APPENDIX 'A'

BACKGROUND REPORTS

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City of Winnipeg – Planning, Property and Development

Riverbank Stabilization along Lyndale Drive– Birchdale Ave. to Claremont Ave.

Preliminary Design Report

Prepared for:
Dr. Kendall Thiessen
Riverbank Management Engineer
City of Winnipeg - Planning, Property and Development Department
Unit 15 - 30 Fort Street
Winnipeg, Manitoba
R3C 4X5

Project Number: 0015-038-00

Date: November 5, 2020

November 5, 2020 Our File No. 0015-038-00

Dr. Kendall Thiessen
Riverbank Management Engineer
City of Winnipeg - Planning, Property and Development Department
Unit 15 - 30 Fort Street
Winnipeg, Manitoba
R3C 4X5

RE: Riverbank Stabilization along Lyndale Drive – Preliminary Design Report

TREK Geotechnical Inc. is pleased to submit our final Preliminary Design Report for the above noted project.

Please contact the undersigned should you have any questions.

Sincerely,

TREK Geotechnical Inc.
Per:

Michael Van Helden Ph.D., P.Eng.
Senior Geotechnical Engineer

Encl.

Revision History

Revision No.	Author	Issue Date	Description
0	MVH	October 6, 2020	Draft Preliminary Design Report
0	MVH	November 5, 2020	Final Preliminary Design Report

Authorization Signatures

Prepared By:

Ruslan Amarasinghe
Geotechnical Engineering Intern

Michael Van Helden Ph.D., P.Eng.
Senior Geotechnical Engineer

Reviewed By:

ENGINEERS GEOSCIENTISTS MANITOBA
Certificate of Authorization
TREK GEOTECHNICAL INC.
No. 4877 Date:________

Ken Skaftfeld, M.Sc., P.Eng.
Senior Geotechnical Engineer

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1.0 Introduction

This report summarizes the preliminary design of riverbank stabilization and active transportation alternatives developed by TREK Geotechnical Inc. (TREK) and our sub-consultant team along Lyndale Drive from Birchdale Avenue to Claremont Avenue. TREK was retained by the City of Winnipeg Planning, Property and Development Department (the City) to complete a preliminary design, detailed design and contract administration for riverbank stabilization works along this stretch of the Red River. TREK retained HTFC Planning and Design (HTFC) for landscape architecture and active transportation design, and Bison Historical Services Ltd. (Bison) for a Heritage Resource Impact Assessment (HRIA) as part of preliminary design. The current report forms the primary deliverable for the preliminary design phase of the project.

The terms of reference for this work are based on the scope of work identified in TREK's proposal to the City dated August 12, 2020 and the City of Winnipeg Bid-Opportunity 510-2020.

2.0 Background and Existing Information

Project Area

Lyndale Drive Park is located along two meanders of the Red River in the Norwood Flats community where the river provides an aesthetically pleasing green space. An abundance of pedestrians and cyclists regularly frequent the park, and a number of people enjoy other recreational activities such as roller-blading and fishing. Active transportation and recreational use of the park is of paramount importance to the local community. Figures 01 and 02 show an overview of the Lyndale Drive Park with the locations of historical riverbank instabilities and previous stabilization works and current active transportation facilities.

The park is nestled between the river's edge and Lyndale Drive which in this area, serves as both a residential street and a portion of the City's primary dike. A well-established pathway for about 750 m from Highfield Street to Birchdale Avenue providing access along the upper-bank within the western portion of the park, however this pathway is absent between Birchdale Avenue and Ferndale Avenue. A gravel pathway (constructed in 2017) recommences in the upper-bank at Ferndale Avenue and extends east to Claremont where it transitions to a 4.6 m wide asphalt pathway behind a a timber pile retaining wall. The retaining wall terminates 115 m east of Claremont between Monck Avenue and Tache Avenue where the pathway splits into a 2.6 m wide roadside asphalt sidewalk and a 3.0 m wide mid-bank gravel pathway which converge again in the upper-bank area at Gauvin Street.

The project site (Birchdale to Claremont) is located along a transition from an upstream outside bend to a downstream inside bend of the river. A majority of the project site is failure controlled, whereas the downstream ~50 m is considered a transition between failure and erosion controlled banks. The failed slopes generally sit at 5H:1V, however appear to be steeper at about 4H:1V in the reach from approximately Claremont Avenue to Lawndale Avenue. Much of the riverbank is vegetated with a mixture of grasses and willows, and large native tree species in the mid to upper-bank areas.

Site History – Upstream of Project Site

A timber pile retaining wall was constructed from Monck Avenue to Gauvin Street in 1976 to protect the dike and roadway from ongoing riverbank instabilities. In 1978, the wall was partially extended upstream towards Metcalfe Avenue but a failure of a portion of the wall during construction prevented its completion. Riprap erosion protection was also constructed at this time, extending from Monck Avenue upstream to Gauvin Street.

In 2000, a more robust timber pile retaining wall was constructed from Monck Avenue to Claremont Avenue (contract WW-2). The WW-2 wall consists of timber piles driven to refusal in till, soil tie-back anchors and a concrete pile cap. The face of the WW-2 wall was covered in timber stringers and brick facing. Riprap erosion protection was subsequently installed as part of a separate contract (WW-3).

In 2013 and 2017, TREK was retained to design emergency and planned slope stabilization works and active transportation improvements from the WW-2 wall (Monck Avenue) upstream to Gauvin Street. This project resulted in stabilization of the 450 m stretch of riverbank with more than 400 rockfill columns, supplemental riprap below the winter ice level, construction of a new 3.0 m wide mid-bank pathway (Photo 1) and widened upper-bank sidewalk, and the decommissioning of the original (1976) timber pile wall.

Photo 1 - Mid-bank pathway / stabilized riverbank from Monck to Gauvin

Site History – Current Project Site

The current project site is the only remaining stretch of Lyndale Drive that poses an immediate threat to the primary dike, roadway, and utilities due to retrogressive slope instabilities. Also, in its current condition, it forms a gap in the active transportation network. Existing stabilization works including rockfill columns, granular ribs and riprap erosion protection are present in the project site, at the locations shown on Figure 01.

In early 1997, 1.0 m diameter rockfill columns were constructed along 30 m of riverbank between Birchdale Avenue and Lawndale Avenue to address localized (shallow) upper-bank instabilities and deep-seated global instabilities. Following construction, drawdown after the 1997 flood triggered retrogression of movements at this location farther into the roadway. Due to the magnitude of movement, e it is likely that the columns have been sheared off and their contribution to future stability should be neglected.

In 1998, compacted granular ribs were constructed along a 140 stretch of the lower-bank from Birchdale Avenue to midway between Lawndale Avenue and Ferndale Avenue to address ongoing slope movements (contract WW-3). This contract also included riprap erosion protection for the entire reach from WW-3 to WW-2 areas (Birchdale Avenue to Monck Avenue). No major stabilization works have been constructed in the stretch of riverbank between the downstream end of the WW-2 wall (Claremont Avenue) and the upstream end of the granular ribs (upstream of Lawndale Avenue).

Pavement depressions are evident downstream of Lawndale and tension cracks are evident in the pathway as far as 30 m upstream of Lawndale within the extent of the WW-2 granular ribs. The upper-bank in this area is steep (about 2H:1V) and therefore it is possible these movements are localized to the upper-bank area (similar to those observed prior to the 1997 flood).

Discontinuous tension cracks along the pathway between Ferndale Avenue and Claremont Avenue may also be indicative of creep and/or retrogressive bank movements although environmental effects (i.e. shrinkage) cannot be ruled out as a cause of the cracking. Geotechnical instrumentation consisting of three slope inclinometers (lower, mid and upper-bank) and a mid-bank vibrating wire piezometer nest equipped with a datalogger were installed at this location by TREK in 2015 and routinely monitored for two years. The monitoring results did not show any significant slope movements.

The riprap through the entire reach is generally in good condition, however the extent of riprap below the river level cannot be confirmed.

Project Objectives

The main project objective is to provide stability improvements to address movements that may be affecting the roadway and primary dike, and to provide sufficient stability improvement to accommodate a future pathway connection between Claremont Avenue and Lyndale Drive Park. It is understood that any bank grading (cuts or fills) required to accommodate the pathways are to be included in the current design, however final design of the pathways are excluded from the current scope of work. Any supplemental riprap erosion protection will be limited to the areas above the ice scour line (defined as the mean winter river elevation and minus the ice thickness), given that project funding schedules precludes environmental studies and approvals necessary to place riprap any farther into the channel.

Another key design objective is to respect and where possible, enhance the natural environment along the riverbank. The City of Winnipeg's Best Management Practices Handbook for work along Waterways and Watercourses and the Ecologically Significant Natural Lands (ESNL) Strategy and Policy are key resources in the planning of the riverbank works. Riverbank areas are always considered ESNL given that they are extremely important for aesthetics, aquatic and terrestrial habitat, and erosion control.

Existing Information

Pertinent background information reviewed for this project is summarized below (not included in Appendix due to size):

- Wardrop Engineering Inc. (1997) - Riverbank Stability report – slope stability analysis and recommendations for lower-bank and upper-bank shear keys (rockfill columns) near Lawndale Avenue
- UMA Engineering Ltd. (1998) – WW-2 Riverbank Stability Report – slope stability analysis and recommendations for tie-back retaining wall and erosion protection from Claremont Avenue to Tache Avenue
- UMA Engineering Ltd.(1998) – WW-3 Riverbank Stability Report - slope stability analysis and recommendations for compacted granular ribs from Birchdale Avenue to between Lawndale Avenue and Ferndale Avenue, and erosion protection from Birchdale Avenue to Claremont Avenue
- UMA Engineering Ltd. (2000) - WW-2 and WW-3 Riverbank Stabilization and Erosion Protection Record Drawings
- TREK Geotechnical Inc. (2013 to 2015) – test hole logs and instrumentation monitoring at Claremont Avenue

Topographic surfaces for the project site were also obtained from 1998 aerial photography (orthorectified), and from 2011 and 2015 LiDAR surveys.

Project Funding Constraints

We understand that all funding for the planned works are from a Province of Manitoba Disaster Prevention and Climate Resiliency Program grant, which requires that all funds be used by March 31, 2021 (i.e. the funding deadline). Accordingly, several work items cannot be completed before the funding deadline (i.e. in freezing conditions) or cannot be completed without significant additional costs. These work items are:

1. Riverbank restoration works including topsoil and seeding, shrub and tree plantings can only be completed after stabilization works and cannot be completed in freezing conditions.
2. The least costly material for pathway embankment fill is clay, which cannot be placed and adequately compacted in freezing conditions. Clean granular fill could be placed in freezing temperatures but would be more costly than clay fill (perhaps up to three times the unit price).

As a consequence of using granular fill, additional stabilization measures would be required to offset the increased unit weight (as compared with clay fill).

3. Pathway granular base course and asphalt pavements cannot be constructed in freezing conditions.
4. Cellular concrete lightweight fill materials that may be considered to improve stability must be cast and cured on site in non-freezing conditions which may require hoarding and heating at a considerable extra project cost.

Design Constraints

Due to the accelerated timelines, public consultation will not be possible for potential design considerations or implications such as:

- Potential obstructions of the river view created by traffic barriers or pathway guardrails.
- Drop-offs steeper than 3H:1V will require barriers within 2.5 m of the edge of roadway to satisfy requirements for vehicle recovery or collision, as well as pedestrian safety.
- Roadway realignment or changes in use including parking restrictions, on-street pathways or roadway re-alignment or narrowing.

Without the benefit of public consultation, but recognizing concerns raised during the construction of previous stabilization works in the area, design options to be considered for the current study will be limited to pathways at sidewalk level located immediately adjacent to the south curb of Lyndale Drive, with side slopes no steeper than 3H:1V. The pathway corridor will need to include space for existing curb-side streetlights, or include relocation of the streetlights to the south side of the new pathway.

3.0 Site Conditions

3.1 Sub-surface Investigation

A sub-surface investigation was undertaken on September 15th and 16th, 2020 under the supervision of TREK personnel to provide supplemental information on soil stratigraphy and groundwater conditions in key areas. Test holes TH20-01 and TH20-02 were drilled in the mid-bank area between Birchdale Ave. and Ferndale Ave. using a track-mounted geotechnical soils rig using 125 mm diameter solid-stem augers at the locations shown on Figure 01. The test holes were advanced to respective depths of 12.2 m and 10.2 m below ground surface. Vibrating wire (VW) piezometer VW-01A and VW-01B were staggered vertically in test hole TH20-01 and slope inclinometers were installed in each test hole (SI20-01 and SI20-02).

Sub-surface soils observed during drilling were visually classified based on the Unified Soil Classification System (USCS). Samples retrieved during drilling included disturbed auger cuttings. All samples retrieved during drilling were transported to TREK's testing laboratory in Winnipeg, Manitoba. Laboratory testing consisted of moisture contents on all samples and Atterberg limits on select samples. Laboratory testing results are included in Appendix A. Soil stratigraphy encountered in the test holes are overlaid on the nearest topographic cross-section in Figures 03 to 07.

A brief description of the soil stratigraphy and groundwater conditions encountered during drilling is provided in the following sections. All interpretations of soil stratigraphy for the purposes of design should refer to the detailed information provided on the attached test hole logs.

3.1.1 Soil Stratigraphy

The soil stratigraphy consists of 0.4 m to 0.6 m of clay or silt (topsoil) overlying silty clay and silt till. The clay is firm to stiff becoming soft to firm below about 4.5 to 5.5 m, and is of high plasticity. A silt layer 0.2 and 0.7 m thick was encountered between 1.7 to 4.0 m below ground surface and is loose and of low plasticity. Silt (till) was reached 9.2 m to 10.7 m below ground surface. The silt till layer is dry to moist and compact to dense, becoming dense to very dense below 10.0 to 12.2 m where power auger refusal (PAR) was encountered.

The stratigraphy encountered in the test holes, including depth of till, is consistent with previous investigations by TREK, UMA and Wardrop within the project site.

3.1.2 Groundwater and Sloughing Conditions

Seepage and sloughing was observed at 1.5 m below ground surface in the silt layer in TH20-02.

The groundwater observations made during drilling are short-term and should not be considered reflective of (static) groundwater levels at the site which would require monitoring over an extended period to determine. It is important to recognize that groundwater conditions may vary seasonally, annually, or as a result of construction activities.

3.2 Instrumentation Monitoring

Instrumentation monitoring for TH20-01 and 20-02 is pending at the time of report preparation. Existing instrumentation installed in 2013 near Claremont Avenue was monitored on September 7, 2020 with the results included in Appendix B. Negligible horizontal displacement is evident since the last monitoring event in 2015 and piezometric levels are generally consistent with previous readings. SI15-01 could not be located at the time of monitoring.

3.3 Bathymetric and Topographic Survey

A bathymetric and topographic site survey using RTK GPS equipment was performed by GDS Surveys in September 2020. Information collected includes pertinent site features such as roadways, curbs, streetlight poles, manholes, catchbasins, key topographic cross-sections, breaklines such as edge of riprap, river level or tension cracks, as well as trees anticipated to be within the areas of impact due to fill placement.

Cross-sections A through M are located as shown in plan view on Figures 01 and 02, and in section on Figures 03 to 07. The cross-sections show the changes in bank geometry from the various years of survey (1998, 2011, 2015 and 2020).

3.4 Heritage Resource Impact Assessment

A Heritage Resource Impact Assessment (HRIA) has been completed by Bison Historical Services Ltd. in September 2020. The final report for the HRIA is not yet finalized at the time of TREK's draft report submission but will be appended to the final submission as Appendix C (Appendix C is blank for the preliminary design report).

3.5 Forestry Assessment

A forestry impact assessment was performed by HTFC Planning & Design and Chris Lepa of the Forestry Branch in September 2020. The assessment involved a visual inspection of the site, including identification of areas of tree removal and habitat impacts associated with anticipated temporary site access, fill placement and stabilization works. Potential tree removals were flagged in the field and surveyed; these trees are shown in plan view on Figure 01. Due to the preliminary nature of the current assessment, tree and habitat impacts may vary significantly from current estimates, however these estimates will be refined in detailed design. Forestry impacts are further discussed in Section 7.1.

3.6 Riprap Condition Assessment

A visual assessment of existing riprap was performed by TREK between September 3rd and 7th 2020. Although some minor weathering and frost degradation was noted, the riprap is in generally good condition with voids infilled with river sediment. In the upstream (unstabilized) portion of the site, various historical lower and mid-bank instabilities are evident based on aerial photography, topographic contours and shoreline geometry. We suspect that some slope movements have occurred since riprap installation in 2000 as evidenced by a bulging shoreline between Ferndale and Claremont as shown on Figure 02. Although these movements may have disturbed the blanket, there is no evidence that it has been compromised or undercut and therefore no significant long-term impacts to global stability are anticipated.

4.0 Active Transportation Design

The active transportation connection through the project site will incorporate an upper-bank pathway immediately adjacent the south curb of Lyndale Drive as shown on Figures 08 and 09, which will require fill placement is some areas where the bank slopes steeply away from the roadway. In consultation with the City of Winnipeg Public Works Department, the preferred width for a multi-use pathway is 3.5 m, however a minimum width of 2.5 m can be considered within constrained circumstances, which would reduce the impacts to bank stability. Due to the presence of existing streetlights located approximately 0.5 m off the south curb of the roadway, the pathway corridor must include either a 1 m sod strip between the roadway and the pathway, or the relocation of the streetlights to the south side of the pathway such that the pathway can be constructed tight to the roadway curb (which would reduce fill on the bank). In summary, the following pathway options are considered acceptable in terms of user safety and minimum design standards (as shown on Figure 10):

Option A: 2.5 m wide pathway off roadway curb, streetlights relocated to south edge of pathway, 0.5 m rounding off edge of pathway to a 3H:1V slope (overall corridor width 3 m)

Option B: 2.5 m wide pathway offset 1 m from roadway curb, maintain existing streetlights, 0.5 m rounding off edge of pathway to a 3H:1V slope (overall corridor width 4 m)

Option C: 3.5 m wide pathway off roadway curb, streetlights relocated to south edge of pathway, 0.5 m rounding off edge of pathway to a 3H:1V slope (overall corridor width 4 m)

Option D: 3.5 m wide pathway offset 1 m from roadway curb, maintain existing streetlights, 0.5 m rounding off edge of pathway to a 3H:1V slope (overall corridor width 5 m)

Options A through D are in order of increasing negative impact to riverbank stability, with Option A involving the least fill on the bank, and Option D the most. Similarly, those options involving the most fill will also result in the largest forestry and habitat impacts due to disturbance and tree removals required for construction.

Option C is considered the preferred pathway alternative with respect to the desired user experience, involving the desired 3.5 m width, with streetlights relocated beyond the pathway, thereby reducing obstructions to pedestrian and cyclist traffic and reducing roadside vehicular hazards. Options D, A and B are subsequently ranked in terms of decreasing preference for vehicular safety and active transportation design considerations as the pathway width decreases.

5.0 Riverbank Stability Assessment

A riverbank stability assessment was performed to back-analyse incipient failure conditions of the riverbank, to evaluate various slope stabilization options, to recommend a preferred stabilization design. Stabilization options considered include lightweight fill and rockfill columns (equivalent shear keys). Traditional shear keys involving open-cut excavations, and backfill and compaction using traditional excavation equipment were not considered due to the potential for riverbank movements during and following construction that could cause further damage to pavements and the primary dike.

5.1 Numerical Model Description

A slope stability analysis was conducted using a 2-dimensional limit-equilibrium slope stability model (Slope/W) from the GeoStudio 2016 software package (Geo-Slope International Inc.). The slope stability model used the Morgenstern-Price method of slices with a half-sine inter-slice force function to calculate factors of safety (FS) along potential slip surfaces.

Cross-sections E and K are considered critical sections within the previously stabilized (downstream) and unstabilized (upstream) reaches of the site, at the locations shown on Figure 01. These cross-sections are consistent in location to the cross-sections previously analysed by UMA; cross-section K is at the location of cross-section B from the WW-2 analysis, while cross-section E is at the location of cross-section A (critical section) from the WW-3 analysis. Cross-section K is steeper at approximately 4H:1V, in comparison to cross-section E which sits at approximately 5H:1V. The soil stratigraphy assumed in the model is based on UMA's and TREK's test holes.

Groundwater conditions were represented in the model using a static piezometric line based on measured piezometric levels for normal conditions, and a hypothetical higher level representative of a post-flood rapid drawdown scenario for short-term extreme conditions. Short-term extreme groundwater conditions and material properties were established by back-analysis of existing conditions, or in the case of the previously stabilized section, post-1997 flood conditions.

The soil stratigraphy included in the model includes lacustrine clay (residual and intact zones), silt till and engineered materials such as riprap, rockfill and the composite properties of rockfill ribs. The lacustrine clay layer was divided into zones of residual and intact strength, with the residual zone present downslope of the slope crest or active head scarp. In this regard, the head scarp is assumed to extend into the roadway at cross-section E (location of historical movement), whereas the top of slope offset from the roadway was used for cross-section K. The material parameters assumed in the model for each soil unit are summarized in Table 1. The back-analysed groundwater levels and strength parameters at cross-section E and K vary considerably, with lower groundwater levels and higher strengths required for a back-analysed factor of safety of unity at cross-section K. It should be noted that UMA's previous analysis for the WW-2 (upstream) assessment also resulted in a lower groundwater level and higher strength parameters in comparison to the downstream section (cross-section E). The selection of higher strength parameters at cross-section K is considered reasonable due to the lower magnitude of riverbank movements at this location, as well as some limited testing results indicating lower moisture contents and plasticity indices in the upstream reach of the project site. These differences in properties may also result in improved bank drainage and reduced impacts of post-flood drawdown on bank stability.

Table 1 - Soil Properties used in Stability Modeling

Soil Description	Unit Weight (kN/m^3)	Cohesion (kPa)	Friction Angle (degrees)
Lacustrine Clay - Residual			
XS E	18	1.5	11
XS K	18	2	14
Lacustrine Clay - Intact	18	5	17
Silt Till	20	0	40
Riprap	19	0	45
Existing Granular Ribs (1:1.3 rock-clay ratio)	19	1	26
Clay Fill	17	2	20
Rockfill Columns	21	0	50
Rockfill Ribs (upper-bank)	19	1	29

<u>Localized Upper-bank Stability</u>

While it is considered likely that pavement subsidence and cracking at cross-section E are primarily associated with a global (deep seated) slip surface, it is also possible that the over-steepened upper slope may be contributing to the distress due to a localized (smaller) slip surface. As will be discussed in a subsequent section, the upper-bank factor of safety is much lower than unity and lower than the critical global factor of safety in all cases. As such, a separate back-analysis of localized upper-bank slip surfaces was required along cross-section E to achieve a back-analysed factor of safety of unity under short-term extreme groundwater conditions. This separate back-analysis resulted in increased shear strength parameters of c'=3.5 kPa and ϕ'=14°.

5.2 Design Criteria and Groundwater Cases

A target factor of safety of 1.30 was selected for the critical slip surface under short-term extreme groundwater conditions, which translates into a 30% improvement over the back-analysed case. Separate analyses were performed for global and localized upper-bank stability, recognizing the differences in shear strengths outlined previously. A target factor of safety of 1.50 was selected for global slip surfaces extending into the roadway (primary dike) under normal groundwater conditions. Table 2 summarizes the river level and bank groundwater elevations used for each case. As noted previously, a lower-bank groundwater level was necessary at cross-section K for a back-analysed factor of safety of unity. For consistency, the short-term extreme river level selected for design was Elev. 223.7 m, which is lower than the back-analysed river level (Elev. 225.0 m) at cross-section E.

Table 2 – Groundwater Cases and Target Factors of Safety

Groundwater Case	River Elev. (m)	Bank Groundwater Elev. (m)	Target Factor of Safety	Applicable Slip Surfaces
Short-term Extreme XS E XS K	 223.7 223.7	 229.2 227.2	1.30	Critical slip surface
Long-term Normal	223.7	225.0	1.50	Roadway / Primary Dike

5.3 Slope Stability Analysis Results

Preliminary slope stability analyses were performed to compare various stabilization alternatives with the worst-case pathway geometry (Pathway Option D) consisting of a 5 m wide overall corridor and 3H:1V slope. Subsequently, a sensitivity analysis was performed to determine if the preferred slope stabilization alternatives could be reduced as a result of a narrowed pathway corridor and a reduced amount of embankment fill.

Table 3 and Table 4 summarize the calculated factors of safety for all analysis cases associated with cross-sections K and E, respectively. The slope stability analysis results are shown on figures included in Appendix D, as noted in Table 3

5.3.1 Cross-section K (unstabilized)

Back Analysis / Existing Conditions

A back-analysis was performed on existing conditions under a short-term drawdown condition as shown in Figure D-1. The calculated factors of safety between the critical slip surface (mid-bank) and the edge of roadway range from 0.99 to 1.05, indicating that the slope is considered unstable to marginally stable. Under normal groundwater conditions, the factors of safety increase to between 1.07 and 1.15 (Figure D-2).

Fill Placement

Given that the top of slope (historical head scarp) is offset approximately 8 m from the edge of roadway, the 5 m wide pathway corridor required for Option D can be achieved through only minor downslope fill and flattening the head scarp as shown in Figures D-3 and D-4. The factors of safety reduce slightly due to fill placement (-2% to -3%).

Lightweight Fill

Cellular concrete (CEMATRIX) lightweight fill was added beneath the pathway area to reduce the driving force and improve global bank stability. It was assumed that the cellular concrete would be placed in lifts of 0.6 m thick or less, would be covered by 0.5 m of granular fill as ballast (beneath the pathway) and with at least 0.3 m of cover along the slope, and that a temporary excavation slope of 1H:1V would be required off the edge of the roadway (back of curb). With this configuration, approximately 2 m of cellular concrete (half the unit weight of water) is expected to adequately resist uplift (flotation) when fully submerged.

A 2 m thick cellular concrete configuration beneath the pathway fill results in a minor improvement to stability at the roadway with a calculated factor of safety of 1.11 under short-term extreme conditions (Figure D-5). Since most of the lightweight fill is located beyond the historical head scarp, this option has a negligible effect on the critical slip surface stability. Without significantly deeper excavations and tie-down anchors to resist buoyant forces, cellular concrete is not feasible for this section.

Rockfill Columns (Equivalent Shear Key)

As shown in Figures D-7 and D-8, a 2.95 m wide shear key results in a 30% improvement to the critical factor of safety under short-term extreme conditions, however the factor of safety at the roadway under normal conditions is only 1.44 (less than the target 1.50). This shear key width is equivalent to two rows of 2.1 m diameter rockfill columns at 0.6 m clear spacing. As shown in Figures D-9 and D-10, three rows of rockfill columns or an equivalent 3.85 m wide shear key satisfies both the short-term extreme and normal conditions design criteria.

5.3.2 Cross-section E (previously stabilized)

Back Analysis / Existing Conditions

A back-analysis was performed on the current bank geometry but excluding existing compacted granular ribs; note that as shown on Figure 04, the pre-construction lower toe geometry was steeper, and therefore using the current ground surface profile for a back-analysis is considered conservative. The back-analysis results in a critical slip surface extending approximately 2 m into the roadway, which is consistent with observed pavement distress and historical reports, with a factor of safety of 1.00 under short-term extreme conditions (Figure D-11). Under normal conditions, the critical factor of safety is 1.17.

The compacted granular ribs and riprap blanket were then added to the model based on dimensions shown on the record drawings (average rib width of 10 m) to evaluate the stability improvement offered by these existing works. As noted previously, any stability improvement offered by the existing rockfill columns has been neglected in the model. The factors of safety under short-term extreme and normal conditions are 1.26 and 1.45, respectively, with the granular ribs and riprap (Figures D-13 and D-14).

A grouping of localized upper-bank slip surfaces has calculated factors of safety below unity when fully residual parameters are applied. We consider this result to be a model inaccuracy and anticipate that the degree of strain weakening along these localized upper-bank slip surfaces is likely insufficient for fully residual properties to be assumed. A separate back-analysis and stabilization design was performed for these localized upper-bank slip surfaces, as will be described in a subsequent section.

<u>Fill Placement</u>

Significant fill placement is required to construct the pathway embankment, which results in a negative impact to slope stability. The critical slip surface which extends into the roadway has a factor of safety of 1.17 and 1.33 under short-term extreme and normal conditions (Figures D-15 and D-16), respectively, representing a reduction in stability over existing conditions, but an overall improvement over back-analysed (pre-stabilization) conditions. Supplemental stabilization works are required to improve the level of stability at the roadway to satisfy the design criteria.

<u>Lightweight Fill</u>

Similar to the analysis along cross-section K, a 2 m thick cellular concrete configuration beneath the pathway fill results in factors of safety of 1.22 and 1.40 under short-term extreme and normal conditions, respectively, representing approximately a 20% overall improvement to stability when combined with the effect of the existing granular ribs (Figures D-17 and D-18). As such, lightweight fill alone is not considered sufficient.

<u>Global Stability Improvement (Rockfill Columns)</u>

As shown in Figures D-19 and D-20, one row of rockfill columns (2.1 m diameter, 0.6 m clear spacing) or an equivalent 1.25 m wide shear key increases the critical factor of safety to 1.30 and 1.48 under short-term extreme and normal conditions, respectively, which essentially satisfies the design criteria. A single row of rockfill columns is considered the minimum recommended geometry and consequently, combining rockfill columns with lightweight will not result in a cost saving.

<u>Localized Upper-bank Stability Improvement</u>

A separate back-analysis of localized upper-bank stability was performed, with a critical upper-bank factor of safety of 1.01 and 1.43 under short-term extreme and normal conditions, respectively (Figures D-21 and D-22). The addition of the pathway option D embankment fill decreases the factors of safety to 0.92 and 1.39, representing a 3% to 9% reduction in stability (Figures D-23 and D-24). Granular ribs approximately 5 m deep were assessed to provide an improvement to upper-bank stability through mechanical reinforcement (increased strength) and improved drainage. With granular ribs, the factors of safety increase to 1.30 and 1.61, respectively, which satisfy the design criteria.

5.4 Sensitivity to Pathway Corridor Width

As previously discussed, pathway option D is the worst-case geometry considered with an overall pathway corridor width of 5 m. Pathway options B and C have an overall width of 4 m, while option A has the narrowest width at 3 m. A sensitivity analysis was performed to determine the impact of narrowing the corridor (thereby reducing fill loading or increasing offloading) on shear key width. Table 5 and Table 6 summarize the calculated factors of safety along cross-sections K and E, respectively, for the various corridor and shear key widths.

At cross-section K, a 3.85 m wide shear key is required to achieve the target factor of safety of 1.50 under long-term normal conditions; the short-term extreme criteria is exceeded in this case. The reduction in corridor width by 1 m (Options B and C) improves the factor of safety slightly (under 5%) and therefore may result in a small increase in rockfill columns spacing but not the number of rows or a reduced diameter. The reduction in corridor width by 2 m (Option A) results in a shear key width of 2.95 m which allows one row of rockfill to be eliminated.

At cross-section E, only a minor improvement to the global factor of safety (1.48 to 1.55) is realized by narrowing the corridor from 5 m (Option D) to 3 m (Option A) with the minimum width shear key (1.25 m). As such, it is not considered feasible to significantly reduce the degree (and cost) of stabilization works at this location by narrowing the pathway corridor.

Table 3 - Summary of Calculated Factors of Safety – Cross-Section K

Geometry Case	Groundwater Case	River Elevation (m)	Bank Groundwater Elevation	Slip Surface	Description	Factor of Safety	Change (%Change) in Factor of Safety	Figure No. (Appendix D)
Back-Analysis / Existing Conditions	Short-term extreme	223.7	227.2	SS#1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	0.99 1.01 1.05	(Baseline)	D-1
	Long-term Normal	223.7	225.0	SS #1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	1.07 1.01 1.05		D-2
Pathway Option D **5 m wide** pathway corridor	Short-term extreme	223.7	227.2	SS #1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	0.96 1.01 1.03	--0.03 (-3%) no change -0.02 (-2%)	D-3
	Long-term Normal	223.7	225.0	SS #1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	1.04 1.09 1.13	-0.03 (-3%) +0.08 (8%) +0.08 (8%)	D-4
Pathway Option D & Lightweight Fill (Cellular Concrete) 0.5 m granular ballast over 2 m thick LWF	Short-term extreme	223.7	227.2	SS #1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	0.97 - 1.11	-0.02 (-2%) - +0.06 (6%)	D-5
	Long-term Normal	223.7	225.0	SS #1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	1.05 - 1.20	-0.02 (-2%) - +0.15 (14%)	D-6
Pathway Option D & Shear Key **2.95 m wide** equivalent shear key (160 rockfill columns) 3 straight rows, 0.6 m clear spacing, 2.1 m diameter columns	Short-term extreme	223.7	227.2	SS #1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	1.31 1.32 1.33	+0.32 (32%) +0.31 (31%) +0.28 (27%)	D-7
	Long-term Normal	223.7	225.0	SS #1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	1.40 1.41 1.44	+0.33 (31%) +0.40 (40%) +0.39 (37%)	D-8
Pathway Option D & Rockfill Columns Equivalent Shear Key **3.85 m wide** equivalent shear key (210 rockfill columns) 3 straight rows, 0.6 m clear spacing, 2.1 m diameter columns	Short-term extreme	223.7	227.2	SS #1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	1.41 1.41 1.41	+0.42 (42%) +0.40 (40%) +0.36 (34%)	D-9
	Long-term Normal	223.7	225.0	SS #1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	1.51 1.51 1.51	+0.44 (41%) +0.50 (50%) +0.46 (44%)	D-10

Table 4 - Summary of Calculated Factors of Safety – Cross-Section E

Geometry Case	Groundwater Case	River Elevation (m)	Bank Groundwater Elevation	Slip Surface	Description	Factor of Safety	Change (%Change) in Factor of Safety	Figure No. (Appendix D)
DEEP-SEATED GLOBAL STABILITY								
Back-Analysis No existing granular ribs	UMA Back Analysis	225.0	227.2	SS #1	Critical (Observed) / Edge of Roadway	1.00	(Baseline)	D-11
	Long-term Normal	223.7	225.0	SS #1	Critical (Observed) / Edge of Roadway	1.17		D-12
Existing Granular Ribs 10 m wide average width 1:1.3 rockfill:clay ratio	Short-term extreme	223.7	227.2	SS #1	Critical (Observed) / Edge of Roadway	1.26	+0.26 (+26%)	D-13
	Long-term Normal	223.7	225.0	SS #1	Critical (Observed) / Edge of Roadway	1.45	+0.28 (+24%)	D-14
Existing Granular Ribs & Pathway Option D **5 m wide** pathway corridor	Short-term extreme	223.7	227.2	SS #1	Critical (Observed) / Edge of Roadway	1.17	+0.17 (+17%)	D-15
	Long-term Normal	223.7	225.0	SS #1	Critical (Observed) / Edge of Roadway	1.33	+0.16 (+14%)	D-16
Existing Granular Ribs & Pathway Option D & Lightweight Fill (Cematrix) 0.5 m granular ballast over 2 m thick LWF	Short-term extreme	223.7	227.2	SS #1	Critical (Observed) / Edge of Roadway	1.22	+0.22 (+22%)	D-17
	Long-term Normal	223.7	225.0	SS #1	Critical (Observed) / Edge of Roadway	1.40	+0.23 (+20%)	D-18
Existing Granular Ribs & Pathway Option D & Supplemental Shear Key Equivalent Shear Key 1.25 m (60 rockfill columns) 1 straight row, 0.6 m clear spacing, 2.1 m diameter columns	Short-term extreme	223.7	227.2	SS #1	Critical (Observed) / Edge of Roadway	1.30	+0.30 (+30%)	D-19
	Long-term Normal	223.7	225.0	SS #1	Critical (Observed) / Edge of Roadway	1.48	+0.31 (+26%)	D-20
LOCALIZED UPPER-BANK STABILITY								
Back-Analysis / Existing Conditions	Short-term extreme	223.7	227.2	SS #1	Critical (Observed) / Edge of Roadway	1.01	Baseline	D-21
	Long-term Normal	223.7	225.0	SS #1	Critical (Observed) / Edge of Roadway	1.43		D-22
Pathway Option D **5 m wide** pathway corridor	Short-term extreme	223.7	227.2	SS #1	Critical (Observed) / Edge of Roadway	0.92	-0.09 (-9%)	D-23
	Long-term Normal	223.7	225.0	SS #1	Critical (Observed) / Edge of Roadway	1.39	-0.04 (-3%)	D-24
Rockfill Ribs Base elevation 226.3 m Partially drained 1:1 rockfill:clay ratio	Short-term extreme	223.7	227.2	SS #1	Critical (Observed) / Edge of Roadway	1.30	+0.29 (+29%)	D-25
	Long-term Normal	223.7	225.0	SS #1	Critical (Observed) / Edge of Roadway	1.61	+0.18 (+13%)	D-26

Table 5 – Sensitivity Analysis to Pathway Corridor Width – Cross-Section K

Geometry Case	Groundwater Case	River Elevation (m)	Bank Groundwater Elevation	Slip Surface	Description	Factor of Safety	Change (%Change) in Factor of Safety	Figure No. (Appendix D)
Pathway Option D & Shear Key **5 m wide** pathway corridor **3.85 m wide** equivalent shear key	Short-term extreme	223.7	227.2	SS #1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	1.41 1.41 1.41	+0.42 (+42%) +0.40 (+40%) +0.36 (+34%)	D-9
	Long-term Normal	223.7	225.0	SS #1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	1.51 1.51 1.51	+0.44 (+41%) +0.50 (+50%) +0.46 (+44%)	D-10
Pathway Option B or C & Shear Key **4 m wide** pathway corridor **3.85 m wide** equivalent shear key	Short-term extreme	223.7	227.2	SS #1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	1.44 1.45 1.44	+0.45 (+45%) +0.44 (+44%) +0.39 (+37%)	D-27
	Long-term Normal	223.7	225.0	SS #1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	1.47 1.55 1.55	+0.40 (+37%) +0.54 (+53%) +0.50 (+48%)	D-28
Pathway Option A & Shear Key **3 m wide** pathway corridor **3.85 m wide** equivalent shear key	Short-term extreme	223.7	227.2	SS #1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	1.44 1.48 1.47	+0.45 (+45%) +0.47 (+47%) +0.42 (+40%)	D-29
	Long-term Normal	223.7	225.0	SS #1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	1.47 1.58 1.58	+0.40 (+37%) +0.57 (+56%) +0.53 (+50%)	D-30
Pathway Option A & Shear Key **3 m wide** pathway corridor **2.95 m wide** equivalent shear key	Short-term extreme	223.7	227.2	SS #1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	1.37 1.39 1.38	+0.38 (+38%) +0.38 (+38%) +0.33 (+31%)	D-31
	Long-term Normal	223.7	225.0	SS #1 SS#2 SS#3	Critical Edge of pathway Edge of roadway	1.45 1.49 1.48	+0.38 (+36%) +0.48 (+48%) +0.43 (+41%)	D-32

Table 6 - Sensitivity Analysis to Pathway Corridor Width – Cross-Section E

Geometry Case	Groundwater Case	River Elevation (m)	Bank Groundwater Elevation	Slip Surface	Description	Factor of Safety	Change (%Change) in Factor of Safety	Figure No. (Appendix D)
Existing Granular Ribs & & Pathway Option D & Supplemental Shear Key **5 m wide** pathway corridor **1.25 m wide** equivalent shear key	Short-term extreme	223.7	227.2	SS #1	Critical (Observed) / Edge of Roadway	1.30	+0.30 (+30%)	D-19
	Long-term Normal	223.7	225.0	SS #1	Critical (Observed) / Edge of Roadway	1.48	+0.31 (+26%)	D-20
Existing Granular Ribs & & Pathway Option B or C & Supplemental Shear Key **4 m wide** pathway corridor **1.25 m wide** equivalent shear key	Short-term extreme	223.7	227.2	SS #1	Critical (Observed) / Edge of Roadway	1.34	+0.34 (+34%)	D-33
	Long-term Normal	223.7	225.0	SS #1	Critical (Observed) / Edge of Roadway	1.52	+0.35 (+30%)	D-34
Existing Granular Ribs & & Pathway Option A & Supplemental Shear Key **3 m wide** pathway corridor **1.25 m wide** equivalent shear key	Short-term extreme	223.7	227.2	SS #1	Critical (Observed) / Edge of Roadway	1.36	+0.36 (+36%)	D-35
	Long-term Normal	223.7	225.0	SS #1	Critical (Observed) / Edge of Roadway	1.55	+0.38 (+32%)	D-36

5.5 Summary

In summary of the slope stability assessment:

1. Improvements to global slope stability are required along the entire study reach, in areas where existing stabilization works are present and where they are not.
2. Lightweight fill to improve riverbank stability is not considered practical given construction considerations (winter construction) and the potential for flotation (buoyancy at high river levels), which limits the stability improvement that can be achieved.
3. A 2.95 m to 3.85 m wide equivalent shear key using rockfill columns is required along the unstabilized (upstream) reach to achieve a target factor of safety of 1.50 at the roadway under normal conditions, depending on the pathway corridor width. The target factor of safety for short-term extreme conditions is exceeded in this case.
4. A 1.25 m wide equivalent shear key using rockfill columns is required just upslope of existing compacted granular ribs to achieve a target factor of safety of 1.50 and 1.30 at the roadway under long-term normal and short-term extreme groundwater conditions, respectively. The degree of stabilization required is governed by the minimum practical dimensions of the equivalent shear key (i.e. a single row of rockfill columns) and the narrowing of the pathway will not reduce the stabilization works required.
5. Localized upper-bank stability is a concern in the over-steepened area where pavement distress is observed and the extent of pre-sheared (residual) soils may extend beneath areas of planned pathway embankment fill. Stabilization of the upper slope using 5 m deep rockfill ribs prior to construction embankment fills is recommended for adequate long-term performance. Such works are likely not required in other areas where the extent of historical riverbank movements is farther offset from the pathway corridor.

6.0 Hydraulic Assessment

A hydraulic assessment was performed to evaluate the impact of pathway embankment fill on channel hydraulics for the Red River, to confirm minimum riprap sizing requirements, and to evaluate impacts on channel hydraulics associated with potential changes in grade associated with supplemental riprap placement upslope of the existing riprap blanket. The hydraulic assessment report is included in Appendix E. The overall finding of the hydraulic assessment is that the fill necessary for Option D and a minor raising of grades up to 0.3 m upslope of the existing riprap will result in imperceptible changes to river levels at flood stage relative to existing conditions.

7.0 Evaluation of Alternatives

All pathway options are considered acceptable in terms of accessibility, roadside safety, CPTED and aesthetics. Options with a wider 3.5 m pathway (Options C and D) are preferred in terms of user comfort and accessibility due to the increased pathway width that better accommodates various modes of active transportation on a single facility. Options that involve streetlight relocation to the downslope edge of the pathway (Options A and C) are preferred from a roadside safety perspective, since permanent roadside hazards (streetlight poles) are offset farther into the roadside clear zone. Overall, the various pathway options are not significantly differentiated, except for habitat and tree loss, and construction costs for required stabilization works, which are discussed in the following sections.

7.1 Estimated Compensation Value of Tree and Habitat Loss

The stabilization work will require the removal of mature trees and understorey shrubs and forbs on the upper-bank. Option A (2.5 metre multi-use path, no grass boulevard) has the least impact, affecting 53 trees, while Option D (3.5 metre multi-use path with 1 m grass boulevard) has the highest impact at 64 trees. In general, the affected tree stand is very healthy with little evidence of Dutch Elm or other major diseases within the project area. The sizes of trees to be removed in Options A and D are summarized in Table 7 and Table 8, respectively.

The City of Winnipeg has several ways to calculate the value of trees and habitat. According to the City's posted Tree Removal Guidelines, if they are considered part of a 'natural stand', compensation is calculated at $740 per tree between 50 mm and 124 mm dbh (diameter at breast height). One additional replacement tree is required for every additional 75 mm of dbh above 50 mm (*i.e.* removing one 125 mm dbh tree is equivalent to two replacement trees at $740 per tree, or $1480 total). Once the calculations are complete, trees may be adjusted in value using the ISA Species rating (*e.g.* oak trees typically have higher value than Manitoba Maple) but this determination should only be done by Forestry or other qualified tree appraisers. This work is still underway with Forestry.

Table 7 – Estimated Compensation Value for Tree Loss – Pathway Option A

Tree DBH (mm)	No. of Trees Removed	Value of Tree Loss	Subtotal
100 +	21	$740	$15,540
175 +	25	$1,480	$37,000
250 +	2	$2,220	$4,440
325 +	0	$2,960	$ -
400 +	1	$3,700	$3,700
475 +	3	$4,440	$13,320
550 +	1	$5,180	$5,180
Approximate Tree Loss Value			**$79,180**

Table 8 – Estimated Compensation Value for Tree Loss – Pathway Option D

Tree DBH (mm)	No. of Trees Removed	Value of Tree Loss	Subtotal
100 +	21	$740	$16,280
175 +	25	$1,480	$39,960
250 +	2	$2,220	$15,540
325 +	0	$2,960	$ -
400 +	1	$3,700	$25,900
475 +	3	$4,440	$13,320
550 +	1	$5,180	$5,180
Approximate Tree Loss Value			**$116,180**

Based on the un-adjusted values presented in the tables above, the approximate compensation value of trees ranges from approximately $80,000 for Option A to just over $120,000 for Option D.

It is assumed at this time that there will be no additional valuation for habitat lost (for example for the understorey plants) since this stand is somewhat disturbed and isolated, however further follow up with Naturalist Services will be done to confirm.

7.2 Construction Cost Estimates

Class 3 cost estimates for the four preliminary design pathway options are provided in Appendix D. The estimates include allowances for roadway repairs (30 m length) and curb replacement (50 m length), and pathway construction. An allowance of 12% was added to the construction cost estimate for engineering, contract administration and post-construction services. A contingency of 15% was added to the subtotal of construction and engineering costs based on previous projects of this nature.

Overall, the construction cost estimates range from $3.65 M (Option A) to $4.29M (Option C). The primary cost differential is associated with streetlight replacement (estimated $30,000) or relocation (estimated $100,000), and the degree of slope stabilization and embankment works required ($2.43M to $2.90M).

Table 9 – Summary of Class 3 Construction Cost Estimates

Work Component	Option A	Option B	Option C	Option D
Roadworks	$78,950	$82,450	$78,950	$82,450
Pathway Construction	$35,660	$35,660	$47,540	$47,540
Relocation of Street Lighting	$90,000	$30,000	$90,000	$30,000
Riverbank Stabilization Works	$2,431,830	$2,894,250	$2,894,250	$2,898,110
Site Restoration, Landscaping and Architectural Features	$198,500	$218,500	$218,500	$238,500
Total CC (Construction Cost Estimate)	**$2,834,940**	**$3,260,860**	**$3,329,240**	**$3,296,600**
Engineering - DDCA (12% of CC)	**$340,193**	**$391,303**	**$399,509**	**$395,592**
Contingency (City Specified Value - 15%)	**$476,270**	**$547,824**	**$559,312**	**$553,829**
TOTAL CLASS 3 COST ESTIMATE (to nearest $1,000)	**$3,651,000**	**$4,200,000**	**$4,288,000**	**$4,246,000**

7.3 Comparison of Options

Option A (lowest cost) is not preferred in terms of user experience and accessibility, due to the narrowed width. However, Option A poses the least risk to riverbank stability, would require the least amount of stabilization works and has the lowest estimated construction cost.

Option B (2nd lowest cost) less desirable than Option A in terms of user experience and accessibility since the presence of streetlight poles between the pathway and the roadway may reduce the effective width users "feel" when travelling on the facility. The additional cost for Option B relative to Option A is approximately $0.55M primarily due to the increase in rockfill columns required to satisfy stability criteria with increased fill on the bank. These additional costs are therefore not warranted given essentially no improvement over Option A in terms of user experience, tree loss and riverbank risk.

Option D (2^{nd} highest cost) is improved over Option A in terms of user comfort and accessibility by increasing the pathway width, however Option D maintains existing streetlight locations (less desirable for traffic safety and aesthetics) and also poses the greatest risk to riverbank stability. The additional cost for Option D relative to Option A is approximately $0.60M primarily due to the increase in stabilization works required.

Option C (highest cost) is the preferred alternative in terms of roadside safety, accessibility and user experience as it includes the larger pathway width and relocation of the streetlights beyond the pathway edge. The total estimated construction cost for Option C is $4.29M, approximately $0.64M higher than the lowest cost option (Option A), and is only $0.04 higher than Option D which is the next lowest cost.

TREK recommends that the City proceed with the detailed design of Option C given that all project objectives are satisfied within the available project budget.

8.0 Detailed Design Considerations

The following items should be addressed in detailed design:

1. All riverbank monitoring instrumentation will be monitored before and after the fall drawdown of the river to measure current groundwater conditions and riverbank movements.
2. The final extent of fill placement should be confirmed once the preferred pathway option is determined. Similarly, there may be opportunities to offload the bank upslope of the tree line in areas where the top of bank is offset from the edge of the proposed pathway. Offloading in these areas should only be performed if damage to tree roots can be avoided, so as to not result in further tree loss.
3. The rock column layout should be refined to avoid areas of existing riprap, rockfill columns, granular ribs, and mature healthy trees. Any potential obstructions to rockfill column installation that cannot be avoided should be noted on the tender drawings. Additional sub-surface investigations could be performed to confirm the extent of the riprap, time permitting.
4. We understand that slight narrowing of the roadway near Lawndale Avenue (by up to 0 5 m) may be possible without the need for public consultation. The preliminary design assessment did not examine any changes to roadway alignment or width. If desired, TREK can retain a sub-consultant to design any changes in the south curb geometry and confirm any reductions to the riverbank stabilization works and associated cost savings. A scope change will be required as this scope was not included in the assignment.
5. A pre-approved temporary access ramp and working platform should be designed to satisfy temporary riverbank stability requirements for construction. Typically, a slight benefit to global stability is desired. Conflicts with existing utilities may influence the possible location of access ramps, thereby impacting the extent of required roadway repairs.
6. Traffic Services should be consulted to confirm if a single-lane or full street closure can be implemented during construction, both of which have been implemented on previous riverbank stabilization projects along Lyndale Drive.

7. If streetlight relocation is to be performed, TREK will need to retain a sub-consultant for design of streetlight relocation works. A scope change will be required as this scope was not included in the assignment.
8. If pathway construction is to be included in the current tender, TREK will need to retain a sub-consultant for design of these works. A scope change will be required as this scope was not included in the assignment.

9.0 Closure

The geotechnical information provided in this report is in accordance with current engineering principles and practices (Standard of Practice). The findings of this report were based on information provided (field investigation and laboratory testing). Soil conditions are natural deposits that can be highly variable across a site. If subsurface conditions are different than the conditions previously encountered on-site or those presented here, we should be notified to adjust our findings if necessary.

All information provided in this report is subject to our standard terms and conditions for engineering services, a copy of which is provided to each of our clients with the original scope of work or standard engineering services agreement. If these conditions are not attached, and you are not already in possession of such terms and conditions, contact our office and you will be promptly provided with a copy.

This report has been prepared by TREK Geotechnical Inc. (the Consultant) for the exclusive use of City of Winnipeg – Planning, Property and Development (the Client) and their agents for the work product presented in the report. Any findings or recommendations provided in this report are not to be used or relied upon by any third parties, except as agreed to in writing by the Client and Consultant prior to use.

Figures

Sub-Surface Logs

Appendix A

Laboratory Testing

Appendix B

Riverbank Instrumentation Monitoring Results

Appendix B

Riverbank Instrumentation Monitoring Results

Appendix C

Heritage Resource Impact Assessment report (pending)

Appendix D

Slope Stability Analysis Results

Appendix E

Hydraulic Assessment Report

Appendix F

Class 3 Construction Cost Estimates