Submitted to:

FLOOD PROTECTION DESIGN

820 CLOUTIER DRIVE

WINNIPEG, MB

MARCH 2020

FILE NO. 16-217-04

"Engineering and Testing Solutions That Work for You"

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1.0 INTRODUCTION

ENG-TECH Consulting Limited (ENG-TECH) completed the flood protection design for the existing house at 820 Cloutier Drive, Winnipeg, Manitoba. ENG-TECH understands that the riverbank is under risk of bank failures, with tension cracks and scarps observed at the neighboring properties of 824 and 830 Cloutier Drive. The purpose of this assessment was to investigate the overall riverbank stability in order to provide riverbank stabilization measures and flood protection for 820 Cloutier using clay dike and double-sided concrete flood wall system to meet the required flood protection level set by the City of Winnipeg.

1.1 Scope of Work

Outlined below is the scope of work completed:

- A review of existing ENG-TECH slope stabilization and flood protection projects along the river near the property to assess overall soil conditions and riverbank stability.
- A survey of the bank in the area of the existing house and cross sections of the river bank using a total station and GPS survey equipment. The river bottom soundings profile was referenced from our previous 2017 survey work at the subject property.
- Assessments of riverbank stability using a two-dimensional finite element analysis computer program to determine the impact the proposed flood protection designs would have on the riverbank stability.
- Installation of one (1) slope inclinometer, one (1) vibrating wire piezometer, and one (1) stand pipe piezometer.
- Development of conceptual drawings detailing the flood protection design.
- An engineering report outlining the assessments and designs.

1.2 Background

ENG-TECH previously completed flood protection designs for adjacent houses at 820, 824 and 830 Cloutier Drive using a variety of design options regarding the proposed flood protection works and slope stability assessments which can be found in ENG-TECH's 2016 report 16-217-04. The purpose of this report was to finalize the flood protection design for the property at 820 Cloutier Drive, and provide remediation measures in order to improve the existing riverbank stability and ensure the proposed flood design would not have a negative impact on the existing riverbank stability.

The soil stratigraphy was based on the visual inspection/classification of soils collected during the drilling for the installation of the groundwater and slope stability instruments at the subject property. ENG-TECH supplemented the soil stratigraphy data with the previous soil investigation conducted on a nearby property at 730 Cloutier Drive. The combined data provided adequate subsoil condition information for conducting the slope stability assessment at the subject property. The soil stratigraphy can be referenced from the previous slope stability assessment report 16-217-04, conducted in 2016.

2.0 INSTRUMENTATION INSTALLATION

ENG-TECH supervised the drilling of two (2) test holes (TH1 and TH2) at the site as shown on Figure 1. Both test holes were drilled by Maple Leaf Drilling Ltd. on June 5, 2019 using a track mounted B37X drill rig equipped with 125 mm solid stem auger and were advanced to auger refusal at 13.1 m below existing grade.

One (1) slope inclinometer, SI-1, was installed and extended into the silt till at TH1 to measure riverbank movements. One (1) vibrating wire piezometer (VW-1) and one (1) standpipe piezometer (STP-1) were installed in TH2 as VW-1 located within the overburden clay and STP-1 was installed in the silt till to measure groundwater conditions. All the piezometers and slope inclinometer monitoring results are shown on Figure 5.

3.0 SITE INVESTIGATION

3.1 Site Geometry

ENG-TECH revisited the subject property on April 9, 2019 and surveyed two (2) cross-sections and several spot elevations in order to examine any changes of the riverbank geometry relative to the survey data that was collected in 2016. No significant bank movements (less than 50 mm) were observed in the past three years after comparing to the 2016 survey data. The river bottom soundings were obtained from our previous bathymetric survey that was conducted in 2016. The City of Winnipeg flood protection level (FPL) along this stretch of the river is 232.46 m.

Numerous tension cracks and headscarps were observed at the top of bank between the properties of 824 and 830 Cloutier Drive. The cracks also meandered along the shoreline and spread eastward to the lower bank at 820 Cloutier Drive, having a crack depth of approximately 0.5 m as shown in the sections overlay on Figure 2. There are several mature trees along both sides of the property; with no trees near the shoreline. Photographs of the properties taken at the time of the survey are shown in Figure 1.

3.2 Piezometers Results

ENG-TECH supervised the installation of one (1) VW piezometer in overburden clay and one (1) standpipe piezometer in the silt till in order to check groundwater levels at the subject property. Both piezometers have been monitored a total of four (4) times since their installation and indicate the groundwater level (GWL) changes in response to river levels. As the river level ranged from 224.8 m to 224.0 m from early June to late June, the piezometer in the clay was reading from 227.3 m to 225.8 m respectively. For the same river levels, piezometric readings from 225.9 m to 225.2 m were observed in the silt till, rising up to approximately 226.0 m when the river level elevated to 225.8 m in late July.

Based on the readings, riverbank instabilities can occur in the spring time due to significant drawdown causing a large head difference between the GWL and river level. During the summer time the riverbank is more stable as the head difference becomes less. Figure 5 demonstrates the seasonal range in the GWL and River levels.

3.3 Slope Inclinometer Results

One (1) slope inclinometer was installed into the dense silt till in order to monitor bank movements within the riverbank during the period of June through to December, 2019. The SI was located in TH1 near the lower bank just upslope of the tension crack, as shown on Figure 1. A total cumulative movement of 1.5 mm at a depth 6.5 m below grade was observed since the June installation date, which is at an elevation of approximately 222.0 m in the clay layer.

From the results, the slip surface geometry could be a deep-seated slide, which initiated from the upper bank, passing through the slicken sided clay and extended into the river channel. The potential slip surfaces (PSS2, PSS3 and PSS6) in cross section 1 on Figure 2 could represent the typical geometry of the slip surface having a FS (factor of safety) value equal to or less than 1.0 under both normal and extreme conditions. More detailed assessment with regard to the PSSs is presented in the slope stability section. The slope inclinometer data and the profile for the SI is outlined on Figure 5.

4.0 SLOPE STABILITY

4.1 General

Slope stability analysis was completed using Slope-W, a two-dimensional finite element analysis computer program. The analysis was completed to assess the existing riverbank stability and the stability after the proposed flood protection infrastructure was in place. The soil and geometry conditions used in the analysis are shown in Figure 2.

4.2 Assessment Criteria

The assessment criteria used for riverbank stability consisted of the following:

- The soil shear strength parameter values used in the analysis should be representative or slightly conservative based on the existing condition observed on site;
- The groundwater level (GWL) should be representative of groundwater conditions observed at the site or a conservative estimate of the static level; and
- The combined groundwater and river level used in the analysis should be representative of site conditions in order to determine the impact of the proposed flood protection improvements.

4.3 Soil and Groundwater Values

Based on the site visit there was evidence of tension cracks and head scarps on both the subject property and nearby neighbours. As such, the shear strength values used in the analysis were both post peak and residual values, with the residual values used on the riverside of the upper bank and post-peak values used for the up-slope as shown on Figure 2. The bulk density of all the soil layers was evaluated based on typical density values for this location. The analysis was modelled using a design groundwater level at 226.2 m in combination with R.S.R.L (regulated summer river level) of 224.0 m and U.W.R.L (unregulated winter river level) of 222.0 m to represent the normal and extreme conditions, respectively. Outlined below are the soil shear strength parameter values and water levels used in the model.

Brown/Grey Clay (post peak)	Unit Wt = 18 kN/m^3,	c’ = 5 kPa,	ϕ’ = 14˚
Brown/Grey Clay (residual)	Unit Wt = 18 kN/m^3,	c’ = 2 kPa,	ϕ’ = 8˚
Silt Till	Unit Wt = 20 kN/m^3,	c’ = 1 kPa,	ϕ’ = 25˚
Rock Fill Column	Unit Wt = 20 kN/m^3,	c’ = 0 kPa,	ϕ’ = 40˚
Rip-Rap	Unit Wt = 20 kN/m^3,	c’ = 0 kPa,	ϕ’ = 5˚

GWL (Groundwater Level)	226.2 m
U.W.R.L. (Unregulated Winter River Level)	222.0 m
R.S.R.L. (Regulated Summer River Level)	224.0 m

4.4 Methodology

The slope stability analyses were completed using Morgenstern-Price circular slip surfaces with a half-sine interslice force function to estimate the FS of the potential slip surfaces (PSSs). Two (2) cross-sectional profiles were obtained within the subject property as shown in the sections overlay profile outlined in Figure 2. Cross section 1 was selected for the slope stability analysis as both cross sections represent a similar bank geometry.

A back analysis was completed on cross section 1 in order to determine the post-peak and residual shear strength values of the brown/grey clay deposit. The back analysis was conducted to assess the values ϕ' and c' assuming a quasi-stable (FS = 1.0) condition at the location where the tension cracks have occurred. The results show large deep-seated failures initiated, ranging between the upper bank and mid-bank and extending into the shoreline. The obtained shear strength parameters were then applied to other cross-sections of the riverbank to perform stability assessments.

The above assumptions are considered a realistic condition that shows why the failure occurred based on the information obtained to date. The effect of vegetation and soil suction on the stability of the bank was not accounted for in the analyses. They would provide a positive benefit to shallow potential failures (sloughing), but based on the analysis would not have a significant impact on deeper slides where the failure plane extends below the base of the tree roots.

4.5 Assessment

Overall Stability

Overall, the riverbank is prone to slides and potential large retrogressive type slope failures, which is considered as quasi-stable since the PSSs (PSS2, PSS3 and PSS6) which initiated from the upper bank and extended into the shoreline have FS values near 1.0. Large failures or retrogressive failures can occur until the riverbank attains a stable angle of repose. With the proposed conditions, the FS values for all PSSs on cross section 1 were improved ranging between 10% and 60% depending on the PSSs location. All parameters used in the analyses and potential slip surfaces are shown on Figure 2.

Caissons & Erosion Protection

The use of limestone rip-rap to minimize shoreline erosion is the most viable option. The rip-rap can be placed starting at an elevation of approximately 225.0 m, and extend into the river channel to an elevation of approximately 222.0 m as shown in the sectional and plan views on Figures 2 and 3, respectively. The rip-rap alone would have a slight positive impact on riverbank stability, although its intended use is for shoreline erosion protection.

Some stabilization options were reviewed, with the most viable being rock filled caissons based on the depth of the potential failure planes, effectiveness of stability improvement, depth to till and construction cost. Improvements between 10 and 60% to riverbank stability resulted from installing caissons having a diameter of 2.1 m constructed at the lower bank in combination with a rip-rap blanket along the shoreline. A total of 22 caissons would be required throughout the property, which includes 4 caissons to be installed on the neighbour's property at 824 Cloutier Drive to ensure no riverbank instabilities would occur from the neighbour's property. The site plan for caissons and rip-rap layout are shown in plain view on Figure 3 and in section view on Figure 2.

5.0 FLOOD PROTECTION

ENG-TECH evaluated three (3) common flood protection design methods: 1) concrete flood wall and 2) clay dike with one side retaining wall and 3) double sided retaining wall. The geodetic flood protection level at this stretch of the Red River is 232.46 m and the proposed flood protection designs shall be in accordance with the City's requirements regarding the final design elevation of 231.0 m and the minimum acceptable FS after proposed construction.

After several discussions and meetings with the City, we believe the double-sided retaining wall is the best viable option in order to meet the City's requirements. While a single-sided wall is less expensive for construction, the amount of fill required to be placed on the riverbank for this design option, and the associated need to mitigate impact on slope stability, makes this option more expensive and less desirable than a double-sided retaining wall. The double-sided wall option assists in limiting the amount of clay fill to be placed on the upper bank and reduces impact on the existing riverbank stability. The inside concrete wall will start from the southwest corner of the house and follow a path along the perimeter of the existing deck. The top of the constructed flood wall will be at an elevation of 231.0 m. The outside wall will be initiated in the neighbour's property approximately 3.0 m beyond the west property line and extend 4.5 m beyond the east property line within the City's property. The setback distance between the inside and outside wall is 3.9 m and will be placed with medium to highly plastic clay having a finished grade of 231.0 m. There will be less impact of the clay fill on the neighbouring properties as the side slopes at both property lines will be 2H:1V (Horizontal to Vertical). It is understood that emergency sandbagging during an extreme flood event will be required to meet the flood protection level of 232.46 as shown on Figure 3. The weight of the emergency sandbag dike has been included in the slope stability analysis.

6.0 CONCLUSIONS

Based on the assessment ENG-TECH concludes the following:

- The clay shear strength obtained from back analysis was considered reasonable to represent the existing site conditions as the FS values from the analyses were consistent with site observations.
- The shoreline is prone to both erosion and slope failures. Shoreline erosion protection (rip-rap) and riverbank stabilization (caissons) will be required to improve the overall stability of the bank, with the rip-rap used for erosion protection and the caissons used for stabilization.
- Full time resident inspection and monitoring by an experienced geotechnical engineer during construction of the proposed works is required to ensure the constructed rockfill caissons are installed into the dense till, QC for concrete works and compacted fill placement.
- A riverbank monitoring program should be conducted during and after the construction of the proposed works in order to confirm the stability of the riverbank. A vibro compactor should be used after the completion of each caisson in order to densify the rock fill material.
- In extreme flood events an emergency dike will have to be placed on top of the double-sided retaining wall dike.

7.0 CLOSURE

This report was based on the scope of work outlined for the purpose of the investigation, and was prepared in accordance with acceptable professional engineering principles and practices. If you have any questions, please contact the undersigned.

Sincerely,
ENG-TECH Consulting Limited

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