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Embankment dams in Canada

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SYNOPSIS The paper reviews the development of embankment dams in Canada. By using selected case examples, it demonstrates that problems in design and construction of embankment dams in Canada are largely related to geology, particularly glacial geology, and to severe climatic conditions. Solutions to these problems have been innovative, advancing the state-of-the-art and providing valuable experience to the profession.

Introduction

The first dam of any size constructed in Canada was the 19 m high Jones Falls masonry dam on the Rideau River in Ontario, completed in 1832 and credited to Lt. Col. John By (Cancold., 1984), but it was not until 1895 that the first earthfill dam of any substance was built. This was a water retaining structure, the Goldstream Dam, for the Greater Victoria Water District, British Columbia. For the next 50 years prior to 1945, few earthfill or rockfill dams were constructed and those built were of comparatively low height. The St. Mary Dam in Alberta (Peterson., 1945) was the first significant earth fill dam in Canada where modern soil mechanics technology was applied in design and construction (1945 – 1950). However after about 1950, the number and size of embankment dams markedly increased. This is illustrated in Figure 1, which plots the number of embankment dams more than 15 m in height built per decade since 1900. The increase in number of embankment dams constructed since 1950 is remarkable, as is the trend of increase in height since that date. An example of this is that by the decade 1950 to 1960, the highest fill dam to date was the Kenney Dam in British Columbia (Jomini., 1954), a 104 m high rockfill embankment. Since then 10 fill dams exceeding this height have been built, the highest being Mica Dam, an earthfill structure 243 m high completed in 1972, also in British Columbia.

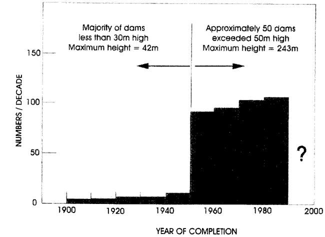


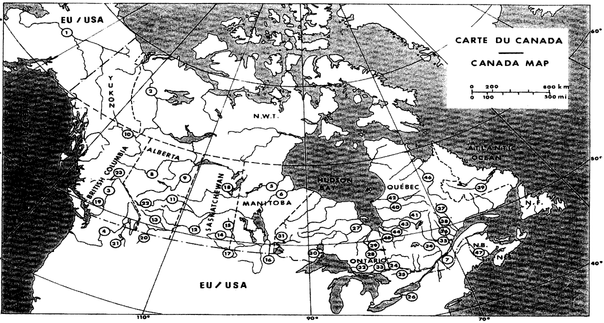
Fig. 1. Embankment dams (higher than 15 m) built per decade in Canada since 1900

The majority of the dams now built are of the embankment type. Most have been built for hydroelectric power generation. Two projects accelerated the design and construction of fill dams. In the period 1968 to 1972, some 88 earth and rockfill dams and dykes totalling about 64 km in overall length were built for the Churchill Falls Power Project in the Canadian Shield of Labrador, only 50 per cent of these being over 15 m in height (McConnell et al., 1973). In the period 1976 to 1982, over 80 earth and rockfill dams and dykes were constructed in Québec, in the heart of the Canadian Shield, as part of the James Bay Hydroelectric Project (Paré et al., 1997). Many of these dams were of significant size, the highest being LG-2 Dam, a 168 m high rockfill completed in 1978 on La Grande Rivière.

Supplemental accounts of the history and trends of dam design and construction in Canada have been recorded in reports by the Canadian National Committee on Large Dams – CANCOLD. (Nancarrow., 1964 and 1967; Baribeau., 1970; Watson & Berry., 1973; McConnell., 1976; Gordon., 1979; Anderson & Keira., 982; Mendes & Ares., 1985.) Papers concerning some incidents and failures of embankment dams in Canada have been described by Peterson et al (1957) and by Seemel & Colwell (1976).

Influence of Physiography and Geology

Canada may be divided into four main physiographic and geologic regions distinguished by variations in topography and geology. These are the Canadian Shield, the Appalachian Region, the Plains Region and the Cordilleran Region (Figure 2). The Canadian Shield, which extensively covers much of the northern and eastern parts of the country, is a peneplained region comprised of igneous, metamorphic and some volcanic and sedimentary rocks, all of Precambrian age. At its southerly and western border, these Precambrian rocks dip beneath thick beds of sedimentary shales, limestones and sandstones of Paleozoic and Mesozoic age which constitute the Plains region. The Cordilleran rocks to the west comprise deformed younger sedimentary and metamorphic rocks as well as volcanic intrusions together with igneous rocks.



Légende

1 Fleuve Yukon
2 Fleuve Mackenzie
3 Fleuve Fraser
4 Fleuve Columbia
5 Fleuve Churchill
6 Fleuve Nelson
7 Fleuve St-Laurent
8 Rivière La Paix
9 Rivière Athabasca
10 Rivière Liard
11 Rivière Saskatchewan nord
12 Rivière Saskatchewan sud
13 Rivière Bow
14 Rivière Qu'appelle
15 Rivière Assiniboine
16 Rivière Winnipeg
17 Rivière Souris
18 Rivière Reindeer
19 Rivière Bridge
20 Rivière Ste-Marie
21 Rivière Pend'oreille
22 Rivière Brazeau
23 Rivière Nechako
24 Rivière Des Outaouais
25 Rivière Madawaska
26 Rivière Niagara
27 Rivière Albany
28 Rivière Matagami
29 Rivière Abitibi
30 Rivière Nipigon
31 Rivière Des Anglais
32 Rivière Montréal
33 Rivière Mississagi
34 Rivière St-Maurice
35 Rivière Saguenay
36 Rivière Bersimis
37 Rivière Manicouagan
38 Rivière Outardes
39 Rivière Churchill
40 La Grande Rivière
41 Rivière Eastmain
42 Rivière De La Grande Baleine
43 Rivière De Rupert
44 Rivière Broadback
45 Rivière Nottaway
46 Rivière Caniapiscau
47 Rivière St-Jean

Legend

1 Yukon River
2 Mackenzie River
3 Fraser River
4 Columbia River
5 Churchill River
6 Nelson River
7 St. Lawrence River
8 Peace River
9 Athabasca River
10 Liard River
11 North Saskatchewan River
12 South Saskatchewan River
13 Bow River
14 Qu'appelle River
15 Assiniboine River
16 Winnipeg River
17 Souris River
18 Reindeer River
19 Bridge River
20 St. Mary River
21 Pend'oreille River
22 Brazeau River
23 Nechako River
24 Ottawa River
25 Madawaska River
26 Niagara River
27 Albany River
28 Matagami River
29 Abitibi River
30 Nipigon River
31 English River
32 Montréal River
33 Mississagi River
34 St. Maurice River
35 Saguenay River
36 Bersimis River
37 Manicouagan River
38 Outardes River
39 Churchill River
40 La Grande River
41 Eastmain River
42 Greatwhale River
43 Rupert River
44 Broadback River
45 Nottaway River
46 Caniapiscau River
47 St. John River

Fig. 2. Major rivers in Canada (From CANCOLD., 1984)

Past geologic processes on the North American plate have created a condition that has great significance to the development of dams throughout much of Canada. The condition is that the present valley floor levels are generally from 30 m to as much as 400 m above the rock base of the ancient valley floors. At exceptional locations, the present river may occupy a young rock canyon with its floor at or only slightly above bedrock. These serve as nearly ideal sites for concrete or embankment dams. Invariably however, the deeper ancient valley will exist on one side or the other of the young rock canyon; examples of these are: Bennett Dam (Taylor., 1969) and Seymour Falls Dam (Ripley & Campbell., 1964) in British Columbia. Otherwise the current channel will lie above the ancient valley which is deeply buried by a complexity of glacial, inter-glacial, and Holocene deposits, the latter consisting variably of thick lacustrine and / or marine silts and clays or highly pervious alluvial materials such as at Mica Dam (Webster, 1970).

In the Cordilleran province, the buried infill is made even more complex by multiple stages of advance of montane glaciers and by interfingering of Holocene river materials with coarse cones of talus from lateral tributaries on the valley walls (Seymour Falls Dam). This complexity of the deposits in buried valleys presents challenges to the engineering dams throughout Southern Canada: first, of obtaining an approximate picture of the complexity and properties of the buried infill, second, of managing problems of underseepage beneath dams and problems of lateral seepage from reservoir rims, and third, of managing problems of deformation with compressible foundations and problems of stability and lateral movement within weak beds in the foundations.

In the Plains physiographic province, the foundations and abutments present their own special problems even though the height of embankment dams is generally less than 80 m, the majority being about 50 m or less. The problems are related to the relatively soft highly plastic beds in the sedimentary rocks and to highly plastic overburden derived from the sedimentary rocks in the region. The shales, siltstones and sandstones are mostly compaction rocks and are relatively uncemented. Being compaction products, they have been weakened by historic rebound due to load reduction. In the case of the more plastic shales, the rebound may not yet be complete as at Gardiner Dam (PFRA., 1979). Historic load reduction factors include the removal of overlying beds during multiple glaciation, removal of thick continental ice, and erosion of the valleys by river downcutting.

Combined vertical and lateral rebound within the valley walls toward the valley have resulted, in many cases, in vertical shearing of lesser extent and lateral shearing of major extent (Gardiner Dam). Moreover, the motions of continental ice fronts during the multiple glaciations have produced additional lateral shearing within both the underlying sedimentary rocks and the overburden as at Squaw Rapids Dam (Klohn., 1967).

Several serious problems at dam sites and reservoir rims arise from these conditions:

i) weak planes beneath the valley floor within the sedimentary rock and within plastic zones within the overburden. Shear strengths of such planes are commonly at or close to residual values. Large deformations on the planes can be initiated by dam fills with very flat slopes. In extreme cases, dam loads may induce very high pore pressures with little or no reduction in decades (Gardiner Dam).

ii) instability of valley walls induced by minor activities both upstream and downstream of a dam due to inherent factors of safety at or close to 1.0 on the weak planes (Gardiner Dam).

iii) rebound leading to slope instability and / or large deformations at cut slopes for spillways, intake and tailrace channels, as well as longitudinal stretching at tunnels and conduits as at Gardiner Dam. Similar problems also were experienced at Oldman Dam (Chalcroft & Baggott., 1989).

Embankment dams, to be economical, need local availability of suitable construction materials. As over 95 per cent of the land surface in Canada has been glaciated, glacial deposits, fluvio-glacial and glacio-lacustrine sediments provide most of the construction materials available, together with quarried rockfill. The most common glacial deposit is till, which forms perhaps as much as 75 per cent of the surficial deposits in Canada even in areas later subjected to late-glacial or post-glacial marine or lacustrine inundations. Tills are complex deposits and their properties are largely determined by the nature of the parent bedrock up-ice from the local till deposit and by the mode of its deposition. Till that has been deposited under the ice and has been over-ridden by the glacier is called "basal till". It is usually very dense, of hard consistency and heavily pre-consolidated. Till that has been deposited from within, or from the top of the glacial ice as the ice melts, is much less dense and normally consolidated and is called "ablation till". Being a generic term, the word till is related to mode of deposition and does not describe the soil texture. Composition of tills vary widely from fines-rich material containing a small content of sand, gravel and larger sizes, to coarse granular tills with a variable fines content. The content of fines for coarse tills varies from more than sufficient to insufficient fines to bind all particles. The fines-rich tills predominate in the Plains region and the coarse granular tills predominate in the other physiographic provinces. It is the variable nature of these tills and associated glacial sediments together with the foundation rock and the effects of climate which engender problems in the construction of embankment dams.

Failures at small clay dams on clay foundations in the 1940s and 1950s in Western Canada (Seven Sisters Dikes and North Ridge Dam) as described by Peterson et al (1957), prompted research and better understanding of the deteriorating influence of strain on the strength of clays.

Dams on Weak and Compressible Foundations

Embankment dams in Canada have been founded on a variety of soil types that are both weak and highly compressible. The soils types range from sensitive marine clays to varved silts and clays, to post-glacial lacustrine materials, to muskeg and to combinations of both compressible and pervious materials. Each type presents its own particular challenge.

Terzaghi Dam (Terzaghi & LaCroix., 1964; Taylor., 1969) had a particularly complex foundation as did the Seymour Falls Dam, both in British Columbia (Terzaghi was the designer of the former and the reviewer of the latter). The foundation of each was both compressible and pervious. Each site was such that Terzaghi recommended that the construction contract should be a force account type, in order to facilitate design changes during construction to accommodate the revealed site conditions. Terzaghi Dam is a 61 m earth and rockfill structure, built on the Bridge River in British Columbia and founded on over 150 m of alluvium comprising soft clay underlain by sand and gravel. Completed in 1960, it involved a deep clay-cement grout curtain and an upstream soft deformable clay blanket to inhibit seepage. Of particular interest is the use of polyvinyl chloride sheeting, almost 10,000 sq.m. in area, to sheath the clay blanket on the upstream face of the dam so that the clay could squeeze into any transverse cracks which developed. This is possibly one of the earliest uses of geosynthetics in major dam construction.

The precedence for building on sensitive marine clays was probably Bersimis 2 in Québec, the auxiliary dams being built in 1957 to 1959 (MacDonald., 1997). Weathered marine clay was also used as core material.

On the James Bay project in Québec (Paré et al., 1978; Paré and Levay., 1997), many kilometres of glacial till dykes were also successfully constructed on deposits of sensitive marine clays. Another notable example is given by the dykes of the Manicougan-Outardes Complex, known as Outardes 2 (Arbour & Paré., 1978).

Possibly one of the most striking examples of a fill dam safely accepting settlement is that of the Duncan Dam (Gordon & Duguid., 1976), built on the Duncan River in British Columbia, 1964 to 1967, over glacio-lacustrine deposits several hundred metres thick, containing extensive silt layers. Significant settlements along the dam axis had been anticipated during design, but severe cracking occurred during construction due to differential settlements of over 4 m adjacent to the left abutment where the deposits were only marginally over-consolidated. (The dam at maximum height is only 39 m high, measured above foundation level). Large transverse cracks through the core developed which necessitated sealing, re-design of the dam section, relocating the core and flattening the slopes. It is a credit to those concerned that the dam has performed satisfactorily to the present.

The Fairweather Dams (Whitman & Hardy., 1962) at Atikokan, Ontario are founded on lacustrine normally consolidated varved silts and clays that had been submerged prior to drainage of the lake in which they had been deposited. The analysis of the compressibility and shear strength of the varved material for support of the dams was without precedent in Canada. Laboratory values of shear strengths were modified by the results of field studies. With heights of 27 m and fill slopes of 4.5 H:1 V, the dam has performed well.

The 48 m high Forty Mile Coulee East Dam (Chin et al., 1991) in Alberta was founded on 60 m of highly plastic clay. Despite extensive laboratory tests, conservative assumptions and the benefits of an instrumented test fill, pore pressures exceeding the predictions by up to 35 per cent were recorded and large foundation movements occurred during construction. The final design that was tendered for construction had 8H:1V upstream and downstream slopes, with internal drains. It was found necessary to add toe berms, as allowed for in the initial design. Extensive instrumentation was beneficial to the successful completion and operation of the dam.

A 25 m high cooling pond dike for the Genessee Power Project in Alberta (Kack et al., 1989) was founded on highly plastic slickensided clay. Foundation drainage wicks and stage construction were used to permit safe completion of the embankment to full height.

Dams on Compaction Shales and Sandstones

Studies of the behaviour of the compaction shales and sandstones of the Plains physiographic region began with the initial investigations for the Gardiner Dam (PFRA., 1979) on the South Saskatchewan River in Saskatchewan in the mid-1940s. Intensive studies of these soft rocks have continued through several decades to the present, consisting of University research projects (Morgenstern & Eigenbrod., 1974); regional studies (Peterson., 1958); design studies for dams in the region such as Oldman Dam (Davachi et al., 1991) and performance measurements at completed projects (Jasper & Peters., 1979).

While, in comparison to examples in the previous section, the Gardiner Dam, a 64 m high earthfill structure built on the South Saskatchewan River, cannot be considered to rest on a "soft" foundation, the Bearpaw shale on which it is founded is nevertheless highly plastic and deformable with pre-existing shear planes. About 2 m of vertical settlement and over 2 m of lateral displacement occurred under the downstream shoulder at the central section of the embankment during construction, 1964 to 1968. Post-construction lateral deformations of the downstream shoulder are related mainly to annual cyclic changes in reservoir level and have now decreased to the order of 10 mm/year to 20 mm/year. The design of this dam is remarkable in that it preceded much of our current understanding of residual shear strength and the behaviour of highly overconsolidated clay-shales.

At the Dickson Dam on the Red Deer River, valley rebound had produced differential movements of beds of soft siltstones and shales, that were manifested in mylonite seams between beds. In order to confirm indications of mylonite seams from drill holes, a test tunnel was driven. It indicated the presence of slickened surfaces in the clayey beds and provided other information that had significant impact on the design of the projects (Phelps., 1981). Test tunnels have also become a useful exploration method at other sites (Gardiner Dam, Peace River Site C, Oldman River Dam) - for assessment of swelling characteristics, in-situ rebound features and tunnel construction methods in soft compaction shales and sandstones. The use of large diameter drill holes (1 m approximately) for in-situ examination of sedimentary rocks, and for recovery of samples of bedding plane features, has been employed at sites of both compaction and cemented sedimentary rocks (Gardiner Dam, Oldman River Dam).

Weak ice-thrust shear planes within soft sedimentary shales and sandstones underlying the till have resulted in significant downstream movement of a relatively low embankment at the Rafferty Dam in Southern Saskatchewan.

Dams on Cemented Sedimentary Rocks

Problems with foundations of this type fall into two categories. The first and more common category has been the buckling of flat-lying sedimentary beds at and beneath the valley floor due to rebound and horizontal strain. The buckling is accompanied by separation of bedding planes and the creation of voids. In some instances, due to horizontal thrust, shear zones have been observed near the centre of the channel. Conditions of this type create open paths for foundation seepage, and create weak planes along which downstream sliding may be critical. Thorough investigation is necessary in order to design a grouting program to control foundation leakage and to assess sliding resistance (Imrie., 1991).

Large grout takes attributed to rebound movements in cemented sedimentary rocks have been encountered at sites of both concrete and embankment dams. Bennett Dam (Taylor., 1969) and Peace Canyon Dam on the Peace River in British Columbia, the St. Mary Dam on the St. Mary River and the Bearspaw Dam on the Bow River in Alberta are examples. Laboratory shear tests on smooth and natural bedding plane surfaces were carried out for the Bennett Dam, the in-situ sliding resistance being dependent on the friction angle for smooth surfaces, the inclination angle associated with the surface roughness and the inclination angle of the beds.

At the Carillon project on the Ottawa River, an in-situ test on an altered shale feature in the bedrock was carried out to determine the sliding resistance (Pigot & Mackenzie., 1964).

Exploration procedures used by British Columbia Hydro at the Peace River sites in order to identify and examine the nature of the bedding plane openings include borehole cameras in conventional sized diamond drill holes. Churn drill holes with diameters up to 1 m were used for visual inspection and sampling of infill material.

The second category of problems involves karstic limestone. The Grand Rapids project on the Saskatchewan River in Manitoba is founded on karstic limestone and required a grouting program that took several years to complete (Rettie et al., 1962). It has few parallels in North America and represents the largest grouting program carried out in Canada. Over 25 km of dykes required grouting with Portland Cement and various admixtures to inhibit leakage through underlying karstic dolomitic limestone. The project schedule required much of the work to be done at sub-zero temperatures in winter.

Dams on Glacial Till Foundations

Basal till has low compressibility. It is normally considered to be a relatively problem-free foundation for embankment dams as long as it does not contain pervious lenses or lobes of highly plastic clay (Paré et al., 1982). Pervious beds and weak sheared material beneath the till can be encountered. These defects require special attention and can create significant problems. Lobes of sheared highly plastic clay within or between till sheets were encountered at Squaw Rapids Dam on the Saskatchewan River (Klohn., 1967) and Nipawin Dam on the same river (Mathewson et al., 1987).

The Paddle River Dam (Thiessen & Ramage., 1986) in Alberta, about 35 m high, deserves special mention as an extreme case of problems encountered with highly plastic slickensided clay in the foundation beneath glacial till, and with similar highly plastic clay in the embankment. It is a zoned embankment with a wide clay core and sand and gravel shells. A horseshoe shaped conduit through the dam rests on fill, 1 m above foundation level. The foundation consists of 3 m of pervious alluvium, above 10 m of stiff clay till, above 10 m of highly plastic slickensided clay, above sand and gravel, above sandstone bedrock. The alluvium was removed beneath the upstream shoulder. Pre-design exploration had revealed the presence of the weak slickensided clay in the foundation. Extensive laboratory and in-situ tests were carried out to estimate the shear strength and deformation properties of the slickensided clay. The design section included toe berms based on the shear strength data. Guided by the observed behaviour, the dam was built in stages over a period of 5 years, using extensive instrumentation to monitor the pore pressure response in the weak foundation, and the horizontal deformations. Significant upstream movement of the upstream shoulder was a surprise. The seat of this movement was in the 1 m layer of highly plastic clay fill on which the conduit rested. In anticipation of horizontal foundation deformation, 1,900 mm long external collars were used at each joint in the conduit. The wisdom of this measure became apparent with the combined horizontal deformation in the foundation and in the fill. One joint was extended by 380 mm and two others were extended by 200 mm. The total extension amounted to 1,150 mm. Relief wells were installed in the foundation at the downstream toe to reduce hydrostatic pressure in the pervious sub-till sediments which connect with windows in the till in the reservoir.

Dams on Pervious Foundations

Pervious foundations demand the use of one or several expedients to control seepage and to inhibit potential instability caused by seepage. Positive cut-off walls, downstream drainage, relief wells, upstream blankets, complete excavation and replacement of the pervious foundation materials, and soil and rock grouting have all been used.

Hugh Keenleyside dam on the Columbia River (Golder & Bazett., 1967) is an example of an embankment founded on deep pervious deposits, but where continuous flows in the river were not just permitted but mandatory. No cut-off was provided for the dam. Underseepage flows were controlled by means of an upstream blanket, downstream drainage and relief wells.

For the case of the Terzaghi Dam, previously mentioned, it was necessary to inhibit underseepage in the variously pervious foundation and this involved the construction of a 150 m deep clay cement grout curtain through alluvium to bedrock using techniques new for Canada at that time (1960). Similar techniques were used at Outardes No. 4 Dam (1968) in Québec to form a grouted cut-off in talus material some 25 m thick containing large blocks of rock up to 3 m diameter, the voids between blocks being incompletely filled with fluvial sands, gravel and silt (Brown & Comeau., 1970).

In 1972, construction of the Bighorn Dam, a 92 m high earthfill embankment on the North Saskatchewan River in Alberta, necessitated construction of a concrete cut-off through 65 m of alluvial sands and gravels (Fores et al., 1973). The 600 mm thick wall was constructed using a bentonite slurry trench excavated by clamshell and percussion tools. During winter, construction was continued from within a heated enclosure.

The same method was initially considered for the Manicougan 3 Dam in Québec, to be founded on over 120 m of sands, gravel and boulders (Dreville et al., 1970). The depth of cut-off, 121 m, exceeded the maximum height of the earthfill embankment, 107 m, and was to be the deepest such cut-off ever installed to that time (1975). After several trials, the design adopted was a double cut-off concrete pile wall installed using bentonite slurry and comprising two rows of piles and panels. In the deepest centre section each row consisted of interlocking piles, 600 mm diameter; on the flanks, panels 600 mm wide and 3.5 m long were formed. The rows were about 2.5 m apart and were joined by cross-walls where the piles and panels met. It was intended that, where required, the alluvium between the rows would be then grouted. To demonstrate the practicability of the method, a similar wall 75 m deep was first built beneath the upstream cofferdam at Manicougan 5, 120 km upstream of the site.

In contrast to the examples discussed above, the use of a cut-off was not adopted at the site of Lower Notch Dam on the Montreal River in Ontario (Tawil., 1979). The dam, a 123 m high rockfill embankment with a central core, was located within a deep narrow gorge. Below riverbed level, the sediment-filled gorge was 76 m deep with a width of only 24 m at the top reducing to less than 6 m wide at the bottom. All of the pervious gorge infill sediments were excavated and replaced with impervious and other fills. The excavation was complicated by the near vertical walls of the gorge containing shear zones, areas of loose weathered bedrock and open joints. Hence there was need for extensive rock treatment and modification of the rock geometry to minimise potential core cracking.

Dams on Unusual Foundation Conditions

The Daisy Lake Dam (formerly Cheakamus Dam) is a 28 m high earth and rockfill embankment in British Columbia that is founded on a 150 year old, large landslide deposit of collapsed volcanic debris (Terzaghi., 1960). The slide debris came from the left side of the valley, and swept part of the pre-slide forest against the right rock wall. The right end of the dam rests against the right rock wall, whereas the main body of the dam rests entirely on the landslide debris. The slide deposit is variable in thickness, texture and relative density. Particle sizes range from silt to large boulders. The main components of the dam cross section are a wide upstream core zone of the landslide debris, and a downstream zone of rockfill. Transition and filter layers separate the rockfill zone from the core and from the foundation. Special precautions were taken to seal the contact of the landslide debris against the right rock wall upstream of the dam, and to filter the contact downstream of the core zone.

The Brazeau Power Project in Alberta (Hardy., 1968) was located in an area where a mantle of muskeg of thickness varying from 1.2 m to 3.0 m covered almost the entire project area. The project involved 18 km of dykes founded on the muskeg. Removal of the muskeg would have considerably increased the project cost. By use of a test section prior to letting the contract for the dykes, a procedure for construction of the dykes was developed whereby the muskeg was removed only at a cut-off beneath the core section of the dykes. Construction of a haul road and excavation of the cut-off was carried out in winter, when truck access over the muskeg was possible. Dyke fill was placed in summer from the haul road. Height of the dykes ranged up to 9 m, but was typically 4.5 m to 6.0 m. Settlement gauges were installed at some 20 locations. Surveillance of the performance revealed failures at a few locations during the first year. These areas were repaired and toe berms were added without further problems. The success of this project is a credit to the designers, and is an excellent example of the application of Terzaghi's observational method.

Glacial Till as a Dam Construction Material

Glacial till is used widely in Canada as a core material in embankment dams, being one of the most plentiful soils available for dam construction (Milligan., 1976; Loiselle & Hurtubise., 1976; Pepler & Mackenzie., 1976; Peters & McKeown., 1976).

In the Plains region the glacial tills are generally plastic and fines-rich, being derived from glacial grinding of plastic to non-plastic sedimentary rocks. They usually have a sand and gravel content of about 30 per cent, the remainder being silt and clay, with the odd cobble and boulder. Due to the abundance of till in the region as compared to available supplies of sand and gravel, the till usually forms the major part of the dam cross section. Where sand and gravel are scarce, these materials are used judiciously in chimney and downstream blanket drain zones, the remainder of the section being the plastic glacial till. In the arid southern reaches of the plains, the till exists at a water content below optimum. This necessitates the addition of water by irrigation of borrow pits prior to excavation, and / or by sprinkling and moisture conditioning each layer on the fill. Compaction is usually done with heavy sheepsfoot type rollers in layers no thicker than 30 cm. Permeability of the compacted till is in the range of 10^{-6} cm/sec to 10^{-8} cm/sec. This results in many wide core zones remaining unsaturated for decades.

In the Appalachian, Precambrian and Cordilleran regions, the glacial tills usually have fines contents of 30 per cent or less, and in some cases they have barely sufficient to insufficient fines to bind all particles. The fines are commonly low to non-plastic, being derived from glacial grinding of hard crystalline rocks. Cobble and boulder content can be sufficiently high to require separation over a bar grille (grizzly), before placement of the finer sizes in core zones. The finer portion of the till is usually quite moisture sensitive, exhibiting dilatant weaving under the traffic of construction equipment. The till in these regions can be placed and compacted in thicker lifts using heavy rubber-tired equipment, in contrast to the necessity for compacting the fill of the plains region in lifts of no more than 30 cm.

Segregation of till with low fines content during placement is an inevitable reality. Hence, permeabilities of low fines content tills can be high and variable within a compacted core. This aspect has led to piping problems where adjacent downstream filter zones have been broadly graded. The coarser zones of segregated filter no longer meet the conventional Terzaghi filter criteria, even though the gradation curve for the unsegregated filter material may satisfy the criteria. It is now generally accepted in Canada that filter zones adjacent to till cores should consist of sand-rich narrowly graded material in order to avoid segregation and to block migration of core fines into and through the filter.

Use of Other Fill Materials for Embankment Dams

Mention should be made of types of materials other than till that are used in embankment dams in Canada. The cross section of an embankment dam is normally dictated by economical use of locally available soil and rock materials. Many of the high embankment dams in Canada are located in the Precambrian and Cordilleran regions, where hard crystalline rock is plentiful. Dams of various cross sections have been used in these regions, both earth fills and rock fills: rock fills with sloping and / or central earth cores, and with upstream concrete or timber faces; the latter type face being relatively rare; and earth fills where soils are more economically available. In these regions sands and gravels are usually plentiful, rainfall is often high in the construction season, and use of clay or till core material is problematic, even if plentiful, on account of difficulties of compaction of till in wet weather.

At the Bennett Dam (Taylor., 1969), the water content of the available clay deposits was so far above the optimum as to be unusable. The design took advantage of a plentiful source of deltaic sands and gravels close to the site, in what was described as the Portage Mountain moraine. This source supplied all of the fill material, except the dry silt that was added to the core material, and except for the riprap. The section consists of a wide core with shell zones of sand and gravel, and with internal near-vertical chimney zones of transition, filter, and drain, the latter connecting with a downstream blanket drain.

The construction schedule for the dam required fill placement rates higher than at any previous dam. In the absence of a suitable natural core material, the specifications required the addition of local dry silt to the sand and gravel for the core zone. To meet the construction schedule, the contractor chose to use a screening plant for processing the core, filter and drainage zones, combined with a conveyor belt for delivery of the pit run sand and gravel to the process plant. A unique feature of the dam is that the various zones other than the drainage zone have nearly equal and very low stress-strain properties.

Use of Fill Dumped through Water for Dam Construction

In a number of instances throughout the world, cofferdams have been constructed by progressively end-dumping rockfill and finer materials directly into water. In Canada, this technique has been extended by end-dumping till and sand-gravel to form permanent dam structures.

Early experience gained in building cofferdams of till in water up to 10 m deep or less, for the St. Lawrence River Project in the 1950s, was extrapolated to construction of the Hugh Keeleyside Dam (Arrow Lakes) on the Columbia River in British Columbia in 1960 to 1968 (Golder & Bazett., 1967). At the site, the river bed consists of sands and gravel alluvium over 150 m deep containing lenses of open-work gravels. As controlled dewatering was not considered to be economically practicable, the construction of the dam (to be 59 m at final height) involved placement of substantial portions of the dam fill under water. Till was bottom-dumped from barges of large capacity (230 m^3), hinged to split longitudinally, to form an upstream blanket. Till was also placed by end-dumping from trucks on the face of a previously dumped sand-gravel portion of the dam. The till was dumped in windrows at the edge of the sand-gravel fill. The till was then heaped by bulldozers until a slide occurred. A general fill level 1.5 m above river level was found most suitable for construction, the depth of water being about 10 m to 15 m deep. The till was placed at a high air content and at a water content virtually that of the borrow pit, which was close to Proctor optimum. Dry unit weights of the till placed under water were about 95 per cent of the unit weight of the same till compacted by rolling. After almost 30 years of operation the performance of the dam is satisfactory.

Another example of this form of construction, as applied to winter conditions, is given by the Twin Falls hydroelectric project on the Unknown River in Labrador. (Pepler & MacKenzie., 1976). The most practicable means of construction for the dam, less than 15 m in final height, was to dump a rockfill section across the river and then dump locally available till along the upstream face of the rockfill until a seal was obtained. Because of the limited period of suitable weather available for fill placement, the rockfill was placed through the winter of 1961 to 1962 and dumping of till commenced in April 1962. Ice in the river had to be broken up by blasting and care had to be taken to minimize the possibility of burying blocks of ice in the fill. End tipping of till from trucks continued on a round-the-clock basis despite blizzards. Temperatures generally prevailed below freezing until June when they moderated. At that time, it was possible to place and compact till above water level to bring the dam close to its final height. The final upstream slope of the dumped glacial till was about 3H:1V. When water levels were raised the following year, sink holes developed in the till and there were substantial losses of water. The sink hole development is attributed to thawing of the dumped till and possibly to some ice blocks trapped in the mass. Further progressive dumping of till upstream was sufficient to fill the depressions and provide a watertight zone to the dam.

The technique of dumping fill under water was used to initiate the construction of the Gardiner Dam (1964), previously discussed, by dumping sand and gravel into the river, less than 3 m deep. It was also used to form the Eastmain Dam, upstream of the L6-2 reservoir in the James Bay Project (Loiselle et al., 1982). The dam was relatively modest in height (31 m), but construction of the Eastmain riverbed section was complicated by the presence of relatively weak fine-grained soils in the foundation. Sand and gravel fill was dumped into the river to form a base for the dam, the rate of placement being controlled by monitoring piezometric response in the underlying weak soils. When the fill was above river level, it was compacted by vibroflotation to ensure a relatively dense base on which to construct the remainder of the conventional earthfill embankment (1980).

An approximate synthesis of Canadian experience in dumping tills under water to form dams (and cofferdams) is illustrated by Figure 3. The range in gradation of tills commonly dumped is plotted. In general, such tills performed satisfactorily both during construction and on drawdown of 10 m to 15 m water depth. The underwater stability of most of the tills dumped is attributed to low initial porewater pressures resulting from placement at relatively low degrees of saturation, but progressive slips occurred during subsequent dumping causing increasing saturation of the till. Slips appear to be more frequent when "finer" till is dumped and pore pressures tend to be high. The final slopes resulting from such progressive slips tend to approach 8H:1V, as compared to slopes of 3 to 5H:1V when coarser material is dumped. Based on experience, a tentative guideline for the division between "fine" (a) and "coarse" (b) is suggested in Figure 3.

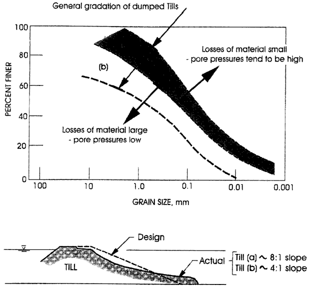


Fig. 3. General experience with dumped tills under water

Effects of Frost

Climate is a major factor in embankment dam construction. Few countries suffer the effects of cold weather more than Canada and particularly the eastern Canadian Shield region. Frost and freezing weather affect dam construction in a number of ways - during borrow excavation and fill placement, by its effect on the core after placement and by the effect of thawing of frozen foundations, such as permafrost. A climate restricting the placement of fill sometimes to only 2 months a year stimulates innovative solutions to the optimum utilization of machines and men.

At Manicougan 3 (Dreville et al., 1970), an earthfill structure 108 m high on the Manicougan River, it was necessary to place glacial till core material in prevailing cold, rainy, short construction seasons. The till was non-plastic and wetter than the optimum water content. To extend the construction season, the water content of the till was reduced by the use of a rotary kiln dryer. Light vibratory smooth-faced rollers were used to compact the till, producing a moist deformable core in-situ. This has performed satisfactorily to date.

Similarly, the dykes of the Outardes 2 hydroelectric scheme, adjacent to Manicougan 3, were constructed with an impervious core comprised of a local clay mixed with sand which had been dried in a rotary kiln to develop a mixture at near optimum water content (Hammamji et al., 1982).

Hydroelectric development of the Nelson River in Northern Manitoba was complicated by the widespread presence of discontinuous permafrost. Ice in the form of lenses or inclusions was conspicuous in lacustrine deposits. Where encountered in glacial till it was difficult to detect because of the very dense nature of the till. The presence of ice in till which must be excavated makes removal even more difficult than normal, necessitating drilling and blasting. When the materials are then exposed to summer temperatures they soften quickly exacerbating problems of handling and placement.

Frozen foundation soils below low dykes were left in place. Reservoir impoundment induces thawing of the underlying frozen soils. Lacustrine soils overlying dense till are ice-rich in contrast to frozen dense tills. Hence they contribute most of the thaw-settlement, for which the dykes must be designed. An example of the design and construction of such dyke fills is the Kelsey project in Manitoba completed in 1960 (MacDonald., 1963). Sand drains were installed in the foundation to improve stability during thawing and the dykes were built of sand to accommodate settlement. These dykes provided experience for subsequent work at Kettle Rapids in 1969 (Macpherson et al., 1970) and Long Spruce in 1977.

Not all tills affected by frost are equally dense. The broadly graded silty tills on the Labrador Plateau exist at a low density, probably as a result of their method of deposition. These loose tills, combined with high groundwater conditions and a cold wet construction season, are difficult to handle. Problems of instability of these tills were encountered during the construction of the Sandgirt-Lobstick dykes of the Churchill Falls, Labrador Project in 1969 to 1970, causing costly disruption to normal construction operations (Seemel & Colwell., 1976). Frost penetrations of over 5 m were measured. The fact that about 65 km of dykes, ranging in height from 10 m to 30 m high, were successfully completed by 1971 is a tribute to both engineers and contractors.

A striking example of the long-term behaviour of an embankment dam built on discontinuous permafrost is Waterloo Dam on the Charlot River in Northern Saskatchewan (Noonan et al., 1982). This 19 m high structure built in 1961 with a core of uniform silt behaved well under initial impoundment. With time, permafrost in the left abutment thawed inducing subsequent leakage and sloughing of slopes. These were remediated. However, over the next two decades, repeated annual severe winter freezing caused almost permanent ice lensing and ice boils in the downstream portion of the silt till core material. The situation was remedied by partial excavation and replacement of the core, together with the provision of styrofoam insulation to the upper part of the central section of the embankment.

Another example is given by the Snare Rapids Hydro Development situated on the Snare River about 150 km north-west of Yellowknife (Eckenfelder., 1998). Construction began in 1946 and was completed in 1948. The main dam is a zoned earth structure with thick quarried granite-gneiss shells. The dam has a maximum height of 19 m and a length of about 80 m.

The site is in the continuous permafrost zone of Northern Canada, the depth of permafrost being probably in excess of 100 m. Although the area was completely glaciated there were no significant till deposits. The only fine material was a fine silt, locally called "rock flour," found in depressions in the predominantly exposed bedrock under a cover of moss and grasses. The silt is impervious, its permeability being in the range of 10^{-8} cm/sec to 10^{-9} cm/sec. Except for the top 15 cm, the silt was permanently frozen. In the early spring, as soon as the mid-day sun provided some heat, the surface of the deposits was peeled off with bull-dozers as it thawed and then stockpiled. The object was to allow it to dry, but because of the low permeability, the extent of drying was negligible.

Placing of the core began in normal fashion by compacting with sheepsfoot rollers, but the supply of dry silt was soon exhausted. Thereafter, the only way to proceed and finish the dam was to cast the semi-liquid mud into the core with draglines and clamshells. There was an ample supply of clean sand of almost any grading desired. The core was flanked by thick zones of progressively coarser sand. Furthermore, the thick outer zones, as mentioned earlier were composed of high quality quarried rock. Thus, while the process breached all of the accepted rules for the construction of stable embankments, the result was quite acceptable. Pore pressures have not been measured and it may be that, even today, after 50 years, the pressure at the centre of the core may be what is normally considered excessive. Some cracking occurred along the crest a few years after completion, but after being filled with sand, the cracks have not re-appeared. Recent inspections indicate satisfactory conditions for the dam.

Stability of Reservoir Rims

Increasing attention from the 1970s onward has been given to potential instability of reservoir rims due to submergence of the toes of the valley walls during reservoir impoundment. It is now common practice for assessments to be made of areas of potential instability and of required stabilization measures. This is a difficult and complex matter, due to the sparsity of accurate information on the subsurface conditions along the reservoir rim. The general procedure is to install instrumentation consisting of pore pressure and deformation apparatus at areas showing indications of previous instability well prior to reservoir filling, and to monitor the effects of reservoir filling. Several years of deformation measurements may be required to determine if a slide is moving. The principal concern about rim instability is whether or not a slide movement will have sufficient size and velocity to create a wave that will overtop the dam and result in sudden release of the reservoir.

In the Cordilleran region, scars of former landslides in both rock and overburden are fairly easily identifiable. Two of many large rockslide masses in the Columbia River valley deserve mention. These are the Downie Slide, which is located at mid-point of the reservoir of the Revelstoke Dam, and the Dutchman's Ridge slide, which is located upstream of and close to Mica Dam (Lewis & Moore., 1989). The Downie Slide mass contains about 2 billion cu.m. of rock. The Dutchman's Ridge Slide contains about 500 million cu.m. of rock. Pre-flooding studies of the rims of these reservoirs had identified many areas of former rockslide activity, in the form of definite scarps and of linear troughs that suggested downslope sagging. The probable cause and timing of the forming of these features was problematic: - were they the result of glacial widening and deepening of the valley, or indicative of movement in the Holocene, and had they occurred suddenly or as long-term creep? The adopted procedure was to install deformation meters at all such locations, supplemented by pore pressure measurements at locations considered to be critical.

At the Downie Slide evidence of current creep was determined by surface deformation measurements, supplemented by slope indicators installed in drill holes. When creep became evident, two drainage tunnels were installed within the slide mass prior to reservoir filling. These were combined with fans of drainage holes drilled from the tunnels. The achieved objective was to lower the pore pressures within the mass by an amount that would more than offset the destabilising effect of flooding of the toe of the slide.

At the Dutchman's Ridge Slide (Moore., 1990), the rate of the creep was so slow that 10 years of slope indicator measurements after the reservoir filling were required to confirm that the mass was creeping. A drainage tunnel with fans of drilled drainage holes was then installed, similar to Downie Slide. The drainage measures for each case have been effective in major lowering of the pore pressures within the masses over long distances within the slide masses.

Safety of Dams

In the 1980s, the Canadian Dam Safety Association (CDSA) was formed, with representation from all provinces. The association has worked closely with the Canadian National Committee of the International Commission on Large Dams (CNC / ICOLD). A detailed and comprehensive document entitled "Dam Safety Guidelines" was published by CDSA in January 1995, with input from each of the provincial regulatory agencies, from major dam owners, and from dam design consultants. The objectives of the document were threefold. First, to define requirements and outline guidelines so that safety of existing dams can be investigated and identified in a consistent and adequate manner across Canada. Second, to enable the consistent evaluation of dam safety deficiencies leading to the construction of improvements which contribute to dam safety. Third, to provide safety legislation and regulation.

A synopsis of six statements of safety requirements in the guidelines is as follows:

i) Each dam shall be classified in terms of the reasonably foreseeable consequences of failure.

ii) The dam along with its foundation and abutments shall have adequate stability to safely withstand extreme loads as well as the normal loads.

iii) A Dam Safety Review shall be carried out at regular intervals for dams and associated facilities, including their operation, maintenance, surveillance, and emergency plans, to determine if they are safe in all respects.

iv) The review of design and construction shall be sufficiently comprehensive to demonstrate whether the dam, discharge facilities and reservoir slopes meet all currently applicable safety requirements.

v) The review shall determine if the appropriate level of emergency preparedness exists and is adequately documented.

vi) Dams shall be designed and evaluated to withstand ground motions associated with a Maximum Design Earthquake (MDE) without release of the reservoir. Selection of the MDE for a dam shall be based on the consequences of dam failure.

The safety of dams in Canada is a matter of jurisdiction by the provincial rather than the federal governments (Dascal., 1991). Dam safety legislation and programs in the provinces of British Columbia, Alberta, Saskatchewan, Manitoba, Ontario, Québec, Newfoundland and Labrador were presented at the 1986 Dam Safety seminar in Edmonton.

An initial dam safety measure was the retention of an independent review board for oversight of design and construction. It had been adopted by owners of the major dams in Canada as early as the 1940s and well prior to the CDSA guidelines. This has been standard practice for major dams throughout Canada. The provinces of British Columbia and Alberta have tended to exert earlier and stronger regulation of dams than the other provinces, on account of their mountainous terrain.

Recent developments in the methodologies for assessment of flood and seismic hazards to dams had prompted a general review of the safety of existing dams with respect to these hazards. Two regions of Canada have significant exposure to major seismic events. They are the Cordilleran Region and the St. Lawrence River Basin. In the last 25 years, many dams in these regions have been upgraded to meet current requirements. The work relative to improvement of seismic stability and control of seepage is on-going, with priority being given to those posing the most serious consequences. Typical remediation activities to meet current safety guidelines are those carried out in British Columbia. This work has included assessment of seismic risk at Elsie Lake Dam (McCammon et al., 1991); replacement of the Alouette Dam, a 20 m high hydraulic fill dam constructed in 1926; addition of a downstream berm at Coquitlam Dam, a 30 m high hydraulic fill dam constructed in 191[illegible]; in-situ deep compaction of a zone of loose material in the foundation at the Daisy Lake Dam (Cattanach & Brunner., 1986); and major cut-offs and foundation drainage at the John Hart Dam (Cathcart., 1989).

Remediation work on dams in Alberta for the same purpose is described by Slopek et al., 1989; Wade et al., 1986; Williams et al., 1982. This work has included among other things, the raising of the cores of several dams such as Bearspaw, Cascade, Ghost by insertion of a plastic membrane between the former top of the core and the dam crest; installation of a partial cut-off; an upstream impervious blanket, and a downstream inverted filter at the Three Sisters Project.

Use of Instrumentation and Monitoring

Installation of instrumentation in embankment dams prior to and during dam construction is now an accepted practice in Canada, for all dams of height greater than about 15 m and for smaller height dams with unusual site conditions, where the consequences of a failure of a small dam would be great. One of the first dams in Canada in which instrumentation was installed was the Pothole Dam in Southern Alberta in 1947. Major advances in types, variety and quality of instruments and in remote data acquisition devices have been made in the past few decades. Experience has shown that instruments should be durable and robust in order to have reliability and long life. Without doubt there will continue to be improvements with time which will necessitate upgrading of existing instrumentation required for long-term monitoring.

Instrumentation is useful for two purposes. The first is to observe the behaviour of the dam, foundation and reservoir rim for comparison with predicted behaviour, and hopefully to detect any anomalous behaviour. Realistically, the observations represent only a small sampling of the entire dam, particularly with respect to piezometric pressures. Nevertheless with intelligent interpretation, the combination of observations of settlement, leakage, and deformation provide a reasonably sound basis for assessment of behaviour. Instrumentation is also extremely helpful when unusual behaviour is encountered to gain an understanding of the cause and mechanics of that behaviour. Additional instruments of selected type and location can then be installed to investigate the mechanics of the problem.

Concluding Remarks

The paper attempts to illustrate that problems of design and construction of embankment dams in Canada are generally related to geology, particularly glacial geology, and to severe climatic conditions. Solutions to these problems, as typified by the examples in this paper, have been progressive and innovative, advancing the state-of-the-art of building embankment dams and providing valuable experience to the profession world-wide.

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