Peto MacCallum Ltd.
CONSULTING ENGINEERS

CROOKS HOLLOW DAM
INTEGRITY ASSESSMENT
CROOKS HOLLOW ROAD
TOWN OF FLAMBOROUGH, ONTARIO
FOR
HAMILTON REGION
CONSERVATION AUTHORITY

Distribution:

25 cc: Hamilton Region Conservation Authority
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February 4, 1993 Our Ref: 92HF137

Mr. B. Scott Konkle, OALA
Hamilton Conservation Authority
838 Mineral Springs Road
P.O. Box 7099
Ancaster, Ontario
L9G 3L3

Dear Mr. Konkle

Crooks Hollow Dam Integrity Assessment
Crooks Hollow Road
Town of Flamborough, Ontario

We are pleased to present our report on the integrity assessment of the Crooks Hollow Dam.

We have found that the concrete in the dam is in poor condition, suffering from severe concrete delamination and cracking, a result of the disruptive forces of freeze-thaw attack and wetting and drying over the past 80 years of its life. The exposed buttresses are in worse condition than the main spillway areas of the dam.

Our analysis of the overall stability of the gravity structure reveals that it is stable under present operating conditions. In addition, it is capable of withstanding the additional load of a short term flood that results in a water level rise up to 1.5 m above the spillway level.

It is our conclusion therefore that while the concrete of the dam is in poor condition the dam is still functional and stable at its current operational loads.

If the dam is to be operated at its maximum potential, using stop logs above spillways 1, 3 and 4, then major reconstruction will be required.

On the other hand the dam may be operated at its current reduced levels, i.e. just below the threshold of spillways 1, 3 and 4 for several years if a relatively minor restoration and planned preventive maintenance program is instituted.

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In this regard, we do not believe it is necessary to repair the existing cracks immediately. It is considered sufficient to institute an ongoing inspection programme with provision for maintenance on an as required basis to maintain the shotcrete surface on the dam and minimize the potential for further degradation of the concrete in the structure.

Further comments concerning measures that could be implemented to upgrade the structure and/or reduce future maintenance requirements are provided in Section 5 of the report. The projected cost of these measures is outlined in Appendix C.

Since this work is classed as a maintenance activity, we believe it will not be subject to a Class Environmental Assessment.

It should be noted that while our analysis indicates the overall stability of the concrete structure has an acceptable factor of safety against failure in the event of a major storm, some damage to the structure may occur. The hydraulic implications of this, downstream of the structure, should be investigated.

We trust we have conducted this investigation and reported thereon within our terms of reference. We look forward to meeting with you to discuss the findings and conclusion of this investigation.

Sincerely

Peto MacCallum Ltd.

Gerry Pacitti, P.Eng.
Managing Director, Hamilton

GP/DWK:rz

25 cc: Hamilton Region Conservation Authority

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1.0 Introduction

Peto MacCallum Ltd. was retained by the Hamilton Region Conservation Authority to conduct a structural integrity analysis of the Crooks Hollow Dam at Christie Conservation Area in the Town of Flamborough, Ontario.

The Crooks Hollow Dam is a 36.6 m long, 6.1 m high concrete gravity dam with four 3.7 to 4.3 m wide overflow spillways. It was originally constructed in 1913 to supply water to the Town of Dundas. It presently provides water for irrigation purposes at a local golf course.

The results of previous investigations conducted since 1968 have raised concerns regarding the stability and safety of the dam. Minor repairs have been carried out in the past 10 years.

The purpose of the current study is to investigate the integrity of the concrete within the dam, to evaluate the stability of the existing structure, and if appropriate outline preliminary engineering solutions to the problems identified.

Several pertinent reports were provided by the Conservation Authority for our review:

1. Crooks Hollow Dam, Report on the Investigation of the Safety of the Dam; William L. Sears and Associates Limited, 1968.

   This report presents the results of an analysis carried out to assess the stability of the dam. Site specific testing to define the properties of the concrete was not conducted during this analysis.

2. Crooks Hollow Dam, Manual of Operating Procedures, 1972.

   This document provides a brief description of the geometry of the dam and operating depth. It is noteworthy that the current operating water level is coincident with the top of the concrete spillways. The water level is lowered in the winter to the concrete level in spillway 2.

3. Dam repairs to the Crooks Hollow Dam on the Spencer Creek; William L. Sears and Associates Limited, 1977.

   The document contains the tender for reconstruction of the north abutment and wingwalls of the dam.

4. Concrete repairs to Crooks Hollow Dam; Singleton & Associates Inc., letter-report dated 1988.

   Specifications for refacing of the dam with shotcrete are contained in this document.

In addition, correspondence from Willam L. Sears and Associates issued 1976, 1977, 1978 prior to and following reconstruction of the north abutment of the dam as well as a 1988 letter from Singleton and Associates Inc., while repairs were being carried out, were provided.

2.0 Procedure

The complete above water portion of the dam was visually examined on November 23, 1992 for evidence of deterioration such as cracks, areas of spalling, scaling, existing repairs, wet spots, leaching etc. The water level was about 4.0 m below the top of dam at the time of the inspection. Portions of the dam below water level were not examined. Photographs were taken to record the results of the visual survey and assist in reporting. Selected photographs are provided in Appendix A.

A total of seven 260 to 1003 mm long, 100 mm diameter, cores were drilled at strategic locations in the dam using an electrically operated diamond tipped core drill. In addition a 7.62 m long 50 mm diameter core was retrieved from the central portion of the dam by coring a hole vertically through the north buttress into the underlying bedrock using a Winkie drill. The locations where the cores were retrieved are shown on the attached Drawings 1 and 2.

A piezometer, sealed just above the concrete/bedrock interface, was installed in the hole drilled through the north buttress to permit monitoring of the groundwater level below the dam.

All coreholes were filled with concrete. A site survey was also carried out to document the dimensions of the dam and the water depth.

The corehole samples were returned to our laboratory for detailed visual and microscopic examination, logging and testing to determine the condition of the concrete at depth and the quality of the concrete at the rock/concrete interface. The following tests were conducted on the concrete:

- Compressive strength tests
- Density tests
- Microscopic examination

Microscopic examinations were conducted on selected specimens prepared from core No.'s 2 and 4 after polishing to a high finish.

3.0 Observations

3.1 Visual Survey

The reconstruction work carried out on the north end of the dam, downstream wall, and the valve chamber in the summer of 1977 was evident during examination of the dam.

It is apparent that the work conducted in 1988 involved application of a coating of shotcrete to the surface of the dam, including the buttresses, abutments and spillways above the water level. While the shotcrete surfaces are still intact, it is apparent that deterioration, mainly in the form of cracking, has occurred following these repairs. Typical examples of this deterioration can be seen in Photograph No.'s 1 to 10.

These photographs show that the cracking is primarily horizontal and vertical with some secondary diagonal cracks. The cracks commonly are lined with a deposit of white calcium carbonate ($CaCO_3$) (Photograph No.'s 2, 6, 7 and 8). These deposits indicate fluid movement along the cracks is leaching calcium (Ca^{+2}) from the cement paste and redepositing it at the crack surface as $CaCO_3$. We believe these cracks reflect the condition of the concrete beneath the shotcrete skin. The nature of the cracking suggests the deterioration is a result of freeze-thaw and wetting-drying.

As shown in Photograph No.'s 2, 7, and 9 the most extensive cracking is present in spillways 1, 3 and 4, the south face of the north abutment wall, and the upstream face of the north abutment and wingwall. These areas also show the most intense leaching/re-precipitation along the cracks. Water seepage through these cracks was noted along a portion of a crack in the south face of the north abutment at the same elevation as the top of the downstream wingwall (Photograph No. 9). Near the top of the spillway 4 water seepage was also noted along a portion of a horizontal crack (Photograph No. 6). Although other sections of the dam also show cracking and leaching/re-precipitation, the intensity of these features is considerably less.

3.2 Core Drilling

Drawings 1 and 2 show the location where the core was retrieved. Core No.'s 1 through 7 are 100 mm in diameter and range in length from 0.26 m to 1.0 m. Core No. 8, located in the north buttress was drilled vertically 7.62 m into the bedrock. The core from this hole was less than 50 mm in diameter.

Photographic logs along with a brief description of the cores are presented in Appendix B.

With the exception of Core No.'s 3 and 4, all of the cores intersected the outer layer of shotcrete. This layer ranges in thickness from 33 mm to 105 mm and typically is well bonded to the underlying concrete. In core No.'s 2 and 6 which were drilled at the toe of spillways 1 and 4, respectively, the shotcrete appears to have debonded from the underlying concrete. In core No. 2 a thin layer of $CaCO_3$ covers the surface of the delamination. It should be noted however that the shotcrete at the bottom of the spillways is still intact.

Good quality concrete was intersected below the surficial layer of concrete in spillways 1 and 4. Occasional horizontal cracking was intersected in both cores with $CaCO_3$ deposits present along all of the crack surfaces. The lower 0.1 m of core No. 2 intersected a very soft section of concrete in which the cement paste and fine sieve sized fractions of the sand have been removed. This material is weakly cemented.

From 0.75 to 0.82 m in core No. 6 a layer of reddish brown mud was intersected. This layer is intimately mixed with the concrete suggesting the mud had contaminated the concrete during its initial placement.

Core No. 6, which was advanced to a depth of approximately 1 m intersected bedrock at 0.9 m. The bedrock-concrete contact was weakly bonded and broke during extraction of the core. The contact surface shows minor $CaCO_3$ deposits. Bedrock was not contacted in core No. 2 at a depth of 0.9 m.

Core No.'s 1 and 7 were drilled horizontally into the north and south buttresses, respectively. The concrete underlying the shotcrete layer in both buttresses is severely distressed such that the drilling operation reduced the cores to rubble. The poor condition of the concrete restricted drilling in both buttresses to less than 0.5 m.

Core No.'s 3 and 4 were taken from the upstream faces of the north and south abutments below shotcrete repair patches, just above the present water level. Core No. 3 reveals good quality sound concrete in the north abutment. Core No. 4 fractured upon extraction from the south abutment, the cracking occurring along the paste-aggregate contact of two cobble-sized particles in the core. The fracture surfaces show extensive leaching and $CaCO_3$ re-precipitation.

Core No. 5 was drilled into the downstream face of the south abutment. The concrete underlying the shotcrete repairs is intensely cracked such that over half of the extracted core is rubble.

No evidence of reinforcing steel was detected in the core samples. A wire screen was noted locally in the shotcrete.

3.3 Laboratory Testing

3.3.1 Physical Testing

A limited number of physical tests were conducted on the core specimens retrieved. Results of the compressive strength and density tests are presented below and indicate that the concrete is of good intrinsic quality in areas unaffected by intense cracking.

Core No.	Compressive Strength (MPa)	Density (Kg/m^3)
2	32.5	2 457
3	23.6	2 425
6	29.0	2 458

3.3.2. Microscopic Examination

Visual and microscopic examination of the concrete in the cores show that the concrete is typical of the type of concrete produced at the time the dam was constructed in 1913. The concrete consists of a gap-graded coarse aggregate in which large (> 200 mm) granitic and dolostone gravel particles are intermixed with smaller sized (< 50 mm) crushed, grey shaley dolostone, brown crystalline dolostone and grey fossiliferous limestone. The carbonate rock types appear to have been locally derived, possibly during construction of the dam. The large gravel particles are probably local river bed material. The coarse aggregate appears to be of adequate quality for this type of construction although the shaley dolostone and brown crystalline limestone appear to be prone to cracking.

The fine aggregate consists of a poorly graded natural sand composed primarily of carbonate and silicate rock types. This aggregate also appears to be of suitable quality for this type of construction. Deleterious proportions of shale, chert, etc. are not present in the sand fraction.

Irregular shaped water voids are very common throughout the concrete, especially in the heavily fractured concrete. Typically the voids occur along the underside and side of coarse aggregate particles. In addition, small (<1 mm) diameter spherical voids are present. Occasionally these voids are partly to completely filled with a calcium sulfoaluminate (ettringite) and/or calcite ($CaCO_3$). These voids comprise approximately 1 % of the concrete.

In addition to the large scale cracking noted in the cores, micro-cracking is also present. This micro-cracking forms a network of fine, irregular cracks which primarily cut the cement paste. These cracks also cut coarse aggregate particles, in particular the shaley dolostones and the brown crystalline limestones. This micro-cracking is present even in apparently good concrete such as in core No.'s 2 and 6. The cracking occurs both perpendicular and parallel to the major horizontal cracks intersecting the cores. These micro-cracks show only minor carbonation of the cement paste with scattered deposits of $CaCO_3$ along their length. The concrete is non air entrained.

4.0 Discussions

4.1 External Stability Analysis

A global analysis was carried out to evaluate the stability of the dam under current operating procedures (water level at top of concrete spillways). A discussion of the various parameters employed for the analysis is provided below:

Parameter	Discussion
Dam Profile	The dam profile shown on Drawing 1 and the cross-section shown on Drawing 3 prepared from site measurements, was used for stability computations.
Abutments	The effect of the abutments was not considered in the analysis. The portion of the dam between the outside edges of spillways 1 and 4 was considered. This assumption is considered to be conservative.

Key	Coring in the spillways and through the buttress indicated that the dam is keyed into the rock some 0.8 to 1.0 m. It is assumed a 0.8 m thick concrete base exists below the entire dam.
Concrete Weight	The unit weight of the concrete measured in the laboratory ranged between 2 425 to 2 458 kg/m^3; a unit weight of 2 450 kg/m^3 was employed in the analysis.
Friction Factor	Considering the relatively flat bedding planes of the bedrock and potential/ observed mud inclusions in the dam concrete, a conservative rock/concrete friction coefficient of 0.6 was employed.
Water Level	In accordance with current operating procedures, the maximum water level behind the dam was assumed to be at the height of spillways 1, 3 and 4.
Sediment Pressure	The channel bottom at the upstream face of the wall was measured to be 5.6 m below the top of the buttress.
Uplift Pressure	Analyses were performed assuming the uplift pressure along the base of the dam decreased at a constant rate from hydrostatic at the heel to zero at the toe. Based on the water level measured in the piezometer, it appears that the actual uplift pressures will be slightly below this level.
Ice Pressure	An ice pressure of 73 kN/m acting at the winter operating water level, spillway 2, was assumed.
Seismic Forces	Seismic forces were not considered since local seismic forces in this area are not significant for structures of this type.

Considering a 0.8 m deep key along the full length of the dam and lateral pressure of the vertical rock face of only 200 kPa the computed safety factor against sliding is 2.0. The actual support provided by the rock key is difficult to quantify.

The computed factor of safety against overturning under present operating conditions is about 1.9, which is considered to be satisfactory for the existing dam. The safety factor decreases to 1.3 when the weight of the buttresses is omitted.

Stability with regards to bearing capacity is not expected to be a problem as the dam is founded on relatively sound dolomite bedrock.

Based on our analysis and the observed performance of the dam, we believe the dam is stable under current operating conditions. It is probable that our assessment is more favourable than the 1968 William L. Sears and Associates evaluation due to the incorporation of a 0.8 m thick base/key under the dam which provides sliding resistance on the downstream key face and increases the weight of the dam. In addition the measured water levels may be less than assumed previously.

We have also analyzed the effect on dam stability of increased water levels over short durations (maximum 48 hours) during brief periods of increased headpond elevations. During such periods, spillways 1, 3 and 4 will become operational. For a water level of 0.75 and 1.5 m above the spillways, a computed factor of safety of 2.0 against sliding would require a lateral capacity of 300 and 400 kPa respectively on the vertical face of the rock key. We believe these values are available.

The computed factor of safety against overturning under similar circumstances is 2.1 and 1.6 respectively ignoring ice pressures. A combination of high flows and substantial ice pressures reduces this safety factor to an unacceptably low value of 1.2 when the water level is 0.75 m above the spillway and less than 1.0 at 1.5 m. We believe it is unlikely for extreme flood conditions and maximum ice pressures to be developed at the same time under current operating conditions.

4.2 North Abutment Settlements

Ongoing minor settlement of the ground surface adjacent to the northeast abutment immediately in front of the concrete step have reportedly continued for the past 10 years. After severe rainstorms, a small depression forms which is immediately filled with granular material by maintenance personnel.

The location of the settlement is shown on photograph No. 4.

We note that settlement of the fill behind the wall was first reported shortly following reconstruction work in 1977. Tender documents for this work called for excavations to be backfilled with native materials compacted to 90% standard Proctor density.

Two possible causes of the settlement consistent with the observed occurrences after major rainfalls are postulated:

i) Erosion of materials from within the abutment by rainwater infiltration and seepage.

ii) Poor compaction of the abutment backfill material.

Erosion of materials below the problem area may occur due to washing out of material through cracks in/under the abutment walls. However, no eroded material is evident at the base of the abutment (Photograph No.'s 7 and 9) and the exit location is not apparent. It is noteworthy that the maintenance chamber was flooded; sediment may be accumulating on the base of this room.

Seepage forces and saturation of poorly compacted granular backfill could result in significant settlements. The specified degree of compaction is considered low and compaction of fill in the confined area behind/under the north-abutment wall would have been difficult. However, settlements of this nature would be expected to occur within the first few years rather than an ongoing basis.

The impact of the loss of ground on dam stability is difficult to assess without determination of the cause. Considering the upstream concrete wall and the minor nature of the problem, we do not believe the problem is significant at this time. However, it is recommended that further exploratory work including boreholes within the backfill and examination within the maintenance chamber (following removal of the contained water) be carried out to define the cause.

4.3 Dam Condition

The concrete in the dam has experienced significant deterioration from the effects of freeze-thaw attack and the volume changes caused by wetting and drying over the past eighty years of its life. It is possible that some of the cracking that was intersected during the coring may have been old construction joints, or cracks caused by thermal shrinkage as the concrete cured initially.

Many of the cracks showed re-precipative deposits of $CaCO_3$, evidence that the cracks are old.

The buttresses appear to be in the worst condition. Being pillar type structures approximately 1 m thick, these elements have been subjected to severe freeze and thaw action and wet/dry cycles. This has resulted in severe delamination of the concrete.

This condition is most obvious in the log of core No's 1, 7 and 8. Core No. 8 shows that the upper 4 to 5 meters of the buttress is badly delaminated. It should be noted that the north side of this

We believe major reconstruction of the dam will be necessary if the normal operating water level is raised. The concrete in the upper part of the spillways and buttresses is in very poor condition; hence rehabilitation of the existing structure does not appear to be feasible and is not recommended.

It must be noted that some maintenance of the structure will likely be required on an ongoing basis. The shotcrete applied to the surface of the concrete has been effective to date in minimizing further deterioration of the dam. However significant cracking is evident that could result in spalling and renewed deterioration of the original concrete.

The structure should be inspected annually, preferably in the late fall after the water level is lowered and before the onset of freezing temperatures. We believe the repairs can be limited to restoration of any areas where spalling of the shotcrete surface has occurred. Limited epoxy injection of major reflection cracks may also be required on an occasional basis.

It should be noted that visual observation on site as well as in the core samples reveals that leakage of water through that dam is occurring during the summer months. The degree of ongoing remedial work required to maintain the structure could be reduced significantly by implementing measures to prevent seepage of water through the dam.

We believe the application of a durable waterproofing membrane on the upstream face of the dam will be most cost effective. Epoxy injection of the cracks could also be considered. However, due to the large number of cracks throughout the dam, this procedure will be expensive and may not be successful unless provision is made to conduct a secondary injection program to fill any cracks that may have not been sealed during the initial application.

It must be noted that while we believe the dam is capable of providing continued service at the present operating level, it is possible that a major storm may result in significant damage to the structure. The hydraulic implications downstream of the structure should be investigated.

We trust the information presented in this report is sufficient for your present purposes. If you have any questions, please do not hesitate to contact our office.

Yours very truly,

LICENSED PROFESSIONAL ENGINEER
M. R. ANDERSON
PROVINCE OF ONTARIO

Murray R. Anderson, P.Eng.
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
D. W. KERR
PROVINCE OF ONTARIO

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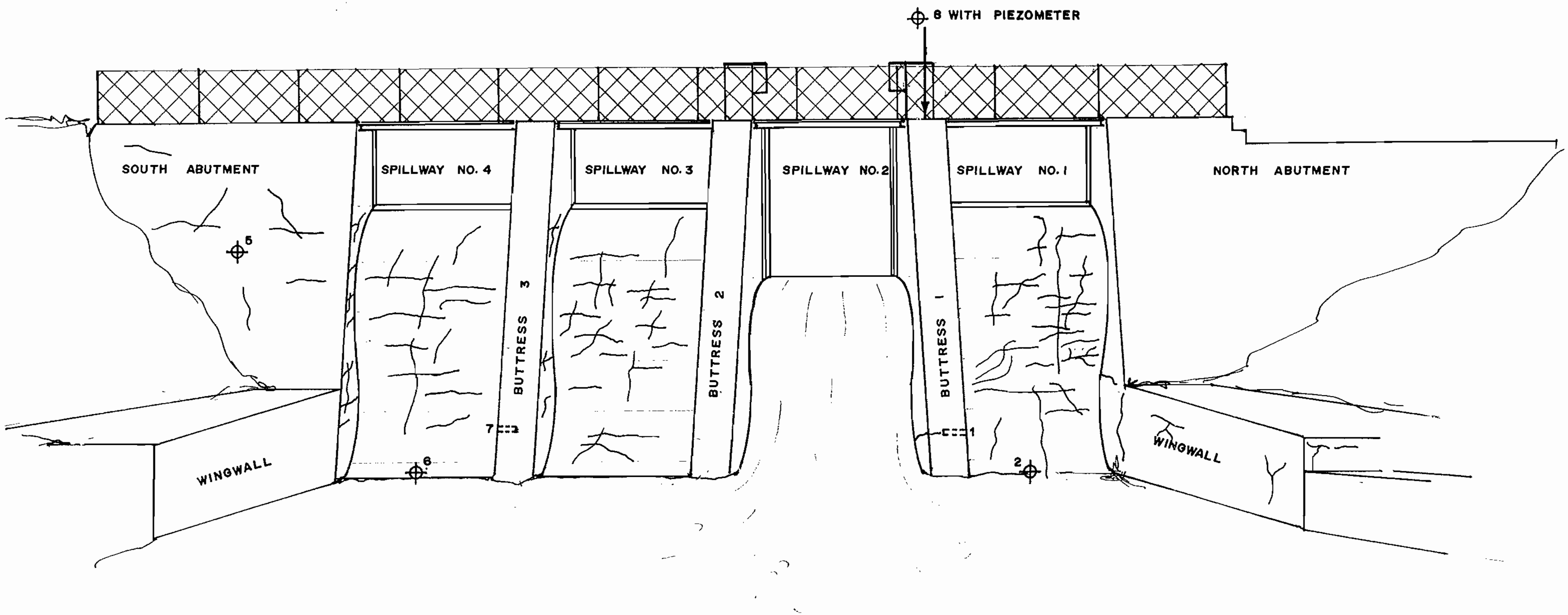


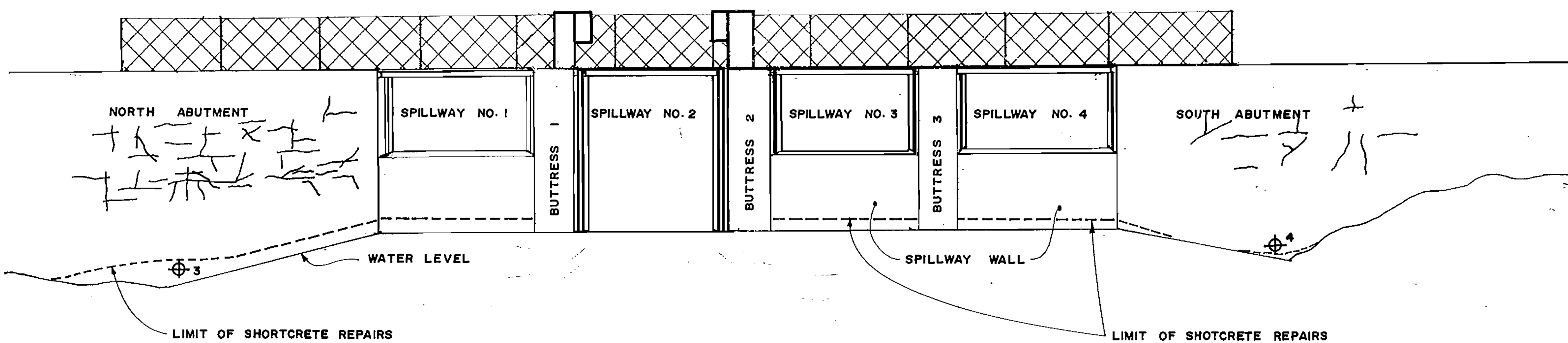
LEGEND

CRACKS

CORЕHOLE LOCATION

0.3m COREHOLE LENGTH

HAMILTON REGION CONSERVATION AUTHORITY					
CROOK HOLLOW DAM INTEGRITY ASSESSMENT FLAMBOROUGH, ONTARIO	PetoMacCallum Ltd. CONSULTING ENGINEERS				
PROFILE - DOWNSTREAM FACE	DRAWN CIB / CHECKED JB / APPROVED JB	DATE	SCALE	JOB NO.	DRAWING NO.
		DEC. 1992	NTS	92HF137	1



LEGEND

CRACKS

2 COREHOLE LOCATION

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CROOK HOLLOW DAM INTEGRITY ASSESSMENT FLAMBOROUGH, ONTARIO					
PROFILE - UPSTREAM FACE	DRAWN CIB	DATE	SCALE	JOB NO.	DRAWING NO.
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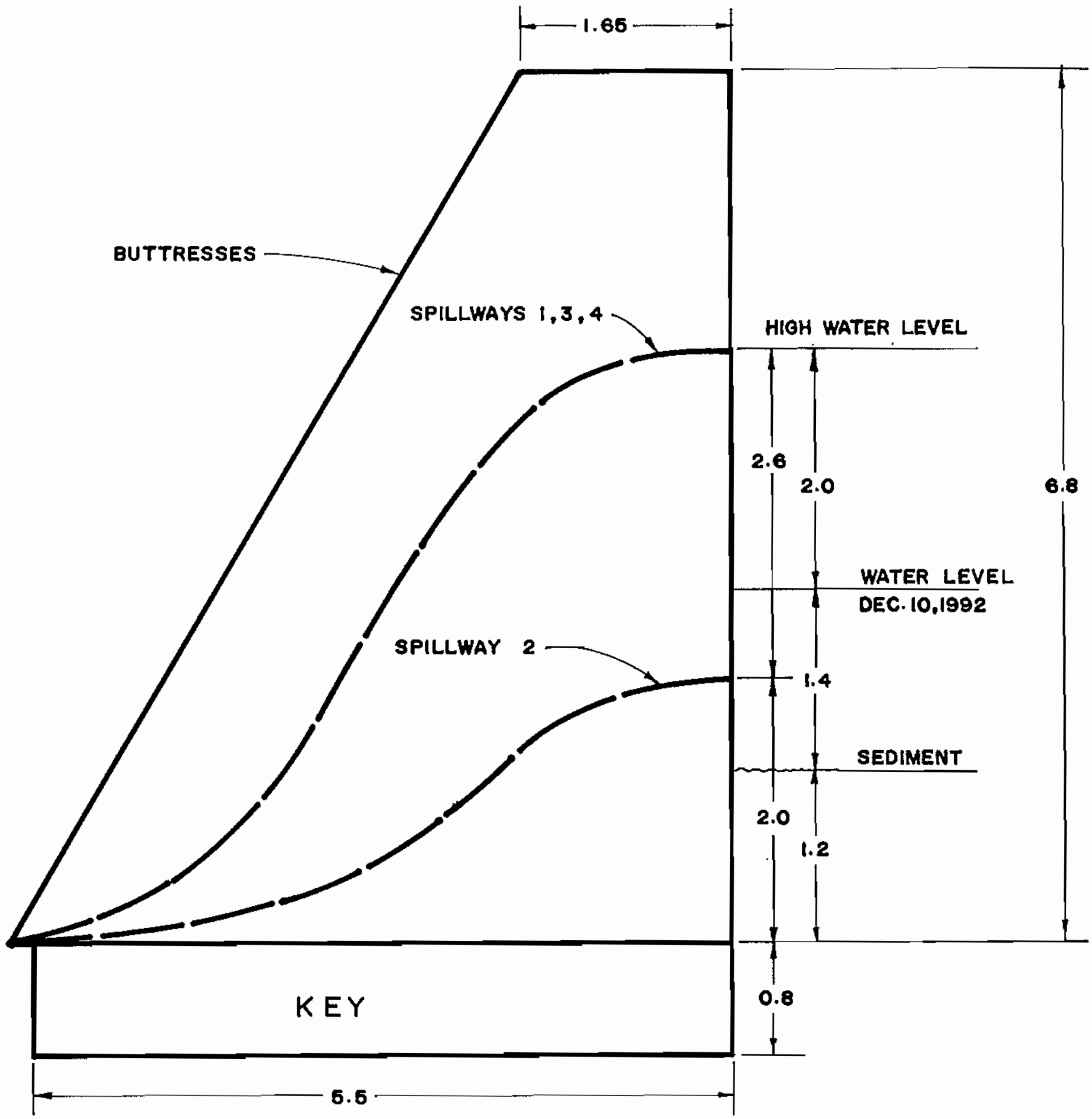

HAMILTON REGION CONSERVATION AUTHORITY

CROOKS HOLLOW DAM INTEGRITY ASSESSMENT
CROOKS HOLLOW ROAD
FLAMBOROUGH, ONTARIO

DAM CROSS-SECTION

PetoMacCallum Ltd.
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DATE	SCALE	JOB NO.	DRAWING NO.
DEC. 1992	1:50	92HF137	3

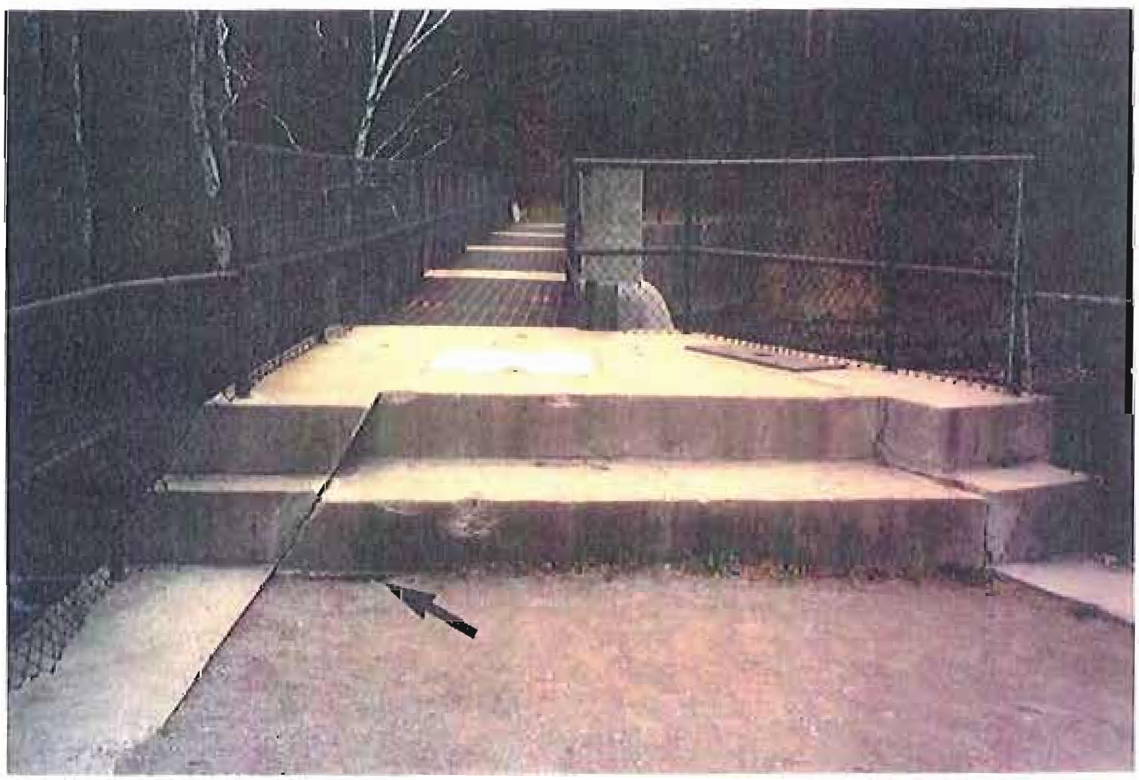
APPENDIX A

Photographs 1 through 10

Photograph No. 1 - Upstream face of North Abutment and wing wall show extensive horizontal and vertical cracking with white $CaCO_3$ deposits along the length. Core No. 3 (arrow) was extracted below the level of shotcrete repairs (dashed line). Note the epoxy injection into buttress 1 as seen through spillway No. 2.

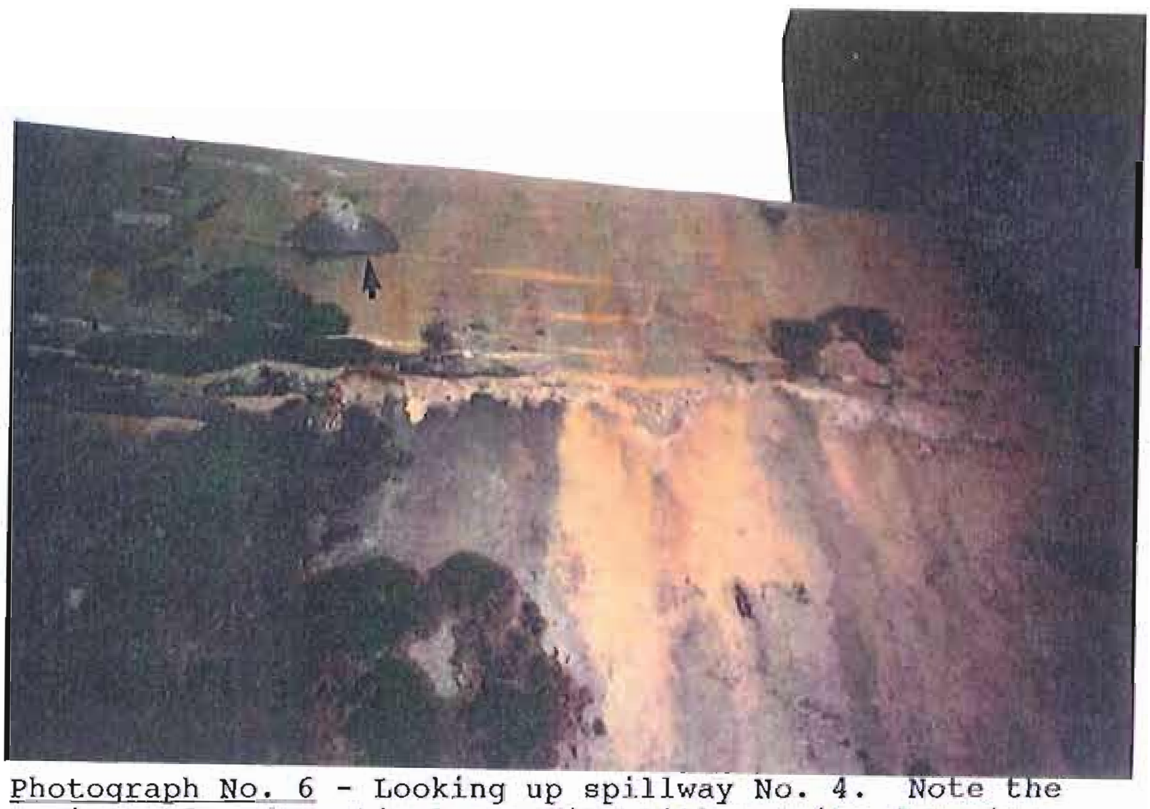
Photograph No. 2 - Upstream face of North Abutment.

Photograph No.3 - Upstream face of South Abutment at right and spillway No. 4 at left. Note that the intensity of cracking and leaching is less on this face than on the North Abutment. The arrow indicates the location of core No. 4.



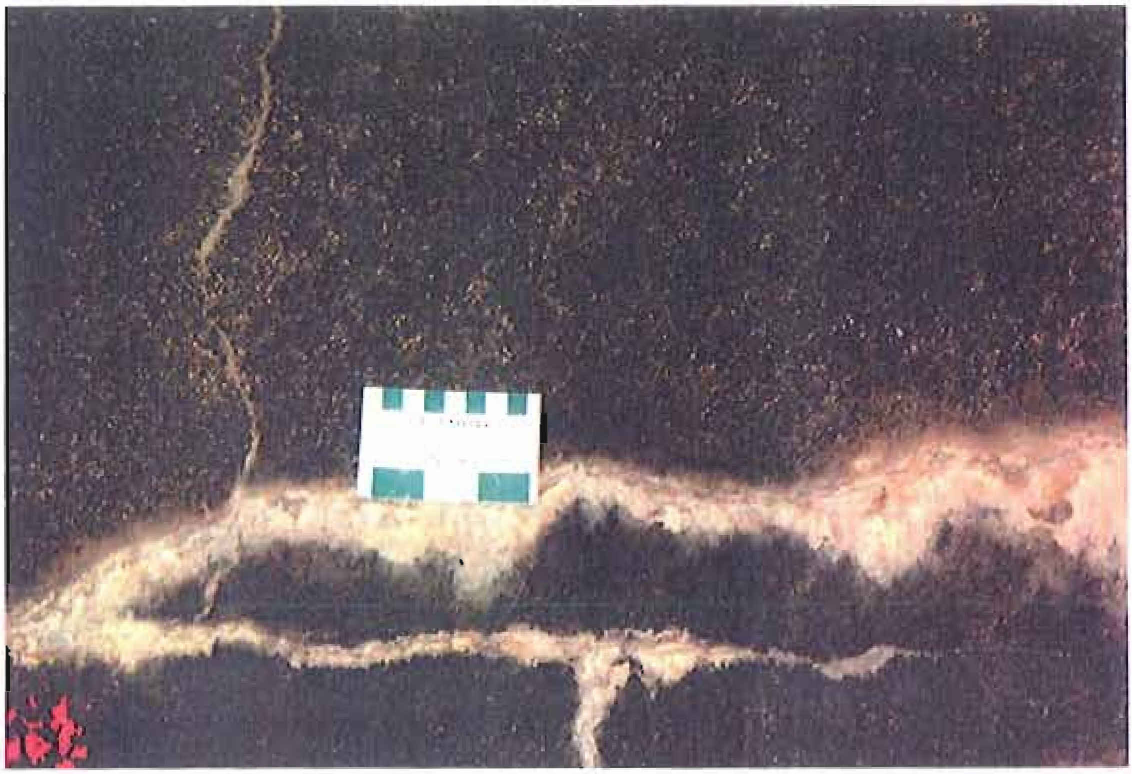
Photograph No.4 - Looking south from the north shore across the walkway over the dam. Note the slight undercutting of the steps at the lower left (arrow).

Photograph No. 5 - Downstream face. South shore. Note the cracking and leaching of the south abutment as well as spillway 4 and 3 (fore ground).



Photograph No. 6 - Looking up spillway No. 4. Note the horizontal and vertical cracking with calcite deposits. Also, the arrows indicate cracks with water seepage.

Photograph No. 7 - Downstream face of spillway No. 1. The arrows show the locations of core No. 1 in the buttress and core No. 2 in the toe of the spillway. Core No. 8 was drilled vertically into the top of the buttress. Note the intensity of cracking on the spillway face as the result of freeze-thaw and wetting-drying effects.



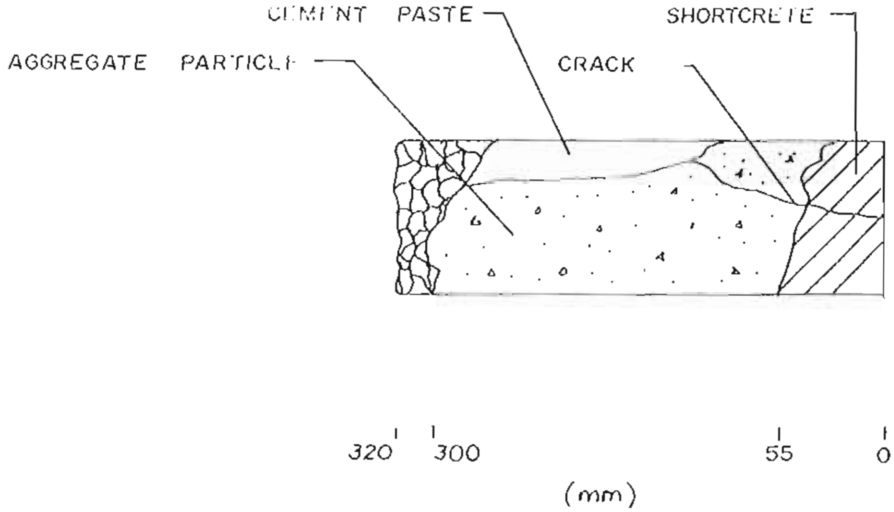
Photograph No. 8 - A close-up of freeze-thaw cracking of the shotcrete repairs with a crust of white calcite. The crust is the result of water migrating along the cracks and leaching Ca^{2+} from the cement paste. As the fluids are exposed to the surface Ca^{2+} is deposited as $CaCO_3$.

Photograph No. 9 - South face of north abutment and wingwall on th downstream side. Note the water seepage (arrows) through cracks in the abutment. Extensive water seepage also occurs over the face of the wingwall (at right).

Photograph No. 10 - Downstream side of wingwalls on north shore.

APPENDIX B

Log of Core Sheet
Coreholes 1 through 8



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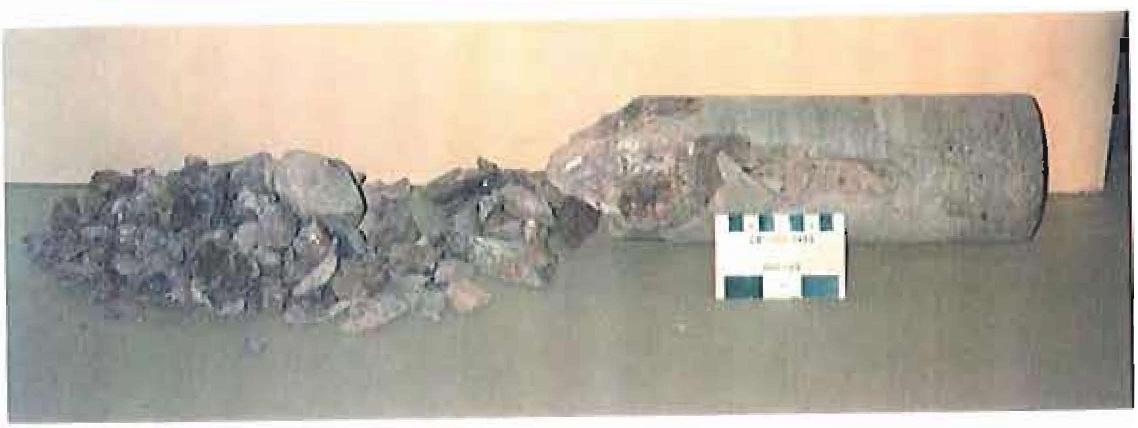

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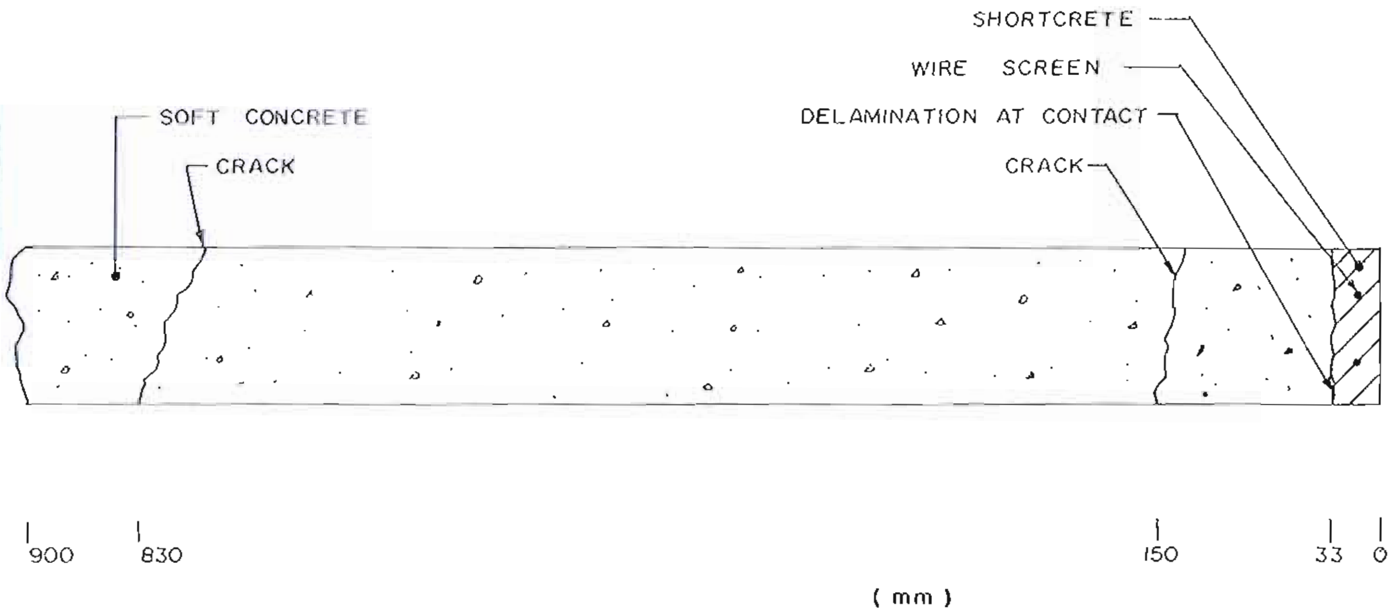

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CORE NO. 3

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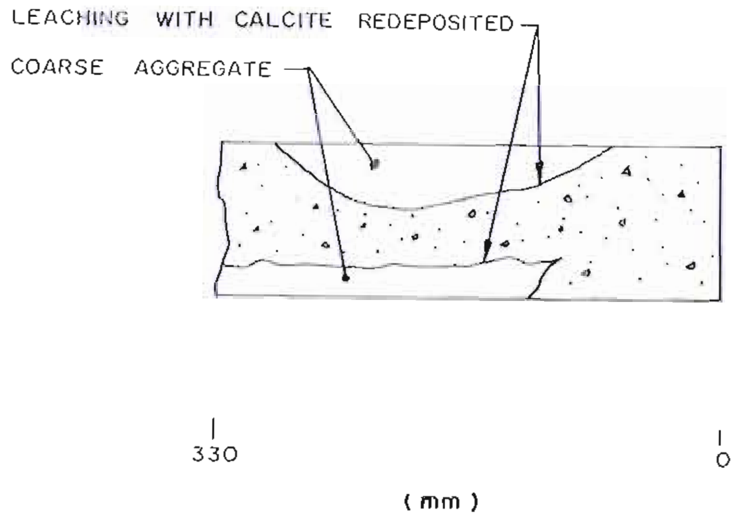

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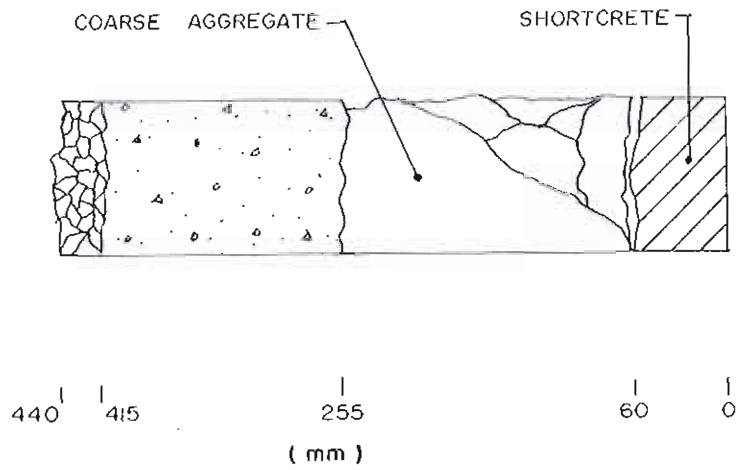

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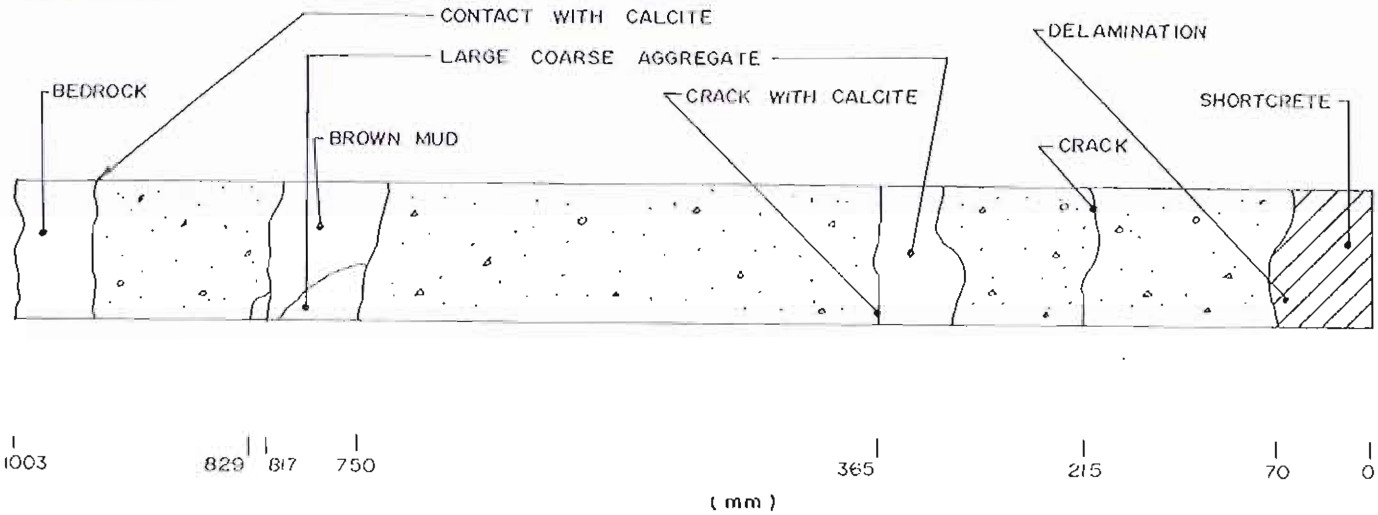

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CONSULTING ENGINEERS

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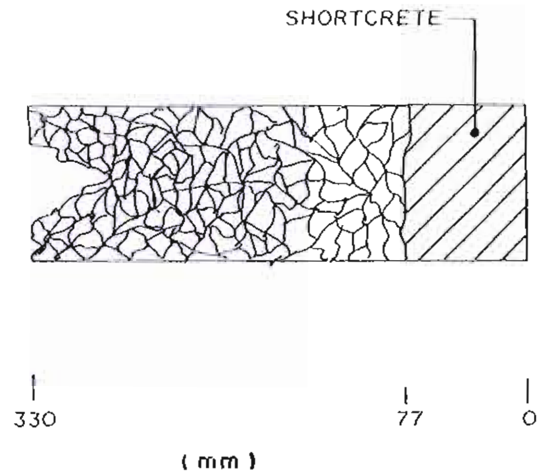
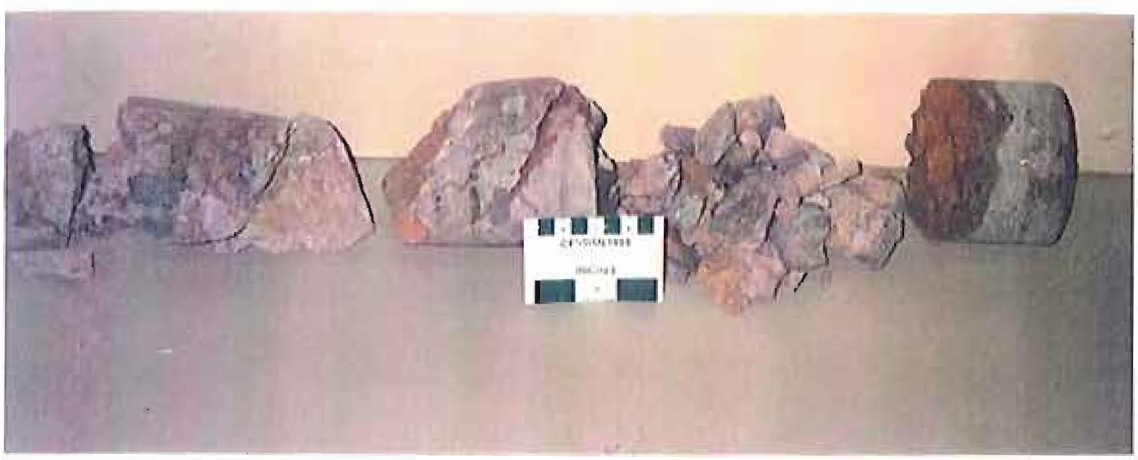
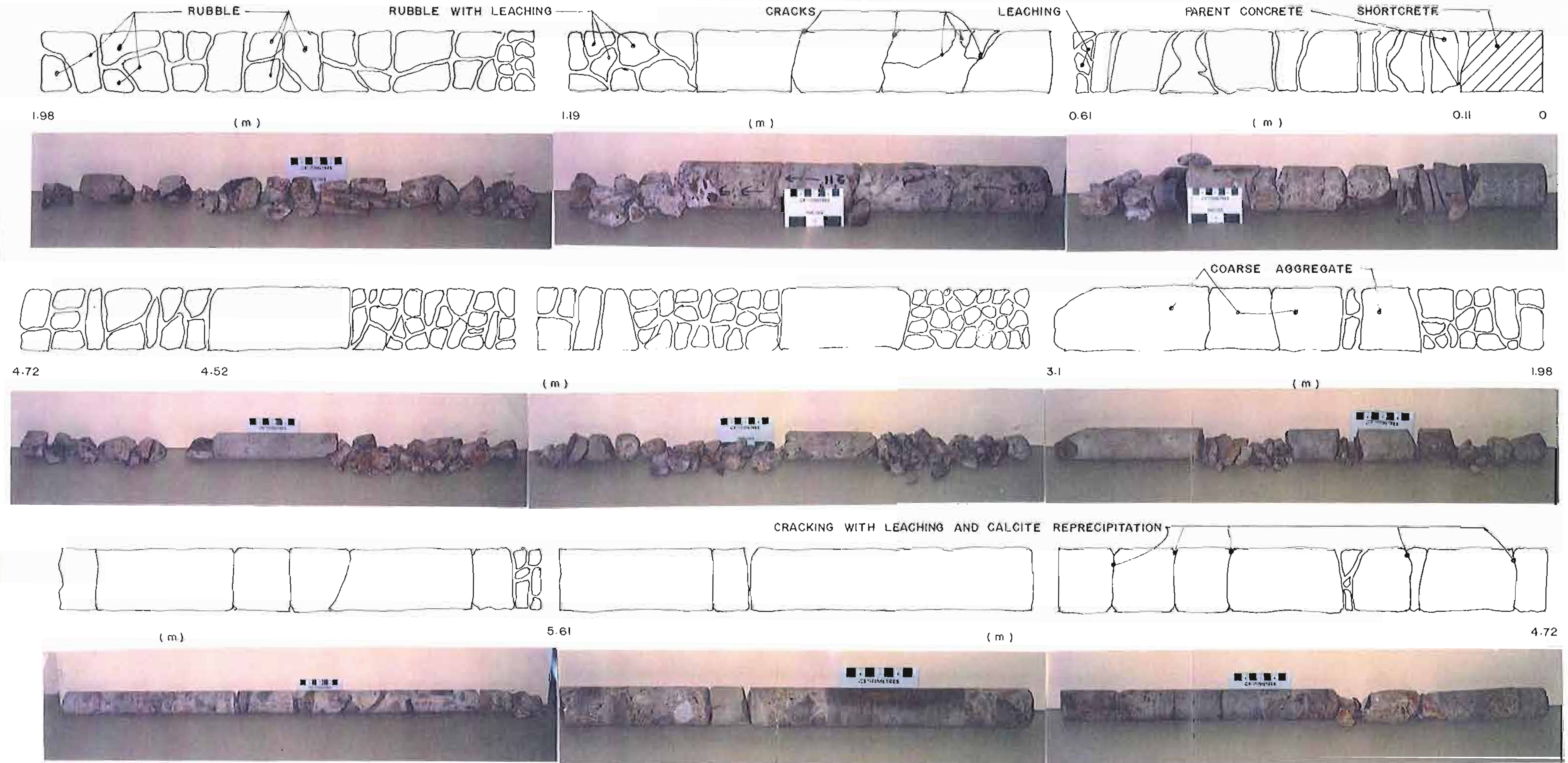


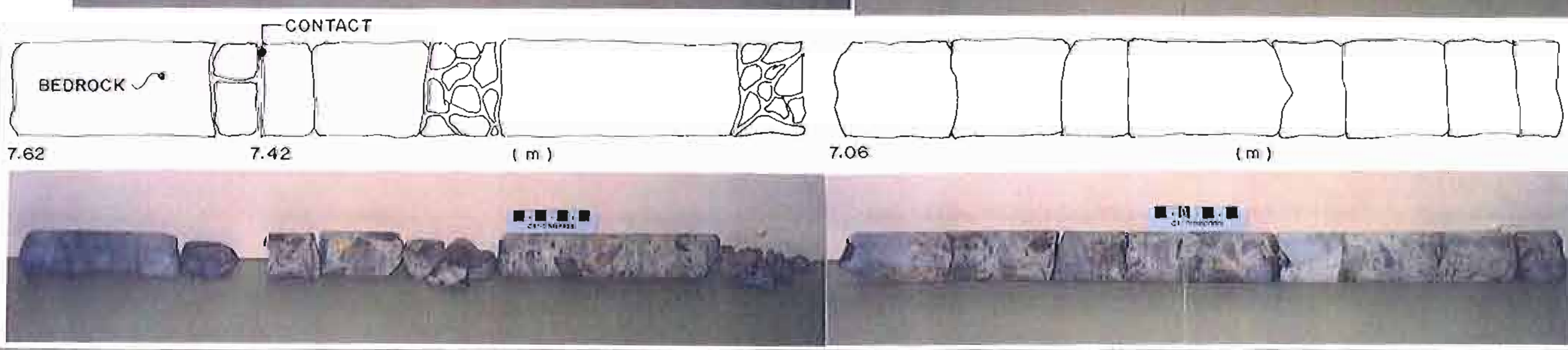
CORE NO. 7



PetoMacCallum Ltd.
CONSULTING ENGINEERS

DATE	SCALE	JOB NO.	DRAWING NO.
DEC. 1992	—	92HF137	—





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		DATE	SCALE	JOB NO.	DRAWING NO.
DRAWN	CIB	DEC. 1992	—	92HF137	-
CHECKED	JB				
APPROVED	JB				

APPENDIX C

Estimated Cost of
Suggested Remedial Works and Engineering Study

Our Ref: 92HF137

APPENDIX C

Estimated Cost of
Suggested Remedial Works

Crooks Hollow Dam
Flamborough, Ontario

A. <u>Rebuild Upper Portion of Dam</u>

- Cost of removal and disposal of upper portion of dam buttress and spillways $40,000.00
- Cost of epoxy injection of the remaining lower portion of the dam prior to reconstruction of the upper portion $40,000.00
- Cost to design, prepare specifications and reconstruct the upper portion of dam . . . . <u>$150,000.00</u>

B. <u>Application of Impervious membrane to upstream face of dam</u>

- Cost to design and prepare specifications for surface preparation and installation of modified bitumen type waterproofing treatment to upstream face of dam . . . $7,000.00

C. <u>Walkway Repairs</u>

- Cost to design, and prepare specification for removal of walkway, construction of reinforced concrete caps and new bearing for walkway. Remodelling, repainting and reinstatement of walkway $30,000.00

Cont'd

Our Ref: 92HF137

APPENDIX C (Cont'd)

D. <u>Localized Repairs</u>

- Epoxy injection of cracks . . . $125.00 per l.m. plus $25.00 per L
- Shotcrete repairs . . . . $200.00 per m^2

E. <u>Hydraulic Modelling in the event of damage during a major storm</u>

- Computer modelling using the "hec 2" program if baseline data is available $5,000.00
- Computer modelling using the "dam brk" program $10,000.00

Price for items A, B, C and D does <u>not</u> include costs for site supervision of the contractors work.