THE USE OF STABLE GROUT AND G.I.N. TECHNIQUE IN GROUTING FOR DAM REHABILITATION

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ABSTRACT

The left abutment concrete gravity dam and the East wall of Spillway # 1 are two of the ten water retaining structures constructed between 1923 and 1926 for the impoundment of Lac St-Jean at the head of the Saguenay River in Quebec.

By 1990, heavy seepage was occurring through construction and lift joints and important concentration of leached material were accumulating on the downstream faces of these structures. At that time, Alcan decided on a global rehabilitation project aiming at ensuring the integrity of the structures for the next 50 years; construction began at the end of 1990 and is now well under way.

One of the key aspects of the remedial works was the grouting of the structures using the recently developed Grout Intensity Number technique (G.I.N.). This paper describes the investigation results, the grout mix testing program, the adaptation of the G.I.N. technique to concrete structures, the design of the grout control curves, the grout hole pattern, the equipment used and finally the grouting results.

RÉSUMÉ

L’aménagement hydro-électrique d’Isle-Maligne est situé à l’exutoire du Lac St-Jean et est constitué de dix ouvrages de retenue dont le barrage latéral nord et le mur est de l’évacuateur, construits entre 1923 et 1926.

En 1990, la détérioration générale des ouvrages, notamment au niveau des joints de reprise engendrait d’importantes fuites d’eau et une lixiviation progressive du béton. Alcan décida alors de réaliser la réfection complète de ces ouvrages dans le but d’assurer leur intégrité pour les prochaines 50 années.

Un des travaux majeurs de cette réfection fut l’injection des ouvrages suivant la technique récemment développé du Grout Intensity Number (G.I.N.). Cet article décrit les résultats des investigations effectuées préalablement aux travaux d’injection, le cheminement suivi pour définir le coulis approprié et pour adapter la technique G.I.N. aux ouvrages de béton, la conception des courbes pression-volume injecté, le choix des emplacements de forage, les équipements utilisés et finalement, les résultats obtenus.
Introduction

Within the scope of the overall rehabilitation of the Isle-Maligne hydroelectric Power complex started by Alcan in 1990, is the grouting of major concrete structures. This paper brings additional information on a recent grouting technique, developed in the past decade in Europe and South America, known as the Grout Intensity Number (G.I.N.). Both normal and micro-fine cements were used to seal open fissures which developed in the concrete dam structures during their 70 years of existence. The use of the G.I.N. technique, which permitted higher grouting pressures with minimum risk of damage to the structure combined with stable fine cement grout, were determinant factors in assuring the success of the grouting programme.

This paper discusses the results of investigations of the left abutment dam, the testing program conducted to select the grout mixes, the adaptation of the G.I.N. technique for grouting the concrete structures, the design of the grout control curves, the grout hole pattern, the equipment used and finally the results of grouting.

Project Description

The Isle-Maligne hydroelectric Complex is located on La Grande and La Petite Décharge, Lac St-Jean, Alma, Quebec, Canada. The complex comprises one concrete gravity abutment dam, six concrete spillways, two earth dikes and a 402 MW powerhouse. The overall rehabilitation of the complex was undertaken in 1991, following a thorough investigation campaign including drilling, in-situ testing and laboratory testing. At the time of writing, rehabilitation works which are evaluated at around 100 millions dollars were 50% completed. The grouting of the left abutment dam and the partial grouting of the spillway no. 1 was completed in the summer of 1993. This paper covers only the grouting of the left abutment dam.

The left abutment dam is a 43 m high by 110 m long concrete structure constructed between 1924 and 1926. The dam is located on the north side of the powerhouse where it completes the closure of La Grande Décharge between the left abutment and intake structure of the powerhouse. Investigations by core drilling revealed that the concrete of the structure was generally sound, except for some local lift joints where heavy leaching was locally observed.

The sealing of the dam with cement grout was one item among the overall rehabilitation works of the left abutment dam which included the installation of pre-stressed anchors during the summer of 1991, the removal of a 30 cm layer of concrete on the downstream face in the winter of 1992-1993, the grouting of the dam in the summer of 1993 described in this paper, and finally, the resurfacing of the downstream face of the dam.
Investigation of the Dam

The investigation was done primarily with diamond drilling equipment, with in-situ water testing and laboratory testing on core samples obtained from the concrete and rock foundation. Underwater inspection of the upstream face of the dam was also carried out. The latest drilling campaign completed in July 1990 established that the body of the dam is composed of cyclopean type concrete with rock boulders of metric size scattered within the concrete mass and along lift joints. This concrete is generally sound but is intersected by sub-horizontal open discontinuities.

The sub-horizontal discontinuities encountered were of two types:

- the first consisted of open lift joints, with leached out zones characterized by oxidized walls. These are water bearing and subsequent water tests indicated absorptions in the range of 40 to 180 lugeons,

- the second type of discontinuities consisted of tighter lift joints with rough surface fractures. Water absorption in most of these fractures was negligible.

The rock foundation consisted of a very hard black brittle anorthosite with few widely spaced generally tight joints. However, some open joints with traces of oxidation were identified just below the rock concrete interface. Water tests through these open joints indicated permeabilities in the range of 100 lugeons.

Results of investigation revealed that concrete deterioration was concentrated primarily along water bearing lift joints in the body of the dam. Interior deterioration was caused by leaching action while at surface, deterioration was caused by repeated freezing and thawing cycles. Based on investigation results, it was decided to rehabilitate the mass concrete with a grouting program designed to seal both the wider and the relatively smaller discontinuities.

The Grouting Program

The Goal to be Achieved and the Selection of Grouting Product

The grouting was designed to create an impervious barrier within the concrete structure so as to prevent all circulation of water through the dam. Two types of openings, with different characteristics, had to be grouted, each required a different approach: the larger open water bearing discontinuities could accept type 30 cement, the fine discontinuities required a micro-fine cement.

A comparative study of costs, durability, advantages and disadvantages of the various products available (elastomers, polyurethane, epoxy, cement) was conducted. The study concluded that grouting with micro-fine cement was the most economical and practical method for sealing the finer discontinuities.
Grouting with a chemical product was discarded not only because of the higher costs but for the numerous problems it raised. The study revealed that the only chemical product having the required durability capable of sustaining repeated thawing and freezing cycles without debonding were epoxy compounds. These products are costly, and are difficult to manipulate and inject in large volumes through 30 to 50 m deep holes.

Nevertheless, provisions for local epoxy grouting, through short holes drilled through specific features both on the upstream and downstream faces of the dam, was included in the contract in areas where micro-fine cement grouting could not be assured.

**Contractual Approach**

The grouting method adopted was intended to minimize risks for both the Contractor and Owner and to successfully complete the sealing part of the rehabilitation program.

To meet these objectives, the following items were included in the Contract:

- detailed description of grouting sequences,
- access to areas of structures to be grouted,
- inclusion of grouting works into the schedule of other activities of rehabilitation works,
- capacity of Contractor to successfully complete grouting works,
- technical support and adequate supervision during grouting operations.

The accepted contractual approach favored a lump sum type contract which allowed for reasonable accuracy in budgeting the cost envelope for the grouting operations. Nevertheless, the Contractor was asked to provide in his bid unit prices for items susceptible to fluctuate due to particular field conditions.

This lump sum approach together with vigilant field supervision, resulted in a successful high quality work at minimum cost to the Owner.

**Grout Hole Pattern**

Considering the geometry of the dam and the need to create an impervious barrier through two different types of discontinuities by grouting, a line of grout holes spaced at 2 m intervals and 2.8 m from the upstream face, downstream of the post-tensioned anchors, was judged to be the optimum hole arrangement (see Figure 1). Grout holes were extended between 3 and 5 m into the rock foundation (see Figures 2 and 3). As
an additional check, three holes were extended 17 m into the foundation to investigate unexplored areas of the post-tension anchor zone.

The grouting of two types of voids dictated that split spacing technique was necessary. The larger water bearing voids were grouted with type 30 cement through primary holes, spaced at 8 m interval and secondary holes midway between the primary holes. The fine voids were grouted with micro-fine cement through tertiary holes located between primary and secondary holes for a final grouted hole spacing of 2 m. To speed up operations, drilling of primary holes was started 2 months prior to spring thawing.

Grouting Method

The G.I.N. method was retained because it provided the most adequate approach for rehabilitating the left abutment dam. The main features of this method include the following:

- a single stable grout mix for the entire grouting process with superplasticizers to increase penetration,
- a steady low-to-medium rate of grout pumping,
- control of grouting process through a limiting curve where the allowable grouting pressure decreases as volume of grout increases with pre-set upper limits on pressure and volume,
- high grouting pressures.

Use of a stable grout, which is defined as a grout exhibiting less than 5% bleeding in 2 hours, has the following advantages compared to thinner grout:

- less sedimentation during periods of low grout take,
- less shrinkage during setting,
- less risk of hydro-jacking,
- greater mechanical strength,
- greater resistance to leaching process,
- greater durability of the grouted section.

However, since the maximum travel distance of a grout slurry is directly proportional to the applied pressure and to the opening of the void, and inversely proportional to the cohesion of the grout slurry, the use of a stable grout mix implies that it is necessary, in order to improve grout penetration, to reduce grout cohesion by adding a superplasticizer, and to increase grouting pressures.
The G.I.N. method was used in ascending stages of 5 m through full depth drilled holes.

Mix Trial Program

Following award of contract, a trial mix program was conducted using the specified colloidal grout mixer. The specified mixer was a high speed colloidal grout mixer with impeller rotation speed of 1750 rpm and a capacity of 270 litres.

Cement from two separate suppliers of type 30 cement and one of micro-fine cement were tested, with and without superplasticizers and silica fumes. The cement and the proportion of different mixes along with results of the tests are given in Tables 1 and 2.

The cohesion was measured with a 10 cm x 10 cm steel plate of 1 mm thickness engraved with square pattern thin grooves, spaced 1 cm apart (Photo # 1). By weighing the plate before and after submerging it into the grout mix, the difference in weight divided by the area of the two sides gives the cohesion value.

The type 30 cement selected for grouting was supplied by Miron (No. 1, Table 2). Although two brands of cement tested produced satisfactory results, the Contractor chose Miron for economical and practical reasons that are specific to him.

The micro-fine cement was supplied by Conbextra. The selected mix (No. 2B, Table 1) produced the optimal combination of bleeding, initial set up time, cohesion and fluidity. As shown on Table 1, above a certain quantity of superplasticizer the initial set-up time is substantially increased.

The Conbextra micro-fine cement is a sulphate resisting blended cement with added superplasticizer, the Blaine fineness is 10,000 cm²/g with a D₁₀₀ inferior to 10 micron and D₅₀ = 3.5 micron.

The grout mixes were specified to develop maximum penetration with minimum loss of grout.

The Grout Limiting Curves

Basic Principles for the Design of the Curves

In order to control the grouting process by this method, it was necessary to select a G.I.N. value compatible with a dam design of monolithic concrete blocks, separated by vertical expansion joints and traversed by leaking horizontal lift joints. Both the vertical and horizontal joints are much more susceptible to uplift than open fissures
# TABLE 1 -- Results of tests on micro-fine cement product

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>Product</th>
<th>Cement (kg)</th>
<th>Water (l)</th>
<th>WRDA-19 l/100 kg</th>
<th>Silica Fume (kg)</th>
<th>Water/cement ratio</th>
<th>Liquid/solid ratio</th>
<th>Bleeding 3 hrs (%)</th>
<th>Initial Set-up (hrs)</th>
<th>Density (g/cm³)</th>
<th>Fluidity (sec)</th>
<th>Cohesion (N/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>Conbextra</td>
<td>81</td>
<td>48.6</td>
<td>0</td>
<td>0</td>
<td>0.6</td>
<td>0.60</td>
<td>1.4</td>
<td>7.5</td>
<td>1.77</td>
<td>47.7</td>
<td>1.9</td>
</tr>
<tr>
<td>1B</td>
<td>Conbextra</td>
<td>81</td>
<td>48.6</td>
<td>0.5</td>
<td>0</td>
<td>0.6</td>
<td>0.60</td>
<td>1.0</td>
<td>12.25</td>
<td>1.68</td>
<td>37.0</td>
<td>1.2</td>
</tr>
<tr>
<td>1C</td>
<td>Conbextra</td>
<td>81</td>
<td>48.6</td>
<td>1.0</td>
<td>0</td>
<td>0.6</td>
<td>0.61</td>
<td>1.9</td>
<td>11.5</td>
<td>1.68</td>
<td>35.5</td>
<td>1.1</td>
</tr>
<tr>
<td>2A</td>
<td>Conbextra</td>
<td>81</td>
<td>48.6</td>
<td>5</td>
<td>0</td>
<td>0.6</td>
<td>0.56</td>
<td>0.0</td>
<td>5.5</td>
<td>1.68</td>
<td>46.5</td>
<td>1.4</td>
</tr>
<tr>
<td>2B</td>
<td>Conbextra</td>
<td>81</td>
<td>48.6</td>
<td>0.5</td>
<td>5</td>
<td>0.6</td>
<td>0.57</td>
<td>0.0</td>
<td>5.75</td>
<td>1.65</td>
<td>34.2</td>
<td>1.4</td>
</tr>
<tr>
<td>2C</td>
<td>Conbextra</td>
<td>81</td>
<td>48.6</td>
<td>1.0</td>
<td>5</td>
<td>0.6</td>
<td>0.58</td>
<td>1.1</td>
<td>9.25</td>
<td>1.65</td>
<td>33.1</td>
<td>1.3</td>
</tr>
<tr>
<td>3A</td>
<td>Conbextra</td>
<td>81</td>
<td>40.5</td>
<td>0</td>
<td>0</td>
<td>0.5</td>
<td>0.50</td>
<td>1.0</td>
<td>6.75</td>
<td>1.73</td>
<td>46.7</td>
<td>2.8</td>
</tr>
<tr>
<td>3B</td>
<td>Conbextra</td>
<td>81</td>
<td>40.5</td>
<td>0.5</td>
<td>0</td>
<td>0.5</td>
<td>0.51</td>
<td>1.0</td>
<td>9.5</td>
<td>1.73</td>
<td>38.3</td>
<td>1.2</td>
</tr>
<tr>
<td>3C</td>
<td>Conbextra</td>
<td>81</td>
<td>40.5</td>
<td>1.0</td>
<td>0</td>
<td>0.5</td>
<td>0.51</td>
<td>1.5</td>
<td>Undetected</td>
<td>1.73</td>
<td>36.0</td>
<td>1.1</td>
</tr>
</tbody>
</table>

Note: Conbextra 10-92 Ultracem is a micro-fine cement product that already contains a superplasticizer, the quantity of superplasticizer shown on the table does not include the pre-added superplasticizer.

# TABLE 2 -- Results of tests on type 30 cement

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>Product</th>
<th>Cement (kg)</th>
<th>Water (l)</th>
<th>WRDA-19 l/100 kg</th>
<th>Water/cement ratio</th>
<th>Liquid/solid ratio</th>
<th>Bleeding 3 hrs (%)</th>
<th>Init. Set-up (hrs)</th>
<th>Density (g/cm³)</th>
<th>Fluidity (sec)</th>
<th>Cohesion (N/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Miron</td>
<td>200</td>
<td>100</td>
<td>1.0</td>
<td>0.5</td>
<td>0.51</td>
<td>2.0</td>
<td>5.75</td>
<td>1.83</td>
<td>34.2</td>
<td>0.680</td>
</tr>
<tr>
<td>2</td>
<td>St-Laurent</td>
<td>200</td>
<td>100</td>
<td>1.0</td>
<td>0.5</td>
<td>0.51</td>
<td>3.0</td>
<td>6.5</td>
<td>1.83</td>
<td>35.3</td>
<td>0.801</td>
</tr>
</tbody>
</table>
areas in the rock foundation. Consequently, specific G.I.N. values were computed for all of the 5 m stages to avoid uplift in the structure. Based on the distribution and type of open horizontal lift joints observed in the exploratory holes, a conservative approach was adopted and one lift joint per stage of 5 m was assumed. Figure 4 shows a typical section of the dam and the assumed position of the open lift joints, the grout holes and also the installed post-tension anchors.

For the calculation of G.I.N. values, the physical properties of selected grout mixes were taken from results of grout mix testing given in Tables 1 and 2. Other parameters such as average fracture aperture, which was taken as 1 mm, were assumed based on investigation results. The relations used to compute the pressure dissipation within a lift joint are those developed by G. Lombardi (Ref. 1 & 2).

The design of the G.I.N. curves shown on Figures 5 and 6 and the following formulas were primarily based on the weight of the concrete structure, the working load of pre-stressed anchors and the potential lift effects due to pressure.

\[ F_{\text{max}} = T_w + W \]

Where:

\[ T_w = \text{pre-stressed anchors working load} \]
\[ W = \text{weight of the concrete} \]

The maximum grouting pressure allowable at the point of injection is given by the following relation:

\[ P_{\text{max}} = \sqrt{\frac{F_{\text{max}} \times 3c^2}{\pi t^2}} \]  \hspace{1cm} (1)

Where:

\[ P = \text{pressure (N/m}^2\text{)} \]
\[ t = \frac{1}{2} \text{fissure aperture} \]
\[ c = \text{cohesion (N/m}^2\text{) of concrete} \]

The maximum volume of grout ( \( V_{\text{max}} \)) that can be injected under a given pressure is:

\[ V_{\text{max}} = \frac{2 \pi P_{\text{max}}^2 t^3}{c^2} \]  \hspace{1cm} (2)

The Grout Intensity Number (G.I.N.) is calculated as follows:

\[ P_{\text{max}} \text{ (Bars)} \times V_{\text{max}} \text{ (litre)} = \text{G.I.N.} \text{ } (1 \text{ Bar} = 100 \text{ kN/m}^2) \]  \hspace{1cm} (3)

Grouting pressure in Bars x Volume of grout injected in litres = G.I.N.
In fact the G.I.N. value is more or less proportional to the "injected energy". It may thus easily be understood that a possible damage to the structure is not related to the grout pressure alone, but to the product of the pressure times the surface on which this pressure may act. This surface is in turn related to the volume of grout not yet set. To avoid damage, not the pressure but the G.I.N. value (or the injected energy) needs be limited.

Example of Calculation

The properties of type 30 cement grout mix are given in Table 2, under mix No. 2. This mix has a water/cement ratio of 0.5 with 1% superplasticizer, a unit weight of 1.83 gr/cm$^3$ or 17.95 kN/m$^3$, and a cohesion $C = 0.000801$ kN/m$^3$ whereby $C_r = C/\gamma = 0.044$ mm.

To compute the G.I.N. curve for stages located between 10 and 15 m in the anchored section of the left abutment, an estimate of the maximum uplift forces was produced (Table 3). For the anchored section of the dam, Table 3 gives a maximum uplift force of 28,704 kN where 21,762 kN is required to lift the dam and 6,942 kN to counteract the pressure exerted by the pre-stressed anchors.

The pressure needed to generate the maximum uplift force, assuming a sub-horizontal crack with 1 mm aperture ($t = 0.5$ mm), is:

$$P_{\text{max}} = 41.3 \text{ kPa} = 0.413 \text{ Bars} \quad (4)$$

The maximum volume of grout that can be injected into a 1 mm fissure at the maximum grouting pressure will be:

$$V_{\text{max}} = 2.089 \text{ m}^3 = 2089 \text{ litres} \quad (5)$$

The Grout Intensity Number is therefore:

$$P_{\text{max}} \times \frac{V_{\text{max}}}{(0.41 \text{ Bars}) \times (2089 \text{ litres})} = \text{G.I.N. (856.5)} \quad (6)$$

A rounded number of 850 was adopted as the G.I.N. number for grouting stages between 10 and 15 m of the anchored section.

The same calculations were carried out with values of cohesion for micro-fine cement, the result was similar to those obtained with cement type 30, consequently the same curves were used for both types of cement.
TABLE 3 -- Left abutment dam
Maximum uplift force (anchored section)

<table>
<thead>
<tr>
<th>Depth of the Stage (m)</th>
<th>Weight of the Concrete 23.96 kN/m³ *</th>
<th>Pre-stressed Anchor Working Load 660 kN/m *</th>
<th>Uplift Force (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 5</td>
<td>2.588</td>
<td>3.960</td>
<td>6.548</td>
</tr>
<tr>
<td>5 - 10</td>
<td>7.025</td>
<td>4.500</td>
<td>11.526</td>
</tr>
<tr>
<td>15 - 20</td>
<td>50.486</td>
<td>9.384</td>
<td>59.871</td>
</tr>
<tr>
<td>20 - 25</td>
<td>98.120</td>
<td>11.826</td>
<td>109.946</td>
</tr>
<tr>
<td>25 - 30</td>
<td>169.580</td>
<td>14.268</td>
<td>183.849</td>
</tr>
<tr>
<td>30 - 35</td>
<td>269.790</td>
<td>16.710</td>
<td>286.500</td>
</tr>
<tr>
<td>35 - 40</td>
<td>403.668</td>
<td>19.152</td>
<td>422.820</td>
</tr>
<tr>
<td>40 - 45</td>
<td>576.135</td>
<td>21.594</td>
<td>597.730</td>
</tr>
</tbody>
</table>

* For each stage in a given grout hole, the weight of concrete and the number of anchors which are solicited in resisting the grout pressure, are calculated for a progressively increasing zone of influence in the longitudinal direction.

TABLE 4 -- Left abutment dam for stages between 10 and 15 metres
Grouting pressure versus volume of grout

<table>
<thead>
<tr>
<th>Volume of Grout Injected Vi (litres)</th>
<th>Effective Grouting Pressure Pe (Bars)</th>
<th>Pressure from the Grout Head Pc (Bars)</th>
<th>Pressure at the Gauge Pe-Pc (Bars)</th>
<th>G.I.N Pe x Vi</th>
</tr>
</thead>
<tbody>
<tr>
<td>230</td>
<td>3.7</td>
<td>2.664</td>
<td>1</td>
<td>851</td>
</tr>
<tr>
<td>106</td>
<td>8</td>
<td>2.664</td>
<td>5.3</td>
<td>848</td>
</tr>
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<td>65</td>
<td>13</td>
<td>2.664</td>
<td>10.3</td>
<td>845</td>
</tr>
<tr>
<td>47</td>
<td>18</td>
<td>2.664</td>
<td>15.3</td>
<td>846</td>
</tr>
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<td>37</td>
<td>23</td>
<td>2.664</td>
<td>20.3</td>
<td>851</td>
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<td>30</td>
<td>28</td>
<td>2.664</td>
<td>25.3</td>
<td>840</td>
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<td>22</td>
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<td>20</td>
<td>43</td>
<td>2.664</td>
<td>40.3</td>
<td>860</td>
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<tr>
<td>16</td>
<td>53</td>
<td>2.664</td>
<td>50</td>
<td>843</td>
</tr>
</tbody>
</table>
The G.I.N. curve or pressure control curve was constructed by placing the volume of grout as abscissa and the grouting pressure as ordinate. Figure 5 represents the G.I.N. curves constructed with values from Table 4. Two curves are shown on this figure: the first is calculated with the effective grouting pressure at the packer elevation and corresponds to the G.I.N. number; the second is calculated with pressure applied at the gauge and was added to facilitate control of the grouting process in the field.

Figure 6 shows the curve for stages from 25 to 50 m depth.

In order to control grout travel within an open fissure when it is not possible to build up grouting pressure, it is necessary to set an upper volume limit. This upper volume limit was selected according to depth of stage being grouted and ranges between 100 and 850 litres: the higher the stage, the smaller the volume necessary to fill the full transverse section of the dam.

Finally, when filling of an open fissure is achieved or where there is no open fissures, the pressure will rapidly build up. In order to avoid such dangerously high pressures and possible hydrofracturing, even with a small quantity of grout, an upper pressure limit needs to be set. This maximum limit on grouting pressure was fixed in order to use a minimum of 20 litres, with an upper limit of 50 bars, for stages below 20 m.

**Field Procedures**

To obtain maximum quality control at site, the following procedures were followed:

- calibrate the equipment (manometer, flowmeter),
- verify with in-situ testing, the grouting equipment proposed by the Contractor,
- verify the drilling equipment,
- verify the products used,
- verify the grout mix formulation,
- verify quality of the water,
- detailed logs of all drilling and absorption quantities for interpretation.
The Grouting Works

General

The grouting program started with the drilling of the first primary hole on May 5, 1993, actual grouting started only after June 21, so as to allow the temperature of the mass concrete to rise above 5 degrees Centigrade. Drilling of the last quaternary hole was completed on September 21, 1993. A total of 1,741 m of drilling spread over 60 holes including 9 uplift indicators and 6 quaternary holes were grouted with a total 50,145 kg of type 30 cement and 10,627 kg of micro-fine cement. The distribution of grout absorptions are summarized on Figures 1, 2 and 3.

Drilling Equipment

All of the 60 production grout holes were diamond drilled in BQ size with a double core barrel. Four test holes were drilled in NQ3 size with a triple core barrel. The specification called for a minimum 10% of grout holes to be cored in NQ3 size to assess the effectiveness of grouting. Although percussion drilling was authorised for this work, the Contractor chose to use diamond drilling equipment in order to assure the specified verticality of 2%. Average drilling rate was in the order of 5 m per hour and 3 hours for moving and setup. Verticality was measured with an inclinometer "rotodip"; all measurements were within the 2% limit.

Grouting Equipment

The Contractor used a tandem grout plant, with double mixers, double holding tanks and double pumps mounted on a single frame. The mixers were colloidal type with impeller speed of 1,750 R.P.M. mounted on 270 litre mixing tanks and connected to a 500 litre holding tank.

The colloidal type mixers were supplied according to specifications for the type of stable mix required. This particular type of mixer is the only one capable, because of its shearing action, of completely separating cement grains for better hydration thus increasing effectiveness of the superplasticizer and reducing cohesion and sedimentation.

The grout pumps were helicoidal screw types with maximum capacity of 50 bars.

The packers were pneumatic rubber inflated with compressed air.

For recording grout absorptions, the specifications called for a magnetic flowmeter equipped with automatic digital recorder. The instrument displayed simultaneously the total volume in litres and the flow rate in litres/minute with a precision of 1%. The
instrument, reset to zero at each stage, gave direct recording of total flow. The use of
an automatic flowmeter is essential when using the G.I.N. method since pressure must
be controlled in relation to volume of grout injected. Volume of grout must therefore
be known instantly to make the necessary pressure correction in time.

Grouting pressure was recorded at the collar of each hole with standard Bourdon type
pressure gauges.

Uplift Control Indicator

As a precaution against uplift and in order to verify assumptions made for producing
the control curves (number of fissure per stage and their aperture), nine uplift gauges
were installed along the crest of the dam, a minimum of two between each vertical
construction joint. Uplift gauges consisted of a steel bar anchored in the rock and
protected by a P.V.C. tubing, (see detail 1 of Figure 4). The void between the hole and
P.V.C. tubing was filled with cement grout to control intercommunications during
grouting operations.

Only four signs of uplift were observed during the entire grouting operations, and in all
cases uplift was inferior to 0.15 mm and of no consequence.

Monitoring during Grouting

The G.I.N. method and the use of a magnetic flowmeter has simplified field control of
the grouting process. With this method, only one grout mix is used and the control of
grouting is done by adjusting the pressure according to rate of absorption.

The evolution of the grouting process for four typical cases encountered during grouting
of the left abutment dam is described below and illustrated on Figure 6.

Case 1

Where grout absorption was relatively high and pressure could not be built up beyond
8 bars.

Grouting was finally stopped when the volume limit was reached. After the grout had
sufficient time to set, the same stage was re-grouted, but this time grout take was
minimal. Refer to line 1, this occurred in stage 36 to 41 m in primary hole P4.
Case 2

When grout absorption diminishes to nil with time and consequently pumping is stopped in order to respect the curve. Grouting was done with micro-fine cement.

Grouting proceeded normally until the pressure reached about 17 bars, from there it was gradually decreased to be maintained under the G.I.N. value. Grouting was terminated when the recorded flow was nil for a period of 2 minutes. Refer to line 2, this case was the one most frequently encountered during the grouting operations. This happened in tertiary hole P3 in stage 35 to 40 m.

Case 3

When grout absorption is so low that pumping is stopped when the upper pressure limit of the curve is reached.

Although the pressure limit was not reached in this case because the capacity of the pump could not exceed 32 bars, the trend of the line indicates that by increasing pumping pressure, the ceiling of the pressure curve will eventually be met. Refer to line 3, this happened in stage 35 to 44 m of secondary hole S1.

Case 4

A situation that normally does not happen when the G.I.N. curve is not exceeded.

The situation is drawn from grouting of stage 35 to 49 m in secondary hole S3, the grouting was pursued beyond the G.I.N. value and uplift in the order of 0.05 mm was observed when the volume of grout reached 1,447 litres at a pressure of 5 bars. The curve was deliberately exceeded in order to verify the validity of the assumptions made for the design of the curves. It can be seen that there is a reasonable factor of security against uplift built in the curve. Uplift gauges were monitored constantly during the experiment and pressure was decreased accordingly at the first sign of uplift, the movement was elastic with the gauges returning to their original position as soon as pressure was decreased. This case illustrates that uplift is not necessarily associated with high pressure but rather with volume of grout injected. When a sufficient volume of grout has been injected, uplift is possible even at lower pressures. At similar stage depth, in case 3, the pressure was increased to 32 bars without risk of uplift. Refer to line 4 of Figure 6.
Figures 7a, 7b and 7c give cement absorptions in kg per meter for each hole. As shown on the figures, the highest grout absorptions are concentrated in the southern part of the structure between holes P₁ and T₂ where major water bearing fissures were intersected between elevations 69 m and 75 m (see Figure 2).

During the grouting of the water bearing zone, grout percolated into the downstream drainage collector pipe buried in the fill material. Because of the thickness of the fill, it was not possible to locate and plug this major leak which is believed to be at the junction of the powerhouse and the dam. Grouting of this zone required a sanded grout mix with intermittent grouting sequences. Most of the other grout takes (92.4%) were below 500 kg of cement per stage.

Table 5 gives the distribution of grout absorption per sequence of grouting and per magnitude range. It can be seen that the majority (58.7%) of the grout takes are inferior to 25 kg of cement/stage. This confirms the good quality of the concrete and that the water bearing fractures are few and local.

### Table 5 -- Distribution of grout takes per grouting sequence

<table>
<thead>
<tr>
<th>Grout Take (kg/stage)</th>
<th>Percentage of Total Number of Grouting Stages for each Sequence</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Primary</td>
</tr>
<tr>
<td>0-10</td>
<td>55.4</td>
</tr>
<tr>
<td>10-25</td>
<td>8.4</td>
</tr>
<tr>
<td>25-50</td>
<td>8.4</td>
</tr>
<tr>
<td>50-100</td>
<td>13.3</td>
</tr>
<tr>
<td>100-500</td>
<td>7.2</td>
</tr>
<tr>
<td>500-1000</td>
<td>3.6</td>
</tr>
<tr>
<td>&gt;1000</td>
<td>3.6</td>
</tr>
</tbody>
</table>

Table 6 summarises the average grout take per meter for each sequence of grouting. The total average grout take per meter should normally decrease from primary to secondary to tertiary sequences. However, this was not the case for grout takes above 500 kg of cement/stage as shown on Table 6, instead there was an increase from primary to secondary sequences and only a slight decrease from secondary to tertiary sequences.

The situation is not much different with grout takes below 500 kg of cement/stage, not counting the high losses of grout through the downstream drainage system. The overall trend is the same and the average grout take increases from 7.9 kg/m in the primary sequence to 9.9 kg/m in the secondary sequence and to 11.9 kg/m in the tertiary.
increase from the primary to the secondary indicates that movement of grout is generally in an upstream-downstream direction and very limited in the lateral direction. This was to be expected considering that leaching along lift joints is in an upstream-downstream direction; furthermore the pre-injection of the post-tension anchor holes created a barrier between grout holes.

The tertiary holes were grouted mainly with micro-fine cement; the increase in the average grout take over the secondary sequence is mainly due to the higher penetration of micro-fine cement grout. This was noticeable during grouting of primary and secondary holes resulting in low grout takes while higher absorptions of micro-fine cement was taking place in the tertiary with moderate pressures (see Figure 3). In several of these cases, the extent of grout travel could be traced to the downstream face of the dam where small flows of water were observed moving away from the hole as grouting proceeded. Most of these seepage occurrences eventually stopped before any loss of grout. It was decided at site not to plug the downstream outlets of these leaking fissures. This enabled visualization of actual grout movements through the structure. It was observed that as the grout moved laterally in the filling process, it was also pushing water towards the exterior.

Figures 8a, 8b and 8c show the average grout take per hole for stages taking less than 500 kg of cement. The more regular distribution of grout takes in the tertiary holes illustrates the greater penetration of the micro-fine cement grout.

<table>
<thead>
<tr>
<th>Grout Take</th>
<th>Primary</th>
<th>Secondary</th>
<th>Tertiary</th>
<th>Quaternary</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>%</td>
<td>kg/m</td>
<td>%</td>
<td>kg/m</td>
<td>%</td>
</tr>
<tr>
<td>Total</td>
<td>100</td>
<td>31.3</td>
<td>100</td>
<td>4.4</td>
<td>100</td>
</tr>
<tr>
<td>&gt; 500 kg/stage</td>
<td>7.2</td>
<td>367</td>
<td>8.7</td>
<td>474.2</td>
<td>8.4</td>
</tr>
<tr>
<td>&lt; 500 kg/stage</td>
<td>92.8</td>
<td>7.9</td>
<td>91.3</td>
<td>9.9</td>
<td>91.6</td>
</tr>
<tr>
<td></td>
<td>93.4</td>
<td>11</td>
<td>92.4</td>
<td>10</td>
<td></td>
</tr>
</tbody>
</table>

**State of the Structure Before and After**

Prior to rehabilitation works, the downstream face of the dam was coated by a thick irregular and rusted colored layer of calcium carbonate, topped by a number of overhanging loose slabs of old resurfacing concrete, giving the structure a ragged and deteriorated surface appearance. During the winter of 1992 and following the removal of the loose concrete slabs, impressive slabs of ice accumulated above the seeping areas along the downstream face of the dam (Photo # 2). Photograph # 3 taken during the winters before and after completing the grouting program show that ice built-up had ceased.
NOTES:
1/ A - GROUT LEAKAGE INTO THE DOWNSTREAM DRUM SYSTEM.
2/ B - GROUT LEAKAGE ON THE DOWNSTREAM FACE.
3/ C - GROUT LEAKAGE ON THE DOWNSTREAM FACE FROM CONSTRUCTION JOINT.
4/ D - THE POST TENSION ANCHORS HAVE BEEN OMITTED FOR CLARITY.
References

1) DON U. DEERE and G. LOMBARDI
Grout slurries - Thick or thin?
Issue in Dam Grouting
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Division of the American Society of Civil Engineers in conjunction with the
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2) G. LOMBARDI, U. DEERE
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Photos

PHOTO # 1 - Plate for Cohesion Measuring
PHOTO # 2 - Ice Accumulation before Grouting Program. This occurred at 8 Places on the Dam.

PHOTO # 3 - State of the Dam during the Winter Following the Grouting Program. Water Seepage only occurred in Arch 2.