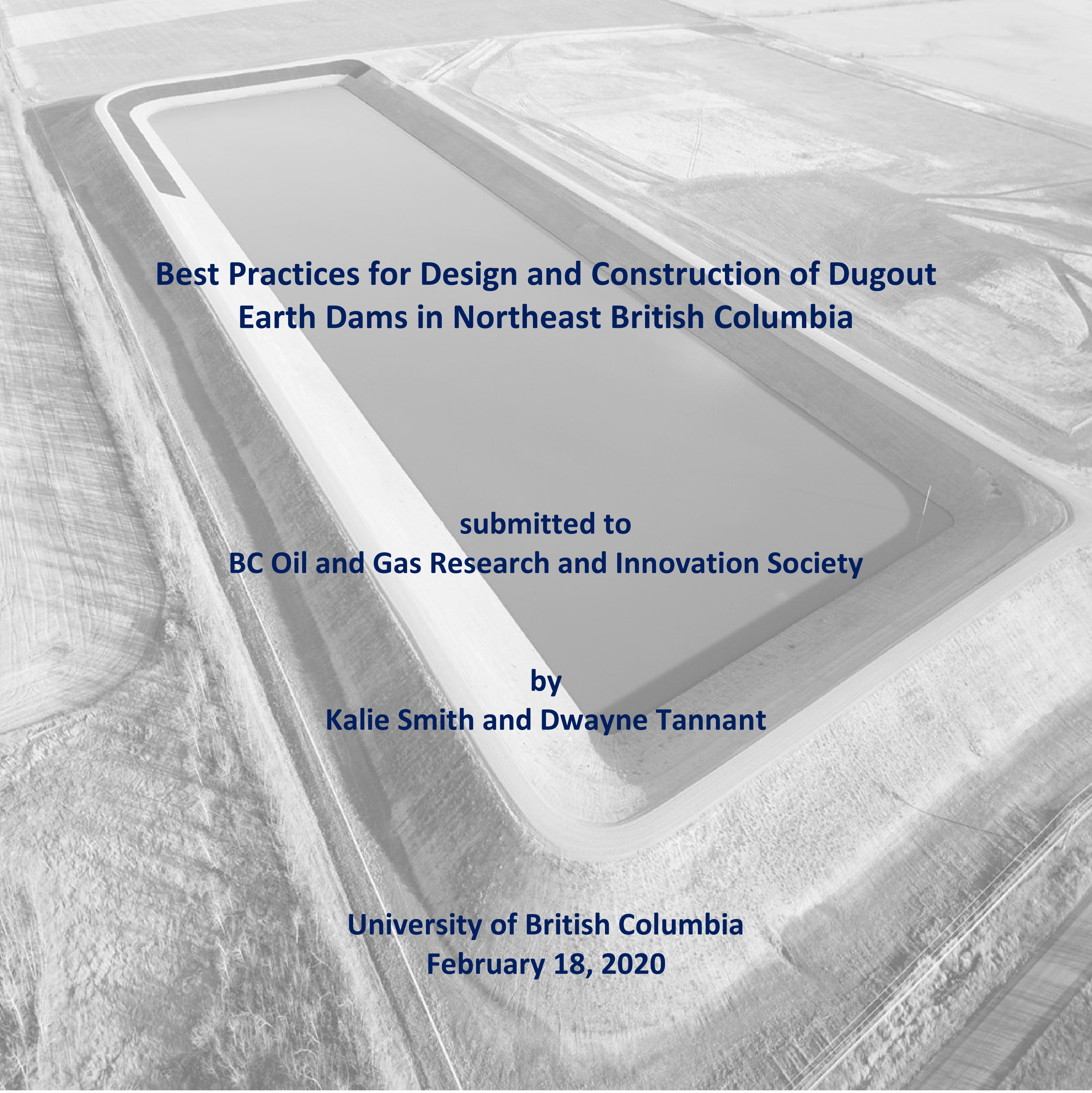

Best Practices for Design and Construction of Dugout Earth Dams in Northeast British Columbia

submitted to
BC Oil and Gas Research and Innovation Society

by
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February 18, 2020

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Executive Summary

A research project was conducted to review best practice documents related to the design and construction of small "dugout" earth dam embankments. The project also involved an examination of seven dams in the Peace River Regional District. Site visits were conducted in August 2018 and May 2019. A progress report was submitted to BC OGRIS in January 2019. The focus of the site visits was to discuss dam and reservoir construction with dam owners, regulators, and contractors, document observations, and make comparisons between as-built conditions and dam designs plus existing best practice guidelines.

The project objective and the purpose of this report is to summarize the recommendations for best practices for the design and construction of dugout earth dam embankments for freshwater storage in northeast BC. The best practice recommendations are organized into seven key aspects of dam design and construction: embankment geometry and stability, hydrotechnical considerations, internal seepage, surface erosion protection, construction, appurtenant structures, and maintenance.

Acknowledgments

The research presented in this report was funded by the BC Oil and Gas Research and Innovation Society (BC OGRIS). The project received guidance from Brian Thomson from BC OGRIS, and Gouri Bhuyan and Dung Nguyen from the BC Oil and Gas Commission. Adam Leece (Encana Services Company Ltd.), Wesley Ferris and Lee Martin (Higher Ground Consulting Inc.), Devon Aaroe (City of Dawson Creek), Robert McLean (BC Ministry of Forests, Lands, Natural Resource Operations, and Rural Development) and Ken McLean (BCOGC) provided information on the case study dams and facilitated site visits. The photographs in this report were taken by Kalie Smith unless mentioned otherwise. The cover photo was provided by the City of Dawson Creek.

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1 Background

The BC Water Sustainability Act does not define a 'dam'. However, the BC Dam Safety Regulation (DSR) defines a dam as a barrier constructed to enable the storage or diversion of water diverted from a stream or aquifer, or both, and other works that are incidental to or necessary for the barrier (BC MFLNRO, 2016).

The dam height, storage capacity, and failure consequence classification determine which parts of the BC Dam Safety Regulation (DSR) apply as illustrated in Figure 1 (BC MFLNRO, 2016). Section 2(1) of the DSR defines a 'minor dam' as less than 7.5 m in height and limited to impounding a maximum live storage volume of water in the reservoir of 10,000 m³ at the full supply level. Minor dams are exempted from the requirements of the DSR except where the comptroller or a water manager believes the dam is or may become potentially hazardous to public safety, the environment, or land or other property. Most dugout dams are less than 7.5 m high, but most of these dams have a live storage capacity of more than 10,000 m³. Thus, these structures are subject to DSR requirements and require licensing under the Water Sustainability Act. The height of a dam is measured from the crest to the lowest downstream elevation.

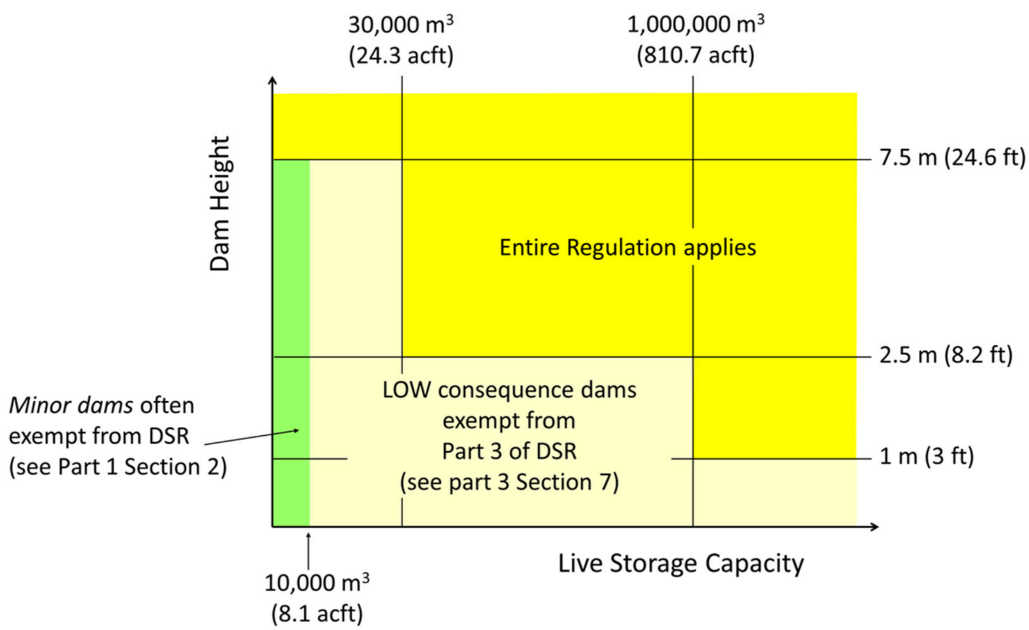

Figure 1. Embankment height and reservoir volume criteria for the BC Dam Safety Regulation.

Figure 2 shows cross-sections through two different typical configurations for an embankment dam and dugout reservoir. The live storage portion of the reservoir is the portion that could drain away if the embankment becomes fully breached.

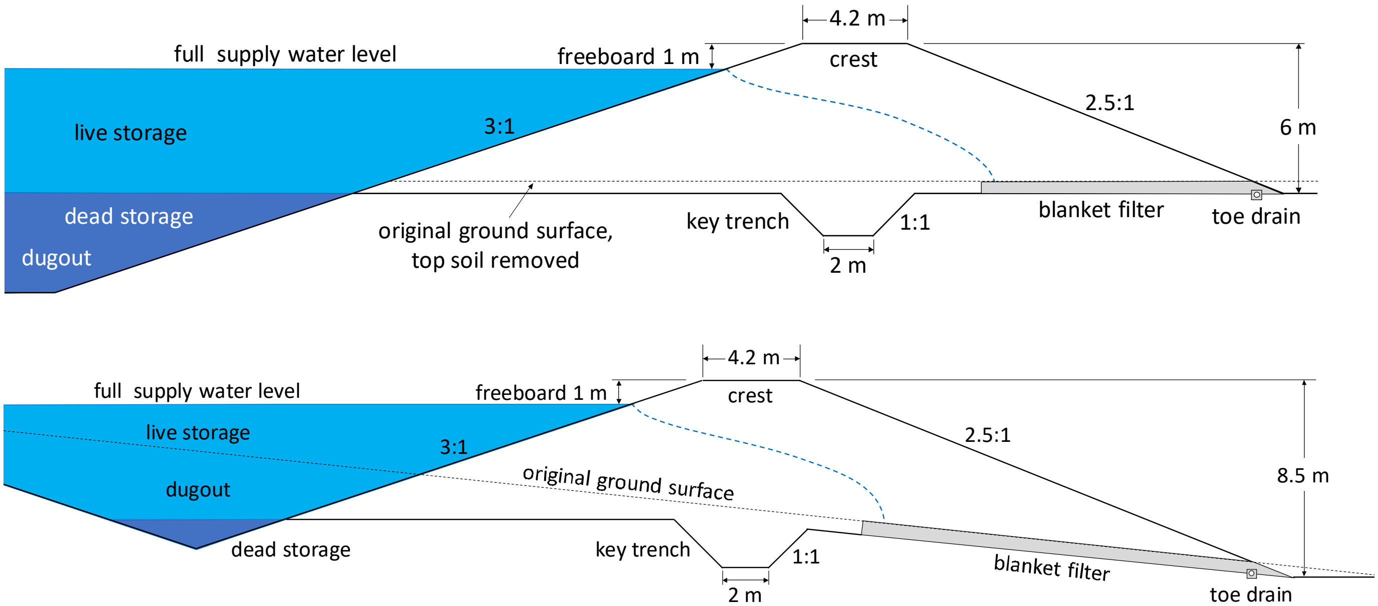

Figure 2. Live and dead reservoir storage in a partially dug out reservoir behind embankment dams on flat or sloping ground.

When a dugout dam is constructed on flat ground, the embankment forms a ring around the dugout. When a dugout dam is constructed on sloping ground, the embankment often forms a C-shape around the dugout. In both cases, there is a desire to use the materials excavated from the dugout to construct the embankment. Often a larger volume is excavated than can be used to construct the dam, and the excess materials are typically stockpiled near the dam. Figure 3 shows two typical examples of these types of structures.

Dams that fall under DSR requirements must undergo a consequence classification that will dictate the frequency of inspection and surveillance and the frequency for dam safety reviews. Dam owners must develop documentation that includes plans for operations, maintenance, and surveillance, and emergency preparedness (BC MFLNRO, 2016). Some existing dams constructed before recent changes to the legislation now fall under DSR requirements.

The purpose of this report is to summarize the recommendations for best practices for the design and construction of dugout earth dam embankments for freshwater storage in northeast BC. The best practice recommendations in this report are organized into seven key aspects of dam design and construction: embankment geometry and stability, hydrotechnical considerations, spillways and outlets, seepage and drainage, erosion protection, construction, and maintenance.

Figure 3. Examples of dugout earth dams on flat and sloping ground (photos: Ben Parfitt & Garth Lenz, The Narwhal, Oct. 22, 2018).

As part of the research, seven dams were identified for site visits and discussions with the design consultants, owners, and regulators. Observations and data collected from these dams are included to supplement recommendations, and these are compared with existing best practice guidelines. These dams provided information on as-built conditions, design specifications, dam performance, and existing practices. A summary of these dams is given in Table 1. The dams are located within the Peace River Region, as shown in Figure 4. The dam numbers are arbitrary.

Figure 4. Locations of seven dams that were investigated.

Table 1. Summary of investigated dams.

Dam	Max. Height (m)	Live Storage (m^3)	Classification	Age (Years)	Soil Type	Slope* U/S (H:V)	Slope* D/S (H:V)	Regulator
1	9.1	64,060	significant	7	CL	3.3:1	2.3:1	BC OGC
2	7.7	75,517	significant	7	CL	2.7:1	3:1	BC OGC
3	6	200,000	high	1	CL	2.7:1	3.2:1	BC OGC
4	5.3	161,800	high	3	CH	2.7:1	4:1	BC OGC
5	11.3	$1.03x10^6$	high	2	CL	3:1	2.5:1	BC MFLNRORD
6	12	379,000 (189,500 x2 cells)	high	44	CL	3:1	3:1	BC MFLNRORD
7	9.6	107,000	significant	3	CL	3:1	3:1	BC MFLNRORD

* As-built measured slopes

2 Dam Geometry and Stability

2.1 Embankment Slopes

BC MFLNRORD (2018) recommends embankment slopes to be constructed no steeper than 3H:1V (upstream) and 2.5H:1V (downstream) unless an evaluation of embankment stability gives adequate factors of safety for steeper slopes. The recommended minimum upstream and downstream embankment slopes from a variety of other best practices documents are summarized in Table 2. There is a consistent recommendation for the maximum upstream slope to be 3:1. For the downstream slopes, the recommendations range from 2:1 to 3:1.

Table 2. Recommended maximum embankment slopes (H:V) from best practice guidelines.

Source	Upstream Slope	Downstream Slope
BC MFNLRORD (2018)	3:1	2.5:1
United States Bureau of Reclamation (2011)	3:1	2.5:1
Eyre Peninsula Natural Resources Management Board (2011)	3:1	3:1
United Nations Food and Agriculture Organization (FAO) (Stephens, 2010)	3:1	2:1
Department of Primary Industries and Water of Tasmania (2008)	3:1	3:1
U.S. Army Corps of Engineers (2004)	3:1	3:1
BC MEM (2002)	3:1	3:1
PFRA (1992)	3:1	2.5:1 or 2:1

The BC MEM (2002) states that grass slopes maintained by tractors or other equipment should not be steeper than 3H:1V. This criterion is for the safety of the operator cutting the grass rather than the dam stability.

Lewis (2014) links the maximum embankment slope to the soil types that the embankment is made of and sits on. The soil types are based on the widely used Unified Soil Classification system. He also considers cases where the embankment is expected to incur rapid drawdown under routine operations. His recommendations are shown in Table 3. The most common soil type at the seven dam locations is silty clay of low plasticity (CL), as seen in Table 1. The slope recommendations in Table 3 are for dams with limited to no engineering and limited quality control during construction. Nevertheless, they indicate that the embankment slope angles should decrease as the clay content or plasticity increases and as the dam height increases. The BC MFLNRORD (2018) requirements are steeper than those list in Table 3 for dams above 3 m in height, suggesting that a higher level of engineering design and construction quality control would be required to ensure embankment stability.

Table 3. Recommended maximum embankment slopes (H:V) for agricultural dams (Lewis 2014).

Dam Height (m)	Slope	GC GM SC SM	CL ML	CH MH
0 to 3	upstream	3:1	3.5:1	4:1
	downstream	2:1	2.5:1	2.5:1
3 to 7	upstream	3.5:1	4:1	4:1
	downstream	2.5:1	3:1	3:1
7 to 10	upstream	3.5:1	4:1	4:1
	downstream	3:1	3.5:1	3.5:1

Field observations during the site visits are consistent with the slope recommendations provided by BC MFLNRORD (2018). For example, Figure 5 shows an aerial photograph of a dam where there are longitudinal cracks on the downstream slope. The as-built slope at this location is too steep at approximately 2H:1V. At other locations around the same dam where the slopes are closer to the design slope of 3H:1V, longitudinal cracking was not observed.

Figure 5. Longitudinal cracking on a 2H:1V slope on the downstream side of the embankment (orthomosaic provided by Higher Ground Consulting).

2.2 Crest

BC MFLNRORD (2018) prescribes a minimum crest width as a function of embankment height using

$$W = 0.2H + 3 \tag{1}$$

where W and H are the crest width and embankment height in metres. Other recommendations for the crest width are listed in Table 4. For the smaller embankments typically associated with dugouts, the recommended crest width is given by Equation 1. A minimum crest width is usually specified so that machinery can work safely on the crest.

Table 4. Recommended embankment crest width from best practice guidelines

Source	Equation	Min. W (m)
BC MFLNRORD (2018)	$W = 0.2H + 3$	3
Lewis (2014)	$W = \sqrt{H} + 1$	2.5
Stephens (2010)	$W = 0.4H + 1$	3
PFRA (1992)	NA	3

All dams that were investigated were designed to have a uniform crest width around the reservoir despite a variation in the crest elevation. For one dam with a maximum height of 6 m, the measured crest width is 4.2 m, which is consistent with Equation 1. Another dam (H = 9.6 m) has an as-built crest width of 8 m, which exceeds the recommended minimum of 4.9 m.

Embankments should be constructed with a uniform crest width based on the maximum dam height regardless of variations in height around the reservoir. A dam crest should have no flat or low spots where water can pond. It is also recommended that crests be sloped to drain into the reservoir at a slope of at least 2%. Stephens (2010) recommends that the crest be inclined to drain toward the reservoir at a slope of 2.5%.

2.3 Slope Stability

The Canadian Dam Association (2007) prescribes the minimum factors of safety for slope stability under different loading conditions, regardless of the consequence classification. These factors of safety are listed in Table 5. Two-dimensional limit equilibrium slope stability analysis methods are often used to calculate the factors of safety. For this purpose, the most critical design cross-section is selected. The critical cross-section typically has the highest and steepest embankment height.

Table 5. Minimum factors of safety for slope stability.

Loading Condition	Minimum Factor of Safety (CDA, 2007)	Ontario Ministry of Natural Resources (2011)
End-of-construction before reservoir filling	1.3	1.3
Steady-state/long-term	1.5	1.5
Inflow design flood	NA	1.3
Rapid reservoir drawdown	1.2-1.3	1.2-1.3
Pseudo-static seismic	1.0	1.0
Post earthquake	NA	1.1

While the as-built or design geometry of the dams is relatively well constrained, the shear strength and consolidation properties of the soils in and under the dam are not. The typical soils used to construct dugout dams in the Peace River region contain a high percentage of clay, which results in low hydraulic conductivity and relatively low shear strength compared to dams with a higher percentage of silt and sand. The foundation for the dams may consist of normally- or over-consolidated soils and these will have different properties. The soil properties have important implications for dam stability. It can take years for steady-state seepage to become established through the dam, and it can take years for consolidation settlements to occur within and underneath the dam.

The shear strength properties used for the stability analysis should be appropriate for the relevant type of analysis: total stress or effective stress analysis. Pore pressures generated by construction, seismic loading, or rapid drawdown conditions will take a long time to dissipate in the clay-rich soils. Therefore, an undrained loading analysis and total stress shear strength properties should be used to analyze the dam stability for end-of-construction, rapid drawdown and seismic loading conditions. The undrained shear strength of the soil can also be used. Stability analysis for steady-state seepage conditions should be performed using effective stress shear strength properties. Obtaining reliable soil properties requires advanced soil testing methods. For total stress properties, unconsolidated-undrained and consolidated-undrained triaxial tests are recommended. If pore pressures are measured within the test specimen, then effective stress properties can also be determined. Consolidated-drained triaxial testing is not practical given the long time needed for pore pressure dissipation in the low permeability clay specimens.

During construction, the addition of the weight of the embankment will likely generate excess pore pressures in the foundation soils for the dam and possibly also within the embankment itself. As the pore pressures dissipate and the soils consolidate, the stability of the dam should increase. This process may take years to occur. The important implication is that a dam's stability under normal operations should improve over time as the pore pressures dissipate.

The dams that were investigated had limited data on the geotechnical properties of their soils, and hence empirical correlations were used to estimate some properties required for stability analyses. These are given in the thesis by Smith (2019). Stability analyses were conducted for dams 1 to 4. The results indicated that these dams met the required minimum factors of safety. However, due to the significant uncertainty in the soil properties and pore pressure distributions, and there is low confidence in these results.

Lack of high-quality geotechnical properties of soils is common in practice and will continue to affect confidence in stability calculations. Therefore it is prudent for dam owners to limit the steepness of the embankments to values that have proven to be adequate from experience. The dam regulators can assist dam owners by collecting and summarizing case histories of good and poor dam performance to establish a database of experience to guide future best practices.

3 Freeboard and Design Flood

3.1 Freeboard

The CDA recommends calculating two different freeboard requirements, normal and minimum freeboard. In general, freeboard is determined based on the principle that 95% of the waves caused by the most critical wind would not cause overtopping when the reservoir is at the maximum level (CDA, 2007). For normal freeboard, the reservoir is taken at the full supply level. For minimum freeboard, the water level is assumed to be that corresponding to the passage of the IDF. Wave height depends on wind speed, and wind data for long return periods are not available. Therefore wind velocities are determined using a statistical technique (CDA, 2007). The USACE (1980) and ICOLD Bulletin 91 (1993) provide examples of some methods for determining the wind set-up and wave run-up to establish the design wave height to prevent dam overtopping.

BC MFLNRORD (2018) recommends that a minimum normal freeboard be 1 m when used in combination with a spillway width of at least 4 m. For narrower spillways, the freeboard must be increased.

The FAO of the United Nations (Stephens, 2010) provides a simple equation to calculate freeboard that depends only on fetch length: $H = 0.014\sqrt{F}$, where *H* is the freeboard in metres and *F* is the fetch length in kilometres. Dugout dams typically have a fetch of less than 1000 m, which yields a freeboard of 0.44 m. One of the investigated dams is used as an example to evaluate freeboard requirements using the methods outlined in the CDA Technical Bulletin for Hydrotechnical Considerations (2007). The maximum wind speed at the Dawson Creek airport recorded in the 1981-2010 Canadian Climate Normals (Environment Canada, 2010) is 87 km/hr. Wind data can be extrapolated to obtain wind speeds at longer return periods, which apply for dam designs. The sum of the calculated design wave height, wind set-up and wave run-up is 0.83 m. Therefore, the 1 m minimum recommended freeboard (BC MFNLRORD, 2018) should be sufficient to handle waves in dugout dam reservoirs. PFRA (1992) developed a graph to guide the choice of a proper freeboard, as shown in Figure 6. This figure also indicates that a 1 m freeboard should be adequate for most dugout dams.

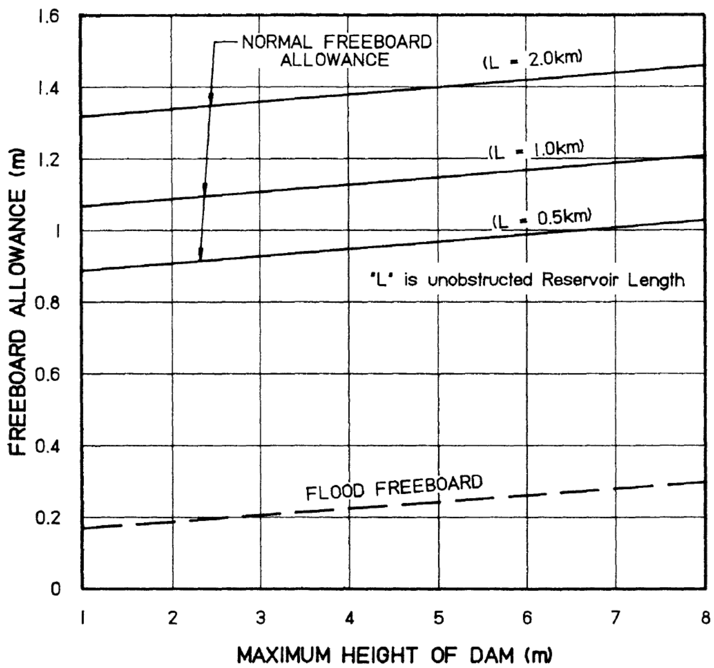

Figure 6. Freeboard versus dam height for various reservoir length (PFRA, 1992)

CDA Technical Bulletin for Hydrotechnical Considerations (2007) also states that the thickness of the material covering the impervious core of an embankment should be sufficient to avoid freezing of the core in winter. Dugout dams are typically homogeneous embankments with no core, but the CDA guidelines suggest that the freeboard should be sufficient to prevent frost penetration to a depth lower than the full supply level elevation. The goal is to prevent cracks from melted ice being located where seepage passes through the embankment. If winter winds blow insulating snow off of the dam crests, the depth of frost penetration may exceed 1 m. Thus, the dam freeboard may need to be greater than 1 m.

3.2 Inflow Design Flood

Dams must be able to retain the reservoir at the design full supply level with an adequate amount of freeboard (CDA, 2007). Depending on the consequence classification of the dam, the Inflow Design Flood (IDF) is determined based on an annual exceedance probability (AEP) or as a function of the Probable Maximum Flood (PMF) based on the CDA technical bulletin on Hydrotechnical Considerations (CDA, 2007). These are summarized in Table 6.

Table 6. Recommended IDF based on dam consequence classification (CDA, 2007).

Consequence Classification	Recommended IDF
low	1/100-year
significant	Between 1/100 and 1/1000-year
high	1/3 between 1/1000-year and PMF
very high	2/3 between 1/1000-year and PMF
extreme	PMF

The suggested IDF prescribed by the CDA (2007) should be used. However, statistical flood analysis is subject to uncertainty when data are limited, and CDA discourages extrapolating precipitation data beyond a 1000-year return period storm event. A hydrologist will use statistical methods to determine the precipitation for the IDF (CDA, 2007).

Precipitation data from Environment Canada weather stations can be used (Environment Canada, 2014). However, these data include all forms of precipitation as water equivalent. To be conservative, these values can be assumed to occur entirely as rainfall on existing snowpack in late spring, which then melts and contributes to runoff. Therefore, snowmelt should be added to extreme rainfalls to compute runoff for a storm with a given return period. For small watersheds with low infiltration, the Rational Method formula can be used to estimate runoff for a given precipitation intensity.

The CDA recommends calculating two different PMF's, one for summer-autumn, and one for spring, as it is unlikely that both rainfall and snow accumulation would be simultaneously extreme. However, Alberta Transportation (2004) states that in Canadian Prairie climates, the late summer PMF caused by convective storms usually exceeds the early spring PMF. The Probable Maximum Flood Estimator for British Columbia (Abrahamson, 2010) does not distinguish between seasons.

In general, the catchment area for the dugout reservoirs is small. Five of the investigated dams had catchment areas that only consisted of the reservoir surface area and part of the dam embankment and crest sloping into the reservoir. Therefore, the most common dam failure mode for earth dams, which is caused by overtopping, should be relatively rare due to the typical lack of a significant watershed providing inflow to dugout dam reservoirs.

For dams with a watershed larger than the reservoir surface area, evaluation of hydrotechnical aspects and the capacity of appurtenant structures should be based on the size and slope of the catchment area. There is a linear relationship between drainage area (or reservoir surface area) and IDF (runoff) because the precipitation intensities for a given return period storm are similar within a small geographic region. One of the investigated dams has ditches directing runoff water into an open inlet channel lined with riprap (Figure 7). This dam had a small external catchment area. If water is collected from a catchment area adjacent to the dam, the water should enter the reservoir along a riprap-lined channel with adequate flow capacity.

Figure 7. Inlet channel for runoff from an external catchment area.

4 Spillways and Outlets

4.1 Spillway

The BC MFLNRORD (2018) recommends that dams have at least one spillway that can pass the IDF and maintain adequate freeboard. Ideally, the spillway should be constructed on undisturbed ground and protected against erosion or damage from debris build-up. When the dam consists of a constructed embankment fully surrounding a reservoir, then the spillway must be constructed within the dam. The elevation of the highest point in the spillway should maintain the minimum freeboard requirements of the dam under IDF conditions (CDA, 2007). It is recommended that no pipes be used as a spillway.

The Alberta Ministry of Agriculture and Forestry (2015) gives maximum spillway side slopes of 2H:1V, but slopes of 4H:1V are preferred. PFRA (1992) recommends side slopes of 3H:1V. When geotextile and riprap are placed on the slopes, a 2H:1V side slope is usually acceptable.

It is a best practice to locate spillways within the reservoir such that they are located opposite from the predominant wind direction to minimize debris and ice from blowing into the spillway. Spillways should also be located such that they are not at the location of maximum embankment height or where dam breaching at the spillway would have significant downstream consequences.

The minimum width of a spillway should be 4 m (BC MFLNRORD 2018). The Eyre Peninsula Natural Resources Management Board (2011) also requires a minimum spillway inlet width of 4 m and requires an outlet width of 6 m, but this is dependent on the anticipated peak discharge and channel slope.

The discharge capacity of a trapezoidal riprap lined spillway with a 4 m bottom width, 2H:1V side slopes, and 1% channel gradient and a 1 m flow height (corresponding to a 1 m freeboard) is approximately 12 m^3/s (assuming Manning's Roughness Coefficients is 0.04). This value increases to 17 m^3/s for a 2% channel gradient. A 4 m wide spillway can safely pass the IDF for all the investigated dams due to their small watersheds.

There should be no objects or structures that can block a spillway or limit its capacity, such as an access road. The BC MFLNRORD (2018) guidelines state that pipes are not to be used as spillways. One of the investigated dams had an access road crossing the spillway channel (Figure 8), and two 760 mm diameter corrugated steel pipe culverts were placed beneath the access road embankment, in the spillway channel (Figure 9). The road and culverts reduced the spillway channel capacity below the IDF.

Spillway channels should direct water coming out of the reservoir away from the embankment. The discharged water should be directed to a channel similar to that used for a low-level outlet. The channels should be lined with riprap (Figure 10). The design of riprap to prevent channel erosion differs from riprap for wave erosion protection. The riprap design should be based on the IDF runoff flow and the channel slope. The design of channel riprap is outlined in the Riprap Design and Construction Guide (BC MELP, 2000), which closely follows the method recommended by the USACE (1991). The USACE (1991) method assumes that the channel side slopes are no steeper than 2.5H:1V and the unit weight of the rock is 25.9 kN/m^3. Riprap should not be placed directly on top of embankment materials; a layer of filter gravel or appropriate geotextile should be placed between the riprap and embankment material, which is not considered in the USACE method. The geotextile provides additional erosion protection for the spillway channel.

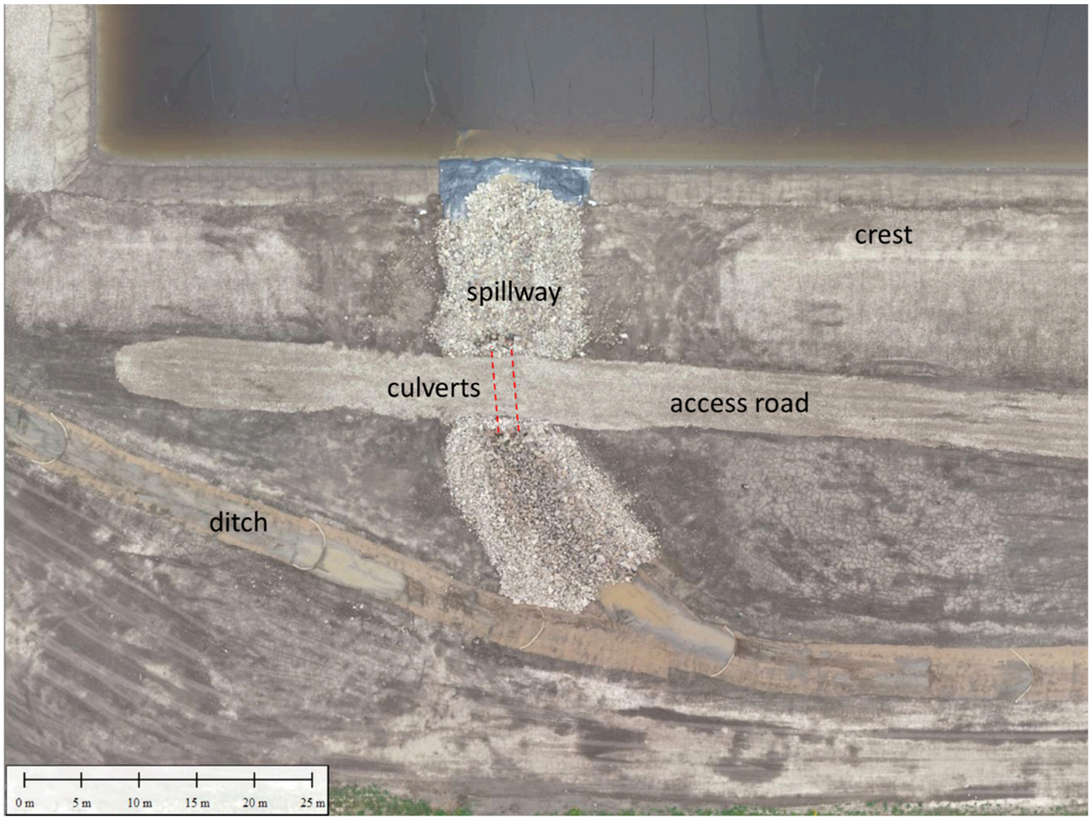

Figure 8. Access road crossing a spillway channel.

Figure 9. Culverts in a spillway channel reduce the channel capacity below the IDF.

Figure 10. Riprap lined channel below a spillway.

4.2 Low-Level Outlet

If a dugout reservoir has a low-level outlet pipe, the pipe should have a minimum diameter of 600 mm and a 2% slope away from the reservoir to promote drainage (BC MFLNRORD 2018). Gates on low-level outlets should be placed on the upstream side of the dam, to prevent a buildup of pressure in the low-level outlet, and to keep the outlet clean and dry, preventing both debris and ice buildup (BC MFLNRORD, 2018).

The Department of Primary Industries and Water of Tasmania (2008) requires low-level outlets and recommends HDPE pipe material, although BC MFLNRORD (2018) suggests using concrete. The material type for the low-level outlet should depend on the anticipated lifespan of the dam. If 20 years is an expected lifespan for dugouts for oil and gas purposes, a corrugated steel pipe may be adequate. For lifespans beyond 20 years, other options should be considered, such as concrete pipes.

A low-level outlet pipe can create a potential area for concentrated seepage through the embankment. To reduce risks associated with piping failure of the embankment, careful construction and compaction practices are required to ensure uncontrolled seepage and migration of fine soils are prevented (USSD, 2011). The use of anti-seepage collars around low-level outlet pipes was recommended in many small dam engineering manuals. These consist of concrete or steel flanges that are placed along the pipe. However, current best practices recommend against the use of anti-seepage collars because it is difficult to achieve adequate soil compaction around the complex geometry associated with these structures.

Three best practices to control and protect against uncontrolled seepage and migration of fine soils are: 1) fully encase the pipe within a concrete structure with a square cross-section, 2) place a compacted sand filter around the downstream portion of the low-level pipe, 3) use bentonite coated aggregate in selected zones along the pipe. A square cross-section for the pipe facilitates good compaction of the soil below, beside, and above the structure. A sand filter can stop the movement of fine soil particles if a seepage channel occurs. Bentonite coated aggregate can be compacted around the concrete-encased pipe to create a modern version of an anti-seepage collar that does not suffer from compaction challenges.

The exit of a low-level outlet must occur within a channel directed away from the embankment and protected from erosion. Riprap should be placed over geotextile to prevent surface erosion and to dissipate the energy in the flowing water exiting the low-level outlet. Figure 11 shows a low-level outlet that should have used even more riprap to surround the outlet pipe.

Figure 11. Low-level outlet directing flow into a channel lined with riprap and geotextile.

5 Seepage and Drainage

5.1 Blanket Drain

The use of a granular filter is not required in existing best practice guidelines for smaller homogenous earth dams. Nevertheless, blanket drains or filters are recommended to manage internal seepage for embankments higher than approximately 4 m. PFRA (1992) suggests a blanket drain is required for embankments higher than 2 to 3 m. A filter increases the factor of safety for the embankment stability under steady-state seepage conditions because it facilitates drainage in the embankment and reduces pore pressures.

The blanket drain should slope at least 2% towards the downstream toe to direct seepage through the filter to the downstream toe. The thickness of the filter material should be 400 to 600 mm. PFRA (1992) recommends a 600 mm thickness of the filter material. The length of the blanket drain depends on the embankment height. The longer that a blanket drain extends into the embankment, the lower the phreatic surface. Figure 12 shows an example where the blanket drain is only used where the embankment was higher than approximately 2 m above the toe elevation. For the same dam, Figure 13 shows how the blanket drain lowers the phreatic surface.

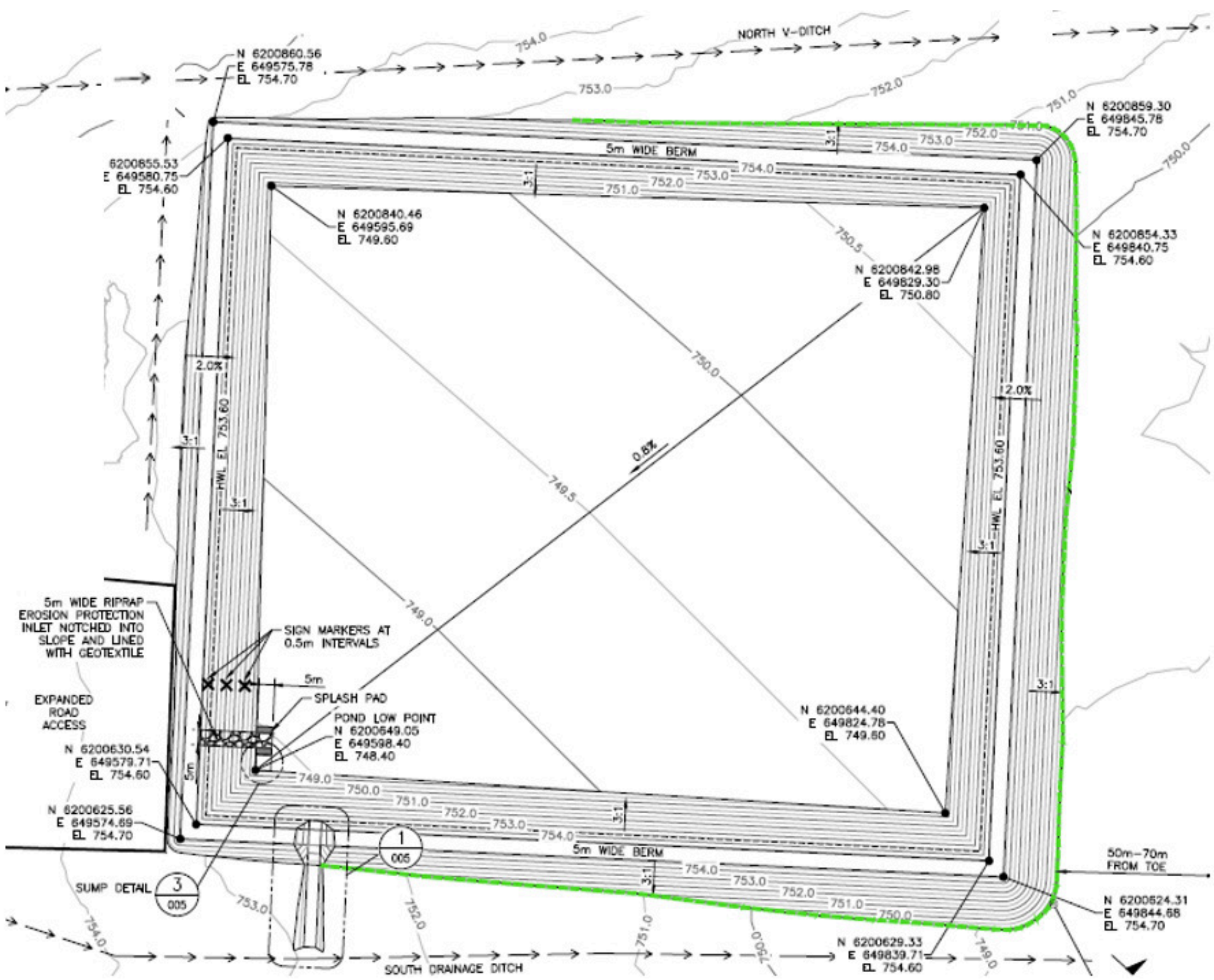

Figure 12. Blanket drain indicated by a green line around a dugout dam with varying embankment height (drawing provided by Higher Ground Consulting).

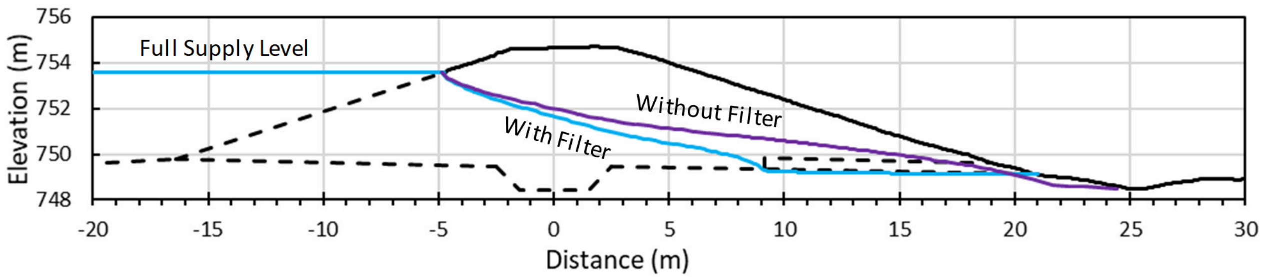

Figure 13. Blanket drain lowers the phreatic surface in the embankment.

Blanket drains normally consist of clean granular filter material (coarse sand or gravel). When the filter soil is in contact with clay-rich soil, the two soil types should be separated by a non-woven geotextile. Figure 14 shows an example of a blanket drain during construction.

Figure 14. Large blanket drain during construction (photo courtesy of the City of Dawson Creek).

Blanket drains have an additional benefit by accelerating post-construction consolidation of the embankment foundation and helping to reduce high water pressures generated in the dam's foundation. For this purpose, there is an advantage of extending the blanket drain back to the edge of the cut-off trench.

5.2 Toe Drain

Blanket drains can be combined with a toe drain consisting of a perforated PVC pipe surrounded by drain rock sloped to provide a collection system that directs the seepage water to discrete exit points from the toe of the embankment. A benefit to having discrete exit points is that seepage can be observed and measured more directly, and seepage collection ditches are not required at the downstream toe. However, ice and debris must not clog the outlets, and the water coming from the exit points must be directed away from the dam. Figure 15 shows a toe drain under construction. The PVC pipe is surrounded by clean gravel and geotextile. Figure 16 shows an example of a discrete exit point for a toe drain. The Federal Emergency Management Agency (2011) recommends that the toe drain should consist of a perforated pipe surrounded by a gravel drain which, itself, is surrounded by a sand filter.

Figure 15. Toe drain under construction (photo courtesy of the City of Dawson Creek).

Figure 16. Toe drain outlet.

5.3 Filter Grain Size Criteria

The CDA recommends using the filter criteria proposed by Sherard et al. (1984) and Sherard and Dunnigan (1989), which prescribes a D_{15} for the filter material based on the embankment soil type and the percentage of fine-grained soil. These are summarized in Table 7. The grain size for which X% of the soil particles is smaller is specified by D_X and d_X for the filter and embankment soils, respectively.

Table 7. Sherard and Dunnigan (1989) filter criteria recommended by CDA (2007).

Soil Type	Criteria
Group 1 85-100% fines	$D_{15}/d_{85} \leq 9$
Group 2 40-85% fines	D_{15} = 0.7 mm
Group 3 0-15% fines	$D_{15}/d_{85} \leq 4$
Group 4 15-40% fines	Intermediate between Group 2 and 3 depending on fines content

Table 8. Filter grain size criteria recommended by U.S. Army Corps of Engineers (2004).

Soil Type	Criteria
Group 1 85-100% fines	$D_{15}/d_{85} \leq 9$
Group 2 40-85% fines	$D_{15} \leq 0.7$ mm
Group 3 0-15% fines	$D_{15}/d_{85} \leq 4$ to 5
Group 4 15-40% fines	$D_{15} \leq \frac{40\text{-}A}{40\text{-}15}\{(40 \times d_{85})\text{-}0.7\}+0.7\text{ mm}^*$

*A = percent passing No. 200 (0.075 mm) sieve after any regrading; when 4 x $d_{85} < 0.7$ mm, use 0.7 mm.

The maximum particle size of the filter material should not exceed 50 mm (CDA 2007). ICOLD (1994) also recommends the use of the Sherard filter criteria, but with minimum D_{10} and maximum D_{90} limits to prevent segregation. ICOLD recommends analyzing the grain size distribution of the embankment soil and adjusting the material to only include the fraction finer than 4.75 mm.

For the high clay content in the soils used to construct the dugout dams, it is difficult to meet the grain size criterion for a granular filter using one material type. Therefore, blanket drains are typically separated from the embankment soils with a non-woven geotextile to prevent embankment soils from migrating into the filter.

If there are dispersive soils in the embankment, CDA recommends that the filter D_{15} not exceed 0.5 mm (2007), while ICOLD (1994) recommends a D_{15} between 0.1 and 0.3 mm. The fines percentage should be limited to 5%, and $D_{15} > 0.1$ mm to ensure the filter will still accept seepage from the embankment. For the dams that were investigated, there was no evidence of dispersive soils. Therefore the design of the filter is simplified in these cases.

5.4 Key Trench or Seepage Cut-off Trench

BC MFLNRORD (2018) requires that embankments be constructed with a key trench adequate to eliminate or minimize seepage through the foundation, although no minimum dimensions or side slopes are mentioned. The recommended key trench depths from a variety of best practices documents are summarized in Table 9. The actual key trench depth should depend on the foundation conditions to minimize the potential for preferential seepage paths or piping mechanisms to form in seams of silt or sand or other unfavourable foundation conditions. If the trench is too shallow, seepage can flow around the key trench and enter a blanket drain.

Table 9. Key trench dimensions from existing best practice guidelines.

Source	Minimum Key Trench Depth
Eyre Peninsula Natural Resources Management Board (2011)	2.5 m with side slopes between 1H:2V and 1H:1V
United Nations FAO (Stephens, 2010)	1 m
Department of Primary Industries and Water of Tasmania (2008)	1.5 x dam height and extends a min. 600 mm into impervious soil or rock
U.S. Army Corps of Engineers (2004)	Width at base ≥ 25% of the difference between the maximum reservoir and minimum tailwater elevations; width at top ≥ 3 m
PFRA (1992)	1 m depth and 3 m base width with side slope of 1H:1V

Figure 17 shows a typical key trench configuration for a dugout earth dam. The sidewalls of the key trench should not be steeper than 1H:1V. A 1H:1V slope can usually be safely excavated at a 1H:1V slope in clay-rich glacial till foundation soils. However, for trenches deeper than approximately 1.5 m, the stability of the trench sidewall should be assessed by a geotechnical engineer to ensure that workers in the trench will not be exposed to sidewall failures.

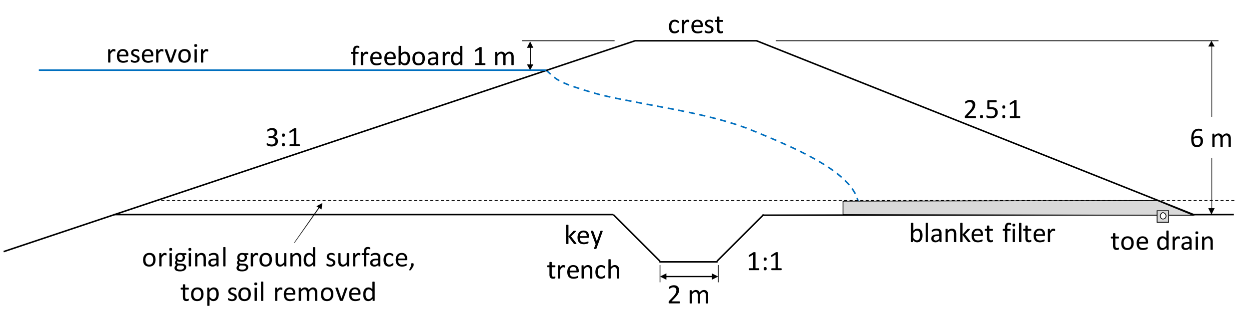

Figure 17. Typical geometry for a key trench in the foundation for a dam.

Leaving the trench empty for extended periods can result in tension or desiccation cracks forming in the soil around the trench that can form seepage pathways. Therefore, as soon as the key trench is excavated, it should be filled with low permeability soil compacted in thin lifts. Figure 18 shows a key trench under construction with a flat bottom and 1:1 sidewalls. The soil excavated for the key trench will be placed back into the trench in thin lifts and compacted to form a low permeability seepage cut-off.

Figure 18. Key trench during construction (photo courtesy of the City of Dawson Creek).

The key trench should be centred under the crest of the dam or slightly upstream of the dam's centreline. A key trench at least 1 m deep is recommended for all dugout dams to ensure that possible seepage through soils with cracks or higher permeability, which are often found near the ground surface, have been cut-off. Another advantage of using cut-off trenches is that they can function as an anti-shear key that ensures that no weak layer passes through the whole width of the dam foundation.

5.5 Dam Seepage

It is normal for some water to seep through a dam. For dam constructed out of clay-rich soil, the seepage quantity should be very small. The presence of increased vegetation at the downstream toe of a dam may be an indication of seepage through the dam. An example is shown in Figure 19. All dams should be visually monitored for changes in the seepage coming through the dam. An increase in the seepage quantity or an increase in sediment in the water may indicate internal piping erosion. Possible piping must be dealt with immediately.

Figure 19. Vegetation indicating seepage through a blanket drain.

If unusual seepage conditions are expected or encountered, then the installation of piezometers is recommended to facilitate an understanding of the pore pressures and their change within the embankment or its foundation.

Water exiting from a blanket or toe drain must be prevented from ponding and accumulating at the toe of the dam. If a toe drain system is omitted, and a blanket drain is present, there should be a seepage collection ditch to divert the water away from the embankment (USBR, 2015). USSD (2011) highlights measures for controlling surface drainage and groundwater seepage during construction. Drainage ditches leading water away from construction activities help to mitigate surface and embankment erosion issues. The British Columbia Aggregate Operators Best Management Practices Handbook (BC MEM, 2002) recommends that ditches be located where stormwater naturally collects and flows. Ditches should be sized to accommodate 110% of anticipated peak flow and be sloped appropriately to drain the water without causing excessive erosion. Drainage ditches or seepage collection ditches should have a consistent gradient, with no low spots. Ditches must be maintained to ensure they are not clogged with sediment or debris. Ditches should be regraded if erosion or ponded water occurs.

6 Erosion Protection

6.1 Riprap Shore Protection

The upstream slope on the dam should be protected from surface erosion due to wave action. The use of riprap can provide effective protection from wave erosion. It is recognized that there are often limited sources for rocks of sufficient size and angularity to function as riprap in the Peace River region. Furthermore, many of the available rocks have poor resistance to freeze-thaw and wet-dry cycles. Thus, the rocks may weather and deteriorate relatively quickly. The durability of the rock should be assessed before it is used as riprap. Figure 20 shows an example of a sedimentary rock that disintegrated in response to weathering.

Figure 20. Sedimentary rock with lower durability.

Riprap is classified according to the rock size or mass. The BC riprap design guides specify different riprap classes, as summarized in Table 10. The required size and weight of riprap for protection from wave erosion can be determined using the method prescribed by USACE (2002). While the BC MTI (2016) has a design guide for riprap for slope protection, as does the BC MELP (2000), these guides and ICOLD (1993) are based on the USACE (1991) riprap design guide.

Table 10. Riprap size and weight gradations (BC MoTI, 2016).

Riprap Class (kg)	Riprap Thickness (mm)	Approx. Average Size (mm)				Rock Gradation Percentage < Rock Mass (kg)		
		15%	50%	85%	<100%	15%	50%	85%
10	350	90	195	280	330	1	10	30
25	450	120	260	380	450	2.5	25	75
50	550	155	330	475	565	5	50	150
100	700	195	415	600	715	10	100	300
250	1000	260	565	815	965	25	250	750
500	1200	330	715	1030	1220	50	500	1500
1000	1500	415	900	1295	1535	100	1000	3000

For a reservoir with a design wave height, the Irribarren-Hudson formula can be used to determine the design weight of the rock, and maximum and minimum rock weights as recommended by ICOLD (1993). For example, given a 0.54 m design wave height, the minimum recommended riprap would be Class 25 kg riprap under the BC MoTI (2016) riprap design guide. In addition to riprap weight, the nominal and maximum riprap stone diameters and minimum thickness of the overall riprap layer should be determined using the methods recommended by ICOLD (1993). An example of recommended riprap layer thickness and rock size for different wave heights is given in Table 11.

Table 11. Recommended dimensions for riprap (French Committee on Large Dams, 2002).

Wave height (m)	Layer Thickness (m)	Rock Size D_{50} (m)
0.30	0.3	0.20
0.55	0.4	0.25
0.8	0.5	0.30

Table 12. Recommended dimensions for riprap (PFRA, 1992)

Reservoir length (km)	Exposure condition	Riprap thickness (mm)	Average rock size (D_{50}) (mm)	Bedding gravel thickness (mm)	Bedding gravel effective size (D_{85}) (mm)
< 1	A	200	130	150	38
< 1	B	250	170	170	38
< 1	C	300	200	200	38
1 – 1.6	A	200	130	150	38
1 – 1.6	B	300	200	200	38
1 – 1.6	C	400	260	260	50

In Table 12, exposure condition A means that the major axis or length of the reservoir is not parallel to the direction of the prevailing winds; exposure condition B means that the length or main axis of the reservoir is parallel to the prevailing winds; exposure condition C is for embankments that face the prevailing winds and are located in southwest Alberta (stronger winds).

Figure 21 shows a design layout for riprap shore protection. The bottom of the riprap layer is designed to fit onto a small bench built into the embankment slope. The riprap layer extends both below and above the maximum range in the reservoir water level.

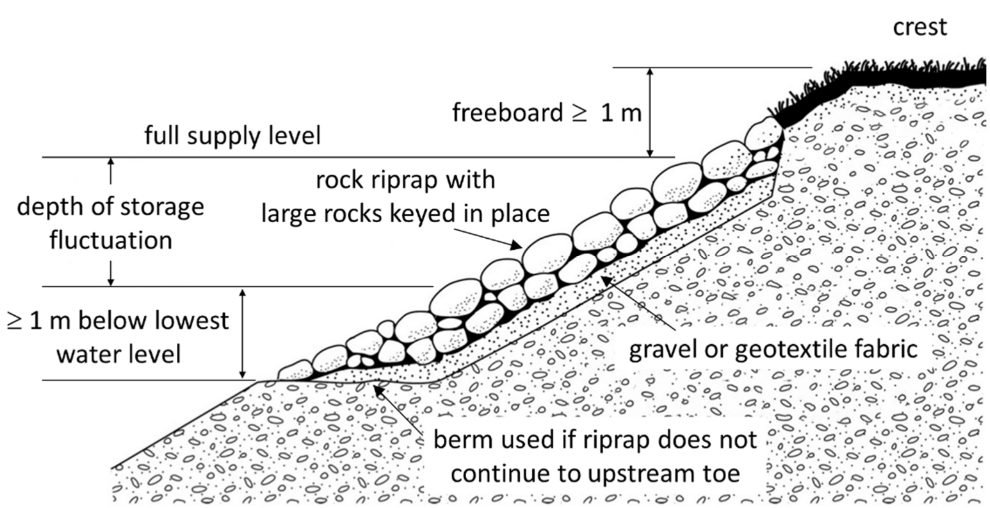

Figure 21. Design layout for riprap shore protection (modified from Lewis (2014)).

In the Peace River region, the dominant wind direction is from the west. Hence, protection from wave erosion is most important on the east side of reservoirs. Figure 22 compares an embankment on the east side of a reservoir one and nine months after construction to demonstrate how wave action on an unprotected slope causes erosion and creates a small scarp and a beach. In contrast, the shore along a dam constructed in 1975 is shown in Figure 23. Riprap was placed along the eastern shore and damage from wave action is minor.

Figure 22. Unprotected slope eroded by wave action one (left) and nine (right) months post-construction.

Figure 23. Shore protected against wave erosion with riprap 44 years after construction.

ICOLD (1993) recommends that a granular filter be placed beneath riprap instead of just placing the rocks directly on the slope. This prevents the migration of fine embankment soils through the riprap and prevents the rocks from sinking into the embankment slope. The design thickness of a granular filter depends on the susceptibility of embankment materials to wave erosion, riprap size and weight, and embankment slope. A 15 cm filter thickness was found appropriate for one of the investigated dams. Riprap filter gradation criteria are provided in ICOLD (1993). As an alternative, a non-woven geotextile similar to the ones used in the dam spillways can replace a riprap filter.

6.2 Erosion Control Blankets

Given the limited availability of riprap rock, erosion control blankets were used in an attempt to protect against wave erosion at a few of the investigated dams. These efforts were not very successful because the water level in the reservoirs dropped below the bottom of the blankets, and wave erosion undermined the blankets. In addition, ice damaged the blankets in the spring. Figure 24 and Figure 25 show two different types of erosion blankets that were ineffective within two years of installation. If erosion control blankets are used, they must extend below the lowest reservoir level, and they must be properly fastened to the embankment slope.

Figure 24. Erosion protection blanket ineffective at protecting the shoreline from wave damage.

Figure 25. Erosion protection blanket above the reservoir level and not fastened properly.

6.3 Booms

Log boom systems can be used to dissipate wave energy and protect reservoir shores from wave action, ice, and debris. However, they should be properly maintained and anchored. BC MFNLRORD (2018) recommends that the logs have a minimum diameter of 300 mm. They provide specifications for chain holes, boom chains, anchors, cables, and anchor chains. Booms that become damaged, saturated, detached, or are otherwise not fulfilling their purpose should be removed and replaced as they can jam spillway channels.

A system of connected plastic booms was installed at one dam in the summer of 2019 (Figure 26). These booms are designed to dissipate wave energy and reduce the size of waves on the shoreline. Initial feedback from the dam owner suggests that these booms are working well. Note that grass was able to become established along the shore where wave action was not severe compared to the far shore, which was exposed to larger waves before installation of the boom.

Figure 26. Plastic chain-link booms installed in summer 2019.

6.4 Crests and Runoff Erosion Protection

If vehicle traffic occurs on the dam crest, an appropriate travel surface should be established to prevent erosion (BC MFLNRORD 2018). The crest should be sloped from the centerline out to both embankment slopes. The USSD (2011) recommends that sand and fine gravel not be placed on embankment slopes as these are susceptible to runoff erosion. If the gravel layer on the dam crest is thin, then a layer of geotextile should be used underneath the gravel layer.

If the dam crest is not meant to function as a roadway, it should be sloped into the reservoir at a slope of 2%, and topsoil should be placed and seeded to promote grass growth. There should be no low spots on the crest and vehicles should not drive on it. Otherwise, vehicles can create ruts in the embankment that form low spots where water can pond, which can contribute to surface erosion.

A comparison of crests at two different dams is shown in Figure 27; the crest on the left was not meant to function as vehicle access, but it has been driven on and has depressions from tire tracks as a result. The crest on the right was built with geotextile and granular road base to function as vehicle access.

Figure 27. Comparison of a dam crest with no vehicle access (left) and a crest designed for vehicle access (right).

Erosion control blankets made of straw, mulch, wood fibre or synthetics can provide temporary protection for soil embankments vulnerable to surface runoff erosion (BC MEM 2002). The intention is to promote grass growth that will eventually become the primary means of erosion protection. Figure 28 shows the upstream portion of a recently constructed dam covered with an erosion control blanket. Grasses and other vegetation are starting to grow through the blanket. For this dam, the crest is covered with a gravel layer to provide a driving surface and to protect the crest from runoff erosion.

Figure 28. Surface erosion protection provided by a gravel layer on the crest and an erosion control blanket above the high water level.

Grassy vegetation should be used to cover and protect embankment slopes from runoff erosion CDA (2007). Efforts to accelerate the establishment of a thick grass cover should begin immediately after the dam is built. To help the grass grow, spread 10 to 15 cm of topsoil that was salvaged during the dugout and dam construction over the embankment slopes. BC MEM (2002) also recommends tillage or roughening the soil slopes such that they have horizontal grooves from a dozer, which creates small pockets that prevent runoff and therefore prevent erosion gullies from forming. This practice also promotes vegetation growth. The slopes should be hydroseeded or broadcast seeded after construction so that grass can begin to grow as soon as possible to protect against surface erosion. The seeding process should promote uniform growth of grass only.

Figure 29 shows topsoil being properly spread on an embankment slope before seeding. Figure 30 shows a different embankment immediately post-construction and one year after topsoil had been placed. The topsoil is vital for good grass growth as it is difficult for vegetation to grow in the low-permeability embankment soils.

Figure 29. Dozer spreading topsoil on an embankment with the track grooves forming small horizontal impressions in the soil (photo courtesy of the City of Dawson Creek).

Figure 30. Embankment slope immediately post-construction (left) and one year after placing topsoil and broadcast seeding (right).

7 Dam Construction

7.1 Foundation Preparation

The BC MFLNRORD (2018) requires foundations for embankments to be free of loose soils and organic materials. The construction of a dugout and dam requires the removal of all topsoil and organic material. Removing the top 0.3 m of soil is a minimum recommendation. The United Nations FAO (Stephens, 2010) recommends clearing access roads, removing trees, and stripping the top 100 mm of foundation area of topsoil and vegetation as decomposition of organic material can affect the stability of the dam by providing pathways for seepage at the foundation. U.S. Army Corps of Engineers (2004) suggests clearing, grubbing to remove stumps and large roots in approximately the top 0.9 m, and stripping to remove sod, topsoil, boulders, organic materials, rubbish fills, and other undesirable materials. The Alberta Ministry of Agriculture and Forestry (2015) says that stripped vegetation and organic soils from the base of the reservoir can either be stockpiled or used on the downstream slope of the embankment to facilitate grass growth.

If the dugout penetrates through a layer of soil with relatively high hydraulic conductivity, a compacted clay liner should be used to cover this soil. Furthermore, the seepage cut-off trench for the dam should extend deeper than this layer.

7.2 Construction Season

The United Nations FAO (Stephens, 2010) recommends constructing dams during the dry season to avoid issues of seepage, water control, and build-up of excess pore water pressure, and to make equipment mobilization easier. In contrast, the Eyre Peninsula Natural Resources Management Board (2011) recommends construction in the spring to ensure that there is sufficient moisture to prevent the soil from drying out and cracking during construction. Figure 31 shows an example of cracks that can occur in the soil as it dries. These types of crack must not get buried within the embankment during construction.

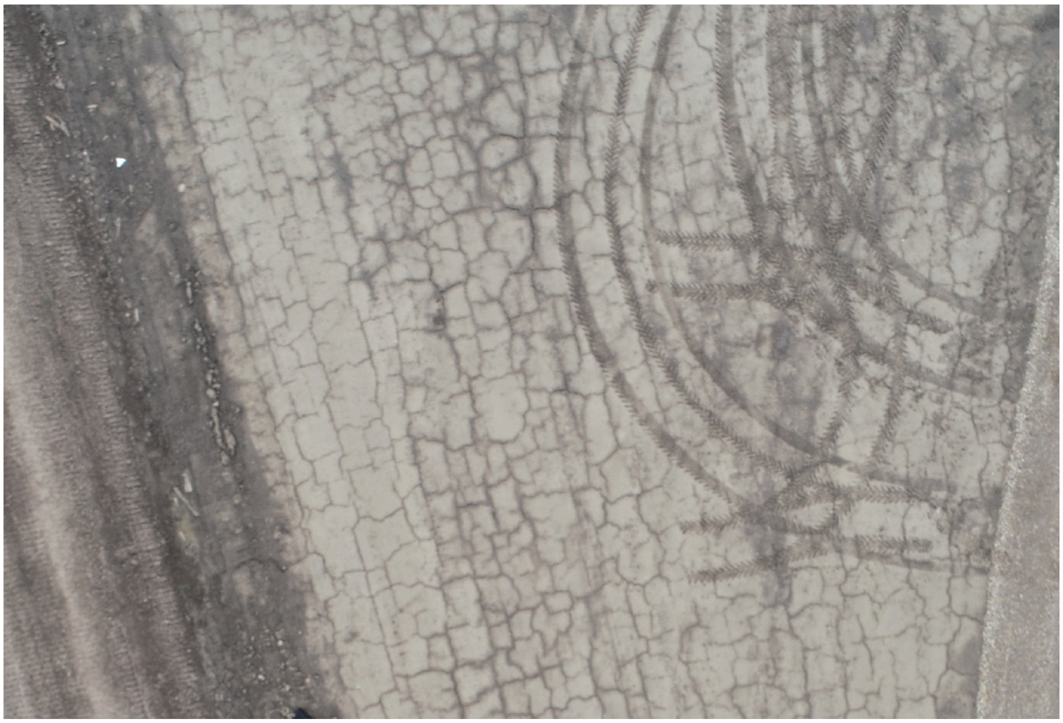

Figure 31. Vertical view of desiccation cracks in clay soil shortly after the construction of a dam.

It is best practice to not construct dams in the winter to avoid freezing conditions, which can result in poor compaction and inadvertent inclusion of snow and ice within the embankment. The moisture content of the compacted soil must be controlled to ensure sufficient compaction and to prevent desiccation cracks from forming on each compacted lift before adding the subsequent lift. If a dam must be constructed in the winter, then remove the top part of the embankment each morning to a depth sufficient to remove frozen soil before adding the next lift of unfrozen soil.

7.3 Embankment Compaction

Recommended moisture levels and lift thicknesses for embankment compaction from different sources are summarized in Table 13. The moisture contents that are listed are relative to the Optimum Moisture Content (OMC) found from Standard Proctor Compaction tests. Clay embankments should be compacted using a sheepsfoot roller in lifts of 20 to 30 cm. Successful compaction occurs when the thickness of the placed soil layer reduces by roughly 25% during compaction (Stephens, 2010). A sheepsfoot roller provides the best kneading action for compacting clays. Scrapers and dozers do not compact clay-rich soils as well as sheepsfoot rollers, and some guidelines recommend against using them (Eyre Peninsula Natural Resources Management Board, 2011).

Table 13. Recommended compaction moisture and lift thickness from best practice guidelines.

Source	Recommended Moisture Content	Recommended Lift Thickness (mm)
Ontario Ministry of Agriculture, Food and Rural Affairs (Shortt, 2016)	near optimum	150-300
Alberta Ministry of Agriculture and Forestry (2015)	near optimum	150
Eyre Peninsula Natural Resources Management Board (2011)	-1% to +3% OMC	200
U.S. Army Corps of Engineers (2004)	-2% to +3% OMC	NA
PFRA (1992)	-1% to +3% OMC	150-250
ICOLD Bulletin 76 (1990)	+2% above OMC	200-300

It is recognized that scrapers and dozers are often the most efficient way to both excavate the dugout and construct the embankment. If scrapers and dozers are used, it would be prudent also to use a sheepsfoot roller to compact the central core zone of the embankment as well as the soil in the seepage cut-off trench. In addition, there should be a higher effort in compaction quality control. The BC MFLNRORD (2018) highlights the importance of and requirements for quality control, construction monitoring, testing, and documentation during construction.

All embankments should be built to have crest elevations greater than the final design level to account for settlement. The amount of over-building needed for the embankment depends on the consolidation properties of the embankment and its foundation, and how much settlement is anticipated. A common recommendation is to overbuild the dam height by about 10% (BC MEM, 2002; Alberta Ministry of Agriculture and Forestry, 2015; Stephens, 2010; Tasmania Department of Primary Industries and Water, 2008). After consolidation settlement occurs, it is important to ensure that there is still adequate freeboard. The consolidation settlements in the clay-rich soils of the Peace River region can take many years to occur. Therefore, crest elevation surveys should be conducted immediately after construction and repeated a few years later.

7.4 Filter Compaction

Granular filter soils should be compacted using a smooth drum roller in thin lifts (maximum 30 cm thick). The USSD (2011) and PFRA (1992) recommends a maximum lift thickness for filters of 0.3 m. Vannobel et al. (2013) recommend a maximum filter lift thickness of 450 mm and compaction of 93-95% Standard Proctor Maximum Dry Density (SPMDD) for embankments higher than 4 m, and 92% SPMDD for heights lower than 4 m. The USBR (1987) recommends compacting filters to 70% relative density.

While the moisture content of granular soils at the time of compaction is not as critical as for finer-grained soils, the water content should be monitored during construction as it affects the degree of compaction. The filter should have similar compressibility as the other embankment materials (USSD, 2011).

7.5 Decommissioning

Dugouts used for an oil and gas purpose should be constructed with a plan for decommissioning. Distinct materials removed from the dam should be stockpiled separately and protected from surface erosion so that when the reservoir and dam are decommissioned, this soil can be replaced in an appropriate sequence. Topsoil should be replaced last and hydroseeded to promote vegetation growth. The post-decommissioning topography should promote natural drainage and prevent water from ponding. Figure 32 shows an example where the excess soil removed from the dugout is strategically stockpiled and stored until the dugout and dam are decommissioned.

Figure 32. Stockpile of excess soil covered with soil and seeded for future reservoir decommissioning.

8 Dam Maintenance

8.1 Vegetation

Well-established grass slopes should be mowed regularly because long grass lies down and is less effective in trapping sediment (BC MA, 2013). It is also easier to identify deficiencies on a slope that is mowed.

A variety of tree species can grow on embankment slopes, and they become more problematic as their roots penetrate deeper. Vegetation management must ensure that trees or large shrubs do not grow on the dam (Province of BC, 2017). Small trees and shrubs should be removed before they grow taller than approximately 2 m because removal of larger trees leaves behind roots that can rot out, leaving seepage paths that can initiate piping mechanisms. Figure 33 shows an example of excessive vegetation growing on a dam. Excessive vegetation can obscure important dam deficiencies. As dams age, the growth of unwanted vegetation often becomes the most persistent maintenance issue for dams.

Figure 33. Excessive vegetation growing on a dam.

Areas of relatively lush vegetation growing near the toe of an embankment are often a good indication of seepage locations. Cattails can also indicate areas of localized seepage.

8.2 Ditches

Drainage ditches should be inspected at least twice yearly, after the spring thaw and before fall when conditions are driest (Shortt, 2016). Ditches and drainage channels should be kept clear of debris, excessive sediment, or large blocks of ice (Province of BC, 2017). Blockages must be removed to prevent overtopping. Drainage ditches require sufficient grade to promote drainage. Embankment slopes, crests, and drainage ditches should be regraded if there are low spots, ponding water, or erosion gullies.

8.3 Animal Damage

Fences are recommended to keep unwanted vehicle traffic and cattle off the embankment. Cattle tend to form pathways on embankments, creating localized low and flat spots for overland flow to accumulate. Cattle can also interfere with uniform vegetation growth on slopes (Province of BC, 2017).

Animals such as muskrats that create burrows in the embankment should be removed. Tunnels or burrows from animals should be repaired (Province of BC, 2017). Geotextile and riprap on the embankment can help to discourage animals from creating tunnels and burrows (USSD, 2011). A wire mesh can also prevent animals from burrowing into embankments. At one older dam that was investigated, muskrats had burrowed into the embankment on the western side of the reservoir where there was minimal wave action. The muskrat damage is shown in Figure 34. The repair plan was to remove the upstream part of the embankment as far back as the animal burrows extend, and to reconstruct the embankment. It is estimated that a trench least 0.5 m wide will need to be excavated into the embankment to repair this damage.

Figure 34. Embankment damage from muskrat burrows.

8.4 Instrumentation and Monitoring

The CDA (2007) highlights the importance of instrumentation and monitoring to detect early signs of problems. Instrumentation is not a requirement, but if instrumentation is used, details on the instrumentation must be submitted as part of the design drawings, plans and specifications (BC MFLNRORD, 2018). Instrumentation should be installed based on expected dam performance and for a specific purpose (CDA, 2007). Instrumentation should be maintained and monitored, and the installation of instrumentation should not initiate any problems in the dam. Instrumentation must be protected from accidental damage and vandalism. For example, Figure 35 shows a protective casing for two piezometers.

Figure 35. Protective casing with locking cap for piezometers.

If significant longitudinal cracks occur in the embankment, a slope inclinometer should be installed downslope of the cracks to monitor the slope movement and to estimate the location of a potential slip surface if a slump or slide has initiated.

If there are concerns about a high phreatic surface or excess seepage, piezometers should be installed near the crest and toe of the downstream slope to measure the location of the phreatic surface. The phreatic surface location should be compared with the expected position as a step in understanding the performance of the dam. If a toe drainage collection system is used, the quantity and colour of the flow exiting the dam should be measured and documented regularly.

Regular visual inspection is the best tool for assessing the performance and safety of an embankment dam. Photographs taken of the same feature from the same perspective, from inspection to inspection, are invaluable to the monitoring process. Photos can be compared across multiple inspections to identify changes in conditions that indicate a developing issue that affects the stability and safety of the dam.

The use of repeated aerial photography using a remotely piloted aircraft system (RPAS) is a powerful way to collect an overlapping sequence of photographs of the dam from which detailed 3D models of the dam can be constructed. RPAS can be used to document conditions during the dam construction and the as-built dam geometry immediately after construction. These data provide a valuable basis for identifying changes in the dam over time. Ideally, the RPAS should provide accurately georeferenced camera locations to facilitate photogrammetric processing of the acquired images into a known coordinate system (e.g., UTM).

9 Conclusions

Many guidelines and recommendations for some aspects of the design and construction of earth dams already exist. Specific recommendations include upstream and downstream slopes, minimum crest width as a function of embankment height, minimum factors of safety for different loading conditions, minimum spillway width and dam freeboard. These best practices guidelines were typically followed for the design of the investigated dams. However, in some cases, the as-built conditions did not match the designs. Examples included overly steep embankment slopes, culverts placed in the spillway, and unusual compaction methods, which do not follow best practices.

Unfortunately, existing best practices documents often contain vague or no reference to some aspects of dam design and construction, such as surface erosion protection. Given the erodible nature of the soils used to construct dams in the Peace River region and the impact of wave action on reservoir shorelines, greater care is needed to manage erosion on the dams.

Some specific findings from this project are as follows.

- The as-built embankment slopes were sometimes steeper than the designed slopes, and upstream slopes were sometimes steeper than downstream slopes. These findings do not meet best practices.
- Dams constructed from glacial till with upstream and downstream slopes of 3:1 and 2.5:1, respectively, should perform well if the soils are adequately compacted during construction.
- High-quality construction quality control is needed to ensure that the design slopes and the design dry density of the soil are achieved. The construction quality control should be documented during the embankment construction.
- Dams with embankment slopes steeper than 3:1 upstream and 2.5:1 downstream need justification with stability analyses using reliable shear strength values for the embankment and foundation soils for both effective and total stress loading conditions. Obtaining these parameters will require advanced triaxial soil testing.

- The embankments were generally constructed using thin lifts (0.2 to 0.3 m), and sheepsfoot rollers were generally used for compaction. These both meet best practices recommendations. If a sheepsfoot roller is not used, a higher degree of compaction quality control should be used.
- Dams should be overbuilt to account for consolidation settlement to ensure that adequate freeboard will be maintained.
- The time required to achieve steady-state seepage and full consolidation settlement can be many years for dams built with and on soils with low hydraulic conductivity.
- Blanket drains should be incorporated into dams higher than 4 m. Blanket drains lower the phreatic surface, reduce pore pressures, and increase the embankment stability. They also accelerate consolidation settlement.
- Blanket and toe drains will generally require a non-woven geotextile to separate the granular filter soil from the clay-rich embankment and foundation soils. Non-woven geotextile should also be used under riprap in spillways and drainage ditches.
- Many dugout reservoir dams do not incorporate a low-level outlet structure. This practice should be appropriate for structures that are typically filled and emptied via over-the-embankment pumping. Eliminating a low-level outlet simplifies the dam construction and removes piping risks that are typically associated with low-level outlets.
- When the drainage area providing water inflow to the reservoir is small, the 4 m minimum spillway width is more than adequate to pass the IDF given the climate in the Peace River region.
- Most dams use a minimum freeboard of 1 m as recommended in various best practices guidelines. This freeboard should be appropriate for typical dugout reservoir sizes (fetch length) and wind conditions in the Peace River region. This finding also assumes that frost penetration does not create ice lenses deeper than 1 m below the crest.
- There was no evidence for the presence of dispersive clay soils in the seven investigated dams. The absence of dispersive clay reduces the risk for piping failures, although the clay mineralogy should be checked at other dam locations.
- Dealing with erosion caused by waves and precipitation runoff is a common issue at most dams. Riprap can provide good protection against wave erosion along the reservoir shore. The performance of erosion protection blankets has been poor due to water levels dropping below the protective layer and anchoring challenges. Embankment slopes and drainage collection ditches require erosion protection. Establishing a grass cover as soon as possible is important for reducing runoff erosion.
- Collecting aerial imagery with remotely piloted aircraft systems can greatly improve construction documentation. The aerial imagery should be used for post-construction dam surveys and performance assessment. Structure-from-motion photogrammetry software can be used to process the images to obtain detailed as-built geometries and to detect changes in the dam geometry. A survey of the dam crest is needed years after construction to ensure adequate freeboard remains around the reservoir after settlement occurs.

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