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DRUMHELLER

RESILIENCY AND FLOOD MITIGATION OFFICE

DRUMHELLER VALLEY

DRFM Dike D - Geotechnical Investigation Report

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1. INTRODUCTION AND BACKGROUND

This report documents the findings and recommendations of a geotechnical site investigation performed by SweetTech Engineering Consultants (SweetTech) on the existing and proposed Dike D footprint in Drumheller. This investigation was undertaken to inform the detailed dike design and forms a key component of the Drumheller Dike D Preliminary Design Process.

Dike D is an approximately 1,200 m long earth fill dike located in Central Drumheller along the right bank of the Red Deer River. Dike D protects the Civic Heart of the Valley, Downtown Drumheller. Critical infrastructure located within Downtown Drumheller including the Fire Department, City Hall, Police Station, the Badlands Community Facility, the Arena, Aquaplex, and vital historical, cultural, and tourism facilities.

The current Dike D was conceived from a provincial Flood Abatement Work Group (FAWG) and was constructed in three phases between 1987 and 1991. In addition, extensive riverbank reconstruction work was completed along Riverside Drive in the late 1960s and 1970s.

This present scope of work involves designing improvements and upgrades to Dike D to reduce flooding risks and protect Drumheller into the 22nd century (for the design flow). Dike D is planned to be upgraded to a design flow of 1,850 m3/s along the Red Deer River with consideration for adaptive emergency response management allowing for an emergency dike raise to manage flows of 2,100 m3/s and greater.

Utilizing the 1,850 m3/s design flow rate, it was determined that the existing dike would need to be raised about 0.8 to 1.25 m along most of the alignment (which includes a 0.75 m freeboard). A portion of the existing dike situated along Riverside Drive, currently has a 0.75 m high concrete jersey barrier that is considered part of the existing flood protection system situated on the crest of the dike and in this area the jersey barrier will be removed and replaced with an earth fill dike and possibly a retention system. Upwards of 2.1 m of earth fill will be required to construct the dike upgrades in this area along Riverside Drive. This area can be seen on Figure 1 in Appendix 1.

This geotechnical assessment was performed to determine requirements and recommendations for raising the dike up to 2.1 m with select earth fill and widening the dike as needed to provide a minimum 6 m top width and 3 horizontal to 1 vertical (3H:1V) side slopes.

The following reports were reviewed in preparation of this geotechnical report:

- SweetTech Draft Concept Design and Feasibility Study (Feb 2021)
- Drumheller River Hazard Study – Open Water Hydrology Assessment Report – Northwest Hydraulic Consultants (2020);
- Drumheller Community Facility – Geotechnical Investigation – Thurber (2006);
- Drumheller Dike D Construction – Field Reports – EBA (1987);
- Landale Development Corporation – Soils Report – Palm Engineering (1999); and
- Geotechnical Investigation - Proposed Hotel and Restaurant Development Lot 2, Block 34, Plan 991 1179, Drumheller Alberta – ParklandGeo (2014).

2. SITE LOCATION AND DESCRIPTION

The existing alignment and proposed Dike D alignment and extents are shown on Figure 1. The existing Dike D is a 1200 m long, earth filled dike topped with a paved asphalt trail for approximately 2/3 of its length, from the Gordon Taylor Bridge to Riverside Drive. The southern portion of the dike along Riverside Drive consists of a concrete pathway and jersey barrier situated on the existing dike crest. The height of the existing dike varies from approximately 0.5 m to 2 m above the adjacent ground for most of the entire length of Dike D.

The existing vegetation around the proposed dike footprint consists primarily of grass, shrubs (willows) and local trees, primarily poplars. The shrubs (and same trees) are on the river side of the dike, between the existing dike and the Red Deer River. The existing dike side slopes have been vegetated with grass and in the park areas the grass was observed to have been trimmed along the dike. Between Riverview Terrace and the Badlands Community Facility the toe of the dike on the land side has been constructed around 4 trees, where timber crib retaining walls have been constructed into the dike to protect the trees.

Some tree and brush clearing was previously performed in fall 2019 along Dike D, primarily at the south end and along Riverside Drive. The river side of the concrete jersey barrier flood wall had not been maintained and there was extensive growth of shrubs and trees. When the geotechnical drilling program was performed, this vegetation had mostly been cleared and was in the process of being removed.

The general topography around Dike D, is a steep riverbank which levels off around Dike D and continues to rise at a shallow slope into the downtown core. Dike D has been constructed on the crest of this bank, a short distance away from the active river channel. In the north, around Centennial Park, the floodplain is broad and has a shallow slope before dropping to the River, and downstream of Centennial Park, the riverbank is steep and is near vertical in some areas due to active riverbank erosion. At the south end of the proposed dike alignment (adjacent to the lift stations), the land slopes down to a lower flat shelf beside the river. This area will require approximately 5 m of fill to construct the dike. There are several large trees and shrubs in this fill zone that will need to be removed.

During recent flood events in 2005 and 2013, Dike D is understood to have been temporarily topped up with extra fill material in a rapid fashion. With exception to the poor performance of the asphalt pathway surface (longitudinal and lateral cracking), no known performance issues have been observed in the past with Dike D.

As seen in Figure 1, the proposed typical dike detail, minimum 6 m top width and 3H:1V side slopes, are suitable for most of the areas along the dike, however there are several areas where the design may need to deviate from this typical detail. These areas are at the northwest corner of the Aquaplex building, a portion of the northeast property line of the Riverview Terrace Condominiums and along Riverside Drive, as identified in Figure 1 in Appendix 1. These areas may need to vary from the typical detail due to space constraints with existing and proposed infrastructure and proximity to the river. The flood barrier along Riverside Drive is currently constructed of concrete jersey barriers with a narrow (<2 m wide) walking path beside it. The portion of the dike alignment with jersey barriers is located between dike station D 0+806 to station D 1+100. If the typical dike detail (trapezoidal 3H:1V side slopes) is utilized in this area and instream river work is to be avoided, the entire width of Riverside Drive falls within the proposed dike footprint. As such, preliminary options for the Aquaplex and Riverview Terrace Condominiums involves the construction of a retention system on the landside of the dike to reduce the proposed footprint. Along Riverside Drive preliminary options include the construction of retaining walls, a combination of retaining walls and reconfiguring Riverside Drive and complete removal of a portion of Riverside Drive to allow for the typical dike section to be constructed. The specific layout, design details and materials for these areas have yet to be selected and further design and decision making from IBI Group and the DRFM is required prior to proceeding with the detailed design for these specific areas.

Image 1 - Riverside Drive Conflict Area

3. GEOTECHNICAL INVESTIGATION

SweetTech performed a geotechnical field drilling investigation and associated lab testing to inform the ongoing pre and detailed designs, allow for transmissivity of the existing dike soils to be addressed, assess the quality and quantity of existing fill soils, define the specific areas groundwater conditions and to verify competency of the underlying soils. The details of the subsurface investigation program are discussed and defined below.

3.1 SURFICIAL GEOLOGY

Based on a review of available surficial geology for the Drumheller area (A. Stalker, Geological Survey of Canada Memoir 370, 1973), the Dike D areas will generally consist of quaternary fluvial deposits (gravels, sands, silts and clays) which overlie cretaceous bedrock from the Edmonton Formation. The bedrock consists of grey, green and brown clay shale, argillaceous siltstone, and sandstone with coal beds and visible ironstone partings.

3.2 FIELD INVESTIGATION

A site reconnaissance was performed on February 19, 2021 by Andres Ocejo, P.Eng., and Thomas Schaepsmeyer, M.Eng., E.I.T. The site reconnaissance was used to plan the drilling investigation, identify potential conflicts with the typical 3H:1V dike section and inform our ongoing design efforts.

On March 17, 2021, 10 boreholes were drilled along the proposed Dike D footprint. The boreholes were spread out along the length of the alignment. The location of these boreholes is shown on the Location Plan, Figure 1. The following sampling and testing procedures were followed during the field program:

- SweetTech completed an Alberta One Call planning application along the entire Dike D footprint to reduce the potential for underground utility conflicts prior to selecting borehole locations.
- The borehole locations were staked out in the field and provided for client review prior to drilling.
- Prior to mobilizing the drilling rig, SweetTech completed an Alberta One Call and cleared the proposed borehole locations of underground utility conflicts.
- A private locate was performed by Line Hunter Locates and cleared the proposed borehole locations of underground utilities.
- Borehole BH-02's location was adjusted based on site conditions and concurrent tree-clearing operations prior to drilling to minimize disturbance to existing flood infrastructure (jersey barrier system).
- The boreholes were drilled by All Service Drilling using a track mounted auger drilling rig (Geoprobe DR7822) using solid stem augers.
- Drilling operations were monitored by SweetTech's geotechnical staff. The soil encountered was visually examined during drilling and logged according to the Modified Unified Soil Classification System.
- Standard Penetration Tests were performed at selected depth intervals in all boreholes.
- At the completion of drilling, 50 mm hand-slotted PVC standpipes were installed in select boreholes. The remaining boreholes were backfilled with auger cuttings and bentonite chips.
- Nested PVC standpipes were installed at two boreholes (BH-04 & BH-05) to allow for in-situ testing of hydraulic conductivity of the existing dike fill.
- Samples were taken in boreholes at 1.0 m intervals or with changes in stratigraphy to determine the soil/moisture profile.
- Soil samples were obtained off the solid stem auger flight (grab samples), from the split spoon sampler, and in 3 Shelby Tubes.
- Selected soil samples were bagged, numbered and delivered to Solum Consultants Ltd. for laboratory testing. All other soil samples were retained and stored at SweetTech's Calgary office for possible further testing.
- Groundwater levels were recorded on the day of drilling (March 17, 2021), one week later (March 25, 2021), and two weeks later (March 31, 2021).

- The location of the completed borehole locations was surveyed by Hunter Wallace Surveys Ltd. using a Trimble GPS receiver and a pole mounted Trimble GPS antenna. UTM coordinates and ground elevations are provided on the borehole logs in Appendix 2.

The area located between Dike D Station D1+080 and D1+195, was not accessible during the March 17th geotechnical investigation due to excessive vegetation. Once this area had been cleared of vegetation, two historical resource test pits were performed in this area on July 16, 2021. TP-01 was excavated at Dike D Station D1+101 and TP-02 was excavated at Station D1+166. During the historic resources assessment, SweetTech performed a bearing capacity assessment to a depth of 2 m utilizing a static cone penetrometer. The location of these test pits is shown on the Location Plan, Figure 1.

3.3 SOIL CONDITIONS

The soil profile along Dike D consisted of varying compositions of sand, silt and clay fill soils, with underlying deposits of fluvial silt and sand with random silty clay partings. Bedrock was not observed in this location and auger refusal was not encountered. Detailed soil conditions encountered for each borehole are described on the borehole logs attached in Appendix 2. The following is a description of the soil types encountered along the proposed and existing dike alignment.

3.3.1 EXISTING DIKE AND UNDERLYING FILL SOILS

The existing surficial soils around the Dike D site are believed to have been filled, recontoured, and changed over the past 100 years, and as such, the existing Dike D has not been constructed on native ground. Thurber reports anecdotal evidence of a landfill and filling of an old side channel of the Red Deer River (Thurber, 2006), while Palm Engineering reported that the elevation of the ground where Centennial Park is located had previously been filled in with 0.9-1.5 m of soil in the 1940's (Palm, 1999). In addition, Palm Engineering investigated the Riverview Terrace property and found that the fill in this area ranged from 1 m to 3.6 m in thickness and generally consisted of a sandy silt fill material.

Boreholes 2, 2A, 3, 3B, 4, 4C, 5, and 6 were all drilled through the existing dike, and were drilled approximately 0.3 m offset from the pathway located along the crest. The dike fill material was found to generally consist of varying mixtures of silt and sand with a low to high plastic clay fraction. Hydrometer testing of samples taken during drilling found clay contents in the material that varied from 21.8% to as high as 28.9%, the silt content varied from 34.3% to 46.3%, and the sand content was predominantly fine grained, varying from 21.3 to 41.3%. Gravel was found in all samples tested and ranged from 2.6 to 4.8% of the grainsize distribution. Liquid limits of the samples also ranged from 27% to 77% in the existing dike fill, with clay shale nodules being found in many of the samples. These results suggest that the fill was randomly mixed with high plastic, highly weathered clay shales and a moderate degree of variability in the fill materials hydraulic conductivity and plasticity is to be anticipated. A thin clay seam was observed in Borehole 2 within the dike fill material, further confirming the variability of the previously utilized fill materials for dike construction. SPT "N" values ranged from 14 to 68 blows per 300 mm penetration, indicating existing dike fill material is stiff to hard. In addition, one Unconfined Compression Test was completed on a sample recovered from

BH-04 at a depth of 0.76 m and resulted in an unconfined compressive strength of 145 kPa. Based on these results, the consistency of the existing dike fill material strongly indicates that the existing dike fill has been placed with an engineered compactive effort and based on a review of the construction information available from 1987, the dike fill material was compacted to ≥98% Standard Proctor Maximum Dry Density (SPMDD). The moisture content of the dike fill material ranged from 4.3 to 23%, with most of the values ranging from 18 to 21%. The moisture contents are indicative of the material being placed at ±3% of the materials optimum moisture content (OMC). The presence of shale in the samples suggests that the dike fill was partially constructed from a reworked shale, which was used for the dike fill construction. This is consistent with field reporting from a 1987 EBA Engineering materials report from the field during construction of Dike D.

The underlying fill material below the existing dike fill was encountered at surface in Borehole 1 & 7 and below surface in Boreholes 2, 3, 4, 5, and 6. The thickness of the underlying fill material varied throughout the Dike D alignment. The underlying fill thickness in the north area of the Dike (Approximately Dike D Station D 0+000 to Station D 0+664) ranged from 1.2 m to 2.0 m. In the south area of the Dike (Dike Station D 0+664 to Station D 1+211) underlying fill thicknesses ranged from 1.8 m to 5.2 m with fill thicknesses increasing as one progresses south on the dike alignment. The underlying fill generally consisted of sand and silt with trace to some quantities of clay. Hydrometer testing of samples taken during drilling found clay contents in the material that varied from 8.3% to as high as 23.5%, the silt content varied from 31.8% to 48.2%, and the sand content was predominantly fine grained, varying from 34.3 to 54.1%. Gravel was found in some samples tested and ranged from 0 to 3.2% of the grain size distribution. The underlying fill soils were found to predominantly have a low plasticity with random medium to high plastic partings noted. Two Atterberg Limit test were performed on this material and resulted in liquid limits of 27% and 47% with corresponding plastic limits of 21% and 24%, respectively. SPT "N" values ranged from 4 to 49 blows per 300 mm penetration, indicating underlying fill material ranges from soft to hard. The majority of the SPT "N" vales were found to range from 4 to 14 and provides a strong indication that these materials were not placed with compaction testing performed and that the degree of compaction attained on these soils is highly variable. The moisture content of the underlying fill soils was found to range from 8.5% to 35.2%, providing further verification of the variability of the fill materials composition and condition at the time of placements.

3.3.2 SILT & SAND

Deposits of silt and sand were encountered below the underlying fill in Boreholes 3, 6 and 7. This soil was composed of silt and fine grained sand, with trace to some clay. The material was found to generally have a slight plasticity and was found to have a highly variable consistency/density in its in-situ condition. The SPT "N" values for the material ranged from 3 to 19 indicating a very loose/soft to compact/very stiff material condition. The material was generally found to be in a loose condition along the north half of the dike alignment and improved to be in a compact condition for the southern half of the dike. The moisture content of the silt and sand ranged from 14.4% to 30.3%.

3.3.3 SILT

Deposits of silt were encountered in Borehole 1, 4 and 7 below the underlying and dike fill materials at a depth of 5.2, 4.3 and 2.6 m respectively. The silt was found to be sandy with trace to some clay and generally had a low plasticity in its in-situ condition. Three SPT "N" values were attained within the silt material and resulted in blow counts of 7, 9, and 32 indicating the material was generally firm to stiff, with the deposit having a hard consistency at the south extent of the dike alignment. The moisture content of the silt was found to range from 8.2% to 26%.

3.3.4 SAND

Deposits of sand were encountered in Borehole 2 and 7 underlying the existing native material, with a depth of 5.3 m to 4.4 m below existing ground surface, respectively. The sand was moderately graded between fine and coarse grained sand and was generally silty and gravely. One SPT "N" value was attained for the sand and resulted in a blow count of 25, indicative of a compact soil. The moisture content of the sand was found to be from 4.2% to 7.9%. This gravely sand layer generally coincides with the river elevation and is assumed to be an extension of the rivers fluvial channel sands and gravels.

3.3.5 CLAY

A deposit of clay was encountered below the underlying fill in Borehole 5 with a depth of 3.4 m below the ground surface elevation. The clay encountered was silty, with some sand, trace gravel and was observed to have a low to moderate plasticity. One SPT "N" value was attained in the material and resulted in 5 blows per 300 mm penetration, indicating a firm relative consistency. The moisture content of the clay was found to range from 17.5 to 23.6%.

3.4 LABORATORY TESTING

Obtained soil samples were organized and select soil samples which covered the different stratigraphic layers observed onsite were submitted to Solum Consultants Ltd. Geotechnical and Material Testing Laboratory in Calgary, Alberta, for index soil testing. Soils laboratory index testing included:

- 33 moisture content tests (ASTM D2216);
- 7 particle size analysis (ASTM D422);
- 7 Atterberg limits tests (D4318); and,
- 1 unconfined compression test (D9728).

Natural moisture contents were provided with particle size analysis and Atterberg limit tests.

The laboratory test results are discussed in this report and are attached in Appendix 3.

3.4.1 MOISTURE CONTENT (ASTM D2216)

Moisture contents were performed on 33 samples. The test results are summarized in Table 1.

Table 1 – Moisture Content

Sample	Depth (mbgs)	Moisture Content (%)	Sample	Depth (mbgs)	Moisture Content (%)	Sample	Depth (mbgs)	Moisture Content (%)
01-02	0.3-0.9	8.9	03-01	0.3-0.8	4.3	06-02	0.8-1.2	19.9
01-04	1.7-2.0	21.1	03-05	2.4-2.7	17.6	06-03	1.2-1.5	20.3
01-05	2.4-2.7	17.5	03-06	3.0-3.5	30.8	06-04	1.5-2.0	12.7
01-06	3.0-3.5	35.2	03-08	4.0-4.3	30.3	06-06	2.4-2.7	14.4
01-08	4.3-4.6	21.0	03-09	4.6-5.0	14.7	06-07	3.0-3.5	14.4
01-09	4.9-5.2	28.5	04-06	3.0-3.5	14.4	07-04	1.5-2.0	7
01-11	6.1-6.6	17.3	04-08	4.0-4.3	30.2	07-07	3.0-3.5	8.2
02-1A	0.8-1.2	23.3	04-10	4.6-5.0	26.0	07-09	4.3-4.6	4.2
02-03	1.5-2.0	27.4	05-01	0.3-0.6	19.7	07-10	4.6-5.0	7.9
02-05	2.4-2.7	20.1	05-02	1.1-1.4	17.9			
02-06	3.0-3.5	21.5	05-06	3.0-3.5	15.4			
			05-08	4.1-4.4	17.5			
			05-09	4.6-5.0	23.6			

3.4.2 PARTICLE GRADATION RESULTS (ASTM D422)

Particle size analysis was performed on seven soil samples to determine the materials gradational characteristics. Moisture content was also completed on the samples. The test results are summarized in Table 2.

Table 2 – Particle Size Analysis

Sample	Depth (mbgs)	Cobble (%)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Moisture Content (%)
02-2A	0.8-1.2	0	3.7	21.1	46.3	28.9	24.6
02-08	4.3-4.6	0	3.2	22.8	47.2	15.8	28.3
03-04	1.5-2.0	0	0	43.5	48.2	8.3	18.2
04-03	1.5-2.0	0	2.6	41.3	34.8	21.8	15.2
05-03	1.5-2.0	0	4.8	34.3	37.4	23.5	17.4
07-02	0.8-1.2	0	0	54.1	31.8	14.1	4.8
07-06	2.6-2.9	0	0	21.8	59.2	19	8.5

3.4.3 ATTERBERG LIMITS TESTS (ASTM D4318)

Atterberg Limit tests were completed on seven soil samples to determine plastic and liquid limits. The test results are summarized in Table 3.

Table 3 – Atterberg Limit Test Data					
Sample	Depth (mbgs)	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
02-09	4.9-5.2	31.5	38	21	17
03-2B	0.6-1.2	8.0	27	21	6
04-01	0.3-.06	21.7	68	39	38
04-02	0.9-1.4	14.1	66	29	37
04-05	2.4-2.7	17.3	77	30	47
05-04	1.7-2.0	13.3	47	24	23

3.4.4 UNCONFINED COMPRESSION TEST (ASTM D2166)

An unconfined compression test was performed on the Shelby Tube Sample, from BH-04. The test results are presented in Table 4.

Table 4 – Unconfined Compression Test – Sample BH-04			
Sample		Test Results	
Diameter (cm)	6.00	Moisture Content (%)	23.4
Height (cm)	14.53	UCS q_u (kPa)	145.03
H/D Ratio	2.42	Shear Strength (kPa)	72.52
Mass (g)	798.4	Axial Fal. Strain (%)	9.45

3.5 GROUNDWATER CONDITIONS

Groundwater seepage and sloughing conditions were only observed in Borehole 2 during drilling. Groundwater levels were measured in the four boreholes with water levels measured at three different times. The recorded groundwater measurements are provided in Table 5 below.

Table 5 – Groundwater Conditions				
Borehole/ Piezometer	Surface Elevation (m)	Depth to Groundwater (m)		
		March 17, 2021	March 25, 2021	March 31, 2021
BH-02	681.9	5.5	5.0	4.8
BH-04	682.1	Dry	Dry	Dry
BH-05	683.1	Dry	Dry	Dry
BH-07	682.9	Dry	Dry	Dry

As the boreholes drilled were close to the river, within 6 to 25 m of the top of bank, it is expected that the measured groundwater table is hydraulically connected to the Red Deer River. Groundwater elevations are expected to fluctuate on a seasonal basis and will be highest after extended rainfall periods or snow melt. Based off the observations during drilling and from our review of previously performed geotechnical investigations in the area, the estimated static groundwater table elevation on the landside of the dike typically ranges from 677 m to 678 m. The groundwater elevation should generally be expected to fluctuate seasonally and with the level of the Red Deer River.

The piezometers installed in Boreholes 4 & 5 are to be utilized to perform field tests of the hydraulic conductivity of the existing dike fill material, with a slug test. These piezometers were installed by drilling to 5 m below the existing ground surface and then backfilling with bentonite to the bottom of the existing dike fill. A 1.5 m length of slotted PVC pipe was then installed with sand backfill extending above the slotted section of pipe for 0.15 m. A solid PVC pipe was then installed to surface, with a 1 m bentonite cap above the sand backfill. Hydraulic conductivity of the existing dike fill can then be tested in the field at these two locations. Given the placement of the two bentonite caps, these piezometers should not be influenced by the existing groundwater table and it was anticipated that they would remain dry.

A further discussion of hydraulic conductivity of the existing soils is presented in section 3.6 below.

3.6 SOIL PERMEABILITY CONDITIONS

The DRFM was to provide hydraulic conductivity of the existing and proposed dike fill materials, however as of the date of this report, this information has not been provided. As such, permeability of soils was examined empirically to estimate a range of hydraulic conductivity values for the existing dike fill and underlying fill materials. Given the variability observed in the existing dike fill materials, several different hydraulic conductivity equations were used to generate an estimate for hydraulic conductivity values. These equations use the grain sizes to estimate the saturated hydraulic conductivity of the fill material. The existing dike fill material was generalized as sandy silt with some clay. The results of this analysis are presented in the table below.

Table 6 – Hydraulic Conductivity of Existing Materials

Material	Hydraulic Conductivity (m/s)
Existing Dike Fill	6.0×10^{-8} to 1.4×10^{-5}
Underlying Fill Material	1.2×10^{-8} to 1×10^{-5}

Generally, results for the existing dike fill had saturated conductivity values around 2×10^{-6} m/s and only one model was an outlier with a far lower hydraulic conductivity. To confirm that the estimated hydraulic conductivity values calculated are within a reasonable range, these values were compared against typical hydraulic conductivity values (Heath, 1983) and were found to be within a reasonable range, as shown in Table 7 below.

Table 7 – Typical Hydraulic Conductivity (Heath, 1983)	
Material	Hydraulic Conductivity (m/s)
Fine Sand	$2x10^{-7}$ to $2x10^{-4}$
Silt, loess	$1x10^{-9}$ to $2x10^{-5}$
Till	$1x10^{-12}$ to $2x10^{-6}$
Clay	$1x10^{-11}$ to $4.7x10^{-9}$

The piezometers installed within BH-04 and BH-05 were constructed to allow for in-situ hydraulic conductivity testing to be completed on the existing dike fill soils. At the time that this report was prepared, this in-situ testing was not conducted and may be completed once the final design alignment layout is available for the section of dike situated along Riverside Drive. The section of dike along Riverside Drive may require the side slopes of the dike to be steeper than 3H:1V and may be the critical design case to be assessed. Once the layout for this portion of the dike alignment has been finalized, in-situ hydraulic conductivity testing on the dike fill soils may be completed.

4. GEOTECHNICAL ASSESSMENT

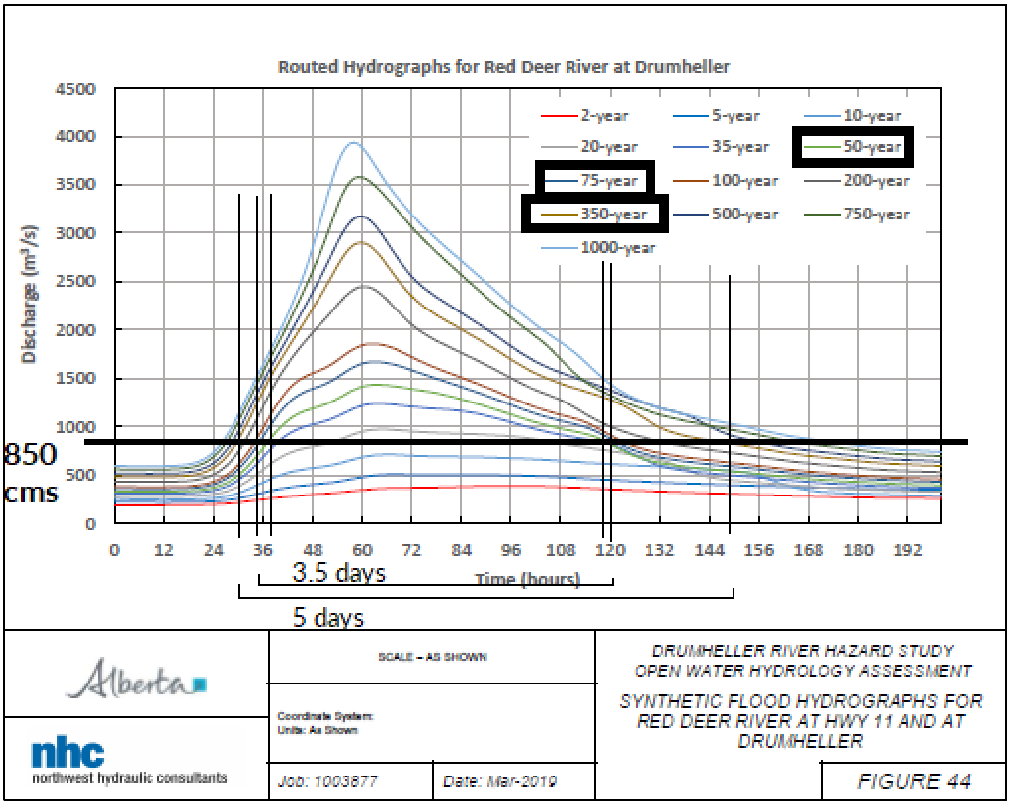
This geotechnical assessment considers the existing dike fill soils and foundation soils in terms of strength, permeability, and density. Seepage/transmissivity and waterside slope stability of the dike were also assessed.

4.1 PREVIOUS FLOOD CONDITIONS AND DURATIONS

Drumheller has experienced at least 11 major flooding events on the Red Deer River since 1900. Recent flood events in 2005 and 2013 provide the only flood events with recorded flow rates and water levels throughout the event. In 2005 the flood waters peaked at a flow of approximately 1,450 m^3/s while in 2013, flows peaked around 1,322 m^3/s. Northwest Hydraulic Consultants (nhc) has modelled flooding on the Red Deer River and estimated that in a flood event of 1,100 m^3/s, failure of the Dike D along Riverside Drive at Second Street would cause inundation of properties (nhc, 2020). During our modeling of the proposed dike and riverbank for transmissivity and slope stability, it was assumed that elevated water levels would be considered and flood conditions present, when flows are greater than 850 m^3/s, which corresponds to an elevation of approximately 681 m at Borehole 6, which is approximately the base of our existing dike. In the 2005 and 2013 event, elevated water levels remained for approximately 2.5 days during both events.

nhc constructed a synthetic hydrograph with different flood events along the Red Deer River at the Town of Drumheller, including flows of 1,870 m^3/s, 2,090 m^3/s, and 3,050 m^3/s. As seen in the annotated figure below, flood flow levels are estimated to remain elevated for approximately 3.4 days for a flow of 1,870 m^3/s, approximately 3.5 days for a flow of 2,090 m^3/s, and approximately 5 days for a flow of 3,050 m^3/s. During modeling, SweetTech has assumed that flood flow levels remain elevated for 7 days to provide a conservative estimate for the risk of failure due to rapid drawdown and to allow for the transmissivity of the dike structure to be assessed. A summary of flood flows and elevated water level durations is provided in Table 8 and can be seen on nhc's hydrograph Figure 44.

Table 8 – Flood Flows and Elevated Water Level Duration		
Flood Event	Peak Flow (m^3/s)	Days of Elevated Flood Duration (> 850 m^3/s)
2005	1,450	2-5
2013	1,322	2-5
Simulated Design Flood	1,870	3-4
Simulated Design Flood	2,090	3-5
Simulated Design Flood	3,050	5



4.2 SOIL DESIGN PARAMETERS

The following material parameters were utilized throughout the seepage and slope stability assessment. During the assessment, cohesion of the soils was neglected.

Table 9 – Material Parameter Summary			
Material	Friction Angle (φ)	Bulk Unit Weight (γ)	Saturated Hydraulic Conductivity (Kx)
New Dike Fill Soil	25°	18.5 kN/m^3	$2x10^{-6}$ m/s
Existing Dike Fill Soil	25°	18.5 kN/m^3	$2x10^{-6}$ m/s
Underlying Fill Soils	27°	18 kN/m^3	$9x10^{-6}$ m/s
Sand/Silt	27°	17.5 kN/m^3	$9x10^{-6}$ m/s
Sand and Gravel	36°	19.5 kN/m^3	$5x10^{-5}$ m/s

Through discussions with the DRFM and Parkland Geotechnical Consulting Ltd., SweetTech was informed to model, assess and design Dike D using a fill soil with a 25˚ friction angle. Upon drilling the existing dike, it has been estimated that the existing Dike D fill soil friction angle is predominantly ≥25˚. Based off laboratory testing and observations made during drilling and test pitting, high plastic soils were only encountered within BH-04, indicating that most of Dike D was constructed using low to medium plastic materials. Once the proposed fill soil has been selected, direct shear testing, hydraulic conductivity, plasticity and dispersity (Crumb) testing is to be performed to confirm the material parameters. For Dike D construction, SweetTech recommends using a fill material with a friction angle ≥25˚ and with a low to medium plasticity. If the materials friction angle is found to be below 25˚, the slope stability assessment will need to be revisited to confirm that the safety factors are within the "Minimum Design Factors of Safety for DRFM Dikes" found within the Geotechnical Design Basis Memo for the DRFM System, issued April 21, 2021, by the DRFM. If a reassessment of the dikes slope stability is required due to the proposed fill materials parameters, it is recommended to complete this reassessment once the final dike alignment has been selected along Riverside Drive. The area along Riverside Drive currently has several design layout options including retaining walls and dike side slopes steeper than 2.5H:1V, and depending on the option selected, additional geotechnical engineering assessment and design details may be required for this dike section.

4.3 SEEPAGE AND PIPING

Seepage through the existing dike was considered using the estimated hydraulic conductivity values for soils with a predicted flood event duration described in Section 4.1. The seepage analysis was performed at one critical section using finite element methods and was performed utilizing SEEP/W software (Geo-Slope, 2021). In a flow event of 1,870 m^3/s, the flood duration is estimated to be approximately 3.5 days. To allow for some conservativism in the modeling of the seepage conditions, a flood duration of 7 days was utilized, with design flows being maintained for 1.5 days. The flood condition was defined as being a flow rate of 850 m^3/s to 1850 m^3/s. Using a hydraulic conductivity of $2x10^{-6}$ m/s for the existing and new dike fill soils, with the specified 7-day flood duration, it was determined that seepage of the flood waters through the typical dike section will not be a significant risk along the understood Dike D alignment. It should be noted that the existing dike material was found to be constructed of moderately variable fill soils and that a range hydraulic conductivity was assessed for the existing and proposed dike fill soils.

Utilizing the estimated hydraulic conductivities of the underlying fill and native soils beneath the subject dike and extending the flood event for a 7-day duration, piping and seepage from under the dike structure was also determined to not occur at the assessed critical section. The groundwater table was found to extend to within 0.5 m from surface on the landside of the dike but did not extend to the ground surface. As the underlying soils are anticipated to have a highly variable fine grained sand content, there is the possibility that piping under the dike will occur where the underlying soils contain a greater than a 50% sand and gravel fraction and higher hydraulic conductivities are present. Additionally, during a flood event, stormwater collecting and runoff from the land side could impact the assumed conditions on the landside of the dike.

Based on the results from the field investigation, laboratory testing, and SweetTech's analysis, transmissivity of the flood waters through the dike should not be a concern unless the defined flood condition (850 m^3/s to 1850 m^3/s) is extended for longer than a 7 day period. Based on the hydrographs from nhc and available historical flood information, these prolonged conditions are not anticipated to occur.

To reduce the risks of under dike seepage, cut-off trenches and sheet-piling have been evaluated by the DRFM and it is understood that these approaches are not currently being considered and that an assessment of these methodologies was not to be performed.

It is to be understood that the provided fill soils for the construction of Dike D must have a maximum hydraulic conductivity of $4x10^{-6}$ m/s as transmissivity of the flood water through and beneath the dike will become a concern when the dike fill soils have a hydraulic conductivity greater than $8x10^{-6}$ m/s. As such, soils containing a high sand and gravel fraction and relatively high hydraulic conductivity will not be a suitable dike fill material.

Additional seepage analysis will be required during detailed design if a retention system is utilized along Riverside Drive.

4.4 SLOPE STABILITY

SweetTech performed a slope stability analysis at the critical section specified on Figure 1 (Station D 0+687). Slope/W geotechnical software (GEOSLOPE, 2021) was utilized to perform the slope stability assessment. The Morgenstern-Price, limit equilibrium method was utilized, and produced circular failure surfaces along the assessed critical section. The circular failure surface was prevented from developing at a depth shallower than 0.8 m below grade as these small failures are difficult to assess due to the variable riverbank topography and can be easily repaired if they happen to occur under rapid drawdown conditions.

The existing dikes constructed with local borrow materials around Drumheller have been constructed to slopes varying from 3H:1V to 2H:1V with varying performance. Steeper slopes in the 2H:1V range have been observed to have surficial cracking (Dike B), while the existing Dike D has only been observed to have longitudinal cracking on the asphalt surface. This cracking of the asphalt surface is attributed to either the condition of the asphalt at the time of placement, volumetric changes from the medium to high plastic soils situated beneath the pathway and possible vehicle traffic loads on the pathway that were beyond the design load and the underlying soils bearing capacity. Some consolidation and fill settlement may have also contributed to the noted damage on the asphalt path but upon reviewing the construction sequence timing of the existing dike, most of this settlement is anticipated to have occurred before the pathway was constructed.

The stability assessment was performed utilizing a dike with 3H:1V side slopes and a 6 m wide dike crest. Cohesion was neglected in the assessment, surcharge loads were not modeled and the flood condition time period was maintained at a total of 7 days. As the worst-case condition for stability of the riverbank and dike is under rapid drawdown conditions, the initial and peak flood condition were maintained for 6 days and then the flood waters were rapidly drawn down over a period of 1 day. This allows for some conservatism in the assessment as the nhc hydrographs show a longer drawdown period during flood events.

Table 10 – Summary of 3H:1V Dike Sideslope Stability	
Model Scenario	Factor of Safety of Entire Escarpment
After 1.5 Days at Flood Peak	2.30
Rapid Drawdown (Midway)	1.34
Rapid Drawdown (End)	1.92
Approximate Steady State	2.56

Based on the performed stability assessment, it was determined that the 3H:1V dike surface was at or slightly above a factor of safety of 1.3 under rapid drawdown conditions. Along much of the alignment of Dike D, the lower portion of the riverbank has become over steepened to a near vertical condition from river flow erosion (in high flow events). This over steepened toe area along the edge of the river was found to typically range from 0.8 m to 2 m in grade change. These over steepened toe conditions were found to impact the resulting safety factors and generally resulted in safety factors under 1.1 when rapid drawdown occurs at the toe of the riverbank. These toe stability safety factors are summarized below on Table 11.

Table 11 – Toe of River Valley Slope Stability	
Model Scenario	Factor of Safety
After 1.5 Days at Flood Peak	1.28
Rapid Drawdown (Midway)	1.19
Rapid Drawdown (End)	1.02
Approximate Steady State	1.45

To avoid these lower safety factors, undercutting of the proposed dike and to avoid further over steepening along the toe of the river valley wall, it is generally recommended to grade and protect the toe of the riverbank utilizing a competent rip-rap material. Where the dike is built within 3 m of the crest of the riverbank, the over steepened bank is to be graded to a 2H:1V and is to be protected with a 1 to 2 m thick layer of keyed-in rip-rap along the toe of the riverbank. This was found to increase the drawdown safety factor to approximately 1.2. This rip-rap placement will allow for the safety factor to increase to above 1.1 under rapid drawdown, as per the "Minimum Design Factors of Safety for DRFM Dikes" found within the Geotechnical Design Basis Memo for the DRFM System. Where the typical dike section is constructed above the riverbank and within 3 m of the crest, The Town of Drumheller could accept the risks of a drawdown induced instability along the toe of the dike and perform critical repairs after a flood event. As the typical dike section consists of a minimum 3H:1V side slopes and a 6 m crest, a complete breach of the dike is not likely with this wider dike section as long as failures are addressed rapidly. Where the typical section is not achieved and is built within 3 m of the crest, keyed-in rip-rap will be required to attain the "Minimum Design Factors of Safety for DRFM Dikes".

Table 12 – Graded and Armored Toe of River Valley Slope Stability	
Model Scenario	Factor of Safety
After 1.5 Days at Flood Peak	1.54
Rapid Drawdown (Midway)	1.34
Rapid Drawdown (End)	1.20
Approximate Steady State	1.46

It is to be understood that the stability assessment performed for Dike D utilized estimated new dike fill material parameters of 18.5 kN/m^3 for the bulk unit weight, 2x10^{-6} m/s for saturated hydraulic conductivity and a 25˚ friction angle. The assessment performed assumed that the dike embankment construction materials consist of a low to medium plasticity, nondispersive clay. If the provided materials friction angle is found to be below 25˚, the slope stability assessment will need to be revisited to confirm that the safety factors are within the "Minimum Design Factors of Safety for DRFM Dikes".

4.5 SEISMIC STABILITY

The seismic hazard of the Drumheller area is anticipated as low by the Geological Survey of Canada. Due to the low-risk susceptibility of seismic instability of the dike, the risk of soil liquefaction is low and no further assessment or consideration for seismic stability was considered in this analysis.

4.6 FOUNDATION BEARING CAPACITY AND SETTLEMENT

The two types of settlement that are typically assessed include immediate and consolidation settlement. It is assumed that immediate settlement would occur during construction as the loads from the construction equipment and fill occur. Consolidation settlements would occur over a longer period as the underlying soils respond to the fill soil placement loads.

The geotechnical investigation confirmed that along the entire Dike D alignment, previously placed fill soils will be situated below the dike structure. Some of these fill soils were found to be placed in an uncontrolled manner with no compaction testing performed. This can be seen from the range of N values attained from Standard Penetration Testing. In addition, the silt and sand layer as well as the localized clay partings encountered beneath the underlying fill soils were found to have a highly variable in-situ consistency and undrained shear strength. The material was generally found to be in a loose condition along the north half of the dike alignment and improved to be in a compact condition for the southern half of the dike. These variable in-situ strength parameters will result in varying degrees of consolidation settlement upon placement and construction of the new dike structure. Settlement of the dike is expected to be primarily caused by compression of the foundation soils, with some settlement of the newly placed fill soils. It should be expected that settlement of new or replaced fill will occur due to "self-weight", particularly where thick fills are placed. For clay fill compacted to 98 percent of the Standard Proctor maximum dry density, the fill settlement is expected to be in the range of 0.5 to 1 percent of the fill height, with most of the settlement occurring during the first freeze thaw cycle.

Along the north half of the proposed dike, consolidation settlement is anticipated to be on the order of approximately 90 mm in the first year with additional settlement of less than 20 mm predicted over the next 10 years. As the underlying fill soils in this area were found to have a highly variable in-situ consistency, differential settlements along the dike alignment from the Aquaplex building to the Badlands Community Facility may be observed within the first year after construction. Along the south portion of the alignment (approx. D0+600 to D1+080), conditions were found to improve slightly and settlements are anticipated to be approximately 75 mm in the first year with additional settlement of less than 20 mm predicted over the next 10 years. Where proposed fills are greater than 2.5 m and where bearing soils encountered are less than those found during the field investigation, settlement magnitudes may exceed the estimates provided above.

From approximately dike station D 1+080 to D 1+195, the topography drops down to a lower elevation along the dike alignment and approximately 4 m of fill is anticipated to be required in this area to complete the dike construction. At the time of the field investigation, this lower area was not accessible to the drilling equipment due to the varying of the topography and the density of the treed area. Upon completion the tree and vegetation clearing in this area, two test pits were excavated and tested for consistency to a depth of 2 m below the current grade elevation. It was determined that this area has a minimum ultimate bearing capacity of 240 kPa. Based off the performed test pit bearing capacity assessment and the results from BH-01, settlement of this area is not anticipated to be a concern for general dike stability. From the two test pits performed in this area, it should be anticipated that approximately 0.75 to 1.0 m of surficial soils will need to be stripped prior to fill placement. This stripping has been accounted for within the 4 m fill thickness estimate. At a minimum, this area is to have the organics and unsuitable surface stripped, and the subgrade is to be scarified to a depth of 200 mm and re-compacted to at least 98% SPMDD. As fill soils in this area are anticipated to be approximately 4 m in thickness, settlements are anticipated to be approximately 80 mm in the first year with additional settlement of less than 20 mm predicted over the next 10 years.

If placement of an asphalt pathway on top of the dike is desired, it is recommended that asphalt placement occur after the dike goes through two freeze and thaw cycles (2 winters) to reduce the risk that settling of the dike causes damage and failure to the asphalt pathway. Alternatively, the pathway surface could be graveled for this initial time period, or a base course of asphalt could be placed with a final asphalt layer placed after two freeze thaw cycles has occurred. High plastic clay fill soils should be avoided beneath the pathway to minimize the risks associated with shrinkage and swelling related movements. Additional measures to ensure that precipitation easily sheds off of the dike materials (and does not get trapped under the pathway) is important to minimize risks of swelling and/or shrinking of the dike fills.

4.7 STRUCTURES IN AND THROUGH DIKE

There are currently electrical utilities, and stormwater outfalls that pass under the existing dike. It is understood that electrical utilities in all areas except by the Gordon Taylor Bridge are planned to remain in place. There are 5 known stormwater cross-drains that extend under the existing/proposed dike alignment. Three of the stormwater cross-drains were found to extend and daylight at or near the rivers typical surface elevation and are out of the anticipated dike footprint. Based off the observations from the time of the field investigation, the remaining two stormwater cross-drains may need to be extended to allow for dike construction. One of the stormwater cross-drains is situated at the west corner of the Drumheller Aquaplex and the other stormwater cross-drain is situated at the southern extent of Dike D, near dike station D 1+174. As the condition of the riverbank is near vertical and heavily eroding for much of the alignment, it is generally recommended to armor the toe of the slope with rip-rap, and this may dictate that additional stormwater outfalls may require extension and or improvements.

4.8 PROPOSED DIKE FILL SOIL REQUIREMENTS

During the slope stability and seepage assessment, it was determined that transmissivity of flood water through the dike would generally not be a concern assuming that the provided fill soils are very similar to the existing dike fill soil and flood durations do not extend beyond a 7-day period. During our assessment it was determined that transmissivity of the flood water through and beneath the typical dike section would be a concern if the fill soils were of a hydraulic conductivity of $8x10^{-6}$ m/s or greater. Based off these findings, SweetTech strongly recommends that the fill material being provided has a maximum hydraulic conductivity of $\leq 4x10^{-6}$ m/s.

The slope stability assessment assumed that the proposed dike fill soils would have material parameters of 18.5 kN/m^3 for the bulk unit weight, $2x10^{-6}$ m/s for saturated hydraulic conductivity and a 25˚ friction angle. The assessment performed assumed that the dike embankment construction materials consist of a low to medium plasticity, nondispersive clay. Once the proposed fill soil has been selected, laboratory testing is to be performed to confirm the materials friction angle, saturated hydraulic conductivity, plasticity, compacted unit weight and dispersivity. SweetTech recommends using a material with a friction angle ≥25˚ and with a low to medium plasticity. If the materials friction angle is found to be below 25˚, the slope stability assessment will need to be revisited to confirm that the safety factors are within the "Minimum Design Factors of Safety for DRFM Dikes". If some dispersive clays are proposed to be utilized, SweetTech can provide recommendations regarding blending of this material, where to use the material within the dike section and techniques that can be employed to significantly reduce the potential risks of internal and external erosion due to dispersive soils (e.g., non-dispersive clay soil cover, vegetation and geotextile utilization).

4.9 RETAINING WALL RECOMMENDATIONS

The local fill material utilized to construct the existing dikes is not suitable or sufficient for construction of retaining walls in the areas identified in Figure 1. These areas will require material with a tighter tolerance for their hydraulic conductivity and a higher residual friction angle. The minimum friction angle (ϕ) for this material is $\phi >= 28^{o}$ and the maximum hydraulic conductivity for this material is $2x10^{-6}$ m/s. A reworked clay till fill would be ideal for the construction of the retaining wall backfill areas. It is generally recommended to construct the retaining walls utilizing Mechanically Stabilized Earth (MSE) construction methods and materials. Most facing units are anticipated to be adequate for the height of wall anticipated, although the emergency response loading condition may require the use of a larger retaining wall block product. Large block facing units generally can accommodate higher live loads and loading conditions situated closer to the back of the retaining wall block. As it is understood that we are to exercise consideration for adaptive emergency response management allowing for an emergency dike raise to manage flows of 2,100 m^3/s and greater, it is anticipated that large trucks transporting fill soils would be traveling on the surface of the dike during a flood event. Large retention blocks should allow for these emergency response live loads to be situated relatively close to the back of the wall with minimal displacement to the facing units. When surficial live loads are situated behind a smaller block retaining wall, pull-out of the upper geogrid layers can occur, and subsequent failure or displacement of the wall system.

5. DISCUSSION

Based on the findings of this geotechnical assessment, it is SweetTech's opinion that the proposed Dike D embankments evaluated generally meet the slope stability, seepage, and settlement requirements. Near the river elevation, lower toe stability safety factors were observed and where the typical dike section is constructed above the riverbank and within 3 m of the crest, these lower safety factors should be addressed. If the toe of the riverbank is graded and is adequately armored with rip-rap, safety factors along the toe of the riverbank could be raised to above 1.1 under rapid drawdown conditions, as per the "Minimum Design Factors of Safety for DRFM Dikes". Once the DRFM has selected the fill material for the construction of Dike D, SweetTech may need to re-evaluate the geotechnical aspects of this project if the material parameters do not meet the requirements outlined within this report.

SweetTech based this report upon the project and documents as described and the information obtained from the exploratory borings and test pits performed during the geotechnical field investigation. The findings and recommendations in this report were based upon data obtained from a limited number of borings, test pits, laboratory testing, observations and information from reports performed by others. Differing geotechnical or geological conditions can occur within small distances and the information attained during our investigation was specific to each location explored. The explored locations may not completely define the subsurface conditions throughout the dike alignment. SweetTech may re-evaluate the conclusions and recommendations presented in this report if the Town of Drumheller or its representatives finds geotechnical conditions that differ from those described herein. This report was prepared only for use by those parties named or described within this report.

6. CLOSURE

We trust this geotechnical report meets your present requirements. This report incorporates and is subject to our attached conditions and limitations. Should you have questions or concerns, or require any additional information, please do not hesitate to contact our office at 587-329-6655.

SweetTech Engineering Consultants
APEGA Permit to Practice #P13638

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