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Behaviour of the LG 4 main dam

Comportement du barrage principal de l'aménagement LG 4

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SYNOPSIS: The 125 m high main dam at LG 4 is characterized by a zoned earth-rockfill section and a steep 70 m high rock abutment. Stress-deformation behaviour of the dam was studied through finite element analyses and the design of the dam was oriented to take into account the likely occurrence of arching and any eventuality of hydraulic fracturing in the till core. The performance of the dam was monitored during construction, reservoir filling and operation stages through visual inspection and instrument monitoring. This paper presents an analytical review of the observed stresses, deformations, pore pressures and seepage quantities and their implications in terms of the arching and the hydraulic fracturing potentials of the dam core. Comments are made regarding the effectiveness of certain design features in eliminating or minimizing these concerns.

INTRODUCTION

The LG 4 dam on La Grande River in Québec, Canada, is located about 1000 km north of Montréal. The 125 m high and 3800 m long earth-rockfill dam with a glacial till core is characterized by features which could render its core susceptible to arching and consequently to hydraulic fracturing. These inevitable features included (i) the interzone contacts between core and filters composed of materials of different stress-strain properties, (ii) a part of the dam resting on a high and steeply sloping rock abutment, and (iii) downstream curvature of the dam axis in the secondary valley. Design of the embankment was suitably adjusted to account for these conditions.

Construction of the dam was carried out between March 1979 and November 1981 and the reservoir filling between March 1983 and December 1983.

This paper presents the observed deformation, stress, pore pressure and seepage performance of the dam during construction, reservoir filling and subsequent operation period. Relevant comparison with the results of the finite element analyses carried out during design are also made.

PRINCIPAL DESIGN FEATURES

As shown on Figure 1, the site topography divides the dam into four segments of which the two major segments are the main river valley and the secondary valley. The 800 m long river valley, where the maximum dam height is 125 m, is characterized by a 70 m high abutment inclined at about 55° to the horizontal (Figure 2). In the secondary valley the dam axis is curved downstream with a radius of curvature of 700 m due to the constraints imposed by topography and the sitings of the intake and powerhouse structures. The dam is founded almost entirely on bedrock composed of granite and gneiss of Precambrian age.

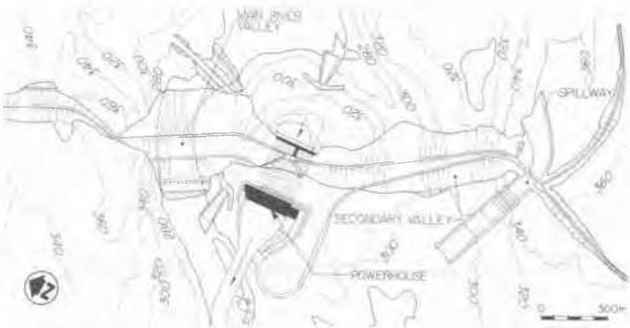

Fig. 1 LG 4 Main Dam - General Arrangement.

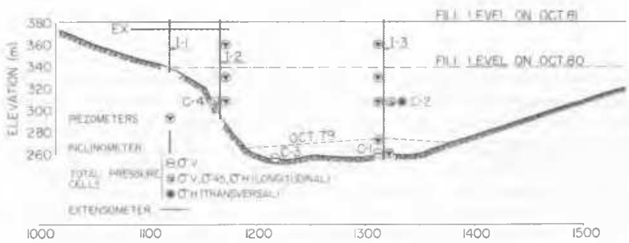

Fig. 2 Longitudinal Profile and Location of Instruments in Main River Valley.

The zoned earth and rockfill embankment dam with a volume of about 19×10^6 cu. m. consists of a wide central till core protected on the upstream and downstream by gravelly sand filter zones (Figure 3). The non-plastic glacial till for the core and the gravelly sand for the filters were well graded materials as shown on Figure 4. The internal and external shell zones were constructed respectively from pit-run sand and gravel and rockfill from powerhouse and spillway excavations. The general specification requirements for placement of these materials were as follows:

Till in general fill: Placed in 45 cm thick loose lifts and compacted to 97% of Standard Proctor maximum dry density at water contents varying between optimum -1% and optimum +2% by 4 passes of 45 tonne pneumatic rollers.

Till in contact zones: Placed in 15 cm thick loose lifts and compacted to densities given above at water contents of optimum to optimum +2%.

Gravelly sands in filters: Placed in 45 cm thick loose lifts and compacted by 3 passes of light (5.5 tonnes) vibratory rollers (Average relative density aimed at 70%).

Sand and gravel in shells: Placed in 45 cm thick loose lifts and compacted by 3 passes of heavy (9 tonnes) vibratory rollers (Minimum relative density aimed at 70%).

Rockfill: Placed in lifts of 1 m and 2 m thicknesses for the internal and external zones respectively and compacted by 4 passes of heavy (9 tonnes) vibratory rollers.

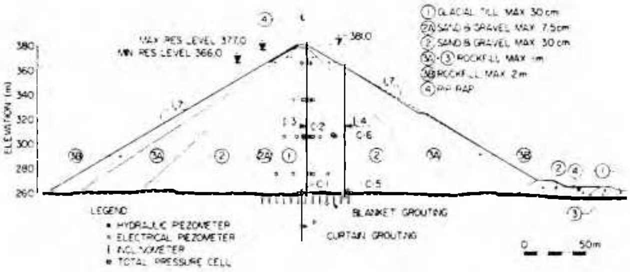

Fig. 3. Typical Section and Location of Instruments in Main River Valley

An extensive quality control programme ensured the execution of foundation treatment and material selection and placement as per the design specifications. Preparation of rock foundation for the core and filter zones consisted of thorough cleaning, slush grouting, shotcreting, dental concreting and blanket and curtain grouting. Details of design, construction methods and quality controls exercised have been provided elsewhere (McConnell et al, 1982 and Paré et al, 1982).

The stress-deformation analyses of the dam embankment in the transverse and longitudinal sections were carried out using non-linear stress-strain relationships and simulating construction in progressive layers as well as full reservoir loading. Details of the analyses, which were performed to study the extent of arching and hydraulic fracturing potentials and also the effectiveness of the remedial measures incorporated in the design have been given by Paré et al (1984).

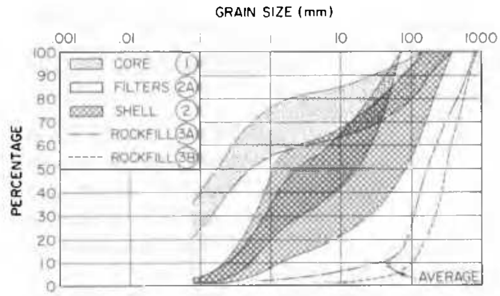

Fig. 4 Gradation of Embankment Materials.

The design of the dam included many features which are considered effective in minimizing cracking of a till core and promoting its self-healing. These features are a wide impervious core, placement of till in bedrock contact zones at higher moisture content and in thin lifts, use of natural sand and gravel for upstream filter, wide filter zones, internal shoulder zones of pit-run sand and gravel, and a thorough and careful treatment of the abutment rock including the elimination of surface irregularities and local slopes exceeding an inclination of 70°. The core and the downstream filter zones were provided extra widths in the vicinity of the steep abutment. Furthermore it was decided to maintain heavy compaction of the core till and to somewhat reduce the compaction of the gravelly sand in filters. The placement details given earlier reflect these modifications.

PERFORMANCE

Performance of the embankment and foundation of the dam has been monitored during construction, reservoir filling and reservoir operation through visual inspection and readings from instruments such as inclinometers, settlement cells, extensometers, surface monuments, hydraulic and electrical piezometers, electrical vibrating wire and pneumatic total pressure cells and weirs. Most of the instrumentation was confined to critical areas such as the steep abutment section, the highest dam section in the main river valley and the curved axis section in the secondary valley.

Due to space limitations, the coverage in this paper is confined primarily to some selected instruments (Figures 2 and 3) in the main river valley of the dam where the chances of interzone arching were maximum and the corresponding steep right abutment where tensile strains were likely to occur. Occasional references to the behaviour in the other parts of the dam are also made to complete this review.

Pore Pressures

The magnitude of pore pressures developped in the till core during construction was small such that the maximum value of the coefficient r_u varied between 0,1 and 0,2. Following the initial reaction to the placement of the first few lifts of fill, the piezometric levels continued to increase with the increasing fill height but at a progressively diminishing rate. Almost complete dissipation occurred during the following winter shutdown. During reservoir filling the pore pressures throughout the dam core increased systematically and simultaneously with the increasing reservoir levels. All changes corresponded well with the rate of reservoir filling and with the relative locations of the piezometers. These observations are illustrated on Figure 5 by typical plots of (a) the transitory saturation fronts corresponding to different levels of the reservoir and (b) the distribution of the equipotential lines corresponding to the maximum reservoir level. These plots are based on the observed piezometric levels. The equipotential line distribution curves developped for similar pond levels attained since the first complete filling have compared well with the plots corresponding to the initial observations.

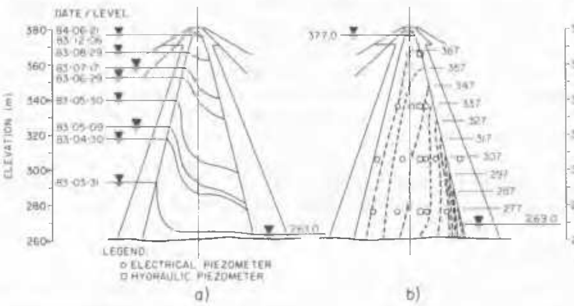

Fig. 5 Seepage through Core in Main River Valley. a) Saturation Fronts during Reservoir Filling. b) Equipotential Lines at End of Reservoir Filling.

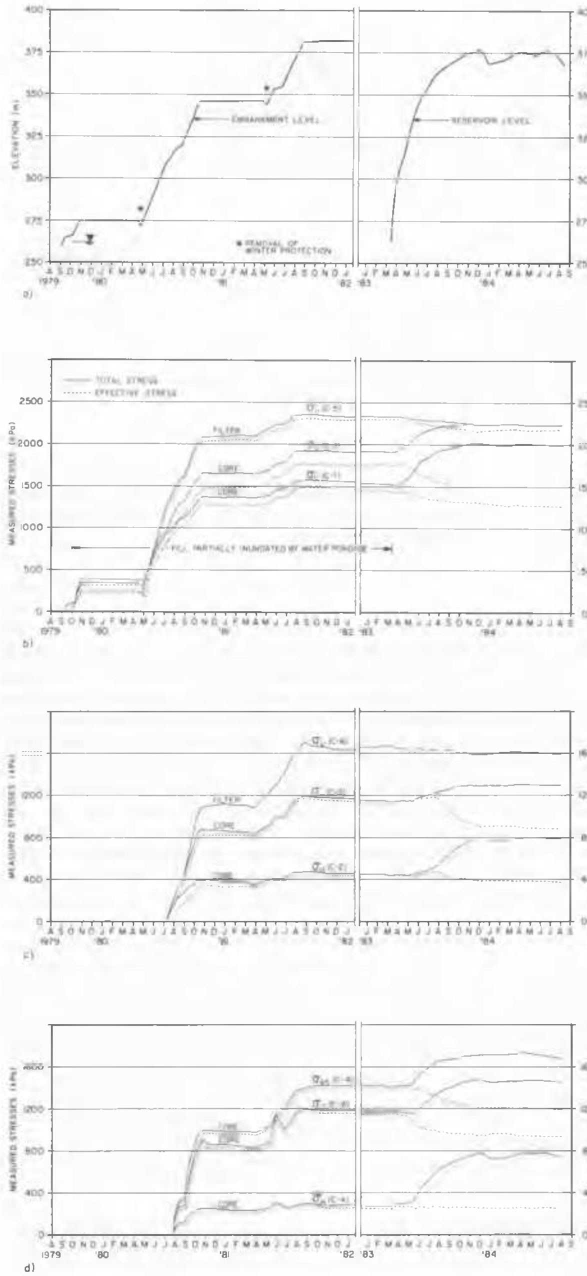

Fig. 6 a) Fill Placement and Reservoir Filling. b) Stresses at Foundation Level in Main River Valley. c) Stresses at Elevation 306 in Main River Valley. d) Stresses at Foundation Level over Steep Abutment.

Stresses

Figure 6 shows development of total and effective stresses within the core and the downstream filter zones as well as at their contact with the bedrock foundation in the main river valley and over the steep abutment. Also shown on this figure are the curves of fill construction and reservoir filling. Figure 7 shows contours of effective vertical stress distribution at the instrumented section in the main river valley as obtained from the finite element analyses for the end of construction and the end of reservoir filling stages. Also indicated on this figure are the locations of the total pressure cells and the corresponding measured vertical stresses.

During construction, the stresses increased gradually and simultaneously with the raising of the fill without any abrupt changes or discontinuities. Based on detailed analyses of the observed stresses, the direction of stress orientations is considered to conform satisfactorily with the design predictions. Thus, the major and minor principal stresses were practically vertical and horizontal respectively in a typical section with flat foundation but were significantly rotated in the vicinity of the steep abutment. The magnitude of the measured stresses, on the other hand, have somewhat deviated from the corresponding predicted values.

As shown on Figure 7 for a transverse section in the main river valley, the analyses had predicted almost equal magnitudes of vertical effective stresses in the core and the filter zones for the end of construction condition with slight core to filter stress transfer. This implied negligibly small development of interzo-

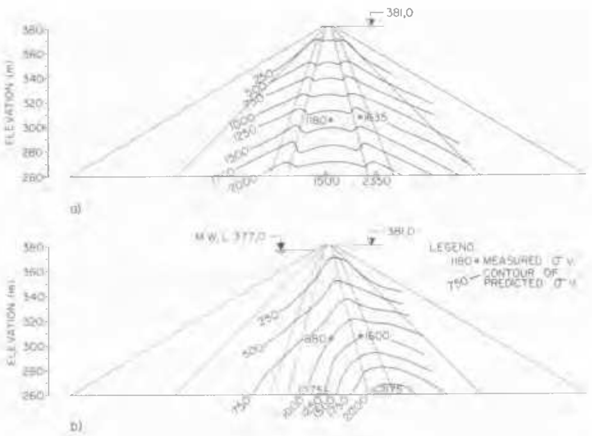

Fig. 7 Predicted and Measured Effective Vertical Stresses at a) End of Construction and b) End of Reservoir Filling.

ne arching. Similarly no interzone arching was indicated for the end of reservoir filling condition. The measured stresses, however, do show some arching effets. A ratio of the effective vertical stresses in the center of the core to those in the center of the filter zones can provide an empirical indicator of the extent of stress transfer and consequently of arching. This ratio at end of construction was about 0,7 both at foundation level and within the fill (elevation 306) compared to a predicted value of about 1, indicating presence of arching at all levels of the fill. For the end of reservoir filling condition, this ratio of measured vertical stresses was about 0,6 throughout the height of the fill compared to the predicted values of about 0,6 at foundation level and about 0,8 at mid-height of fill. It should be noted that the predicted values of less than unity for end of reservoir filling condition reflect the effect of core submergence on the effective stresses and are not related to arching phenomenon. This shows that whereas the arching is virtually eliminated near the bottom of the fill, it is apparently still maintained in the upper parts.

The interzone arching effects discussed above are not considered consequential at any stage insofar as the occurrence of hydraulic fracturing is concerned. In this regard the ratio of effective horizontal (in both transversal and longitudinal directions) and vertical stresses decreased progressively from about 1 to about 0,4 during construction and remained constant at about 0,4 throughout reservoir filling and subsequently. This indicates a structural readjustment with wetting such that the ratio σ_h'/σ_v' remained practically unchanged, ensuring positive effective stresses in all directions. This readjustment within the fill must develop to accommodate the changing stress condition in conformity with the inherant nature of the material and its stress history. Positive values of effective stresses in all directions are indicative of absence of hydraulic fracturing. This condition is applicable to both the main river valley and the secondary valley.

The stress orientations during construction in the bottom part of the fill over the steep abutment deviated by about 30 degrees from those in a typical section. Consequently the measured stresses on a plane inclined at 45° were greater than vertical stresses (Figure 6). The stress orientations in the upper part of the fill were not influenced by the abutment inclination and were essentially similar to those in a typical section elsewhere.

As predicted, significant arching is suspected in the vicinity of the steep abutment. This is evidenced by the aforementioned stress rotations, increased magnitudes of deviator stresses, and finally direction and magnitudes of deformations as detailed later. Excessive arching may reduce effective stresses to negligibly small magnitudes and produce conditions conducive to hydraulic fracturing. However this is not the case at the abutment where positive effective stresses are inferred from measurements. Furthermore, during reservoir filling, favourable effective stress adjustments are evidenced by the values of the stress ratio σ_h'/σ_v' increasing from 0,23 at the end of construction to 0,28 at end of reservoir filling. It was with a view to facilitate these stress readjustments that relatively more humid material in the core to bedrock contact regions was specified in design.

Deformations

Based on inclinometer readings, the settlements during construction in the core and downstream filter/shell zones increased gradually and simultaneously with the increasing fill thickness and maintained the predicted parabolic shape of the height versus settlement curves. The magnitude of the observed and predicted settlements in the core were quite comparable for fill heights of up to about 70% of the final height. However, the measured settlements were significantly smaller than the predicted values for the remainder of the core raising. Thus the measured and predicted maximum settlements at the end of construction were about 19 cm and 29 cm respectively in the center of the main river valley (Figure 8) and about 13 cm and 19 cm respectively in the center of the secondary valley (not shown here) (Garneau et al, 1982). These measured settlements of the core correspond to a total strain of about 0,35 to 0,5%. In the downstream filter/shell zones, the measured settlements at all stages of construction amounted to about half of the predicted settlements. Thus the observed maximum settlements in the main river valley (Figure 8) at end of construction amounted to about 12 cms, i.e. a total strain of less than 0,3%, compared to a predicted value of about 24 cm. Settlements during reservoir filling were negligibly small for both the core and the filter/shell zones.

As can be expected for an embankment with a symmetrical central core, the horizontal displacement during construction were small at all locations except over the steep abutment. Based on readings of the inclinometers (Figure 8) and the surface monuments, transverse displacement for full reservoir condition occurred at or near the dam crest. The magnitude of displacements

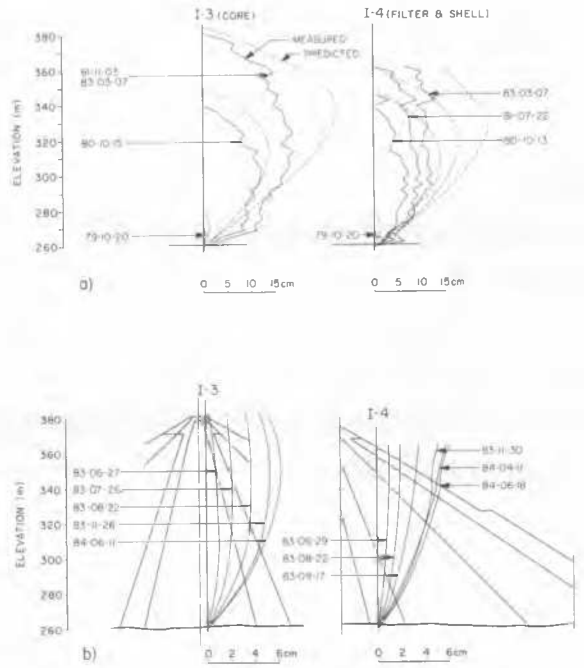

Fig. 8 Embankment Deformations in Main River Valley: a) Settlements and b) Horizontal Displacements during Reservoir Filling.

measured to date has reached a maximum of about 6 cm in a downstream direction compared to predicted values of about 40 cms.

Over the steeply inclined abutment (Figure 9), the settlement behaviour during construction of the upper part of the core was similar to that of a typical section elsewhere without any appreciable lateral displacement. Below about elevation 350, settlement was accompanied by significant lateral displacements in a downhill direction. The differential settlements and the differential horizontal displacements at any given elevation over the two sides of the abutment hump amounted to a maximum of about 12 cms and 6 cms respectively. The general direction of these displacements was confirmed during reservoir filling by surface monuments and also by a chain of extensometers which measured tensile strain of up to 2×10^{-4} between stations 1085 and 1135 and compressive strains of up to 2×10^{-5} between stations 1135 and 1175. The provision of a relatively wet (i.e. flexible) till is believed to have been beneficial in rendering facility of displacement to the till core at the abutment contact zone where local shear movements producing the desired structural readjustment are inferred to have occurred.

Figure 10 shows mean values of the end of construction secant moduli based on vertical strains accumulated over 12 m fill thickness and the corresponding stresses for the core and the filter/shell zones. The observed moduli are based on strains from inclinometer readings and stresses from extrapolation of pressure cell readings. The predicted moduli are derived from the results of the finite element analyses. Values of the predicted moduli for the till core varied from about 300 000 kPa near the bottom to about 150 000 kPa in the upper half of the fill. The magnitude of the corresponding measured modulus was about 240 000 kPa for most of the fill height with some increase in the upper part. In the granular filter and shell zones both the predicted and the measured moduli increased with depth, the measured moduli exceeding the predicted values by about 30 to 100%. The higher values of measured moduli are believed to be related to greater than predicted compaction achieved for the well graded granular materials. (Average in-place relative density of 85% compared to a targeted relative density of 70%). The moduli represent the actual stress-deformation behaviour of the fill materials and provide a convenient means of prediction for embankments composed of similar materials and constructed in a similar way.

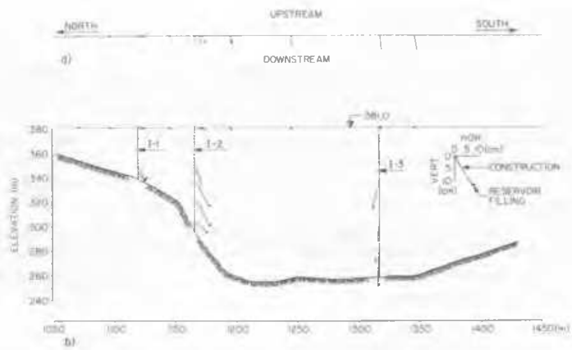

Fig. 9 a) Horizontal Displacements of Dam Crest due to Reservoir Filling. b) Accumulative Movements in Core in Longitudinal Plane.

Seepage

The seepage flow from the river valley segment of the dam is collected and measured at a weir installed in the downstream cofferdam. The measured seepage quantities increased gradually with the increasing reservoir level and achieved a maximum of about 9 l/s for nearly full reservoir level. All seepage observed to date has consisted of clear water. No significant flows have been observed at other dam segments. The low seepage observed is indicative of a relatively impervious core, a good core to bedrock contact and a watertight foundation.

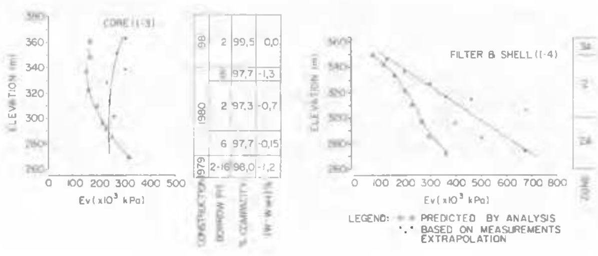

Fig. 10 Secant Moduli Based on Predicted and Measured Settlements in Core and Filter/Shell Zones.

CONCLUDING REMARKS

Based on a detailed programme of visual inspections and instrument monitorings, the performance of the dam embankment is considered to be satisfactory. The instrument results have indicated some interzone arching but no hydraulic fracturing potential in the main river valley and the secondary valley. This arching is attributed to the ease of compaction of gravelly sands which produced filters of high density and low deformability despite the reduced number of roller passes.

Near the steep abutment, although arching is considered to have developped, the risk of core cracking due to hydraulic fracturing is believed to be low. This is based on the fact that the core material appears to behave as a fairly flexible material permitting readjustment of the soil structure with changing stress conditions and thus providing for a material and stress continuity. In this regard the provision of the various preventive and self-healing features is believed to have proven beneficial and effective.

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