor the effectiveness of the drain. This will allow the Ministry to respond to changing conditions and to provide data to justify if additional mitigations measures are required on the riverbank for further protection.

Performance to Date

Since implementation in 2021, the Site has performed well. The Site was inspected in 2022 and received a landslide risk assessment rating of 70, with a probability of failure of 9, and a consequence of failure of 6. The site received an erosion risk assessment rating of 90, with a probability of failure of 9, and a consequence of failure of 10. The Site now falls into the priority risk level, which recommends for inspections once per year, and can be seen in Figure 8.

Figure 8: 2022 GRMP Rating

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7	Inactive landslide with high probability of remobilization, or additional hazards present. Uncertainty level high. Perceptible movement rate with defined zones of movement.	$1.2 > F_S > 1.1$. on basis of effective stress analysis with calibrated data and model. Perceptible signs of movement, or additional hazards present. High probability of landslide.
9	Active landslide with moderate, steady or decreasing rate of movement in defined shear zone.	$F_S < 1.1$ on basis of effective stress analysis with calibrated data and model. Obvious signs of ongoing slow to moderate movement.
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4	Fills and cuts associated with bridges, intersectional treatments, culverts and other structures, high fills, deep cuts, historic rock fall hazard areas. Sites where partial closure of the road or significant detours would be a direct and avoidable result of a slide occurrence.
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Risk Level (RL)	Response Level	Management Approach
>125	Urgent	Inspect at least once per year. Monitor instrumentation at least twice per year in the spring and fall. Investigate and evaluate mitigation measures.
>75 to 125	Priority	Inspect once per year. Monitor instrumentation at least once per year.
27.5 to 75	Routine	Inspect every 3 years. Monitor instrumentation at least every 3 years with an increased frequency for selected sites as required.
< 27.5	Inactive	No set instrumentation monitoring or inspection schedule. Monitored and inspected as required in response to maintenance requests

Water level readings from piezometers installed on site can be seen in Figure 9 and Figure 10 below, which display the groundwater level, and the water level below depth of casing for piezometers PH-19-01, PH-19-03, PH-19-04, PH-19-05, respectively:

Figure 9: Groundwater Level – CS 914-03 piezometers

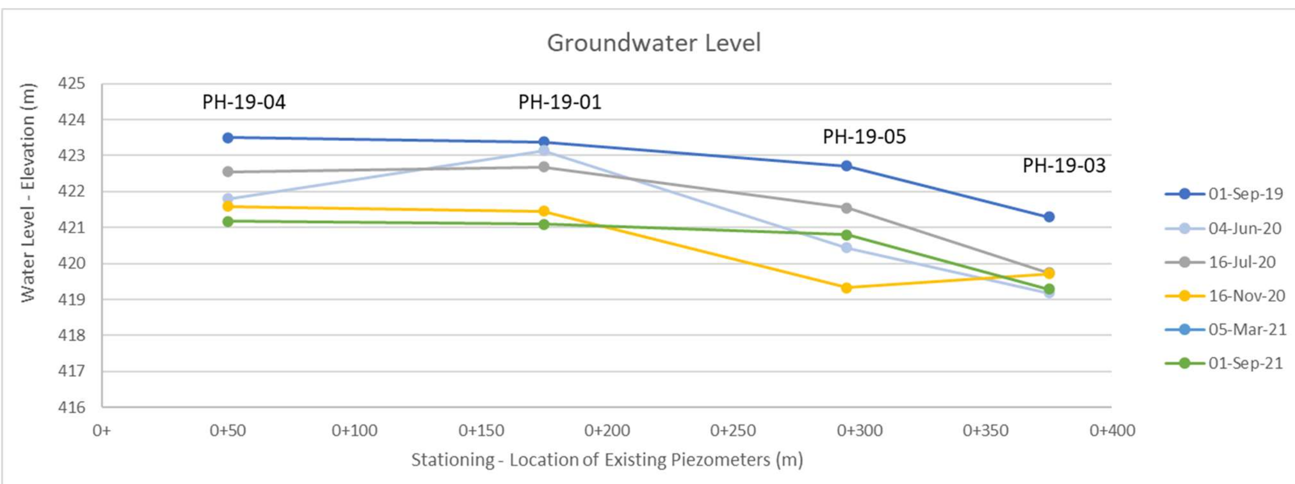
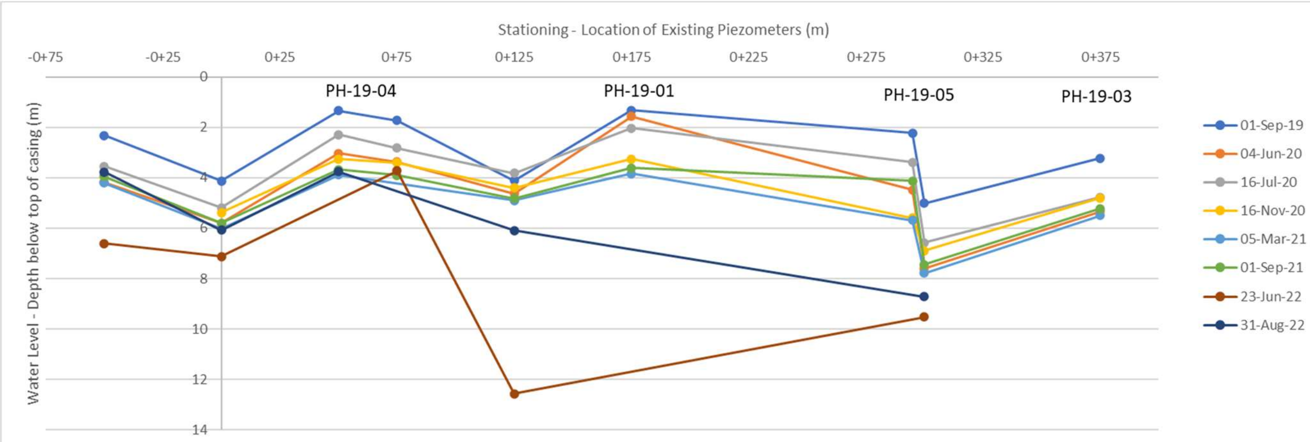


Figure 10: Depth below top of casing - CS 914-03 piezometers



Lessons Learned

Overall, the project had numerous lessons learned which can be used to assist projects of a similar nature in the future. Lessons learned from the investigation phase (Phase I), design and tender phase (Phase 2), and Construction phase (Phase 3) are summarized below.

<u>Investigation phase – Phase I</u>

- Considering the complex groundwater conditions present at the site, it would have been beneficial for a more comprehensive hydrogeological lab testing program to gain a better understanding of the site conditions. Addition K-tests could have been complete which would have increased the confidence in chosen design, and;
- Due to remote setting, complexity of site conditions, and impact of asset, it may have been beneficial to evaluate the design options over numerous iterations to characterize site properties specific to the selected design.

<u>Design/Tender phase – Phase II</u>

- Ensure all possible extraneous items have been considered and covered. Smaller project items neglected cause more headaches in northern remote sites, and;
- Constructability could be considered further during both the design and tender. Structuring the tender process with a better understanding of the possible construction methodologies could negate potential change order items. Complex designs options may require additional investigations, data captures, and surveying to sufficiently plan the design. Once a design has been narrowed in on, the complexities of the design could be explored further to complete the design iteration.

<u>Construction – Phase III</u>

Due to the original dewatering scheme not working as intended, construction costs changed from an approximate $2,500,000.00 at tender award to just over $5,600,000.00 following construction completion. The majority of this increase came from the shift in the dewatering scheme, which cost approximately $2,500,000.00 to complete. As mentioned in lessons learned from Phase I and II, a more detailed site investigation and comprehensive hydrogeological analysis would have increased confidence in site conditions, which may have ultimately changed recommendations. Construction cost overrun may have been minimized if a larger scope was considered during the investigation phase.

In addition to the dewatering scheme adjustment, significant lessons learned arose from the construction phase, which are found summarized below.

Phase I

- Stripping went very smooth but was completed over a long period and some items (wood stacking) were left and then commenced in Phase II, and;
- Expanded the stripping area was a low-cost change, but it could have been foreseen and planned earlier.

Phase II

- Limited accommodations and food options, potential for wildfires, and encounters with wildlife added to planning logistics and supervisor responsibilities. Ensure project manager and site supervisor have planned accordingly for these challenges when working in remote, isolated areas. When possible, provide on-site Wi-Fi or satellite phone to keep lines of communications consistent.

Phase III

- Continual public awareness. Unfamiliar residents concerned with large open area.

Conclusion

This paper presents a case study for a strategic drainage reconfiguration along a northern highway in Saskatchewan.

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