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DIAGNOSIS OF AAR IN CANNING DAM, CHARACTERISATION OF THE AFFECTED CONCRETE AND REHABILITATION OF THE STRUCTURE

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ABSTRACT

The Canning Dam concrete (gravity structure) has shown an upward movement of 18.3 mm and lateral upstream movement of 14.2 mm over the past 15 years of monitoring. These movements have been associated with considerable cracking of the upper parts of the dam and the upper gallery. Investigations have shown that the cause of the cracking was a strong alkali-aggregate reaction in the concrete, brought about by a deformed granitic rock containing strained quartz. Extensive horizontal and vertical cracking in the upper part of the dam wall has necessitated the removal of the section above the floor of the upper gallery level, and construction of a new reinforced concrete section to act as head beam for post-tensioning of the rest of the dam wall.

In addition to the small diameter cores taken from the various parts for diagnostic purposes, a vertical core of 100 mm diameter was taken through the whole thickness of the wall for the determination of the strength properties, alkali content and residual expansion potential. Based on these, a post-tensioning stress of 1.3 MPa has been calculated for restraining the residual expansion of the concrete. The paper also provides information on the performance of a high volume fly ash, triple blend concrete mixture developed for the replaced section of the dam wall.

Keywords: Alkali-Aggregate Reaction, concrete, dam, deformed granitic rock, expansion cracking, post-tensioning, rehabilitation.

INTRODUCTION

Canning Dam, constructed between 1933 to 1940, is a 66 m high concrete gravity dam with a curved crest. A 250 mm thick concrete layer placed on top of the crest forms the wearing course, which is continuous with the mass concrete. Monitoring over the past 15 years has shown that the dam wall has had an upward movement of 18.3 mm and a lateral upstream movement of 14.2 mm in this period. The movement has been associated with cracking in various parts of the dam. The top surface of the dam wall (i.e. the concrete road surface) shows extensive sub-parallel cracking with random cracking between them (Figure 1).

Fig. 1A

Fig. 1B

Fig. 1: Sub-parallel cracking and random cracking between the longitudinal cracks in the crest of the dam wall.

Some other sections of concrete show clear random cracking in localised zones. The concrete panels forming the parapets on the two sides of the road often show spalling at the joints. Cracking is also evident from the inside of the upper gallery.

The authors initially conducted a diagnostic investigation on seven 1 m long cores (55 mm diameter) from five locations in the upper part and two from the lower gallery into the base of the dam. The sketch of the dam wall at the end of this paper shows these locations. A 100 mm diameter core was subsequently taken through the whole dam wall to characterise the concrete with respect to strength properties, alkali content and residual expansion.

In view of the results obtained in these studies and severe vertical and horizontal cracking in the top section of the dam wall, above the floor of the upper gallery, the repair option chosen by the Water Corporation and the consulting engineers has been to remove the badly cracked concrete of the crest (about 4 m), and to replace it with new reinforced concrete designed to be free of AAR and other durability problems. The design involves post-tensioning of the rest of the dam wall such that the new concrete acts as the head beam for the post-tensioning anchors. This paper summarises the results from the study of the cores and those obtained on the performance of the new concrete proposed for the rehabilitation of the dam wall.

Materials Used in the Dam Wall

Construction records show that two types of concrete were used for the dam wall, with the mix compositions given in Table 1.

TABLE 1: Mix Composition (kg/m^3) of the Two Concretes of the Dam Wall

Concrete	Cement	Sand	Aggregate	Water	W/C	Wet Density kg/m^3
Type 1	342	562	1354	181	0.53	2439
Type 2	261	618	1299	169	0.65	2347

Type 1 concrete was apparently used for the upstream section and the upper parts of the dam wall, whereas Type 2 concrete was used at the lower parts and the downstream sections. Some rock pieces in both concretes were of 10-15 kg mass.

EXAMINATION OF CONCRETE CORES

Diagnosis of AAR

The visual cracking symptoms of the concrete together with the vertical and lateral movements of the dam wall had provided strong indications that AAR was present in the concrete. Among the seven cores, cores 3 and 6 were very light in colour with a chalky appearance, as compared to the other cores which were grey and dense. The latter showed many reaction rims around the aggregate particles, and many wet looking patches which indicated AAR gel, as well as some pores filled with AAR products. Sometimes fine cracks were filled with such products. These signs were weak in cores 3 and 6. The more severe AAR had occurred in the Type 1 concrete with the higher cement content, although weak AAR was found in the Type 2 concrete with the cement content of 261 kg/m^3.

Petrographic examination of thin sections showed the aggregate phase to consist of a mixture of granite and gneissic rock types. In both rock types the quartz crystals show undulose extinction angles of around 20 degrees. They are anhedral with microcrystalline quartz at the grain boundaries. The fine aggregate in most cores was angular except in cores 3 and 6, and appeared to contain crusher fines of the coarse aggregate with the same mineralogy. In cores 3 and 6 the sand grains were rounded and more like natural sand rather than rock fragments.

The petrographic examination also showed considerable microcracking in the matrix, partially around some coarse aggregate particles and extending and branching into the mortar phase. Some of the microcracks were filled with AAR gel in the vicinity of the aggregate particles. Very limited microcracking was seen in the thin sections made from cores 3 and 6, where the AAR gel from the aggregate appeared to have diffused into the mortar phase. Detailed examination of many concrete specimens by the scanning electron microscope (SEM) equipped with energy-dispersive X-ray (EDX) analyser revealed the presence of several forms of amorphous to crystalline AAR products, typical of those in advanced cases of AAR in Australia and other countries. The petrographic and SEM examinations identified AAR as the major cause of deterioration of the upper parts of the dam wall.

Characterisation of the AAR Affected Concrete

Strength and expansion properties were determined on the 100 mm diameter vertical core taken from the whole height of the dam wall. It should be noted that differences in aggregate size and distribution in different parts of the core would cause variations in the strength and expansion measured on the different core segments. Physical and mechanical tests such as density, moisture content, elastic modulus, compressive and tensile strengths were conducted on a number of core segments taken from different depths of the core. Thirteen core segments of about 1 m length were selected at approximately 5 m intervals for determination of available alkali content and residual expansion. Each set of data is obviously obtained from a different set of core segments, due to the destructive nature of the tests.

TABLE 2: Water Absorption and Density of 23 Core Segments from Different Depths

Specimen No.	Depth of specimen (m)	Mass of SSD* specimen	Density of SSD* specimen (T/m3)	Water absorption (%)
C1	2.41 – 2.61	3962.0	2.425	5.6
C2	3.77 – 3.97	3909.3	2.432	4.4
C3	7.09 – 7.29	3958.6	2.444	4.5
C4	9.60 – 9.80	3895.7	2.409	4.2
C5	13.10 – 13.30	3818.9	2.424	6.1
C6	16.33 – 16.53	3970.9	2.447	4.7
C7	19.03 – 19.23	3971.0	2.437	4.3
C8	21.50 – 21.70	3676.0	2.393	5.6
C9	24.30 – 24.50	3876.8	2.370	6.3
C10	27.24 – 27.44	3878.8	2.390	6.3
C11	30.62 – 30.82	3640.9	2.384	6.4
C12	34.20 – 34.40	3673.0	2.400	5.4
C13	37.73 – 37.93	3868.2	2.372	6.8
C14	39.55 – 39.75	3689.7	2.314	8.2
C15	43.38 – 43.58	3810.8	2.328	8.3
C16	46.65 – 46.85	3819.3	2.359	6.2
C17	49.15 – 49.35	3789.0	2.330	6.4
C18	51.00 – 51.20	3861.9	2.388	5.9
C19	55.78 – 55.98	3778.6	2.357	6.8
C20	58.82 – 59.02	3752.9	2.323	7.0
C21	60.70 – 60.90	3698.9	2.290	8.0
C22	64.60 – 64.80	3842.1	2.356	7.3
C23	67.00 – 67.20	3762.4	2.342	6.6

* SSD = Saturated surface dry condition

Density and Water absorption – Table 2 gives the water absorption and density of one set of core segments. Generally, water absorption is lower and density is higher for the top 35 m (higher quality concrete). This relates to the types of concrete used in the upper and lower parts of the dam wall.

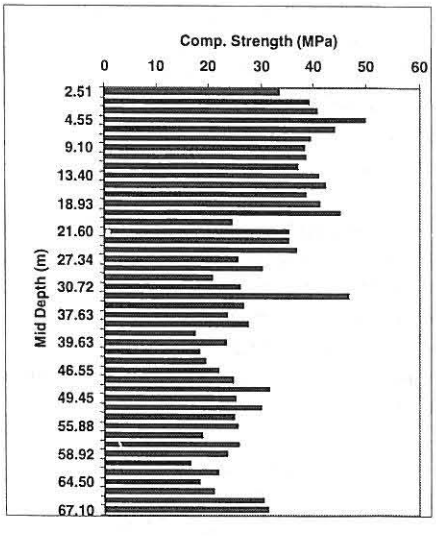

Fig. 2: Variation of compressive strength of concrete with depth.

Compressive strength – Figure 2 shows the variation of compressive strength with depth in the dam wall. The length to diameter ratio of the specimens was very close to 2.0 ranging from 1.84 to 1.98 with most values closer to the latter.

The strength is also generally higher for the top 35 m of the dam wall than the lower 32 m, although some variation of strength is seen within each section. This may relate to localised variations in factors such as cement content, water/cement ratio, porosity (water absorption), and aggregate size. Some of these differences are reflected in the values of water absorption. However, the extent of expansion and cracking in the different core segments, due to AAR, could also have influenced the compressive strength values. Several papers in the Proceedings of the 8th, 9th and 10th International AAR Conferences [for example, Ono (1989), Koyanagi et al. (1992), Takemura et al. (1996)] show that most strength properties of AAR-affected concrete deteriorate compared to those of sound concrete. It must be noted that the strength values determined on sound segments of the core are very optimistic and do not reflect the macrocracking and its effects on the integrity of the wall, particularly at the upper gallery level. For example, the cracking would significantly influence the response of the structure to earthquake by providing planes of weakness, regardless of the strength of the crack-free concrete. Nevertheless, deterioration of concrete arising from internal expansion and microcracking could be detected if the original strength of the concrete is known. For instance, the lower strength of 33.4 MPa at around 2.5 m depth, compared with an average of around 40 MPa for the depth 3.7-9.8 m, may have resulted from differences in AAR expansion and cracking.

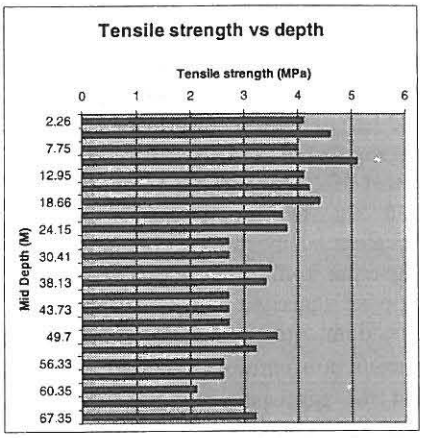

Fig. 3: Variation of splitting tensile strength of concrete with depth.

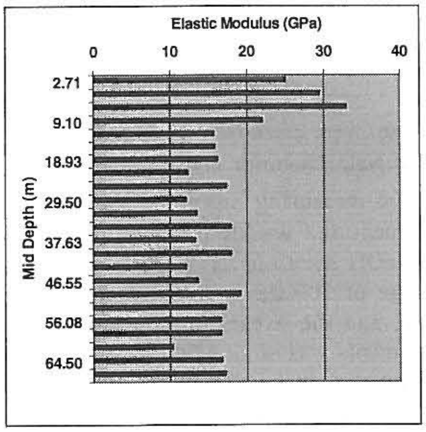

Fig. 4: Variation of Elastic modulus of concrete with depth.

Splitting Tensile Strength

Sixty-eight core segments of 100 mm length and 100 mm diameter were tested for splitting tensile strength. Each result shown in Figure 3 is the average of 3 values for the segments from the relevant mid depth. Clayton (1989) stated that this test does not reflect the true tensile strength of the concrete, but indicates the effects of AAR on the compressive strength of the AAR-affected concrete. However, Bremner, et al. (1995) recommended the indirect tensile test (gas pressure method) for the evaluation of AAR damage to concrete. Although not exactly the same concrete segments were tested the compressive and tensile strength show similar trends. Similar comments made for compressive strength also apply to the tensile strength values. The higher strength of segments from the top of the dam wall reflects the higher quality of the original concrete rather than the effect of AAR

Elastic Modulus

Figure 4 shows the values of elastic modulus determined on 23 locations. The values of elastic modulus in a few locations in the upper part of the dam are in the range of 25-33 GPa or crack-free portions of the core, which is the normal range for undeteriorated concrete of this type. However, in many locations, the modulus has dropped to values as low as 10, 12, 13, 15 and 16. AAR-affected concretes are sensitive to elastic modulus and can suffer losses of modulus as large as 80% (Swamy 1995). The average value of elastic modulus over the whole wall is about 17.5 GPa, although if the three values for the upper parts (depth of 7.6 m) are excluded, the average drops to 15.7 GPa. These values could be used in combination with the measured expansion strains to determine the required post-tensioning stress.

Soluble Alkali Content

Representative portions of each of the 13 core segments taken at 5 m intervals, quarry site were pulverised to pass 75 μm size for the determination of their water soluble alkali content (Table 3). This alkali is considered to be available for further reaction with the aggregate in the concrete.

TABLE 3: Soluble Alkali Content of the 13 Core Segments (kg/m^3)

Core No	Depth (m)	K_2O	K_2O	Na_2O equiv *
1	1.15 – 1.30	0.66	1.82	2.25
2	5.67 – 6.0	0.60	1.69	2.08
3	11.05 – 11.25	0.55	1.56	1.92
4	17.45 – 17.70	0.56	1.56	1.92
5	22.36 – 22.50	0.32	1.43	1.64
6	25.83 – 26.0	0.73	0.56	1.04
7	32.32 – 32.47	0.78	2.73	3.24
8	36.90 – 37.10	1.26	2.08	2.91
9	42.70 – 42.90	0.84	2.39	2.89
10	48.60 – 48.80	0.66	2.47	2.90
11	53.0 – 53.20	0.66	2.34	2.78
12	57.50 – 57.65	0.64	2.34	2.76
13	64.0 – 64.20	0.64	2.21	2.63
Aggregate	-	0.09	0.13	0.19

The contribution of the aggregate of 0.19 kg/m^3 Na_2O equiv. should be deducted from the values given in Table 3. The variation in the alkali content seen in Table 3 depends on the aggregate content and size of aggregate particles in the core segment used. It also depends on the extent of AAR, and the amount of alkali tied up in the reaction products. Accelerated mortar bar testing of fresh aggregate from the abandoned quarry that supplied aggregate for the construction of this dam, using NaOH solutions of various concentrations at 80°C, showed that the aggregate could tolerate concrete alkali contents of up to 2.8 kg Na_2O equiv/m^3. In this case perhaps values above 2 kg/m^3 could be considered sufficient to maintain the AAR and its consequent expansion of the concrete. Direct measurement of core expansion could best reveal the potential for concrete expansion, albeit under unrestrained conditions.

Expansion of Core Segments

Two adjacent sets of core segments (250 mm x 100 mm) were taken from 13 locations on the full core (whole dam wall). One set was stored in 1M NaOH solution at 40°C, and the other at 100% RH (40°C), after being fitted with demec measuring disks and the required pretreatments. The length change of the cores was measured weekly for two months and then monthly. As expected, expansion in the 1M NaOH solution was larger than that at 100% RH and has continued to increase to the age of 500 days, although the rate of expansion was much smaller after about 300 days, and the expansion appears to have levelled off. The average expansion of the 13 core segments was around 0.06% at 500 days, of which 0.02% was the initial rapid expansion due to water uptake. Therefore, the concrete still contains reactive components and can expand by a further 0.04% due to new reaction sites if sufficient alkali is available.

At the age of 200 days, expansion for cores stored at 100% RH (40°C) had largely levelled off and increased slightly up to 500 days, when the average expansion for the 13 core segments was 0.027% of which 0.01% was attributed to the rapid initial water absorption. Therefore 0.017% expansion is attributed to the residual potential for AAR expansion.

Bérubé et al. (1995) suggested that all the parameters determined could be combined to estimate the extent of future expansion of a structure. Using this approach the future expansion potential for the Canning Dam wall is classed as "medium". However, they did not relate the classification to any remedial action.

Assuming that the residual expansion potential of 0.017% (i.e. a strain of 0.00017 m/m) will be realised, and taking the average modulus of elasticity of about 16 GPa, a stress of 2.7 MPa is calculated to be needed to confine the residual expansion. However, because the rate and magnitude of expansion in the field is usually much less (around 50%) than the expansion of drilled cores kept at 100% RH 40°C, (Tomita et al. 1989) a confining stress of around 1.3 MPa is considered to be adequate for suppressing the residual expansion. However, in practice a post-tensioning stress of less than 1 MPa has been designed, which together with the structural confinement is expected to significantly reduce the cracking of the concrete.

In relating laboratory expansion and lack of confinement to the field situation, a few factors need to be considered. First, the laboratory specimens may undergo stress relief and may expand just after the coring so that the remaining long-term expansion may underestimate the expansion potential of the concrete in the field. Second, the structural confinement may mean that the full expansion potential of the concrete would not be realised under the restrained field condition. Third, environmental factors could significantly differ under the two conditions, particularly in cold climates, but in Australia this difference would not be as significant. Ideally, a finite element model that takes all these factors into account should be used for the estimation of stresses and strains. This modelling has proved to be very complex and difficult for AAR. The approach used in this paper is a simplification of the effects of all these factors on the behavior of AAR-affected concrete.

REMEDIAL ACTION

As mentioned earlier, the remedial actions involves replacement of the top 4 m of the dam wall. The level of reinforcement in the new concrete was designed to resist cracking due to heat of hydration, shrinkage, and stresses in the anchorage zone of the post-tensioning tendons. Because the aggregate chosen from the vicinity of the dam could possibly be reactive, precautions against AAR were considered necessary. The concrete was also required to protect the reinforcement steel against corrosion.

A concrete mixture design containing a high volume of fly ash (HVFA) was selected for trial. This mixture was modified to have 5% silica fume (SF) and was labelled a triple blend (TB). Tables 4 and 5 provide details of the chemical properties of the materials and the mixture proportions used. A plain Portland Cement Concrete (PPC) was also used for comparison.

TABLE 4: Some Chemical Properties of the Cementitious Components

Oxides	Cement	Fly ash	Silica fume
SiO_2	19.90	49.6	93.7
Al_2O_3	4.74	25.1	0.37
Fe_2O_3	2.96	10.9	0.20
CaO	64.70	1.9	0.16
MgO	1.52	1.0	0.29
Na_2O equiv.	0.57	0.82	0.35
LOI	2.92	4.60	4.89
SO_3	2.67	0.25	0.10

TABLE 5: The Three Concrete Mix Designs (kg/m^3)

Ingredients*	HVFA	TB	PPC
Portland cement	199	192	370
Fly ash	160	150	-
Silica fume	-	15	-
20 mm Aggregate	481	485	840
10 mm Aggregate	770	770	420
Sand	673	670	600
Water	144	151	155
Water/binder ratio	0.40	0.42	0.42
Na2O equiv.	2.45	2.38	2.11

* naphthalene based superplasticiser used for HVFA and TB mixes at the rate of 800 ml per 100 kg binder.

PERFORMANCE OF CONCRETE MIXTURES

Overall 12 concrete mixtures were assessed, of which 6 are listed in Table 6, together with some of their properties. Note that properties developed in the low alkali (L) mixtures are relevant to field condition. However, high alkali (H) mixes are needed for assessment of alkali-reactivity of the aggregate.

TABLE 6: Properties of Selected Concrete Mixes

Mix designation	Alkali content kg/m^3	Strength (MPa) 180 days	180 day † void content (% volume)	180 day · carbonation depth (mm)			180 day # pH 1:1 slurry
				MC	DC	CC	
PPC	2.11 (L)	53	13.4 ± 1.4	0.5			12.62
	5.5 (H)	44					12.74
HVFA	2.45 (L)	49	15.1 ± 1.8	1.1			12.51
	5.5 (H)	39	14.6 ± 0.6	1.8	6.4	5.1	12.71
TB	2.38 (L)	50	15.6 ± 1.9	1.0			12.44
	5.5 (H)	41	16.6 ± 0.3	1.9	7.2	5.3	12.65

* Superplasticiser added to HVFA and TB mixes at the rate of 800 ml/100 kg binder, to maintain the slump.
† According to ASTMC 642
· MC = moist curing; DC = dry curing; CC = curing compound. 7 days moist curing applied for DC, CC.
\# pH of a 1:1 slurry of pulverised concrete in distilled water.

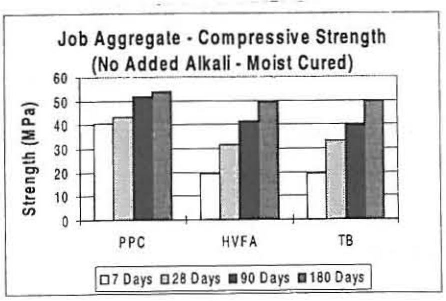

Fig. 5: Compressive strength development for the various concrete mixtures

Figure 5 shows the strength development trends for the concretes. Mixtures with higher alkali were weaker as expected (Shayan and Ivanusec, 1989).

The void contents of the blended cement mixtures was a little higher than that of the PPC mixture, and did not decrease between 90 and 180 days. This was thought to be due to lower workability and compaction for these mixtures and possibly due to a diminished amount of $Ca(OH)_2$, thereby diminished pozzolanic reaction to refine the pore structure. The carbonation depth of the blended cement concretes was also larger than that of the PPC concrete, possibly for the same reason. However, the predicted carbonation depth at 100 years was found to be acceptable. The alkalinity of the concrete was also considered acceptable for the protection of the reinforcement. The shrinkage values were acceptable (~ 450 microstrains) and below the value of 750 microstrains specified in the Australian Standard AS 3600.

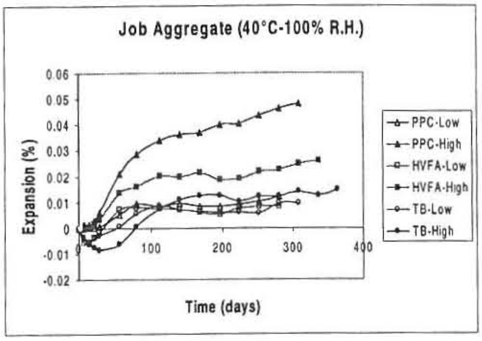

Fig. 6: Expansion curves for the various concrete mixes at low and high alkali contents

Figure 6 shows expansion curves for the concrete prisms made with the three concrete types. "Low" and "High" on Figure 6 refer to the level of alkali (Table 6). Figure 6 shows that the aggregate is potentially reactive in the presence of plain Portland cement alone, and that blended cements, particularly (TB), have successfully suppressed the AAR expansion. On this basis, the TB concrete mixture is recommended to minimise the risk of AAR in the new concrete.

CONCLUSIONS

The concrete from Canning Dam wall has been diagnosed to have suffered severe damage due to AAR expansion and cracking. Consideration of extensive horizontal and vertical cracking, and probable loss of monolithic action, has necessitated the removal of the top section of the dam wall above the floor level of the upper gallery. The integrity of the structure needs to be enhanced by post-tensioning techniques. Expansion measurements and determination of elastic modulus have been used to calculate 1.13 MPa confining stress for the post-tensioning of the concrete. The design post-tensing is < 1 MPa and together with the structural confinement is expected to significantly reduce the cracking in the wall.

A triple blend reinforced concrete (50 MPa) containing 40% fly ash and 5% silica fume has been recommended to replace the damaged parts of the dam wall, and to act as head beam for the post-tensioning anchorage zones.

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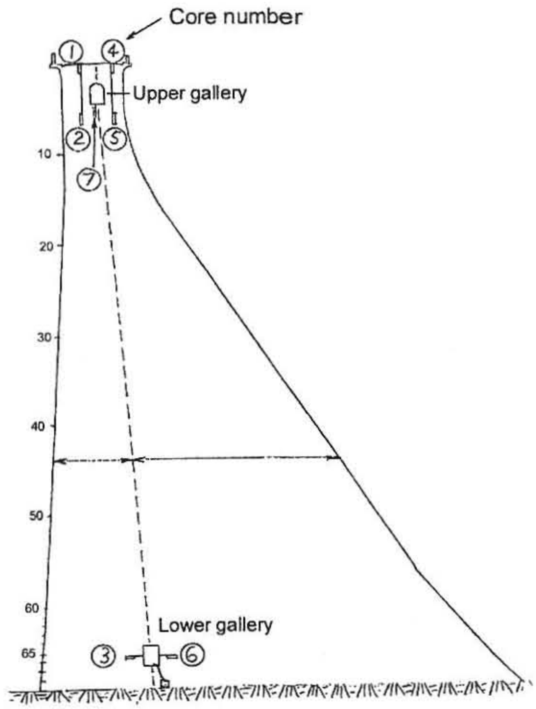

Sketch of dam wall showing positions of the seven cores taken for diagnostic purposes.