020409

CURRAGH RESOURCES CORPORATION

FARO MINE YUKON

REPORT

ON

DESIGN AND PROPOSED CONSTRUCTION

ROSE CREEK WATER RESERVOIR DAM RAISING

MARCH 1986

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1.0 INTRODUCTION

1.1 PURPOSE OF REPORT

Curragh Resources Corporation (Curragh) is currently actively involved onsite, with plans to reopen the Anvil lead-zinc mine and mill operations, near Faro Yukon, by mid 1986. Through the previous Cyprus Anvil Mining Corporation Ltd., (CAMC) and by transfer of ownership to Curragh, an amendment application to the Yukon Territory Water Board existing water licence Y2L3-2226 was submitted in August 1984[1]. The objective of this amendment application was to seek approval for an increase in fresh water supply to approximately 42,900 m^3/day (9,430,000 Igpd $\doteq$ 6550 Igpm).

The reasons for required extra water were documented in the report. Briefly they are as follows:

(a) daily production tonnages are expected to exceed the 1981 expansion design, and be up to 11,500 tonnes per day average.
(b) metallurgical problems will occur related to metal recoveries if efforts are made to recycle "chemical water" that has been through the flotation circuit.
(c) optimum grinding and flotation efficiencies occur at the lowest practical pulp density (i.e. highest practical water addition rate).

Justification for enlarging the main fork Rose Creek water reservoir by raising the existing level by 6.5 metres was given in the submittal to meet the extra fresh water demand. This would require a raising of the existing water retention dam by the same 6.5 metres. Alternatives to this proposal were evaluated and rejected for the reasons stated, they included: (a) use of groundwater from the open pit, (b) recycle of "chemical" water, (c) recycle of tailings impoundment water, and (d) a new reservoir created on the north fork of Rose Creek. Predicted impacts of the increased water use on Rose Creek and the Down Valley Tailings Impoundment were fully identified.

[1]Indicated references listed in Section 9.0

The main purpose of this report is to present the detailed dam modification design as promised in the aforementioned amendment application. This report also consolidates the findings of various 1984 and 1985 geotechnical and hydrological studies carried out subsequent to the amendment applications.

1.2 WATER DAM LOCATION

The location of the Rose Creek Water Reservoir Dam is shown on the key plan given on drawing 820-35-1 in Appendix C. The location plan also indicates the relationship of the dam to (a) the tailings impoundment areas further downstream in the Rose Creek Valley, (b) the mill and mine facilities to the north and (c) the main access road paralleling the water reservoir.

1.3 BACKGROUND INFORMATION

The present Water Dam was constructed in the summer of 1968. The participants in the design, engineering and construction were:

Prime Consultant/Contractor - Parsons/Jurden, New York
Dam Consultant - H.A. Simons - Vancouver, B.C.
Geotechnical Consultant - Ripley Klohn Leonoff - Vancouver, B.C.
Specialist Geotechnical Advisor - Dr. A. Gasagrande
Dam Earthworks Contractor - Ben Ginter Construction Limited, Prince George, B.C.
Subcontractor - Reinforced Concrete - General Enterprises, Whitehorse, Yukon.

The dam was constructed using locally available natural materials obtained from borrow sources located within the valley section to the west.

Construction was carried out generally in accordance with the drawings and specifications prepared by H.A. Simons. A list of the drawings is given in Section 9.0.

Subsequent to the 1981 mill expansion, when the requirement for additional fresh water was recognized, several investigations and reports have been prepared. The reports [2,3,4] include:

Oct. 1984 - A site visit to assess the state of the existing structure.

Mar. 1985 - Hydrology study of the Rose Creek under normal and flood flow conditions; a performance evaluation of the low level conduit and the spillway; a plan for handling flows during the dam raising construction.

Nov. 1985 - An operations report on the water dam including a geotechnical program consisting of 7 boreholes, laboratory testing and recommendations for design. Copies of relevant factual information from this work is included in Appendix B.

2.0 DESCRIPTION OF EXISTING DAM

2.1 GENERAL

The water supply reservoir for the Faro mill was created in 1968 by the construction of a zoned earthfill dam on Rose Creek, approximately 2 kilometers upstream of its confluence with the North Fork of Rose Creek. The dam is approximately 410 m (1350 feet) long and a maximum 20 m (65 feet) high above the valley floor. The crest width is 6 m (20 feet) and the upstream and downstream side slopes are 2.5 horizontal to 1 vertical (2.5 H:1V) and 2H:IV respectively. At its highest section, the dam is founded on up to a maximum about 5 metres of competent predominantly silty sand and gravel glacial terrace and till overburden. The abutment sections rest directly on gradually sloping bedrock surfaces.

The dam cross section consists of three basic earthfill zones:

Zone 1 - an impervious core
Zone 2 - a sand and gravel shell
Zone 3 - a random fill zone acting as a transition between the core and the shell.

The seepage control features incorporated in the original dam included an upstream blanket extension of the core zone which was keyed into the foundation tills. The blanket extended some 35 metres (120 feet) up the valley section to the original upstream construction cofferdam.

The outflow facilities comprise an overflow spillway equipped with stoplogs on the northern or right (looking downstream) abutment, and a low level outlet pipe under the maximum dam section near the south abutment. The spillway is 30 meters (100 feet) wide and consists of a concrete sill with 3.5 meter (11 foot) high wing walls. Stoplogs, not part of the original design, were installed in 1976 and are set in 8 bays supported by steel I beams. The discharge channel is boulder strewn bedrock with eroded overburden banks. The low level outlet system is a submerged drop inlet pipe with outlet control. The main pipe is 1070 mm (42 inch) in diameter, and there are two downstream valves with 610 mm (24 inch) openings.

2.2 FOUNDATION CONDITIONS

The information for the existing dam was investigated in 1967 - 1968 by a series of 18 drill holes identified by the DH100 series on the plan shown on drawing 820-35-02 in Appendix C. In addition a series of Test Pits were put down at the locations shown.

In summary the test program indicated that the depth to bedrock ranges from 6 to 11.3 metres (20 to 37 feet) beneath the valley floor, reducing to about 1.5 metres (5 feet) and less at both abutments. The overburden is described as a sand and gravel glacial till with silt and boulders.

The damsite was selected in an area where the width of the valley floor is narrowed by a long terrace projecting outward from the north abutment. The terrace reflects a buried bedrock spur that is mantled with the remnant of a terminal moraine.

The glacial till overburden was described as a gravelly ablation till consisting of an unsorted to poorly sorted mixture of silt, sand gravel and boulders with variable percentages of fines.

It was determined that the majority of the vertical profile consists of well graded material with high density, low porosity and of low permeability. An estimated maximum 5 percent of the profile consists of slightly sorted and poorly graded material. With a permeability several times greater than the average, seepage through the foundation was expected to occur chiefly through interconnected veins of this slightly sorted material. Since the field permeability of the deposit is governed by the statistical percentage of the area of the deposit that these veins occupy, its reliability and prediction by labratory or field tests must be governed by experience. Ripley Klohn Leonoff (April 20, 1968 report - Section 9.0) estimated the average deposit permeability to be in the range of 5.0×10^{-5} to 5.0×10^{-6} m/sec (10^{-2} to 10^{-3} ft/min.)

The original foundation investigation revealed spotty permafrost at one hole only DH103 located upstream of the initial dam centreline. Permafrost was not encountered in the valley floor or the terrace area where the spillway is located.

2.3 EMBANKMENT MATERIALS

The existing dam has been constructed with materials generally meeting the original gradation limits as shown on H.A. Simons Drawing D1575-058-06. In very general terms, from the examination of existing records as identified in Section 9.0, the core material tended to be slightly on the coarse side but almost entirely with the limits. The downstream shell material, perhaps if anything, had somewhat finer gravel sizes than the expected mid range of the gradation envelope.

The embankment has performed well over the 18 years since construction. The upstream riprap slope has a variable range in sizes from well-rounded fine gravel to cobbles and boulders.

A downstream weighting berm was placed in 1969 in response to seepage events at the toe. The berm consists of mine waste rock.

2.4 SPILLWAY

The spillway and discharge channel on the north abutment (30 metre wide by 3.2 metres high opening) has 20 cm (8 inch) I-beams embedded vertically in the concrete sill and stoplogs added. This has raised the full supply level of the reservoir to within one foot of the top of the embankment core. The bays nearest the concrete spillway walls consist of 1.2 metre by 1.2 metre (4 ft x 4 ft) concrete stub walls capped by four rows of wooden stoplogs. The I beams and stubwalls appear fixed into the structure and hence reduce the discharge capacity of the structure from the original design.

The discharge channel has been excavated through both rock and overburden. The channel bends towards the center of the valley about 150 metres (500 feet) downstream of the dam axis. The channel bed has cobbles over what appears to be sound and competent bedrock.

2.5 LOW LEVEL OUTLET STRUCTURE AND CONDUIT

The valve house for the low level outlet system is located in an excavated channel near the south (left) abutment just downstream from the toe of the dam. A 1070 mm (42 inch) diameter steel pipe from the reservoir enters the upstream end of the valve house, reduces to 610 mm gate valve and control plug valve. Both valves are operated by hand cranked wheels.

The outlet pipe then passes into and dicharges into the downstream chamber, where the flow impinges against the lower downstream inside face of the valve house before discharging to the right. The discharge is measured by monitoring the pressure drop across the control valve and the volume tabulated by an automatic digital counter. There is no method of closing the conduit upstream of the dam core.

As previously reported[2] the control valve wheel is very difficult to open and close. To open the valve from an initial 1000 gpm to fully open took at least 15 minutes and required two men with a 5 foot steel pipe to turn the control valve wheel.

Noise of collapsing cavitation bubbles and vibration increases as the valve opens, until shouting supplemented by hand signals is required to communicate. The noise and vibration is understood to be attributable to cavitation caused by the geometry of the plug valve. The flow passage area through the plug valve, even wide open, is less than the cross-sectional area of the pipe. Water accelerates through the valve and rapidly expands and decelerates immediately downstream. This rapid decrease in velocity creates zones of subatmospheric pressure immediately downstream of the valve which leads to the formation and collapse of cavitation bubbles.

Previous review has ruled out water hammer or slug flow from air due to vortex action through the inlet structure.

3.0 GEOTECHNICAL FIELD AND LABORATORY INVESTIGATION

3.1 FIELD DRILLING AND TEST PIT PROGRAM

The findings of the field investigations carried out in 1984 and 1985 at the dam site are reported in detail in the various reports prepared[2].[4] The work in 1984 consisted of shallow test pits, with four pits at the dam - 3 at the downstream toe and 1 on the dam crest.

A total of seven boreholes were put down in late 1985 to provide a more detailed evaluation of the embankment and foundation materials. The borehole logs and their location in plan are given in Appendix B for completeness. The results of this investigation are given in the operations Report[4] which was recently forwarded to the Yukon Territory Water Board. Test pitting was also carried out at both abutments to confirm the suspected shallow overburden depth of bedrock.

3.2 CONSTRUCTION MATERIALS

The test pitting program has revealed adequate supplies of construction materials. They include:

(a) A source of impervious glacial till fill suitable for Zone 1 located approximately 2.5 kilometers downstream of the site just north of the C.I.L. plant. The material encountered is dry low plastic silty till with some cobbles and boulder oversize.

(b) A sand and gravel pit with former screening facilities within the valley area near the tailings impoundment area.

A variety of options are available for the supply of the outer shell zones. The most cost effective choice, and one which will meet the design requirement of maximizing on fill placement and minimizing on special seepage control measures is to use calc-silicate mine rock from the open pit stripping operations. It will likely be necessary to establish a special drilling and blasting pattern to ensure the pit run mine waste meets the desired gradation range.

3.3 INSTRUMENTATION

Both piezometers and thermistors were installed in the existing dam during the September 1985 field investigation. The piezometer readings are given in Appendix B.

There are two distinct piezometric surfaces at the Water Dam. For those piezometers installed in the embankment materials i.e. 85-3, 4 and 5, the phreatic surface closely relates to what would be expected from a normal flow net through a zoned earthfill. The foundation piezometers, 85-1, 2 and 6 reflect elevated pressures, and prior to freezing in late October 28, 1985, were showing artesian conditions. Readings from all piezometers were observed to remain relatively steady over the period observed, with a trend to a slightly lower reading perhaps influenced by reservoir lowering.

Thermistor readings show the indicated frost penetration occurring with winter.

3.4 LABORATORY TESTING

Laboratory testing on embankment and foundation materials has consisted of gradation tests, Atterberg Limits, natural moisture content; and for the case of potential borrow materials, Standard Proctor compaction tests. The test results of all 1985 laboratory testing is given in Appendix B.

4.0 DESIGN CONSIDERATIONS

4.1 GEOTECHNICAL

4.1.1 Foundation Seepage

In designing for the raising the Water Dam perhaps the single most important consideration is the concern over controlling foundation seepage. The original dam design, based upon the recommendation of the recognized expertise of Dr. Arthur Casagrande, considered that there should not be a total seepage cutoff in term of a slurry trench, or a compacted impervious till key to cite examples. Rather, the design should allow for a "seepage window" within the central valley section of deepest overburden.

On the basis of the original design criteria of not adopting a total cutoff, there is no over riding argument for dramatically changing from this philosophy as a result of a 6.5 metre dam raising based upon the performance to date.

Accordingly the dam design has been based upon maximizing the used of downstream mine waste rock as a stabilizing berm and minimizing on the extent of active foundation seepage control measures. More is discussed on this subject in Section 5.1 where alternative seepage control measures are discussed.

The reported estimated seepage in October 1984 at the downstream toe was 9 litres/sec (120 Igpm).

A flow net seepage analysis was carried out for a dam width of 1370 feet, a maximum head differential of 61 feet between reservoir level and downstream toe level, and an assumed coefficient of permeability of 1 x 10^{-5} metres per second. A value of 56 Igpm was calculated for seepage through the foundation.

Adding to this value a similar amount for seepage through the embankment essentially confirms the range of seepage occuring.

The raising of the dam will result in an approximate 40% increase in the reservoir head. By flow net calculations it was found that the expected increase in seepage flow at the downstream toe will similarly increase by 40 percent. The seepage rate is thus likely to be about 12.5 litres/second (170 Igpm). This quantity of seepage is not expected to pose a problem with collection and control.

4.1.2 Embankment Stability

In the evaluation of the expected factor of safety for the stability of the proposed raised dam a great deal of information can be gained by examining the stability of the existing structure, and by extending the use of similar soil density and strength parameters known to provide a factor of safety exceeding unity for the present dam.

As a basic start, some preliminary manual calculations were carried out using a composite surface slip surface through the embankment and at a shallow depth within the foundation. The following parameters were used for all embankment zones and the foundation materials for ease in calculations:

Unit weight = 2000 kg/m^3 (125 lbs/ft^3)
Effective stress angle of friction $\phi' = 35^o$
Cohesive intercept $C' = 0$
Piezometric surface - as per measured standpipes.

The factors of safety for trial sliding blocks were determined to be in the order of F.S = 1.2. While this factor of safety may seem low, several conservative assumptions were made in these initial checks.

(1) A relative conservative angle of friction ϕ' valve was assigned for the compacted medium dense sand and gravel comprising soils. For example if the ϕ' value is increased to say 38^0, the factor of safety increases to a value in the range of 1.3 to 1.4.

(2) Assigning a cohesion intercept value (c') of zero is also relatively conservative.

These same strength parameters were used to carry out a preliminary check on the stability of the proposed dam using a predicted phreatic surface through both the embankment and foundation.

Without the benefit of pressure relief, factors of safety of the raised dam were determined to be in the 1.3 to 1.4 range by preliminary composite surface stability analyses. Recent computer analyses carried out by Dr. A.Z. Ahmed of Acres International Ltd. of Niagara Falls, Ontario, confirms this range of values, with factors of safety in the order of 1.4.

It is believed that with the benefit of pressure relief, the factor of safety should comfortably exceed a value of 1.5. This value is considered appropriate for static stability, and provides the necessary reserve for seismic design, which has conservatively been based on a horizontal force of 0.15 g (15% of gravitational acceleration).

A continued assessment of stability is planned as more information is gathered on the actual compacted soil strength parameters determined by direct shear tests or other strength testing procedures during the early stages of construction. As discussed elsewhere, the downstream construction allows modifications to be made to the weighing berm appropriate to the desired safety.

4.1.3 Spillway

The spillway has been designed as a rock cut some 6 metres (20 feet) deep at the north face. Based upon the logged description at Borehole 85-7 the bedrock is generally described as hard and competent biotite/muscovite/andalusite schist. Jointing is more prevalent in the upper 20 feet than at depth, with both core recovery and rock quality designation (RQD) increasing below this depth. This suggests that the concrete sill will be founded on a competent and tight bedrock surface. Some attention to the rock slope may be needed upon examination of the excavated face.

A detailed log of Borehole 85-7 is included in Appendix B together with the results of pressure packer tests.

4.2 HYDRAULICS

4.2.1 Low Level Flow Conduit

The design of the new valvehouse arrangement has been based on the recommendations made by Dome Petroleum and Acres Consulting Services in their reports[2,3,4]. In general terms, the new valvehouse will be constructed with improvements made in the form of flow measurement, and ease in valve operation.

Two new valves will need to be purchased in addition to some 50 metres of 24 inch steel conduit and required fittings and reducers. The 24 inch butterfly valve and 16 inch float activated control valve offer a smooth control over regulated flows.

The normal flow is proposed to pass through a modulating float valve. The operation of this valve is controlled by a remotely set float which will maintain a constant preset flow from the reservoir. This flow is also independent of the variance in head of the reservoir water level. Adjustment to flow is made by resetting the level of the float. Should the valve fail, flow would automatically be closed off.

The downstream weir, located 80 metres (270 feet) below the present valvehouse, will be maintained. A sight glass will be installed on the float stilling well and calibrated with the flow across the weir.

4.2.2 Spillway

The design of the new spillway at the north abutment has been premised on the design information provided in the Hydrology Study by Acres Consulting Services(3). The report observed that design floods up to a maximum return period of 10,000 years can be handled by a spillway 30 meters wide - similar to the existing spillway. The flow depth over the spillway for this flood is estimated as 1.05 metres. The spillway walls are designed to be 3 metres high and the top of the impervious core will be 1.5 metres above the spillway sill level. The reinforced concrete spillway design follows good engineering practise and is based upon a similar design to that prepared previously by H.A. Simons International Ltd. (See Section 9.0) Improvements will be made in the operation of the new spillway due to the fact that no stoplogs will be installed, nor will they be necessary.

Drawing 820-35-04 in Appendix C provides a plan and cross-sections of the spillway.

5.0 TECHNICAL EVALUATION OF ALTERNATIVES

5.1 EMBANKMENT AND FOUNDATION

The findings of the September 1985 investigation (4) indicated that one of the major areas to be addressed in raising the dam relates to the control of foundation underseepage.

The alternatives proposed for the foundation to reduce and/or control the high piezometric heads downstream of the dam included:

(a) A total cutoff constructed by means of a slurry trench at the upstream toe of the dam through they key trench in the upstream blanket - This alternative requires that the reservoir be completely drawn down during construction. It is concluded as not being feasible for reasons of schedule, its considerable degree of difficulty, the requirements for mobilizing a specialist contractor, and economics.

(b) A new cutoff core excavated to bedrock at the downstream side of the dam. This alternative would undoubtedly require an extensive wellpoint or other form of dewatering system. Although the work would not require the same level of specialist expertise as installing a slurry trench cutoff, special dewatering equipment would need to be brought to the site. Excavating at the toe of a dam, already having high piezometric levels, would have to be extremely carefully planned and executed and does pose some considerable risk unless the reservoir is practically emptied. Again schedule restrictions and economics dictate that this alternative is not feasible.

(c) Pressure relief wells or drains installed at the downstream toe of the raised dam will assist in reducing the high piezometric head by filtering and centrally collecting seepage water. The disadvantage of this alternative is that there would still be a seepage "window" under the dam. However as previously mentioned, the expected increase in seepage rate resulting from a high reservoir level is predicted to be well within the range of tolerable limits for storage purposes and ensuring adequate mill feed. This alternative combined with the enlarged downstream shell can be constructed economically, safely, and within the schedule constraints imposed.

It is proposed that pressure relief drains be installed while the dam is being raised. The installation of piezometers or standpipes in the dam, near the downstream toe, will assist in determining the total requirements for these drains. The piezometers will also serve to monitor the pressure head in the foundation soon after the reservoir rises above its former maximum level.

There are several alternatives available in the downstream construction and raising of the Water Dam. Insofar as the embankment is concerned, there is a choice in the types of materials that can be used primarily in the shell zone. Sand and gravel imported from the borrow sources downstream can be used successfully in the dam construction. However the reasons for selecting mine waste rock are principally related to economy, with the cost borne to the dam construction budget being only the overhaul cost of mine waste dumping.

5.2 SPILLWAY LOCATION AND DESIGN

As an alternative to the proposed north abutment for the new spillway, considered was given to the south abutment. The south abutment has the potential for being suitable for this purpose in view of the similarly shallow depth of overburden.

The south abutment however has several disadvantages, as listed below, which were deemed sufficiently strong as to make the north abutment a more desirable location:

(1) The south abutment is less accessible for inspection than the north abutment. It would require travel across the dam crest at all times.

(2) The nature of the ground contours on the south abutment suggest a steeper drop over a shorter distance to the valley floor only overcome with the excavation of a lengthy rock cut section to ensure the flow circumvents the planned new valvehouse.

(3) The south abutment does not have the advantage of an existing cut section (existing spillway discharge channel).

A possible alternative to the reinforced concrete spillway section, generally applicable to either abutment, would be to excavate a channel further up the abutment. The spillway would then be created as an entirely rock cut section. At both abutments there are disadvantages:

(1) The rock excavation quantity would be excessively large with a rock cut up to 30 feet deep. The overall length of the spillway cut through bedrock would also be substantial.

(2) In the case of the north abutment the main access road would need to be relocated.

(3) The favourable hydraulic flow properties of a conventinal broad crested weir associated with a conventional reinforced concrete spillway are not afforded by a rock excavation with a higher roughness coefficient.

5.3 LOW LEVEL FLOW CONDUIT

There are a variety of alternatives for the valving arrangement for low level flow regulation. The choice as to valve types is based upon past experience and the intended purpose. The design selected offers flexibility in providing a hydraulic bypass with the use of the relocated plug valve (for any major reservoir lowerings if ever needed) and the control afforded, for the downstream fresh water mill feed demand in the order of 7000 USgpm, by the use of a float operated control valve.

The least expensive alternative would be to simply duplicate the present system leaving only the existing gate valve in place since there is no conceivable way of removing this valve baring the complete emptying of the reservoir and isolation of the currently submerged inlet structure. This is not practical for scheduling and required reservoir reasons. There are several alternatives to the partial dismantling and access structure proposed, and as shown on the accompanying drawings, for the gate valve left in place. These are:

(1) Completely dismantle the existing valvehouse and either bury the gate valve - left completely open - with compacted rockfill placed surround, or create a thick heavily reinforced box structure around the valve and pipe section only.

(2) Break out only the "back" wall of the building for the extension of the 24 inch conduit to the new valvehouse. Reinforce the existing building to create a buried chamber and cover under the extended downstream rock fill shell.

The disadvantages associated with either alternative are that if for some unknown reason problems occur in the new downstream section of pipe between this gate valve and the new installation, or for that matter with the existing gate valve itself, access is forever lost. Albeit, while it is more expensive to construct an access to the gate valve, than to "bury" it, the future advantages of having an isolation valve are believed warranted.

One possible alternative dismissed soon was one of not constructing a new valvehouse at all, but rather extending the low flow conduit out from the back of the existing valve-house to the new dam toe. The present building would need to be incorporated within the downstream fill. This concept posed the problem of having to reinforce the building walls and roof from the adjacent embankment earth pressures. Also access would be severly restricted. The building would also need to be located probably in a depressed section of the rockfill which is not a desirable feature.

6.0 PROPOSED CONSTRUCTION

6.1 CONSTRUCTION PLAN

The raising of the Water Dam will involve three main operations, namely:

(1) Earthworks construction using the mines own forces and equipment to the maximum extent possible. Select mine waste rock from open pit stripping operations is expected to be in ample supply and maximum use will be made of this material. It should be noted that to avoid confusion with the previous embankment zoning, and the proposed new material sources, Zone Number 4 to 7 are used for the fine filter through to riprap materials. Only Zone 1 remains similar to the previous embankment material; Zone 2 sand and gravel shell and Zone 3 random fill are not proposed for the raising.

(2) Reinforced concrete construction for both the new spillway required further up the abutment, and a new valve house. Each of the structures will be very similar to the existing structures with some improvements made as identified previously in this report.

(3) The handling of the 1986 spring/summer runoff by the use of the existing spillway and low level outlet conduit while construction is in progress on the new spillway and valvehouse. Extra pumping and/or siphoning will be provided during the critical times when the normal flow at each of the existing conduit and spillway will in turn need to be stopped for a short period of time.

It is intended that the mine will use their own loading, hauling, excavation and placement equipment and mine site based labour for the majority of the earthworks construction. Reinforced concrete formwork, concrete supply and placement and extension of the conduit and new valving will be subcontracted.

6.2 DESCRIPTION AND SCHEDULE OF CONSTRUCTION ACTIVITIES

The proposed 1986 schedule of construction activities is briefly described below, generally in the chronological sequence that work will be carried out:

Certain activities are not described in the following list, they include the completed original site geotechnical and hydrological investigation work, separately presented in this report, design engineering, and the additional reservoir clearing which is recommended to be done before the ice comes off the reservoir.

(a) A haul road is to be constructed from the south side of the mine waste dumps down to the main access road generally as shown on the key plan drawing in Appendix C. The North Fork Rose Creek will be crossed with no requirements for a new stream crossing. Improvements to the section of road from the main access road down to the Water Dam will be required. In particular two existing culverts located at the crossing of the spillway discharge channel, some distance below the dam, will be removed and properly set in place at an appropriate location and grade to avoid any significant future erosion concerns due to freefall.

A new ramp constructed of mine waste rock will be constructed up and onto the existing dam and raised intermittently with the raising of the dam.

Schedule: late March - end April

(b) The downstream toe berm and base "footprint" area of the new Zone 4 section is to be stripped with all remaining snow, standing water, and organics removed. Some ripping or scarifying and grading will likely be necessary to ensure that there is good integration with the overlying new materials when they are placed and compacted.

Schedule: late April - Early May (as early as possible - temperatures and snow depth will dictate earliest start).

(c) The rock excavation for new spillway can be undertaken in the dry before major earthworks construction proceeds. Careful line drilling and blasting will be carried out to avoid overbreak. If suitable, rock excavated from the spillway channel will be used in the downstream shell zone near the toe.

Schedule: Early May - final excavation for spillway walls and sill footings will be carried out in July prior to concrete placement.

(d) The downstream shell will be constructed using select mine waste rock. The rock will be hauled in by large mine trucks, probably from the Wabco 120 ton fleet. It is expected that by having the trucks travel over the fill and ensuring they do not travel in the same path, good compaction will result. In addition it is expected that a large tractor dozer (CAST D9 or equivalent) will be used for spreading the fill into specified lift thickness and will assist with compaction. Additional compaction equipment will be brought onto the construction as required. The shell will be raised uniformly from the base up, with the exception of the area immediately around the existing valve house.

Schedule: May - late June with final work in late July.

(e) A new valvehouse and a conduit extending from the existing valvehouse will be constructed as early as conditions allow. The intention will be to have all water pass over the existing spillway during the spring runoff while this work is in progress. The gate valve at the existing valvehouse will be closed and the channel below drained. The foundations for the new valvehouse will be excavated and a mat foundation constructed on well drained compacted granular fill. The concrete wet well and valve supports will be constructed first and once everything is in place, the existing plug valve removed and relocated to the new valvehouse. Some dismantling of the existing valvehouse will begin and once a clear opening is created, a new section of 24 inch diameter steel pipeline will be installed from the remaining closed gate valve to connect with the new valving arrangement with hydraulic by-pass and relocated plug valve. The channel will need to be infilled and the new 24 inch pipe suitably buried within well compacted free draining granular fill. Once the hydraulic components are installed and carefully inspected, it may be possible to open the first gate valve and operate the new system before completing the erection of the pre-engineered building. When the new system is operating to satisfaction, a portion of the existing valvehouse will be dismantled and a special riser pipe constructed to provide permanent access to the existing gate valve. The existing building will in part provide the formwork for the cast-in-place section. The riser pipe with a permanent stairwell for access will be brought up with the placement of mine waste rock fill. Details of the new valvehouse and proposed dismantling/access to the existing valvehouse are given on the drawings in Appendix C.

Schedule: May to early June

(f) The new concrete spillway can be constructed as soon as the rock excavation has been completed and temperatures are consistently above freezing. It must be completed before the existing spillway section is later dismantled. The work required will include final bedrock surface preparation and shallow strip excavations for the foundation of the sill and reinforced concrete retaining wall. Some impervious material will likely be required in the new approach to continue the blanketting on the upstream face tying in with the blanket at the existing reservoir edge. A granular layer will be placed over this blanket for erosion protection from reservoir lowering and channel flow during future operations. Riprapping will be required at the approach channel concrete wingwall.

Reinforced concrete walls and the spillway sill will be constructed as shown on the drawings in Appendix C.

Schedule: May to Early June

(g) It is deemed important to excavate the existing dam crest to a depth of about 5 feet or perhaps more. This is required in order to remove the upper frost loosened materials and to expose the Zone 1 core and Zone 3 filter-transition material. These zones will be extended using suitable graded materials. The excavated material can be reused and blended and compacted in with the mine waste rock, Zone ~~2~~ 4 downstream shell, provided it is placed in the upper section (not near the base where filter gradation is needed).

Schedule: mid June

(h) The filter Zone 5 and 6 and core Zone 1 placement will be carried out from the prepared crest sections and brought up simultaneously with the downstream Zone 4 shell zone. Careful compaction control will be exercised to ensure the specified densities are achieved. Every effort will be made to raise each zone uniformly across the entire length of the dam. Special requirements are in order for placement of materials in and around the existing spillway to ensure good compaction.

Schedule: June to July

(i) The extent of the required dismantling of the existing spillway section needs to be evaluated to a certain extent as work proceeds. That portion of the spillway which will end up being in the upsteam shell Zone 4 likely will not need to be totally removed. However potential seepage pathways could occur around the walls or under the sill section, for that portion which will be incorporated into the core zone. This portion of the existing concrete should be demolished and removed (perhaps to be buried and compacted within the upper sections of the downstream shell zone). By field adjustment it may be possible to shift the dam (see Section B-B drawing 820-35-02 in Appendix C) so that the existing spillway can be totally incorporated into the upstream Zone 4 shell zone.

Schedule: mid July

(j) With ample supply of mine waste rock available, <u>riprap</u> Zone 7 will be placed on the upstream face of the newly raised dam, at the approach channel wing wall, and where required for erosion protection and energy dissipation in the new spillway channel.

Schedule: late July

(k) Certain <u>miscellaneous</u> construction activities will occur ongoing with the main earthworks and concrete work. These include:

(a) installation of piezometers and/or pressure relief wells at the downstream toe of the new fill section.
(b) installation of thermistors in the fill.
(c) A new flow measuring weir.

<u>Schedule</u>: intermittent during summer months.

7.0 ANTICIPATED RESERVOIR OPERATION

It is expected that the new raised reservoir will be operated similar to the present reservoir with the following improvement:

(a) The new spillway structure will have no restrictions in the form of concrete stub walls, vertical I beams, and timber stop logs and can safely pass the design storm.

(b) A new control valve with a hydraulic bypass arrangement will be provided in the new valvehouse to provide a smoother operating system, less prone to cavitation. In addition the existing gate valve will be left in place and access will be maintained such that the new valvehouse can be isolated if repairs are need by turning off the original gate valve.

In general terms the future reservoir will be operated similar to the operations over the last 18 years. The spring freshet usually starts in early May at which the reservoir is at its lowest level. The May runoff is usually sufficient to fill the reservoir to the spillway invert. With additional reservoir capacity, the future spillway may not be used until mid June.

During the open water months, this spillage and the flow in the North Fork of Rose Creek are more than adequate to meet the mine's needs.

From about the first of November to the beginning of May the reservoir is drawn down to meet the mill requirements. When the reservoir is at its lowest level there should, as in the past, still be an ice cover on the reservoir at its lowest level. An adequate water depth under the ice is still expected in May for the inlet structure to perform as in the past with no vortex action and slug flow.

A routine will be established to measure ice levels in the reservoir during winter months. A systematic recording will be kept of the volume of water released from the valvehouse.

Further details are available from the hydrological and hydraulic study carried out in March 1985[3] on the flood frequency, reservoir reliability to meet mill fresh water demand, and operational aspects of the low level conduit.

8.0 CONCLUSIONS - OBSERVATIONS

1. It is believed that the Rose Creek Water Reservoir can be safely raised by an additional 6.5 metres (21 feet), and continued satisfactory performance can be ensured provided the dam earthworks, reinforced concrete and valvehouse modification construction is carried out in accordance with good engineering practise adhering to the technical specifications and drawings.

2. The schedule for construction is quite restrictive since the objective is to have the water reservoir in operation by August 1986 to provide adequate reserves for the early spring 1987 mill fresh water feed requirements. Nevertheless, by careful sequencing of construction activities, the dam can be raised in the 3 - 4 month construction season available, and the discharge of the spring runoff can be safely accommodated.

3. The new spillway design is similar in many respects to the present arrangement with no stop logs needed, there will be a greater assurance that major flood flows can be safely handled.

4. The new low level flow conduit valvehouse modifications are believed to offer an improvement to the existing system in that (a) isolation of the new conduit section is provided by retaining access to the existing gate valve and (b) a hydraulic bypass utilizing the existing plug valve - relocated - and the float operated control valve allows for a wider range in operating flows and should provide for a smoother operation with less chance for cavitation occurring.

5. Special pressure relief wells will be installed at the downstream toe to reduce the foundation pore pressures.

6. Contingency measures are available with the design approach taken of maximizing on the use of mine waste rock at the downstream toe. If piezometer readings note higher than anticipated pore pressures in the foundation during reservoir filling, than were estimated in the stability analyses, additional mine waste rock can be quickly brought in to widen and/or raise the height of the berm section.

9.0 REFERENCES

The following references are cited in this report.

1. Cyprus Anvil Mining Corporation, August 1984, "An Application to the Yukon Territory Water Board to Amend the Existing Water License Y2L3-2226".

2. Cyprus Anvil Mining Corporation, Faro, Yukon-Rose Creek Water Supply Dam "Feasability Study of Raising the Height of the Existing Dam" Site Visit Report (October 3, 4 and 5, 1984) Dome Petroleum Ltd. (Geotechnical Group) in association with Acres Consulting Services Ltd. Calgary - October 1984.

3. Cyprus Anvil Mine "Rose Creek Reservoir - Hydrology Study" Acres Consulting Services Limited Calgary Alberta - March 1985.

4. C.A.M.C. Rose Creek Dam Investigation Operations Report - B. Berzins & H.E. McRae November 15, 1985 Dome Petroleum Ltd. (Geotechnical Group) with input from Acres International Ltd. Geotechnical Group.

The following background correspondence, reports and drawings were examined as related to the existing Water Dam for establishing the basic design criteria and as-constructed information.

Correspondence - Letters

1. August 9, 1967: Mr. A.B. Hamilton Ripley Klohn Leonoff (R.K.L.) to Mr. B. Carlson Parsons Jurden Corporation - re: core materials for Earth Fill Dam.

2. November 30, 1967: Mr. H. Kolle Parsons Construction Ltd. to Mr. D. Van de Voort, Parsons Construction Limited (P.C.L.) re: bedrock profile.

3. December 22, 1967: Mr. K.I. Morrison (R.K.L.) to Mr. V.W. Messick - Resident Construction Manager (P.C.L.) re: Dam Core Material.

4. January 4th, 1968: Mr. K.I. Morrison (R.K.L.) to Mr. V.W. Messick (P.C.L.) re: Dam Foundation Investigation.

5. March 6, 1968: Mr. C. Ripley (R.K.L.) to Mr. B. Carlson Parsons - Jurden Corporation re: proposed earthfill dam.

6. May 7, 1968: Mr. Earle Klohn (R.K.L.) to Mr. H.A. Simons (H.A. Simons International Ltd.) re: Summary of discussions with Dr. A. Casagrande.

7. April 24, 1968: Mr. Charles Ripley (R.K.L.) to Mr. D. Van de Voort (Parsons Jurden Corporation) re: borrow materials.

8. October 28, 1969: Mr. Earle Klohn (R.K.L.) to Mr. F.J. Shumes (H.A. Simons International Ltd.) re: site visit report.

Field Records

1.	Field density test results	Zones 1, 2 and 3
2.	Sieve Analysis	Zones 1, 2 and 3
3.	Sieve Analysis	Borrow Pit
4.	Proctor Compaction test	Zone 1

Drawings

The following H.A. Simons as-constructed water storage dam drawings. Those drawings in the list either voided during construction or for work not constructed are marked thus (*):

Drawing No.	Rev.No.	Title
D1575-058-001	4	Hydrograph
D1575-058-002	4	Location Plan #1
D1575-058-003	5	Location Plan #2
D1575-058-004	5	Exploratory Drilling and Foundation Section
D1575-058-005	6	Dam Excavation Details
D1575-058-006	6	Plan and Section
D1575-058-007	6	Spillway Excavation Plan and Section
D1575-058-008	2	Spillway Structure Concrete Outline
D1575-058-009	6	Spillway Structure Concrete Reinforcement
D1575-058-010	6	Diversion Structure
D1575-058-011	6	Upstream Cofferdam
D1575-058-012*	5	Downstream Cofferdam
D1575-058-013	4	Intake Structure
D1575-058-014	5	Intake Structure Reinforcement
D1575-058-015		Intake Structure & Superstructure
D1575-058-016*		Intake Structure Access Bridge
D1575-058-017	4	Intake Structure Valves and Piping
D1575-058-018		Intake Structure Trashrack - Details
D1575-058-019		Diversion Structure Reinforcing Details
D1575-058-020		Miscellaneous Details

The following Ripley Klohn Leonoff drawings:

1. Plan View Typical Cross-Sections of Water Supply Dam
2. Cutoff Profile of Water Supply Dam
3. Section of Initial Centreline of Water Dam
4. Section of Suggested Centreline of Water Dam
5. Dam site topography, location of test holes
6. Approximate bedrock contours
7. Recommended toe drainage.