Deloitte & Touche

Vangorda Creek Diversion Inspection Report June, 2004

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Prepared for:

DELOITTE & TOUCHE INC.
Interim *Receiver of Anvil Range Mining Corporation*
Suite 1900, 79 Wellington Street West
Toronto, ON M5K 1B9

Prepared by:

SRK Consulting
Engineers and Scientists

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Vangorda Creek Diversion Facility Inspection Report, June 2004

Deloitte & Touche Inc.

Interim Receiver of Anvil Range Mining Corporation
Suite 1900, 79 Wellington Street West
Toronto, ON M5K 1B9

SRK Project Number 1CD003.56

Steffen Robertson and Kirsten (Canada) Inc.
Suite 800, 1066 West Hastings Street
Vancouver, B.C. V6E 3X2

Tel: 604.681.4196 Fax: 604.687.5532
E-mail: vancouver@srk.com Web site: www.srk.com

Peter Healey
phealey@srk.com

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Authors:
Peter Healey
Pat Bryan

Table of Contents

List of Tables

List of Figures

List of Appendices

1. Introduction

SRK Consulting (SRK) has prepared this report following a request by Deloitte and Touche Inc. (the Interim Receiver of Anvil Range Mining Corporation) on June 18, 2004 to conduct an inspection of the Vangorda Creek Diversion facility at the Anvil Range mine. Deloitte made the request following a discovery by site personnel of a new groundwater discharge located above the pit wall in the northwest corner of the Vangorda Pit. Deloitte believed that this discharge may have been originating from the drop structure which is an integral part of the Vangorda Creek diversion facility. The location of the discharge zone and drop structure is shown in Figure 1 and Photo 1 of Appendix A. This discovery was made 10 days after a heavy runoff event which peaked on June 8, 2004. This event seemed to concentrate in the Vangorda Creek catchment and had a significant impact on the diversion facility.

A senior engineer from SRK, Mr Peter Healey, arrived on site on June 19, 2004 and carried out the inspection of the diversion flume, the headworks and the drop structure. The site manager, Mr Dana Haggar accompanied Mr Healey.

This purpose of the inspection was to determine the source and cause of the discharge, assess the immediate impact of the discharge on the diversion system and the pit wall stability, and to make recommendations on remedial action. An assessment was also made of the overall Vangorda Creek Diversion facility.

This report provides a brief description of the June 8, 2004 runoff event, an overview of the purpose and design of the Vangorda Creek Diversion facility, a summary of the observations that were made during the June 19 and 20, 2004 inspection. The report also provides recommendations for remedial action and modifications to the overall system that would provide sufficient security until implementation of the Final Closure and Reclamation plan.

2. Background

2.1 History of the Diversion

To develop the Vangorda pit, Vangorda Creek had to be diverted upstream of the pit into an open channel which passed around the northern pit perimeter and then re-enter the existing channel below the haul road as shown in Figure 1. The Vangorda Creek Diversion Facility was constructed to accommodate a 100 year flood event and convey the diverted water with minimal seepage loss. The diversion comprised three main components including:

- The headworks (see Figures 2, 3, 4 and 5; Photos 5 and 6)
- The channel (See Figures 6, 7 and 8); and
- The energy dissipation structure (Figures 9, 10, 11 and 12; Photos 1, 2, 3, 4).

The purpose of the headworks was to divert water from Vangorda Creek to the diversion channel. The channel, which conveys the water from the headworks around the pit to the energy dissipation structure, consisted of the flowing components:

- A transition section at the outlet of the headworks from the culvert beneath headworks embankment to the half round culvert; and
- An open channel section consisting of a half-round CSP culvert installed in an open channel.

The energy dissipation system consists of:

- A plunge pool at the end of the flume;
- A 2000mm dia. CSP culvert which conveys flow to a vertical drop structure consisting of a 3000mm dia. CSP (Multiplate) pipe; and
- A 1600 mm dia. CSP culvert beneath the haul road which conveys flow from the drop structure to Vangorda Creek.

In designing the diversion, it was expected that the facility would have a life not exceeding the design life of Vangorda Pit, which was projected to be mined out in 4 years.

2.2 Original Design and As-Built Configuration

The diversion was designed to accommodate the peak flow from a 100-year flood event. This event was estimated to produce an instantaneous flow of $10m^3/s$. The headworks embankment, which was built over an existing culvert crossing as shown in Figure 3, was designed to a height that would provide a 6.5 m headwater depth above the invert of the existing culvert during this event. The original design was based on an invert elevation of 1161.5m and a culvert diameter of 1500mm. Therefore, to provide for a minimum freeboard of 1m, the crest elevation was established at El. 1169m. During construction, however, it was discovered that the existing culvert was actually 1800mm in diameter and the inlet invert elevation was EL 1161.034. As the 1500mm pipe had already been delivered, a transition section was constructed between the existing 1800mm pipe and the new 1500mm culvert. Under this configuration, the headworks had a higher flow capacity than was originally designed.

The main diversion channel was originally designed as a 2400mm half-round culvert, 800m in length set within a riprap lined trapezoidal section as shown in Figures 7 and 8. The current length of the culvert as shown on the as-built drawings is about 900m. The as-built configuration of the diversion channel varied along the alignment in accordance with the type of foundation material encountered as shown in Figures 7 and 8. The as-built grade of the channel varied from 2 percent in the upper reaches to about 8 percent as it approached the plunge pool. The riprapped channel and flume was built to accommodate the design peak flow of $10m^3/s$ while maintaining a freeboard of 1m. During the mean annual flow (MAF), it was anticipated that in certain sections, the flume would overtop into the riprapped channel by about 0.2m.

Flow from the flume and channel enters a plunge pool located near the haul road as shown on Figure 9. The plunge pool also receives flow from the Sheep Pad Sediment pond, which was constructed to control sediment from the Grum Interceptor ditch. From the plunge pool, the flow enters a 2000mm diameter CSP culvert, which conveys the flow to a vertical 3000mm multiplate drop structure. The drop structure and the plunge pool form part of an energy dissipation system, which reduce the velocity of the flow before it discharges into a 1600mm CSP culvert below the haul road. Water discharging from the 1600mm pipe re-enters Vangorda Creek.

2.3 June 8, 2004 Flood Event

An intense convective storm passed over the Vangorda Creek catchment during the morning of June 8, just a week or so after the spring freshet had peaked. The heavy rainfall, coupled with the already saturated ground conditions, resulted in the largest flood event to be experienced by the Vangorda Creek Diversion Facility in its 14-year life. The flood event was extensively photographed by site personnel near the peak of the event. Photographs 7 to 15 show the flood flows at various locations along the diversion facility (see captions on photographs for details). A view of the drop structure taken during the June 8 storm is included in Appendix A as Photograph 38. A brief report describing the event and actions taken prepared by Deloitte & Touche Inc. is included as Attachment "B".

The objectives of this section are to:

- Estimate the peak discharge rate of this flood event;
- Estimate the return period of the event; and,
- Compile information on past flood studies prepared specifically for the diversion.

The magnitude of the flood event was estimated using a combination of high water marks and hydraulic calculations. Although any number of locations along the diversion could have been used to infer the peak discharge rate, three locations were identified as being most suitable. These correspond with the locations where the stream's flow is conveyed in a closed conduit, namely:

- The culvert passing under the Blind Creek Road (the diversion's headworks);
- The culvert leading from the stilling basin to the drop structure; and,
- The culvert passing under the Vangorda Haul Road.

High water marks at the inlets of these three culverts were used to provide reasonably accurate estimates of flow. The conditions were particularly favourable for making accurate estimates because the inlets of all three culverts were submerged during passage of the flood event. After a culvert inlet becomes submerged, the culvert effectively becomes a sensitive measuring device (i.e., small increases in discharge are associated with large increases in water level in the pond upstream of the culvert inlet).

Table 1 summarizes the flood estimates made at the three locations. In preparing the estimates, it was recognized that the flow through the respective culverts could have either been "inlet" or "outlet" controlled. Inlet control means the inlet of the culvert acts essentially as a weir and controls the amount of water that can pass into the culvert. Outlet control, on the other hand, means the flow through the culvert is dictated by frictional losses along the whole length of the culvert. When culverts are running full, as was the case on June 8, it is not initially known which control may have dominated. To overcome this problem, two sets of calculations were performed, one assuming inlet control and the other assuming outlet control. The calculation resulting in the lower flow value indicates which of the controls governed during the passage of the flood peak. For example, the flow estimates for inlet and outlet control at the headworks worked out to 14.7 m^3/s and 13.6 m^3/s, respectively. This suggests outlet control governed and the peak flow rate was 13.6 m^3/s.

The estimated flows at the three locations range from 11.0 m^3/s to 13.6 m^3/s. As described in Table 1, the estimated flow rate at the diversion headworks may overestimate the true flow rate. The high water mark at the diversion headworks set the amount of energy available to pass the flood. The hydraulic calculations used to produce the values in Table 1 determined how much of this energy was used to overcome frictional forces and how much was turned into kinetic energy. The latter energy term dictates the flow rate. During passage of the flood, rock and debris was deposited in the trash rack upstream of the inlet to the culvert at the headworks. This may have caused more resistance to the flow than allowed for in the hydraulic calculations. Greater resistance would mean less energy was available to be converted into kinetic energy, resulting in a lower flow rate.

The best estimate of the flood peak for the June 8 event was judged to be the average of the estimated flows at all three locations, or 12 m^3/s. This is 20% greater than the design discharge for the diversion facility. The flow estimates in Table 1 suggest that the magnitude of the flood peak may have been reduced as it moved downstream through the system. This seems unlikely because there is only minimal storage in the system and the diversion receives an additional inflow from the Sheep Pad Pond at the stilling basin.

After estimating the flood magnitude, the next step was to estimate the return period of the June 8 event. The diversion was designed to pass the 100-year event. At the time of its design in 1990, the magnitude of this event was estimated to be 10 m^3/s using a flood study prepared as part of the permitting studies for the Vangorda Plateau Development. Since the magnitude of the June 8 flood appears to have been greater than the design discharge, this would suggest the flood had a return period of greater than 100 years.

At least three other potential means are available to infer the return period of the flood. In brief, these are:

- Application of a recent flood-estimation technique developed by Northwest Hydraulic Consultants Ltd (NHC) for the Faro site. This technique is based on a method known as Regional Analysis and can be readily applied to the Vangorda site.

- Characterize rainfall intensities of the June 8 storm using data collected at an automated climate station operated by the Yukon Government at the Grum mine site. This station has operated for only a short period, but the measured rainfall intensities during the storm can be assigned approximate return periods using the intensity-duration-frequency curve of a long-term climate station in the region.
- Estimate the return period of the June 8 event using the streamflow data collected by DIAND at their hydrometric station near the mouth of Vangorda Creek. This would entail fitting a theoretical frequency distribution to the annual series of flood peaks measured at this station, including the one generated by the June 8 rainstorm. The station has been in existence since 1977 but has gaps in its record. The station is believed to have measured the annual peaks in at least 19 of the years in the period from 1977 to 2003.

At the time of preparing this report, only the first method described above was used. A request has been made for the Grum rainfall data and the Vangorda Creek flow data so that the other two methods can be checked.

Table 2 shows how the NHC study was used to provide independent estimates of flood flow at the entrance of the Vangorda Creek Diversion Facility for a range of return periods. In essence, the flood estimates for the Faro Creek Diversion were transposed to the Vangorda site. Details of how this transposition was undertaken are provided in the footnotes to the table. The catchments of the Faro Creek Diversion and the Vangorda Creek Diversion have very similar drainage areas and elevational characteristics.

Given an estimated peak flow rate of 12 m^3/s, the flow estimates in Table 2 suggest the return period of the June 8 event was about 100 years. In interpreting the flow values in Table 2, it should be noted that NHC suggested error bands of about 25% be placed on the flood values generated by their flood-estimation technique.

Having estimated the magnitude and return period of the June 8 event, the final objective of this section is to compile flood estimates made specifically for the Vangorda Diversion. Two studies are known to have been made, the original one used to provide the design flow for the diversion and a subsequent one used to examine potential closure measures for the Vangorda Plateau mine development. Details of these two flood studies and their resulting flood estimates are summarized in Table 3.

A summary of observations that were made and actions taken during the flood event on June 8, 2004 is presented in Appendix B.

3. Observations

3.1 Discharge Zone

Following the June 8, 2004 flood event, several smaller rainfall events occurred, which generated run off in the surrounding drainage ditches and culverts. The drainage ditch, which runs along the haul road towards the Vangorda Pit, flows into a culvert at a low point in the road. From the culvert, the flow discharges via a gully that runs alongside the drop structure into the Vangorda Pit. As the weather conditions improved and the runoff diminished, mine staff discovered a discharge from northwest corner of the pit wall. With further investigation, it was found that the discharge was originating from a spring in the gully that runs alongside the drop structure. The discharge zone was located on the bedrock surface at the edge of the pit (Photo 1). It had not been identified previously because it had been masked by the runoff from the surrounding area. The flow rate was estimated to be about 6L/s (see Photo 16). The drop structure and the gully are shown in Photos 17 and 18.

This discharge raised concerns about the volume of water that was now entering the pit and the additional treatment cost. Furthermore, the source of the discharge was unknown but was believed to be originating from either the drop structure or the adjacent culverts. Therefore it was decided to conduct an inspection of the inside of the drop structure.

On June 19, 2004, water in the flume was diverted, under emergency measures, to Vangorda Pit and an inspection of the drop structure was carried out. At that time, the flow in the flume was estimated to be about $0.5m^3/s$. While most of this flow was diverted to Vangorda Pit, a sizeable flow was still entering the drop structure from the Sheep Pad sediment pond.

Prior to the inspection, a review of the as-built report and drawings was carried out. It was determined from this review that the likely cause of the discharge was a leak from the coupling between the 2000mm diameter CSP culvert and the 2000mm diameter nipple that forms part of the drop structure as shown in Figure 11. Problems with these joints had been experienced during construction and a 200mm perforated CSP drain pipe wrapped in filter fabric was installed below the coupling to collect any leakage and direct to the open pit. This pipe, has overtime been covered over with fill.

Once inside the drop structure, it was discovered that the leak was in fact caused by a separation in the coupling just inside the 2000mm pipe (see Photos 19, 20, 21 and 24). Damage to the crown of the 2000mm nipple was also observed. As water was still flowing into the drop structure at the time of the inspection, no attempt was made to inspect the base of the drop structure below the invert of the 1600mm culvert. However, a visual inspection of the 1600 joint did confirm that there had been no damage to this culvert.

3.2 Drop Structure and Culverts

During the inspection of the drop structure, it was possible to obtain a clear view along the inside of the 2000mm culvert to the entrance at the plunge pool. From this visual observation, it appeared that there had been no other damage to the culvert. No obstructions were observed.

Concerns had been raised about the structural integrity of the drop structure and that maybe it had moved during the June 8, 2004 flood. While there has been some erosion of the material around the structure, there does not appear to have been any movement, tilting or settlement.

As possible remedial measures to correct the damage to the coupling would include replacement of a section of the 2000mm pipe and re-establishing the coupling, the crown of the pipe was exposed at the location of the joint to assess the nature of the coupling and its precise location (See Photos 22 and 23).

It was confirmed that the coupling was about 450mm wide and there was no obvious evidence of a gasket or membrane. No further excavation or disturbance was carried out.

3.3 Headworks Culvert and Embankment

During the June 8, 2004 flood, water was rising behind the headworks embankment at an alarming rate and the decision was made to breach the abutment on the south side to allow water to discharge into Vangorda Creek via the original channel and avoid a possible overtopping of the embankment. Figure 2 shows the approximate location of the breach and an aerial views are provided in Figures 4 and 5. The depth of the excavation is estimated to be about 2 metres below the crest of the embankment. A view of the breach is provided in Photos 25 and 26. The excavation has been left open until a decision is made on whether to backfill or install an overflow spillway. It was reported that the headwaters rose to within 500mm of the crest of the dam as shown in Photo 15.

Upstream of the embankment and the 1800mm culvert, the inlet pool contains a significant amount of debris deposited during the flood. Cobbles and boulders have accumulated around the trash rack and partly restrict the invert of the culvert as shown on Photo 27.

3.4 Flume and Channel

During the peak of the flood, water accumulated in the plunge pool at the outlet of the flume and the headwater rose to about 1.2 to 1.5 metres above the crown of the 2000mm culvert. The rising waters caused erosion of the bank adjacent to the flume access road. Deloitte carried out temporary remedial work on the bank by placing several loads of granular material to prevent further erosion (Photo 28). Two of the end sections of the flume were badly damaged during the flood and have been removed (See Photo 28).

Despite the high flows the flume experienced, overall the facility performed in accordance with the original design (Photo 30). However, many of the cross braces were buckled during the flood and some of the joints between the 6m flume sections have weakened (Photo 29). Some of the anchor ties that hold the flume sections down loosened and lifting of some flume sections was experienced. Photo 13 shows one particular flume that was lifting during the flood and was held down by an excavator. It is currently held down using a heavy steel canopy (Photo 33). It is believed that the lifting was a result of high velocity flow on the uphill side of the new flume section.

Some of the riprap in the channel was displaced during the flood, which has reduced the support for certain sections of the flume.

4. Conclusions and Recommendations

4.1 Drop Structure

Planning for the repair to the damaged joint in the 2000mm pipe should be initiated immediately. Parts and materials to carry out the repair should be purchased as soon as possible. The repair work, however, should be conducted under dry conditions with no flow in the system. Consequently, this work would be carried out during the low flow period, which normally occurs late July or early August.

The plan would be to block off the 1800mm culvert at the headworks and allow flow in Vangorda Creek to discharge into Vangorda Pit via either the existing breach in the south abutment or through the proposed overflow spillway referenced in Section 4.3. At the plunge pool, the 2000mm culvert would also be blocked off and water that accumulates in the pool from the Sheep Pad Pond, would be pumped either into the pit or over the haul road to Vangorda Creek.

The repair to the damaged joint would likely involve excavating around the joint and installing a larger collar with a rubber gasket. An attempt could also be made to straighten and weld the damaged sections of the pipe from both the inside and the outside of the pipe and then encasing the joint in concrete.

During the repair work, an inspection of the base of the 3000mm pipe should be carried out as well as a more thorough inspection of the 1600mm culvert.

4.2 Plunge Pool

Debris that accumulated in the plunge pool during the flood should be removed and additional riprap and filter fabric should be placed around the sideslopes of the pool.

4.3 Headworks Embankment

It is recommended that the breached section in the south abutment of the headworks embankment be replaced with an engineered overflow spillway to the Vangorda Pit. The purpose of the spillway would be to provide additional security for the diversion channel over the next 5 to 10 years until implementation of the Final Closure and Reclamation Plan. It is recommended that the spillway consist of two 1m diameter CSP culverts installed at an invert elevation 4 metres below the crest of the existing embankment. Under these conditions, flow in the overflow spillway would begin to spill into the Vangorda Pit once the flow exceeds the current system design capacity of $10m^3/s$. This would provide additional protection against overtopping of the embankment during a more extreme event than the June 8, 2004 flood. The spillway would also provide additional security for the flume and the energy dissipation system. Figure 13 illustrates the flow capacity of the proposed combined system and the impact of the spillway on the overall system.

In order to prepare a final design for the spillway and to confirm the hydraulic capacity of the existing system, it is recommended that a survey be carried out at the headworks embankment and around the area of the breach.

SRK also recommends that the cobbles and debris that have accumulated at the entrance to the 1800mm diameter culvert be removed.

4.4 Diversion Channel and Flume

Based on the recent inspection of the flume and the overflow channel, SRK believes that the existing facility with the recommended repairs, upgrades and ongoing maintenance presented in this report, would provide sufficient security for the open pit until implementation of the Final Closure and Reclamation plan.

Firstly, it is recommended that a survey be completed along the entire length of the diversion. This survey would be carried out at the same time as the survey of the headworks embankment. Based on the survey, a profile along the invert of the flume will be prepared and cross sections will be drawn up at key stations along the alignment. The capacity of the existing flume and channel will then be assessed on the basis of the original design criteria i.e. peak instantaneous discharge of $10m^3/s$.

Following the assessment, SRK will provide recommendations on restoration of the existing flume and channel. It is anticipated that these recommendations will include replacement of a number of flume sections, reconstruction in some places of the channel section, removal of the obsolete flume sections, replacement of riprap where required and repairs to the damaged flume joints. SRK will also provide recommendation on the plunge pool which will likely include removal of the debris and alluvial deposit in the bottom of the pool and restoration of the riprap protection around the sideslopes.

Prepared by

Peter Healey, P.Eng.
Principal Engineer

Pat Bryan, P.Eng.
Associate Hydrologist

Table 1 Estimated Magnitude of June 8, 2004 Flood

Location	High Water Mark	Estimated Peak Instantaneous Flow (m^3/s)			Comments
		Assuming Inlet Control	Assuming Outlet Control	Governing Hydraulics	
Diversion Headworks (Blind Creek Road)	Approx. 0.3 m below crest of embankment	14.7	13.6	13.6	1) During the flood, rock was deposited in the trash rack upstream of the inlet to the culvert. This may have caused more resistance to flow than allowed for in the hydraulic calculations. If this is the case, the true flow through the culvert was less than the estimate provided in this table. 2) The possibility exists that the flood peak in the stream may have been higher than computed. A ditch was constructed across Blind Creek Road to prevent overtopping of the headworks embankment. If this ditch was completed before the arrival of the peak discharge, then the maximum flow through the culvert was somewhat less than the maximum flow in the stream. Overflow from the ditch reported to the open pit.
Stilling Basin	1.2 m to 1.5 m above crown of 2000 mm diameter outlet pipe	11.0	Not computed	11.0	No estimate of flow rate is provided for the assumption of outlet control. To become the governing hydraulic condition, the flow rate computed for this condition would have to be less than that estimated for inlet control (i.e., less than 11 m^3/s).
Drop Structure	Rim elevation of drop structure (1128.4 m)	14.1	11.6	11.6	The actual flow may have been larger than the value computed using hydraulic calculations. The drop structure was observed to be spilling during the passage of the flood event.

Table 2 Recent Flood Estimates for Faro Creek Transposed to Diversion Headworks on Vangorda Creek

Return Period (years)	Peak Instantaneous Flood Discharge (m^3/s)	
	Faro Creek Diversion above NF Rose Creek [1]	Vangorda Creek at Diversion Headworks [2]
2	1.9	2.3
50	7.7	9.4
100	9.4	11.5
200	11	13.4
500	14	17.1
1000	16	19.5

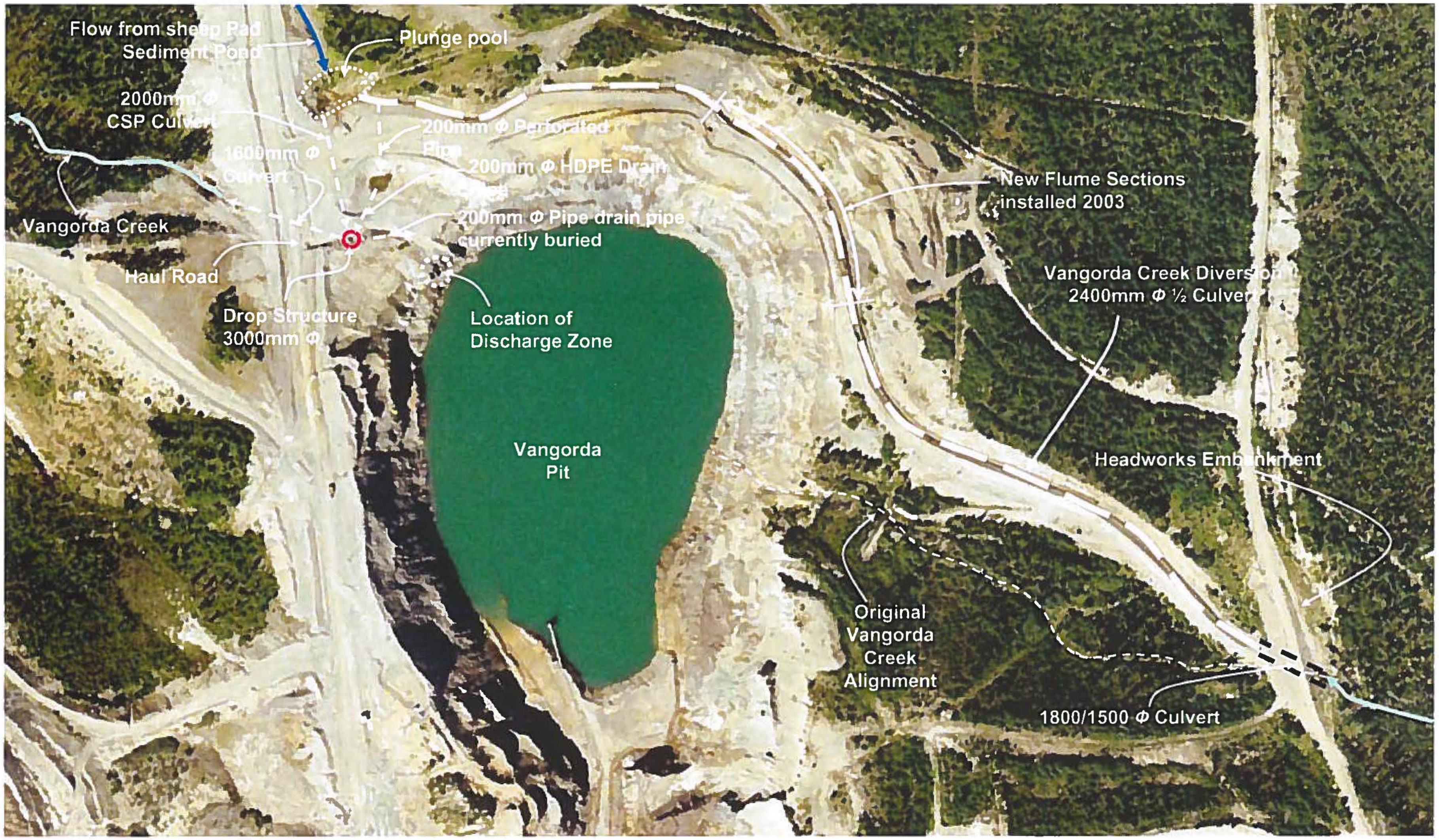
Notes: 1) Flood estimates for Faro Creek were made by Northwest Hydraulic Consultants Ltd. in their report entitled "Hydrotechnical Study for Closure Planning, Faro Mine Site Area, Yukon" (December, 2003). The drainage area controlled by the Faro Creek Diversion is 16 km^2.

2) The estimated floods for the Faro Creek Diversion were transposed to Vangorda Creek using the following ratio: $(A_{Vangorda}/A_{Faro})^{0.87}$, where $A_{Vangorda}$ is the drainage area of Vangorda Creek at the diversion headworks and A_{Faro} is the drainage area controlled by the Faro Creek Diversion. The exponent on the ratio of drainage areas is used to account for observation that unit flood flows tend to decrease as drainage area increases. Unit flood flows are the absolute flood value divided by the contributing drainage area. The drainage area at the headworks is 20.1 km^2. For a given return period, the floods at the entrance to the Vangorda Diversion are estimated to be 122% of the floods conveyed by the Faro Creek Diversion.

Table 3 Compilation of Flood Estimates for Vangorda Creek Diversion

Source of Flood Estimates	Return Period (years)	Estimated Peak Instantaneous Flood (m^3/s)	Estimation Technique
Vangorda Plateau Water Licence Application, Volume I (1990)	2	4.1	Regional Analysis (based on streamflow records from 9 WSC and DIAND stations)
	20	7.6	Regional Analysis (based on streamflow records from 9 WSC and DIAND stations)
	50	9.0	Regional Analysis (based on streamflow records from 9 WSC and DIAND stations)
	100	10	HEC-1 rainfall/runoff model (SCS unit hydrograph, A = 18.8 km^2, t_L = 2.2 h and CN = 80)
	200	13	HEC-1 rainfall/runoff model (SCS unit hydrograph, A = 18.8 km^2, t_L = 2.2 h and CN = 80)
	500	18	HEC-1 rainfall/runoff model (SCS unit hydrograph, A = 18.8 km^2, t_L = 2.2 h and CN = 80)
Anvil Range Mining Complex, ICAP (November, 1996)	500	31	Rational Method (A = 20.1 km^2, C = 0.8 and t_c = 4.9 h)
	PMF[1]	180	Creager Curve (with Creager Constant = 26)

Note: 1) The return period for the probable maximum flood (PMF) is likely in the order of 10,000 years.



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Vangorda Creek Diversion Facility
Inspection Report

General Arrangement Plan

PROJECT	DATE	APPROVED	FIGURE
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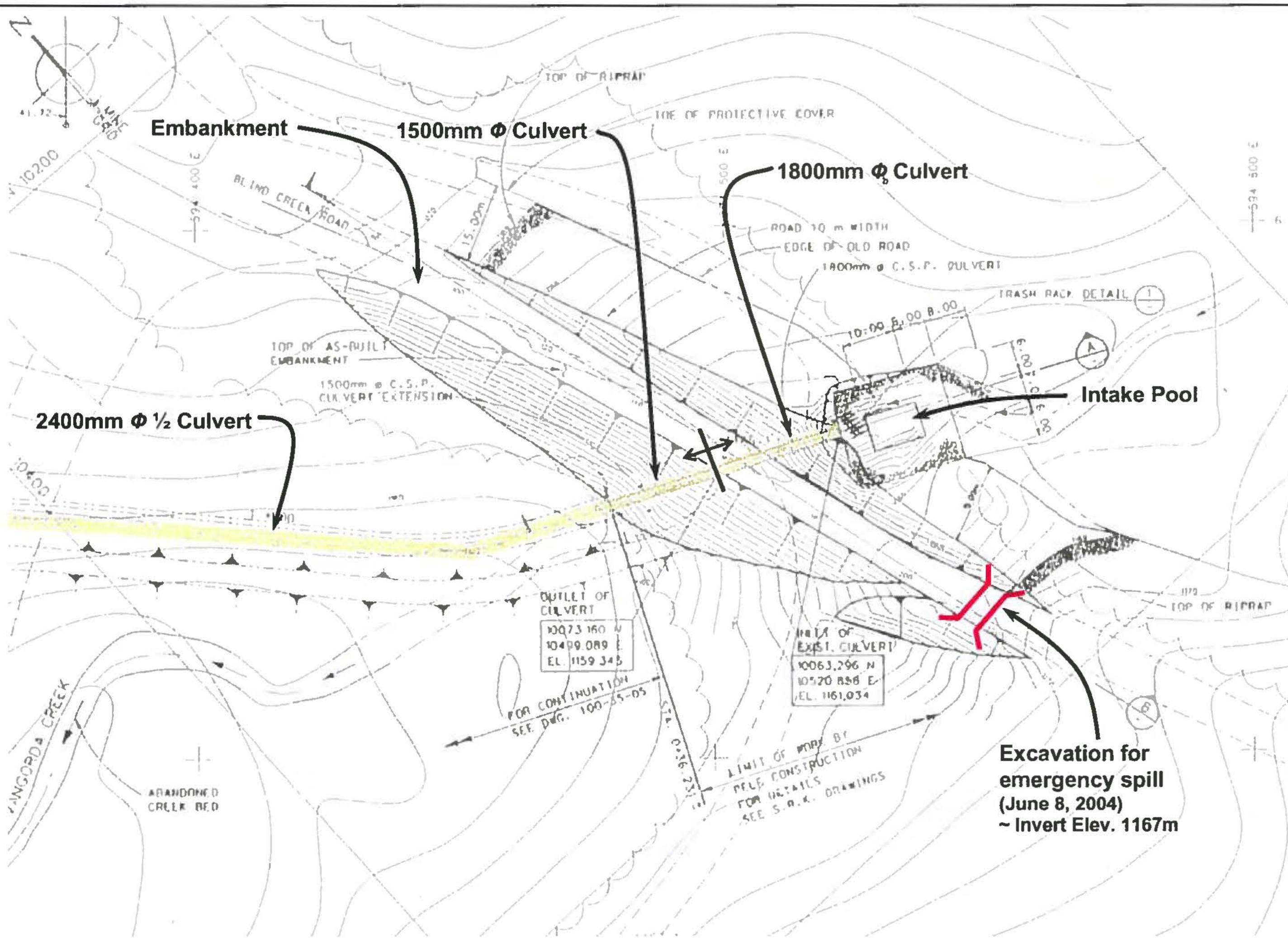

SRK Consulting Engineers and Scientists	Vangorda Creek Diversion Facility Inspection Report
Deloitte & Touche	**Headworks Embankment - Plan**

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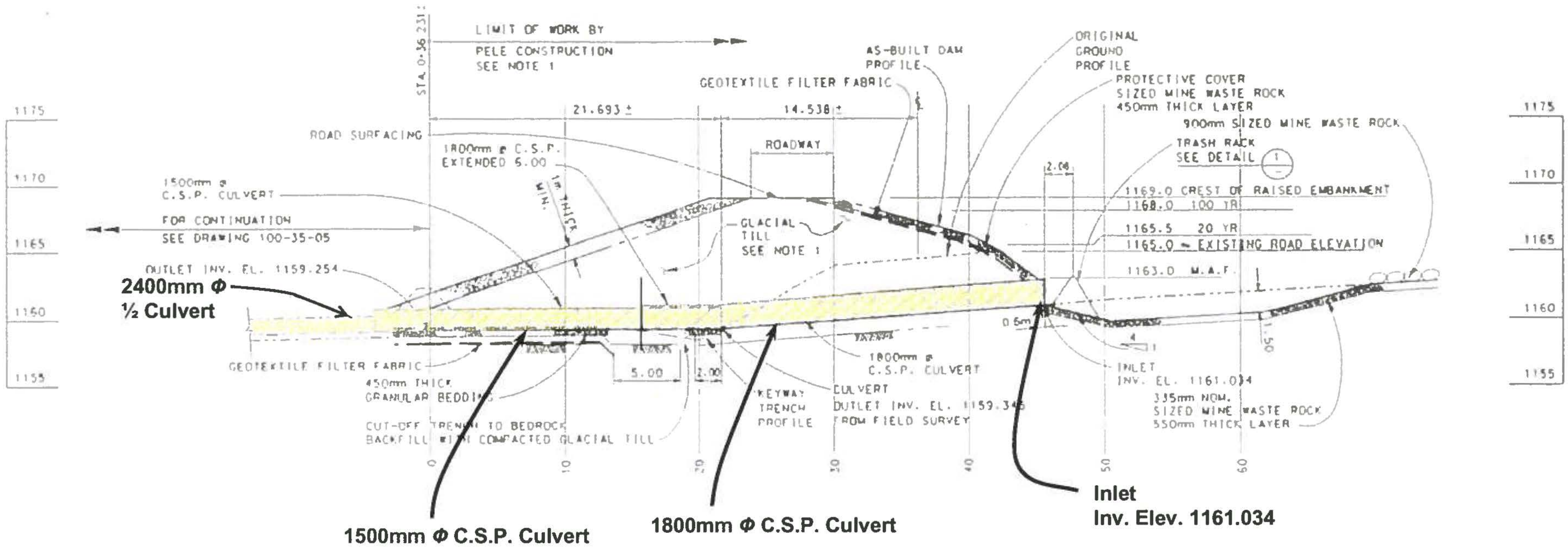
1CD003.56	June 2004		2



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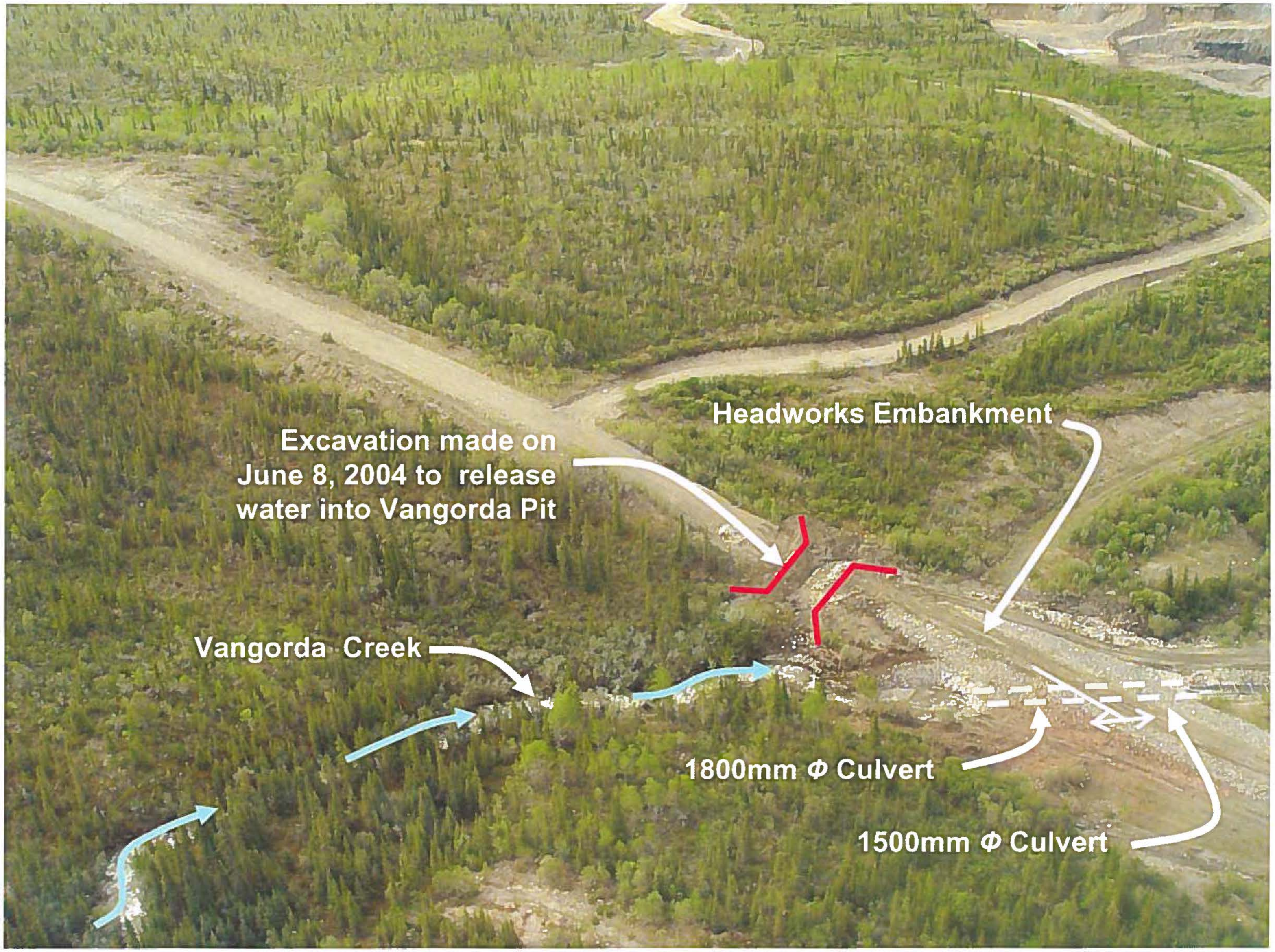
Headworks Embankment - Section

PROJECT	DATE	APPROVED	FIGURE
1CD003.56	June 2004		3



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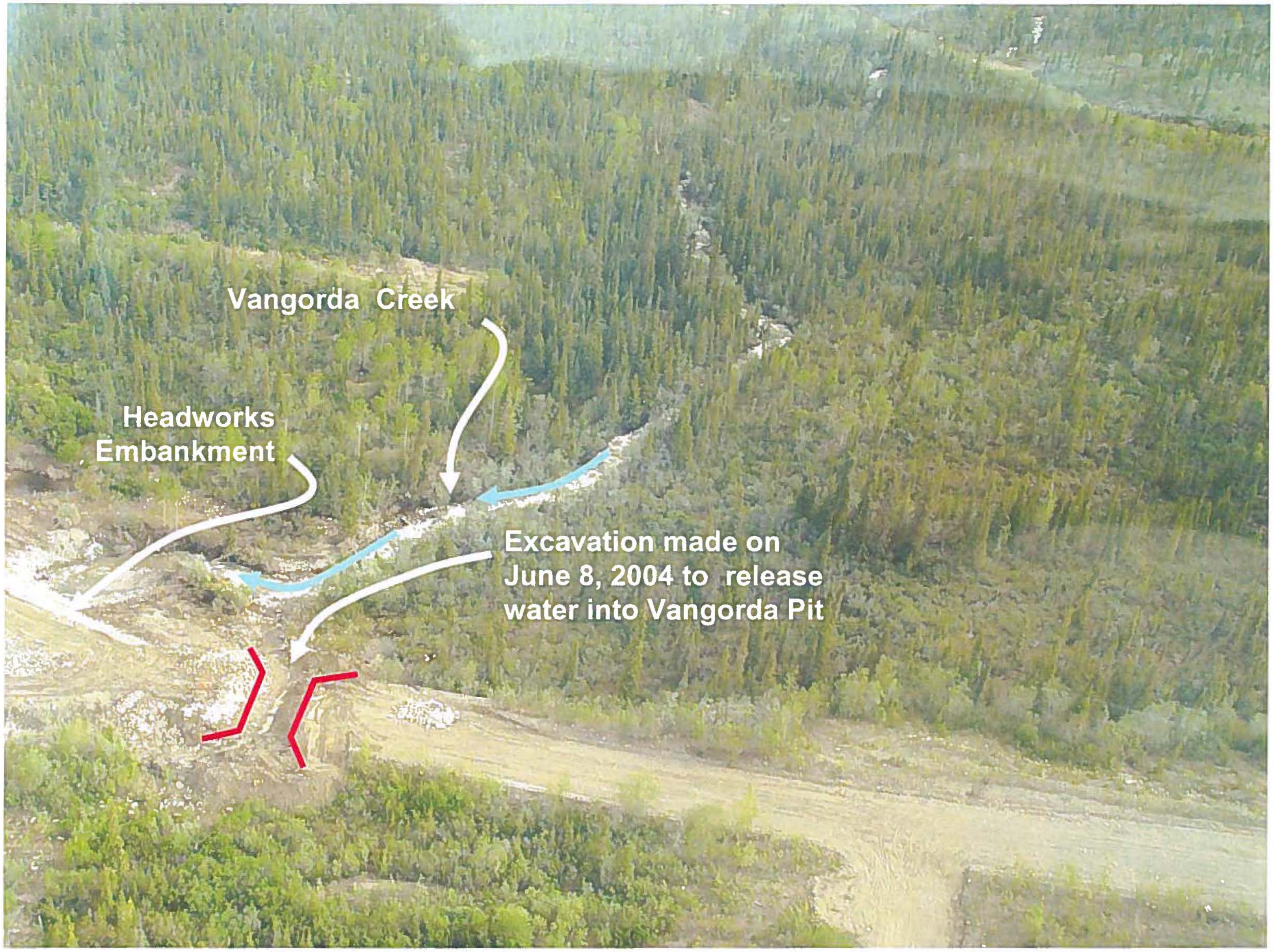
Headworks Embankment View 1
June 9, 2004

PROJECT:	DATE:	APPROVED:	FIGURE:
1CD003.56	June 2004		4



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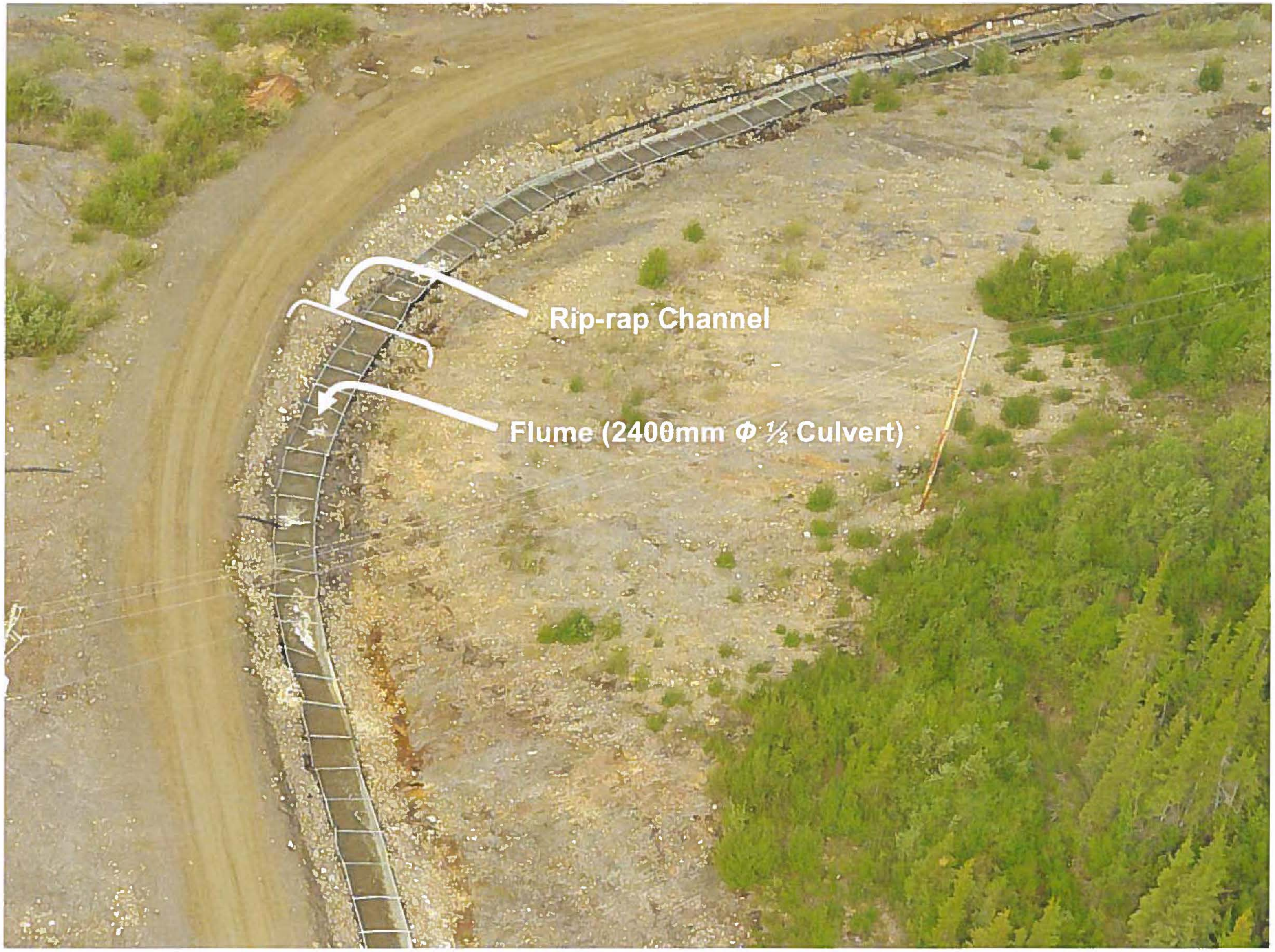
Inspection Report

Headworks Embankment View 2
June 9, 2004

PROJECT:	DATE:	APPROVED:	FIGURE:
1CD003.56	June 2004		5



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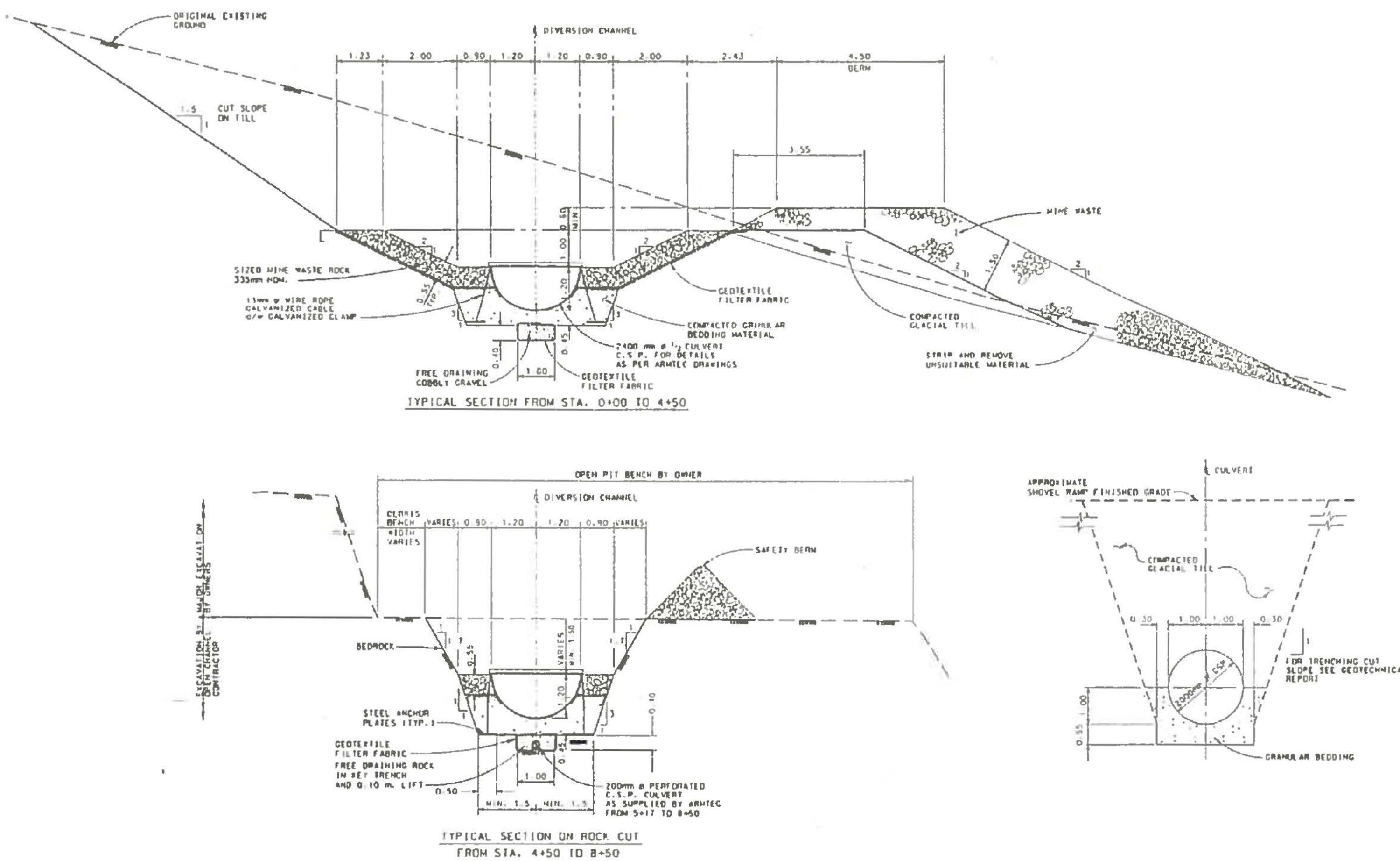
Inspection Report

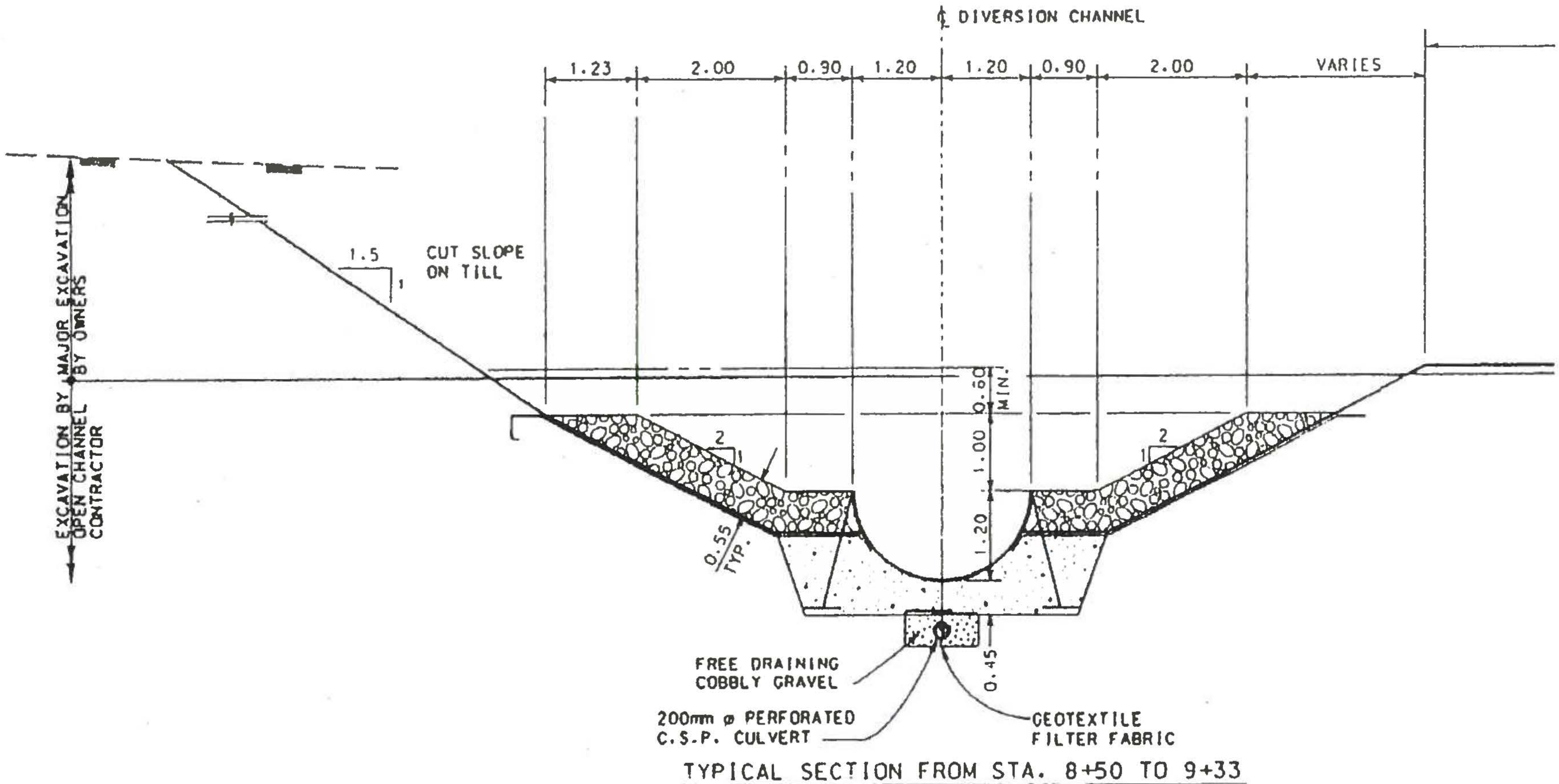
Flume and Rip-rap Channel
June 9, 2004

PROJECT:	DATE:	APPROVED:	FIGURE:
1CD003.56	June 2004		6



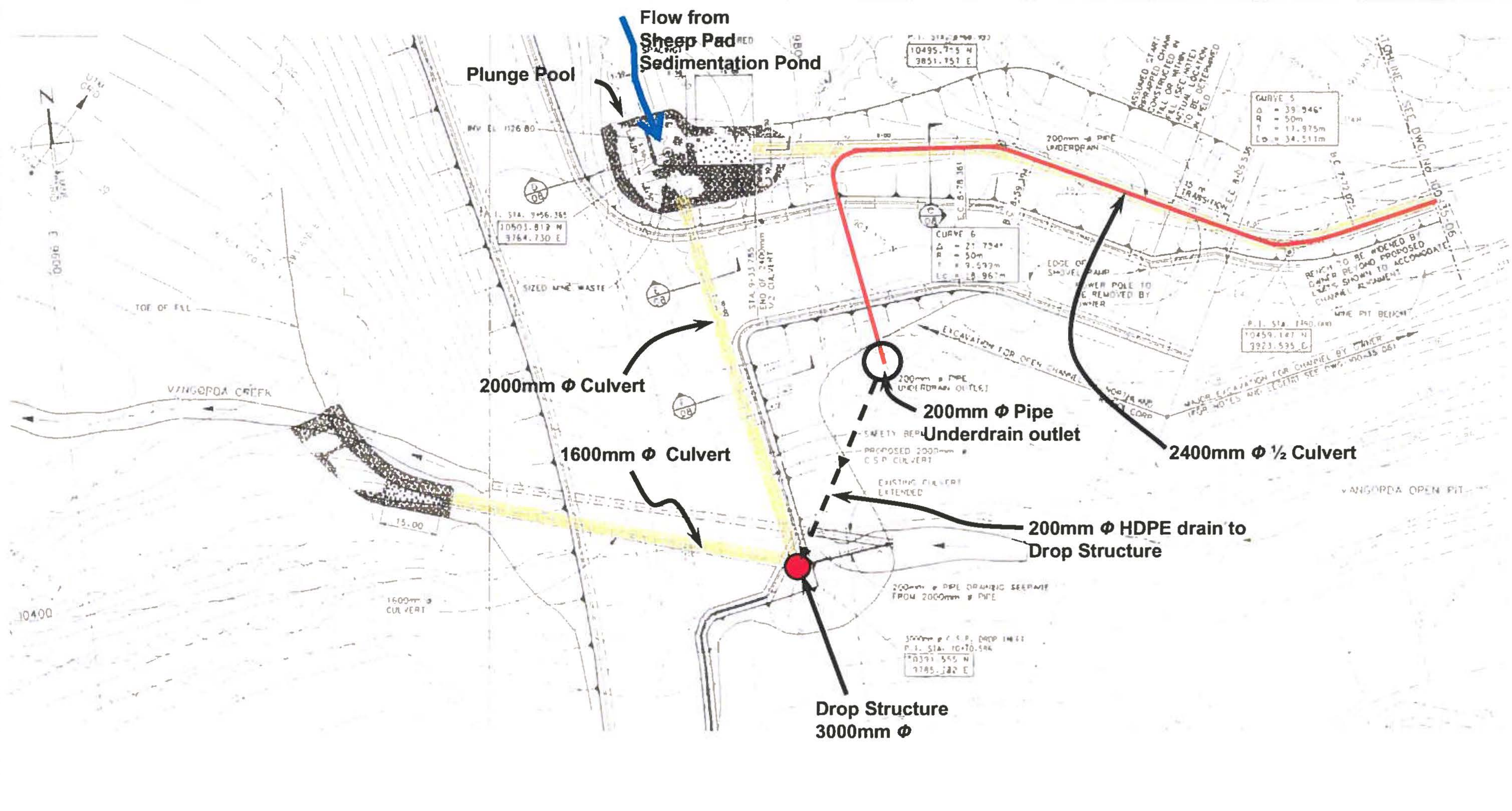
Dwg. Ref.: Kilborn As-builts, 1990

SRK Consulting Engineers and Scientists	Vangorda Creek Diversion Facility Inspection Report			
Deloitte & Touche	**Typical Sections through Diversion Channel**			
	PROJECT: 1CD003.56	DATE: June 2004	APPROVED:	FIGURE: 7



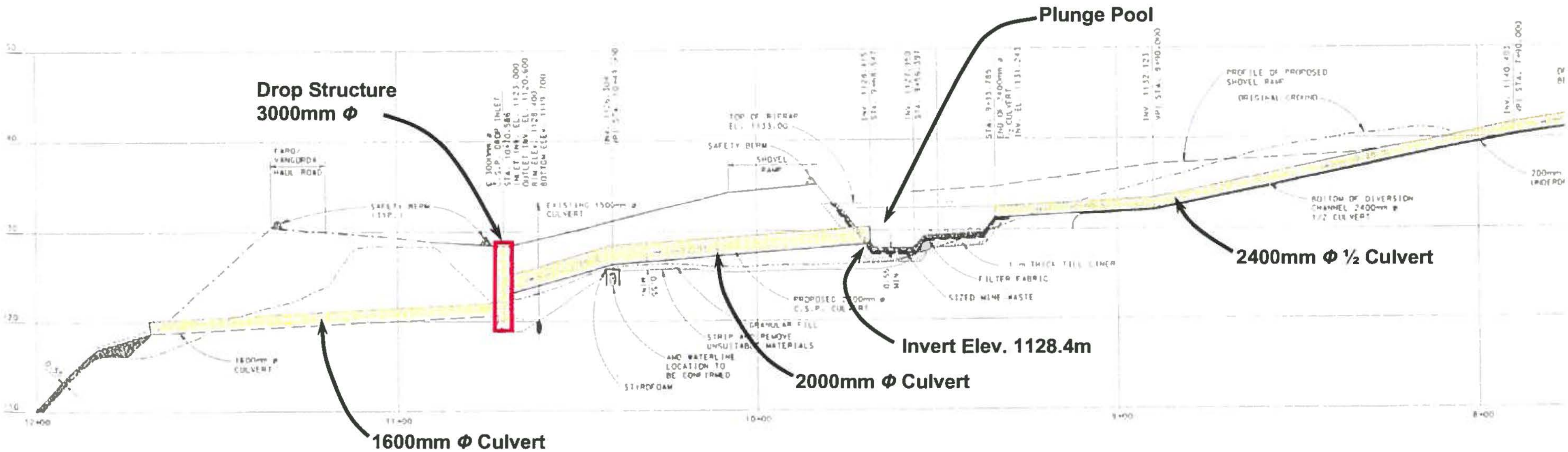
Dwg. Ref.: Kilborn As-builts, 1990

SRK Consulting Engineers and Scientists	Vangorda Creek Diversion Facility Inspection Report			
Deloitte & Touche	**Typical Section through Diversion Channel – Sta 8+50 to 9+33**			
	PROJECT: 1CD003.56	DATE: June 2004	APPROVED:	FIGURE: 8



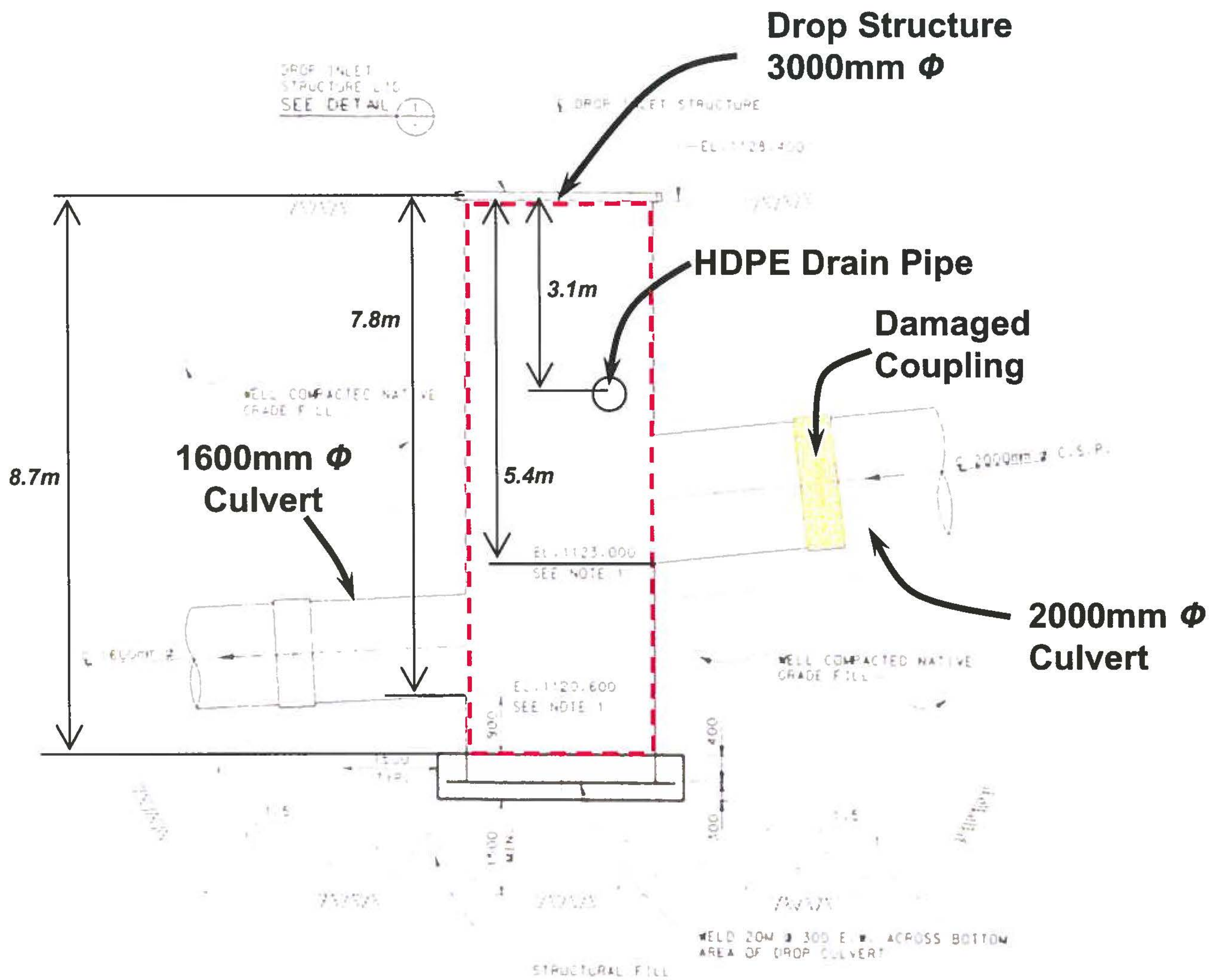
Dwg. Ref.: Kilborn As-builts, 1990

SRK Consulting Engineers and Scientists	Vangorda Creek Diversion Facility Inspection Report			
Deloitte & Touche	**Drop Structure and Plunge Pool - Plan**			
	PROJECT: 1CD003.56	DATE: June 2004	APPROVED:	FIGURE: 9



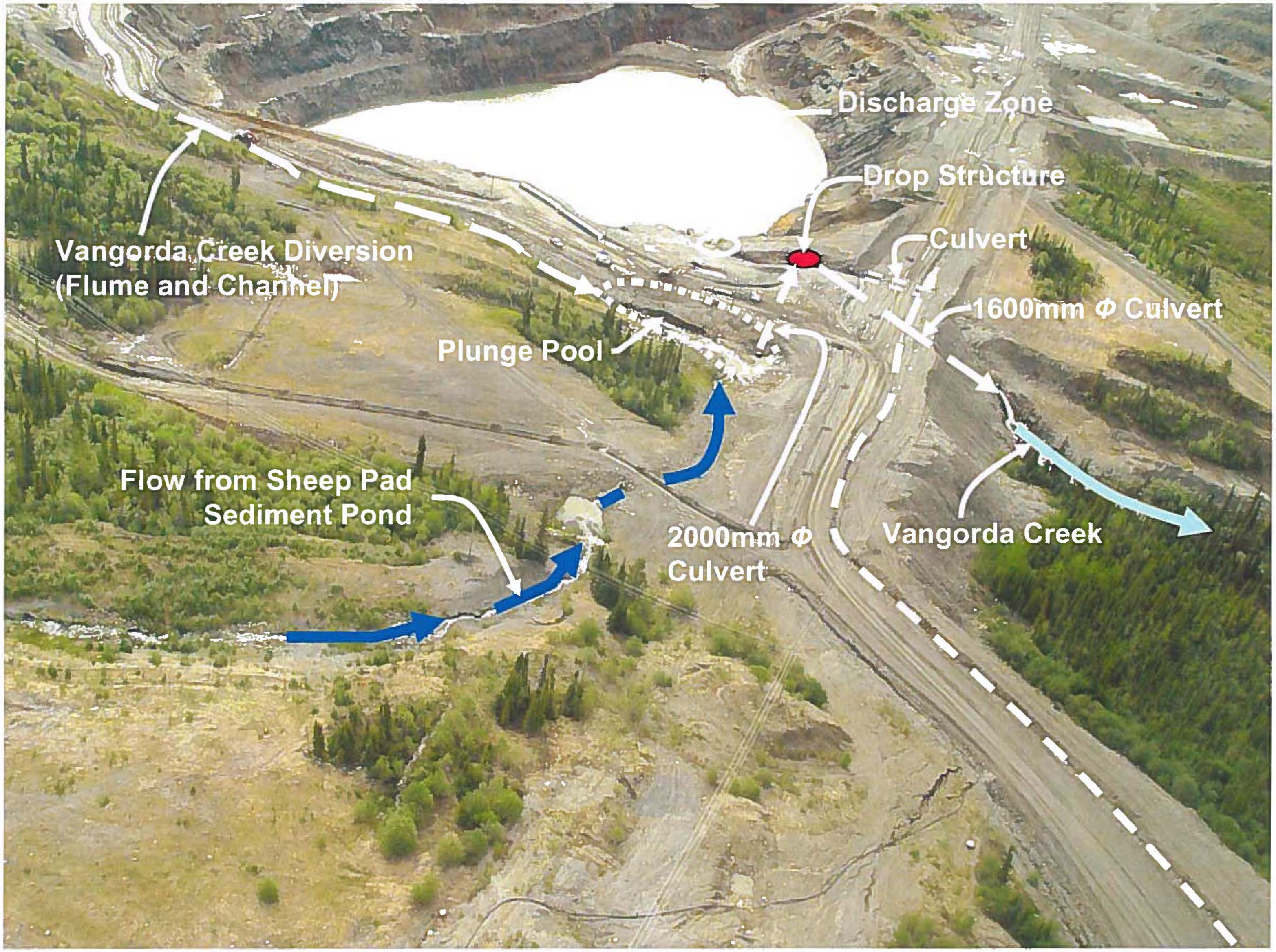
Dwg. Ref.: Kilborn As-builts, 1990

SRK Consulting Engineers and Scientists / Deloitte & Touche	Vangorda Creek Diversion Inspection Report			
	Drop Structure and Plunge Pool - Profile			
	PROJECT 1CD003.56	DATE June 2004	APPROVED	FIGURE 10



Dwg. Ref.: Kilborn As-builts, 1990

SRK Consulting Engineers and Scientists / Deloitte & Touche	Vangorda Creek Diversion Facility Inspection Report			
	Drop Structure			
	PROJECT: 1CD003.56	DATE: June 2004	APPROVED:	FIGURE: 11



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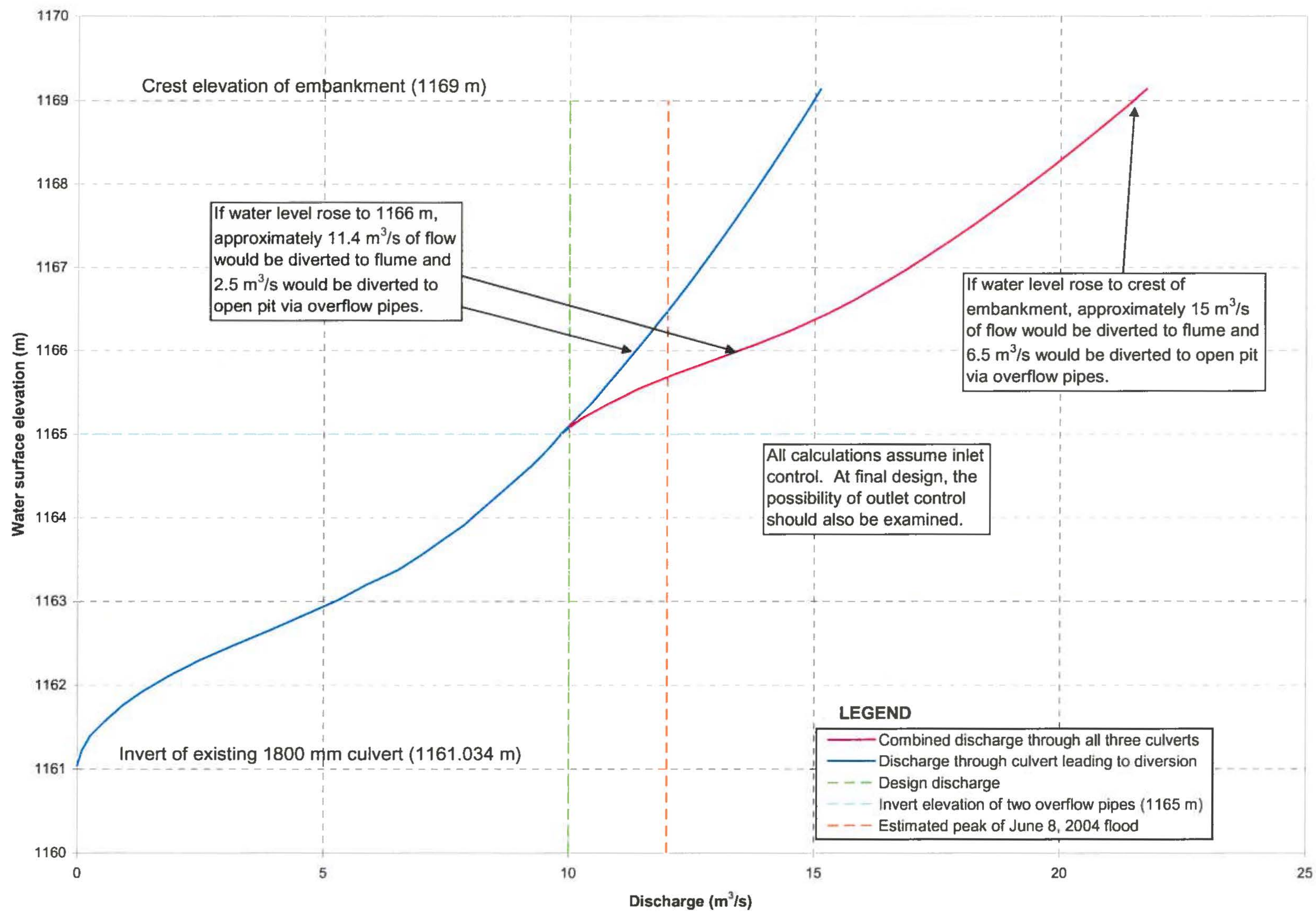
Vangorda Creek Diversion Facility
Inspection Report

Plunge Pool and Drop Structure

PROJECT:	DATE:	APPROVED:	FIGURE:
1CD003.56	June 2004		12



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Vangorda Creek Diversion Facility
Inspection Report

Estimated Discharge Capacity at Headworks with the Installation of 2x1000mm Φ CSP Overflow Pipes

PROJECT:	DATE:	APPROVED:	FIGURE:
1CD003.56	June 2004		13

Photo 1: Drop structure, discharge zone and Plunge Pool.

Photo 2: Drop structure

Photo 3: Outfall from 1600mm Ø culvert. (June 8, 2004)

Photo 4: Inlet to 2000mm Ø culvert at Plunge Pool.

Photo 5: Outlet of 1500mm Ø culvert beneath Headworks Embankment.

Photo 6: Transition from 1500mm Ø culvert to 2400mm ½ culvert.


Photo 7: Plunge Pool (June 8, 2004).


Photo 8: Flume sections at entrance to Plunge Pool (June 8, 2004).

Photo 9: Plunge Pool (June 8, 2004).

Photo 10: Lower reaches of flume and channel. (June 8, 2004)

Photo 11: New flume section with old flume still in place (June 8, 2004).

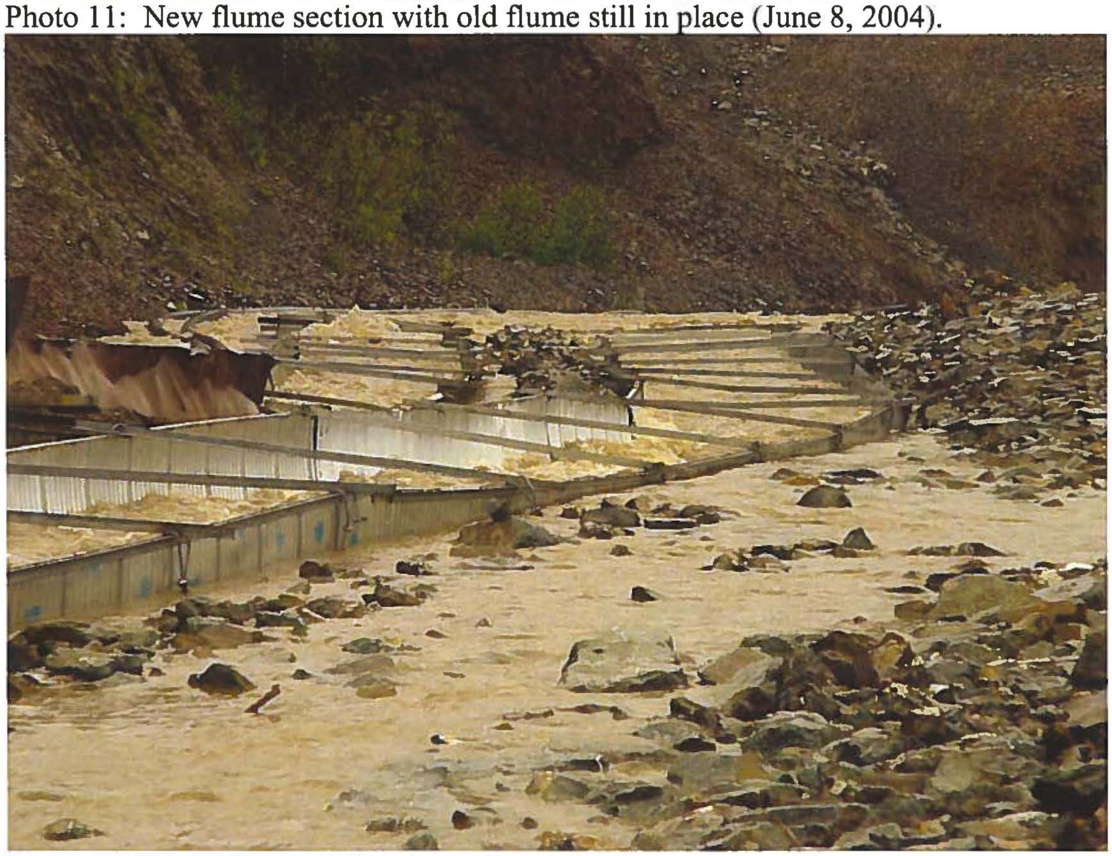


Photo 12: New flume section (June 8, 2004).

Photo 13: Backhoe restraining flume section from lifting (June 8, 2004).

Photo 14: Upper reaches of flume and channel (June 8, 2004).

Photo 15: Ponded water upstream of Headworks embankment (June 8, 2004).

Photo 16: Discharge zone below drop structure.

Photo 17: Drop structure.

Photo 18: Discharge channel below drop structure.

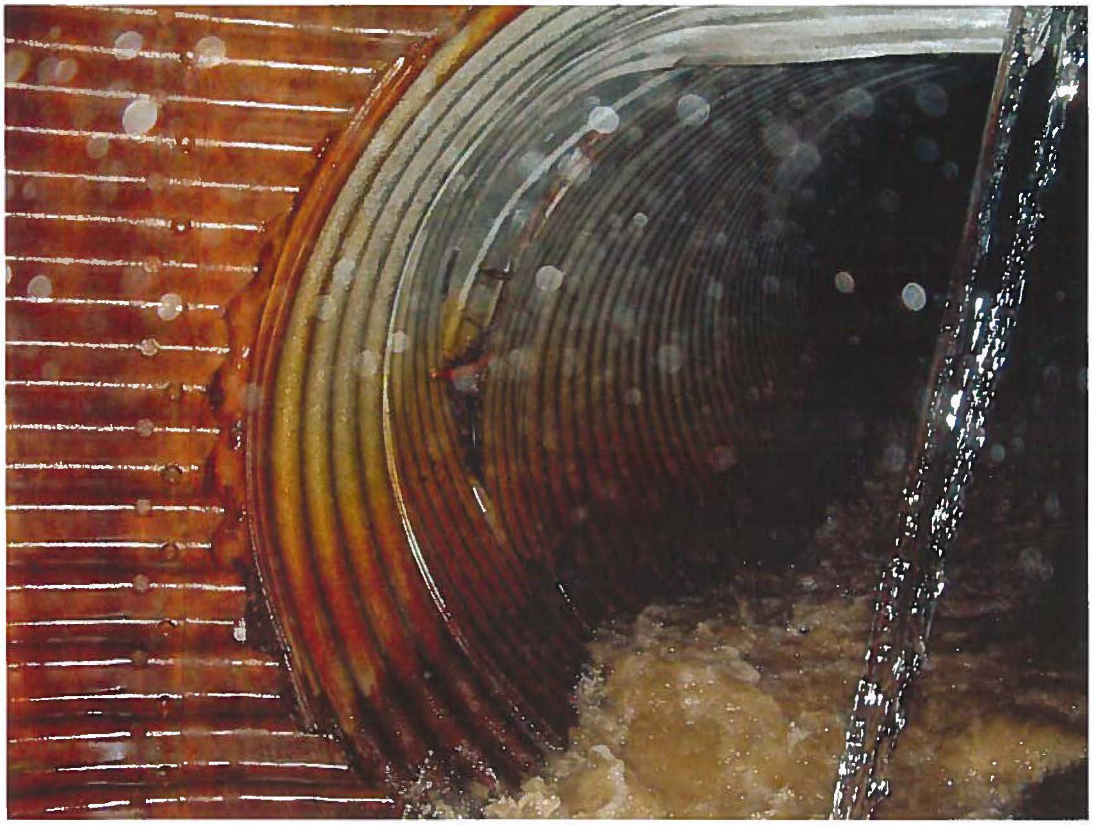


Photo 19: View of joint separation in 2000mm Ø culvert taken from inside of 3000mm Ø drop structure

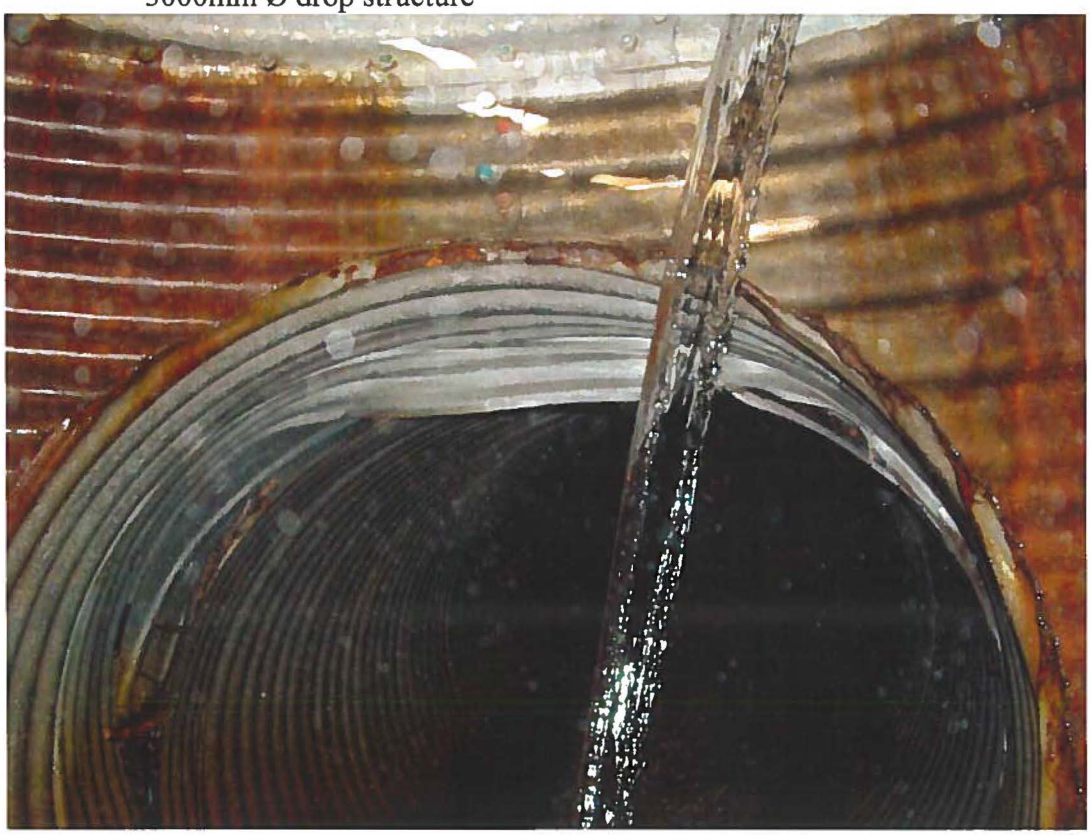


Photo 20: View of buckling in the crown of the 2000mm nipple.

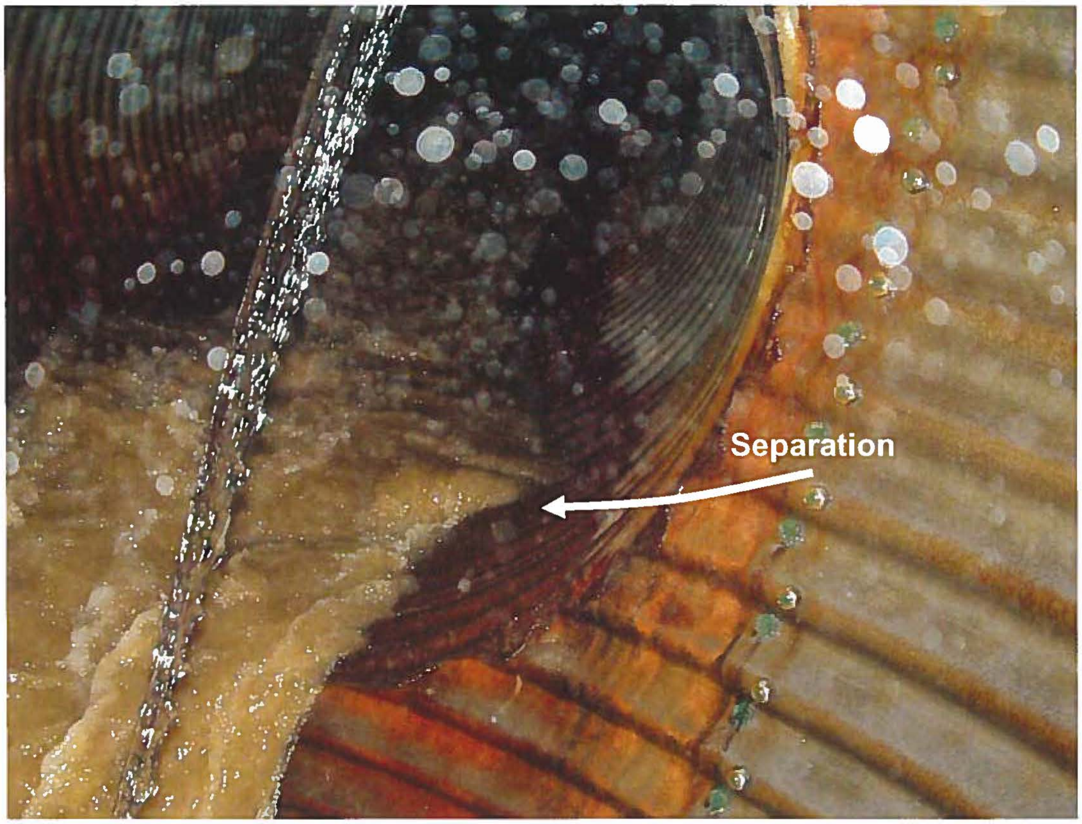


Photo 21: View of joint separations in the 2000mm Ø culvert.

Photo 22: Exposed crown of separated joint in 2000mm Ø culvert.

Photo 23: Coupling at 2000mm Ø joint with nipple.

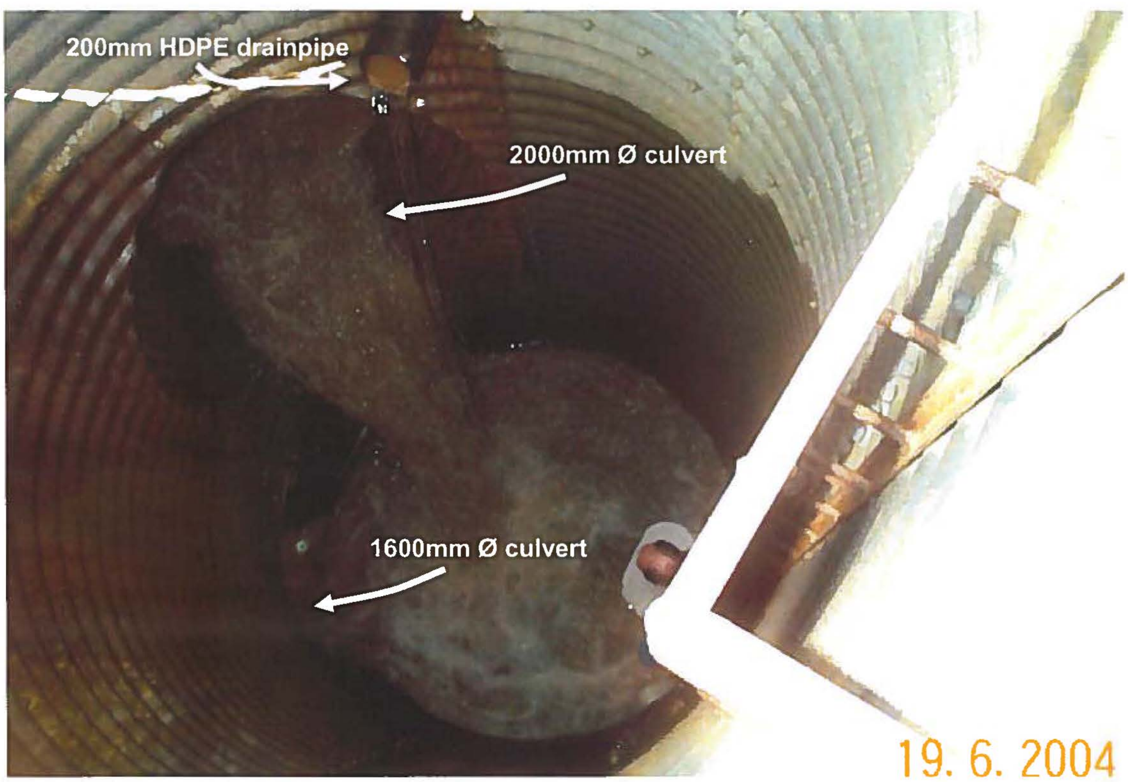


Photo 24: View of drop structure

Photo 25: Excavated channel on south abutment of headworks embankment.

Photo 26: Excavated channel on south abutment of headworks embankment.

Photo 27: Trash rack at inlet to 1800mm Ø culvert at headworks embankment.

Photo 28: Plunge Pool at bottom of flume.

Photo 29: Lower sections of flume and channel.

Photo 30: Transition between lower and middle reaches of flume and channel.

Photo 31: Middle section of flume and channel.

Photo 32: Section of flume that experienced most severe lifting during flood.

Photo 33. Connection between original and recently installed flume sections.

Photo 34: View upstream of recently installed flume section. Original flume section shown to the left of new flume.

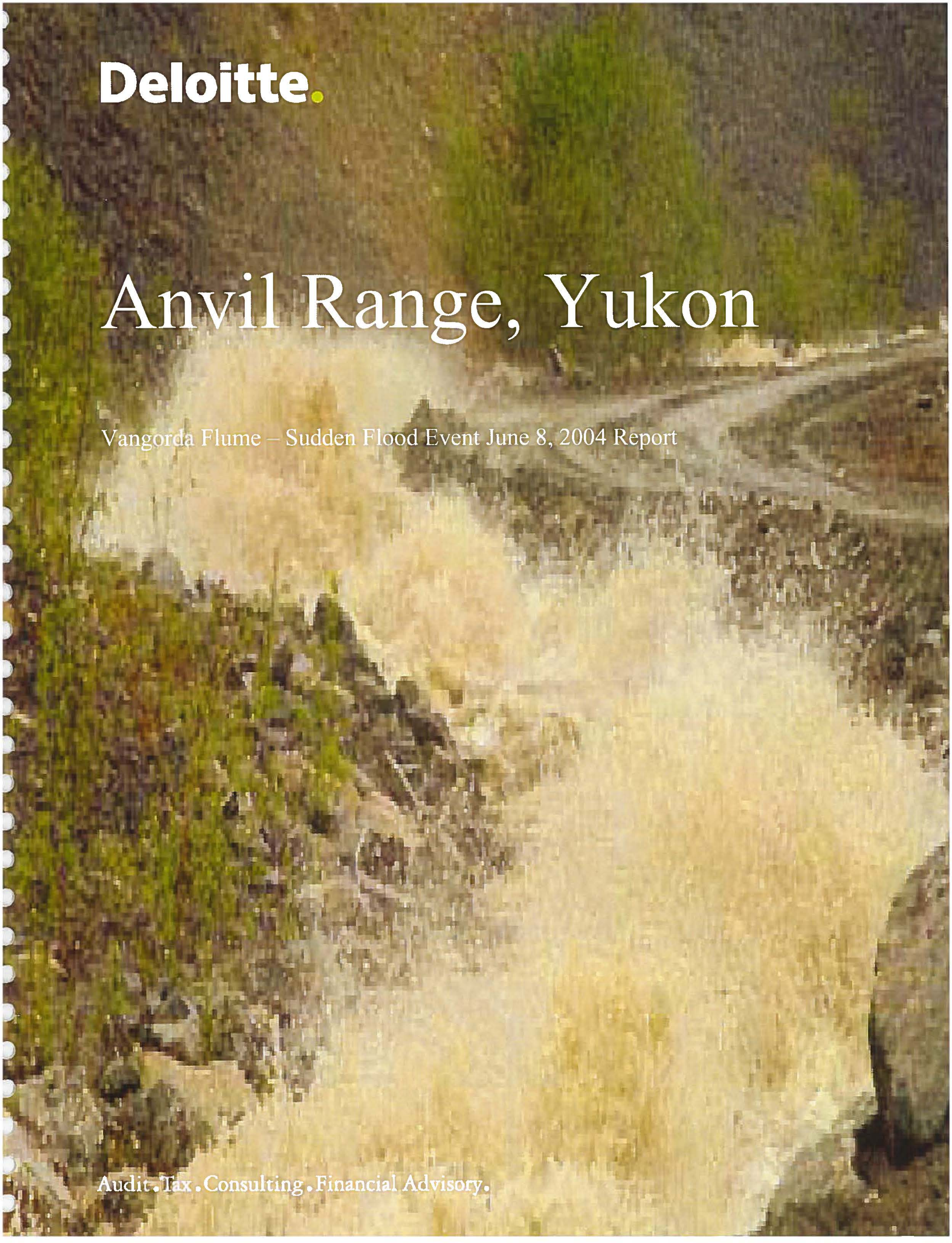
Photo 35: New flume section.

Photo 36: Upper reaches of original flume and channel

Photo 37: Drop structure during flood event. (June 8, 2004)

Photo 38: Outlet of 1500mm Ø culvert beneath Headworks Embankment (June 8/04)

Photo 39: Above Vangorda Headworks (June 8, 2004)



Deloitte.

Anvil Range, Yukon

Vangorda Flume – Sudden Flood Event June 8, 2004 Report

Audit . Tax . Consulting . Financial Advisory.

Table of Contents

A. Background

The Vangorda flume was constructed as a bypass for the Vangorda Creek around the Vangorda pit. The flume infrastructure is composed of:

- A headworks with a main culvert through the base of a small dam approximately 8 metres high at the centre, which also functions as a vehicle road across Vangorda Creek. The dam also contains a small overflow culvert approximately a ½ metre from the crest of the road.
- Half moon flume sections, which are contained in a ditch system armoured with rip rap adjacent to the access road. The flume section is approximately 900 metres in length.
- A plunge pool at the end of the flume section connected to a drop box system that transports the water under the Vangorda Haul Road and the water is discharged into a rip rap armoured receiving channel, which in turn is connected to the original Vangorda Creek downstream.

B. Observations

- Doug Sedgwick of Deloitte & Touche Inc. was attending at the mine site the week of June 7th and was present during the event and has prepared this report having observed the actual conditions.
- It is noted that over 100 photographs were taken relating to the storm event.
- On the evening of June 7th, 2004, the routine inspection of the flume at approximately 7:30 pm indicated that the water was running through the flume at approximately two-thirds full and there was no water buildup behind the headworks dam. Conditions were normal for the time of year.
- The routine morning inspection at approximately 5:45 am on June 8th, 2004, following heavy rains in the area during the night, observed major water overflow of the flume, but contained within the flume ditch system. The bottom sections of the flume had disconnected and were partially relocated in the plunge pool due to the force of water. It was also observed that at approximately half way along the flume system, sections were being lifted by water running under the flume and in danger of being destroyed. There were also large rapids created by the flume crossbar supports and debris in the channel. Small tree trunks washed down the Vangorda Creek through the culvert and into the flume were also observed. The water above the headworks was beginning to pond behind the culvert/dam headworks. Vangorda Creek above the headworks had major water flow and a gauge was placed to measure the rate of rise of the water behind the headworks.
- Between 7:00 am and 8:00 am the water began rising rapidly behind the headworks culvert/dam system. The culvert was at capacity flow with no restrictions or debris observed that would reduce the capacity of the culvert.
- Between 8:00 am and 10:30 am the water continued to rise behind the headworks dam at a rate of approximately 5 cm per minute and by approximately 10:30 am the water was 500 mm below the crest of the road and was within 3 cm of entering into the small overflow culvert. At that time, a side channel had been constructed, armoured with rip rap and opened to allow a portion of the water to flow into the old Vangorda Creek channel and into the Vangorda pit. This action prevented the water from overtopping the road and dam structure. No seeps were observed during this event on the downside face of the dam.
- The water running through the flume ditch was observed to be within 30 cm of overtopping the road running alongside the flume system.
- The drop box was inspected during the event and a significant amount of water (approximately 30%) was flowing from around the drop box into the Vangorda pit as the drop box capacity had been exceeded.
- The overflow from the culvert below the Haul Road into the Vangorda Creek was very strong and no restrictions were observed.

C. Actions Taken

- Immediately following the observation at 5:45 am of the flume condition, Anvil Site Supervision arranged to have the backhoe, which was in the area, walked up to the midpoint of the flume where the flume sections were lifting and the boom was extended out and the bucket was utilized to hold down the flume and prevent it from being destroyed by the fast moving water.
- At 8:00 am following communication to Dana Haggar of the flume issue and the rapid rise of the water behind the headworks, Mr. Haggar drove to the Vangorda flume site arriving at approximately 8:20 am and immediately arranged for the D9 Cat to be walked up to the headworks and for the Link Belt excavator to be trucked to the site from the Faro side and also walked up to the headworks. The loader was also brought to the headworks to enable rip rap, which was stored nearby (for emergency purposes) to be utilized to armour an overflow channel that was being excavated by the above equipment. This would allow water to flow out of the pond created behind the headworks.
- Between 9:15 am and 10:30 am the overflow channel was constructed approximately 1.5 metres deep utilizing the D9 Cat and the link belt excavator with the channel being armoured with rip rap. As soon as the channel was opened at approximately 10:30 am the water level began lowering in the pond.
- By 12:00 noon the buildup of water behind the headworks was eliminated, but the flume capacity was still exceeded and the ditch was running almost full.
- The drop box still had some water overflow into the pit.
- It was estimated that the water originating from the constructed channel ran into the pit for approximately 2 hours with continuing minor seepage for the next 24 hours. It was also estimated that the excess water from the drop box ran into the pit for approximately four to five hours at a decreasing rate. The pit water level went up approximately 0.7 metres during the 24 hour period between 10:00 am on June 8, 2004 and 10:00 am the following day.
- The constructed armoured channel did not erode during the event.
- Regulatory authorities were contacted per the Environment Management Emergency Response Plan documentation. In addition, it was noted that the construction of an overflow channel at the headworks corresponded with one of the alternatives in the Emergency Response Plan.
- SRK was contacted to carry out an analysis of the flume system hydraulic capacity in an attempt to understand the size of the event. It was determined that the headworks culvert had a capacity equivalent to a 100 year flood if the water behind the dam was at a level 1 metre below the crest of the road. In fact, the water was at a level approximately 0.5 metres below the crest of the road, indicating the event may have exceeded the 100 year flood. The water was observed flowing at approximately 10 cm below the top of the flume ditch at one location and it was noted by SRK that the flume ditch (not the flume proper) was designed for a 100 year event.

D. Conclusion

The actions taken by Site Management were appropriate and carried out in a timely manner, which averted a much more serious event. Regulatory authorities from the Yukon Type II Office and YTG Water Resources were present at the site within 48 hours of the event and concurred with the actions taken. The flume system was monitored for the next 24 hours on a full time basis with no further events noted.