CDA ACB

CANADIAN DAM ASSOCIATION
ASSOCIATION CANADIENNE DES BARRAGES

CDA 2009 Annual Conference
Congrès annuel 2009 de l'ACB

Whistler, BC, Canada
2009 Oct 3-8

PERFORMANCE OF A ROCK DRAIN AFTER 20 YEARS OF SERVICE

Gerry Ferris, M.Sc., P.Eng., BGC Engineering Inc., Calgary, Alberta, Canada

ABSTRACT:

Rock drains have been used by the mining industry in Canada since the early 1980's, and they now form part of the engineer's standard 'tool box' for the design of waste-rock dumps. Although there remain some questions concerning the long term performance of rock drains (especially for the closure period), the design methods employed for rock drains are well established and are generally accepted by the industry and regulators.

A rock drain typically consists of a segment of a waste-rock dump that is constructed of large and clean waste rock. The purpose of this segment of the dump is to transmit water, typically from an existing drainage course that will be covered by the waste rock dump, through the dump itself. The majority of rock drains reported in the literature are built near headwaters of valleys and are long structures (1 km or more). These rock drains are typically designed to have the capacity to discharge a 100 to 200 year return-period event, or flows upto about 30 m^3/s. Measured flows are typically 2 m^3/sec and less.

The present paper provides performance details on a rock drain constructed in 1987, a brief background on analysis methods used to predict flow through capacity and presents details from monitoring of the North Fork Rose Creek Rock Drain since 2005. During this period detailed records were kept, and the maximum measured flow through the structure was 22 m^3/s.

RÉSUMÉ:

1.0 INTRODUCTION

Rock drains have been used by the mining industry in Canada since the early 1980's, and they now form part of the engineer's standard 'tool box' for the design of waste-rock dumps. A rock drain typically consists of the lower portion of a valley filling waste rock dump, and consists of the large, clean rock at the base of the dump. The rock drain part of the dump is needed when the dump covers an existing drainage course, such as a small stream or creek. The purpose of the rock drain portion of the dump is to transmit water through the dump. This removes the requirement to install diversion ditches around the dump or culverts beneath.

The majority of rock drains reported in the literature are built near the headwaters of valleys and are long structures (typically 1 km or more) (Claridge *et al.* 1986 and BCMWRPRC 1999). This type of rock drain is formed in the lower portion of a valley filling dump. Typically these rock drains are designed to have the capacity to handle a 100 to 200 year return-period event, or flows of about 20 to 30 m^3/sec (in the relatively small drainage areas). Typical measured flows through these rock drain structures are 2 m^3/sec and less. Although less common in the literature, rock drains are also constructed at the base of haul roads and replace a culvert that would have been used to transmit stream flow under the road. This type of rock drain has somewhat different characteristics than the valley fill type, and more closely resembles a rock fill dam without a water retention element. The present paper provides information about a rock drain constructed in 1987 of this second "haul road" type. It has handled a measured flow through of 22 m^3/s in 2005. It was built as part of the construction of a haul road to the Vangorda and Grum pits at the Faro Mine Complex, Faro, Yukon, Figure 1.

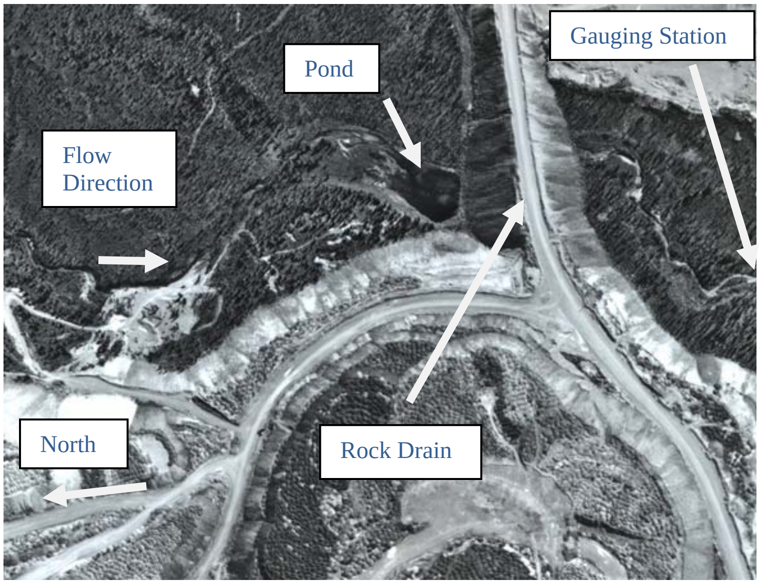

Figure 1 Aerial View of the North Fork of Rose Creek Rock Drain

2.0 BACKGROUND

The systematic study of the flow-through capacity of rockfill and similar coarse porous media began in the 1950's and 1960's at the beginning of the "modern" period of the construction of rockfill dams (earth core or concrete faced). The study of the flow-through capacity of rockfill was in this early period a subset of the study of the stability of cofferdams being overtopped by floods or rockfill dams being subject to flow during construction (Leps 1973, ICOLD 1993).

Between 1934 and 1943, seven earth and rock dams were constructed in Mexico that would allow water to flow-through and/or be overtopped during construction (Leps 1973). This was accomplished by protecting the downstream slope of the rockfill with a grid of steel bars with tiebacks anchored in the rockfill. Of the seven dams protected in this manner, three were overtopped during construction. The depth of water overtopping the rockfill was between 1.8 and 4 meters and the reinforcing measures worked as planned. These results and development of flow through capacity calculations (Wilkins 1956) and reinforcing recommendations (Wilkins 1963) resulted in designers opting to reinforce cofferdams (and the downstream toes of the main dam rockfill section) and allowing them to be overtopped if a large flood event occurred during construction. An early example of this is the Laughing Jack Dam in Tasmania, Australia, constructed in 1957. Laughing Jack Dam did not have a conventional spillway or low-level outlet; the only discharge of water from the reservoir was through the unique ‘in-built spillway’, (shown as Figure 2) whereby all the water discharged from the reservoir flows through the rockfill.

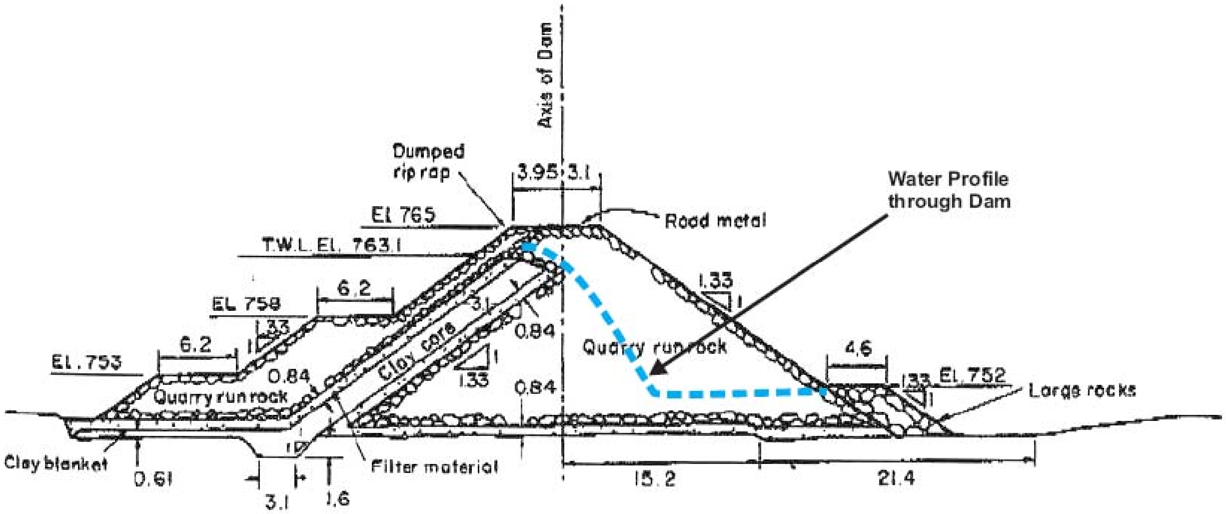

Figure 2 Laughing Jack Dam (Wilkins 1956)

Although these concepts were already in use in Mexico and Australia prior to Wilkins’ work, the theoretical basis provided by Wilkins and extended by Leps (1973) resulted in the spread of the practice. A summary of reinforcing practices for rockfill (ICOLD 1993) revealed 47 case histories. In Canada, these concepts were used in the design of the 61 m high Revelstoke cofferdam (Salmon 2005).

The formal use of rockfill drains by the mining industry in North America did not begin until the early 1980’s (Claridge *et al.*1986). The development of the rockfill drain was related to the placement of waste rock dumps within mountainous terrain. Prior to the use of a flow-through rockfill drain, mining companies would have to create water diversions, place culverts or haul the waste rock to other areas. Based on the work on dumped rockfill dams, observations of accidental flow through and the natural gravity segregation of the dumped waste rock, the idea developed of allowing the rockfill itself to act as a drain. By the late 1990’s, between 20 and 30 rockfill drains had been constructed in Canada (BCMWRPRC 1999) and many hundred “valley fills” where used in the USA (Shank 2003). The practice of using flow through rockfill is now a fairly standard in the mining industries of Canada, USA and Australia.

3.0 THEORY

The method used to calculate the flow through capacity of rockfill, in the mining industry, is almost exclusively the Wilkins equation (Wilkins 1956). A comparison with various two dimensional seepage analyses and other one-dimensional simplifications indicates that this equation is valid over a large range of particle sizes and circumstances (BCMWRPRC 1999). Other equations have been shown to produce a better match to model test data for highly controlled material sizes and densities (Hansen *et al.* 1995). Engineers working on full scale structures have concluded that Wilkins equation produces acceptable results; other relationships are less used in practice (Leps 1973, Beckstead *et al.* 2000). The Wilkins equation, applicable to the nearly fully-developed regime of turbulent flow, may be stated as:

$$Q = n\,A\,W\,m^{0.5}\,i^{0.54} \qquad (1)$$

where:

Q = flow (m^3/s),

n = porosity (dimensionless),

A = cross-section area through which water flows (m^2),

W = Wilkins' empirical constant, 5.243,

m = hydraulic mean radius (m),

i = hydraulic gradient (dimensionless).

The hydraulic mean radius can be calculated according to equation 2 (Hansen *et al.* 1995):

$$m = e\,d / 6\,r_e \qquad (2)$$

where:

e = void ratio (dimensionless),

d = "dominant" particle diameter, (m)

r_e = particle surface-area-efficiency ratio, typically about 1.3 for coarse angular rock (dimensionless)

The "dominant" particle diameter in a granular material is usually thought to be controlled by the smaller sizes, such as the D_{10} (EBL 2005). For rockfill, which is generally devoid of fines, the D_{50} size is generally considered to be the dominant particle diameter (Leps 1973). This pragmatic decision was reached due to the impracticality of estimating the gradation of rockfill *post facto*, and consideration of the effects of segregation of the rock, the likelihood of the washing out of some fines over time and the relative insensitivity of the final result to small variations in rock size.

The Wilkins equation relates the one-dimensional hydraulic gradient to the amount of flow. Flow through rockfill is actually a two dimensional problem (if not 3-D). The actual gradient in a flowing situation also varies with position within the structure (Figure 3). Therefore, the concept of an "effective" or representative hydraulic gradient has been developed (Hansen *et al.* 1995). Calculation of the effective hydraulic gradient can be found using the definitions shown in Figure 3 along with Equations 3 and 4 as follows:

$$A_R = 1/H\,(B_u + B_c + B_d/2) \qquad (3)$$

where:

A_R = aspect ratio (dimensionless),
B_u = width of upstream portion (m),
B_c = width of the central portion (m),
B_d = width of the downstream portion (m).

$$i_{eff} = 0.8\ A_R^{-3/2}\ (h/H)^{1.4} \qquad (4)$$

where:

i_{eff} = effective hydraulic gradient
h = upstream water level (m),
H = height of the dam (m).

For the large and long (up to 2.5 km long) structures extending under valley fills this two dimension effect can be ignored and the hydraulic gradient may be assumed to be equal to the slope of the original creek bed (BCMWRPRC 1999). For the haul-road type of rock drain, this gradient through the structure is thought to dominate.

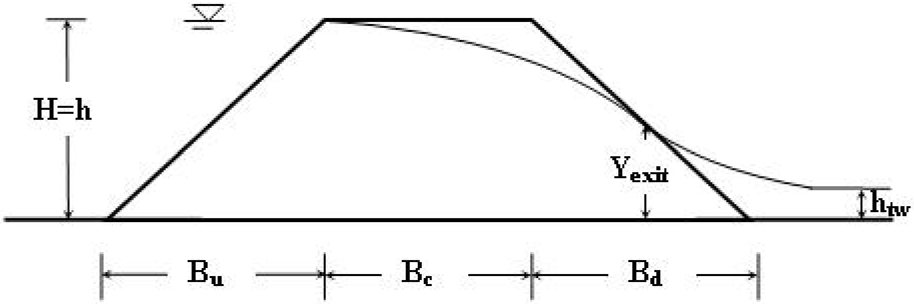

Figure 3 Definition of terms to determine gradient within a Rock Drain

4.0 DESCRIPTION OF ROCK DRAIN

A 12 km long haul road was constructed from the Faro Mill area to the Vangorda and Grum deposits at the Faro Mine complex in 1987-88. Its construction necessitated crossing numerous small creeks and three larger water courses. In two instances, when the depth of fill was greater than about 20 m, a rock drain was used rather than placing culverts in the creeks. The largest of these, located across the North Fork of Rose Creek (Figure 1), is described below.

Most of the haul road was constructed using the available schistose and phillite waste rock. These materials were placed via end-dumping of the rockfill from the final road crest elevation. In general, this process results in the segregation of the rock, with large relatively clean rockfill being found at the base of the dump face. When the dump face is greater than about 15 to 20 m, a very clean and coarse fill results at the base of the dump (Nichols 1986). At the North Fork Rose Creek crossing, a hard calc-silicate rock was used rather than the schistose to form the drain portion of the haul road. It was specified to extend the calc-silicate 35 m on either side of the creek centreline, or that a 70 m wide zone of stronger rock should be used to form the drain portion of the haul road. A view of the upstream face of the rock drain portion of the haul road is shown in Figure 4. This photograph shows the pond that develops on the upstream side of all rock drains; this pond varies in size according to the amount of flow that is passing through the structure.

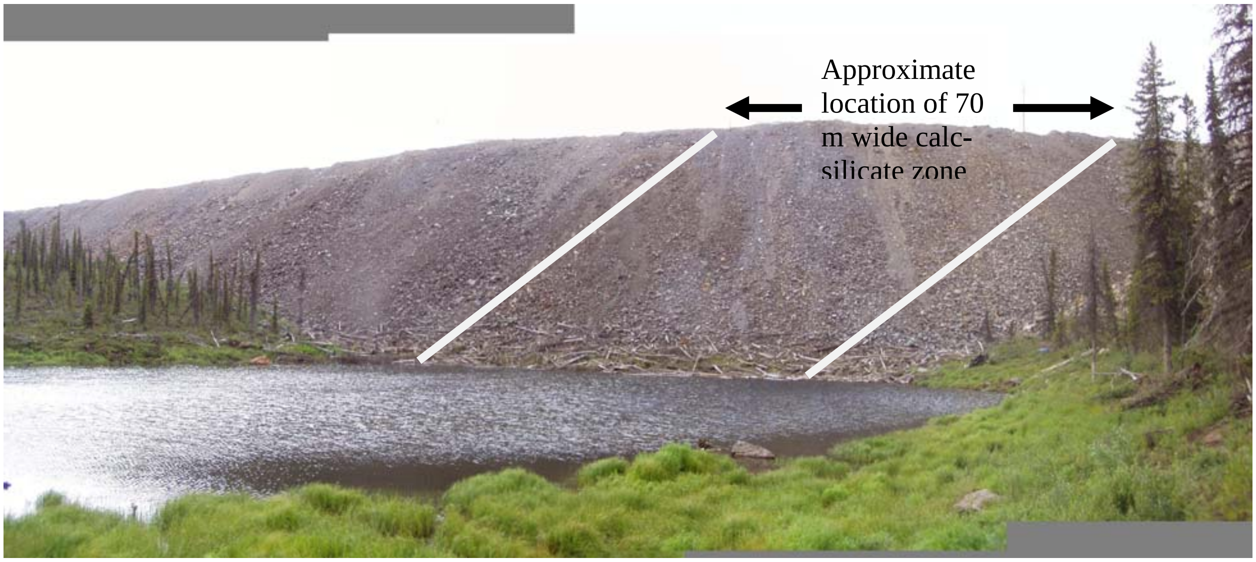

Figure 4 View of upstream pond and face of rock drain

The haul road was constructed to have a generally trapezoidal shape. At the location of the North Fork of Rose Creek the dimensions of the rock drain are given in Table 1.

Table 1 Dimension of Drain

Drain Dimension*	Value
B_u	85.3 m
B_c	30.0 m
B_d	95.3 m
H	54

*As defined in Figure 3

During construction, one site visit was conducted by the design engineer (Golder 1987). An inspection of the rock drain portion of the haul road was undertaken, during placement of the calc-silicate rock. A review of the grain size of the material, with reference to the height above the toe of the dump was made, as summarized in Table 2 (see also Figures 5 and 6). The lower 55 m of the dump face was noted as being ‘remarkably clean”, with “fines” only encountered in the upper 10 m of the dump face.

Table 2 Summary of observed particle sizes during construction

Location	Toe of dump	7 m above toe	10 m above toe	17 m above toe	55 m above toe
D_{max}*	2 m		0.8		
D_{50}**	1 m	0.3	0.4	0.15	0.2

*D_{max} is the maximum particle size

**D_{50} is the median particle size.

Figure 5 Rock 10m above toe (Golder 1987)

Figure 6 Rock 55 m above toe (Golder 1987)

5.0 PREDICTED FLOW CAPACITY

The flow through the structure was calculated according to the theory presented in Section 3.0. As part of using Wilkins equation to relate pond elevation to the flow passing through, a number of assumptions have to be made beyond those described previously. These primarily concerned the dimensions of the structure, bulk density and the porosity, the 'dominant' rock size, and the area through which water flows.

5.1 Dimensions and Flow through area

The dimensions were determined from a digital terrain model (DEM) with 2 m contours, based on 2003 air photographs. This information, along with assumed pond elevation, was used to estimate the effective hydraulic gradient in accordance with equations 3 and 4 to produce an estimate of the flow that would result from a particular pond elevation. The 2003 survey information was combined with information on the pre-haul road conditions to estimate the area available for flow. The pre-construction information was developed based on the 2003 DEM and a second DEM developed from 1973 aerial photographs. The 1973 DEM had 25 foot (7.62 m) contours; these two models were used to estimate the area available for flow passage.

5.2 Dominate Grain size and Density of dumped Rockfill

The hydraulic capacity of rockfill is controlled by three main material properties; the dominant grain size, the porosity, and the angularity. The angularity of the calc-silicate in question was considered to be similar to 'typical' blast rock (spherical on the Zingg diagram but angular), the schistose was more tabular (oblate). Limited information is available on the grain size and its variation through the structure. The typical pattern of segregation of dumped rockfill is however apparent, with the largest sizes at the base, becoming progressively finer-grained towards the crest. The conversion of grain size to hydraulic mean radius was performed using Equation 2.

No direct information was known about the porosity of the fill in the haul road. Estimates concerning the porosity for this rock drain were made based on typical porosity of waste dumps. Using various estimates, five different scenarios were used to predict the pond elevation versus resulting flow relationships for this rock drain. These were:

D_{50} = 0.25 m, n = 0.39
D_{50} = 0.25 m, n = 0.43
D_{50} = 0.25 m, n = 0.35
D_{50} = 0.35 m, n = 0.39
D_{50} = 0.15 m, n = 0.39

The results of this analysis are shown on Figure 7.

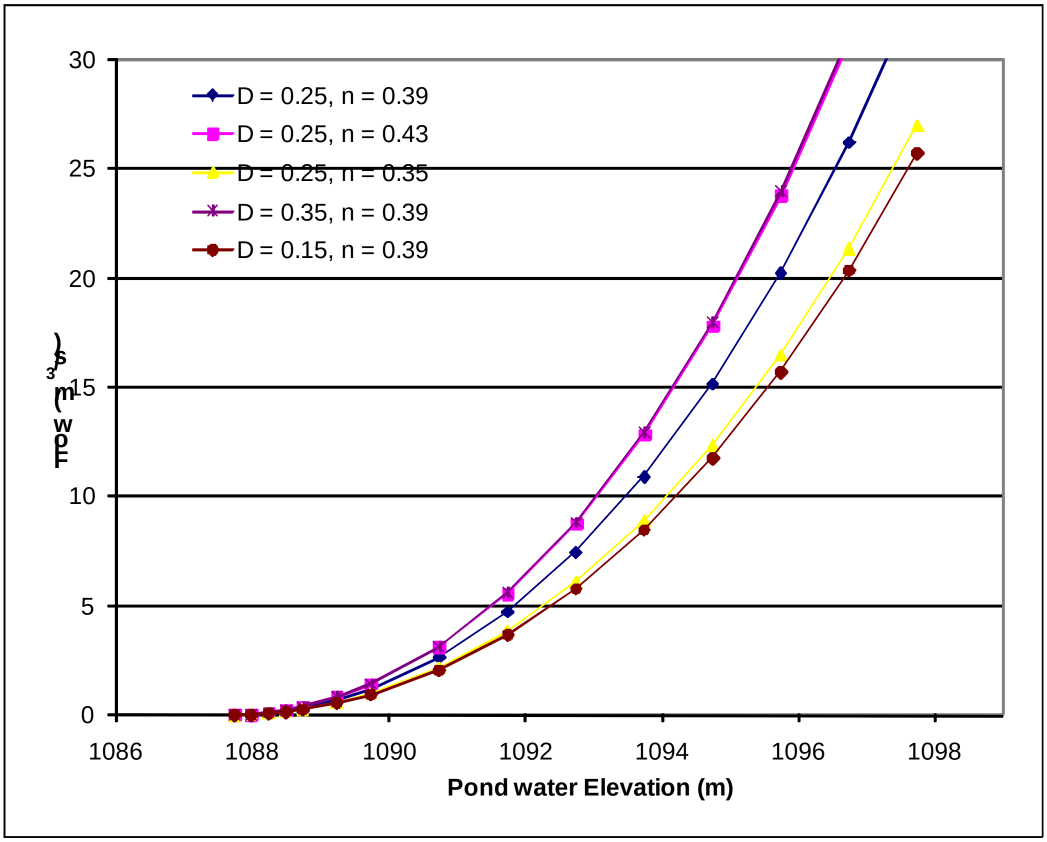

Figure 7 Predicted pond elevation flow relationships

6.0 MEASURED FLOW AND POND ELEVATION

Measurement of the elevation of the water's surface in the pond began in 2004. This was measured by two methods, survey of the water elevation from a benchmark and via the use of a data logger and a water head sensor. The water elevation of the pond was measured at 15 minute intervals with the datalogger and the survey information was collected at varying intervals, but was done every two weeks during the open water season in 2005. The two datasets were compared to ensure that the results were consistent, with the results of the direct survey occasionally used to correct the datalogger information. The results of the pond elevation survey are shown in Figure 7 for the May / June period in 2005. Of note, the estimated pre-construction elevation of the pond was 1087.75 m.

On the downstream side of the rock drain the water elevation was measured once a day using two staff gauges (one installed in 1991 and the other installed in 2005), and on a 15 minute interval using a pressure sensor and datalogger (installed in 2005). These two datasets were also compared to ensure consistency. Additionally, a regular survey of the elevation of the staff gauges was undertaken. These survey results were used to correct the data for relative movements between the sensor and the staff gauges due to settlement or other disturbances. A rating curve was developed to convert the measured water levels to flows. At this station, all the flow, even at the peak of the spring melt period, was confined within the channel making the developed rating curve valid throughout all river stages. The results of the flow measurement are shown in Figure 8 for May - June of 2005. A relationship between the pond level and the resultant flow is apparent from inspection of the data. The peak flow measured during this period was 22.5 m^3/s when the pond elevation was 1096.4 m, by the end of June 2005 the flow and the water elevation in the pond had reduced to more typical levels, 2 m^3/s and 1090 m, respectively.

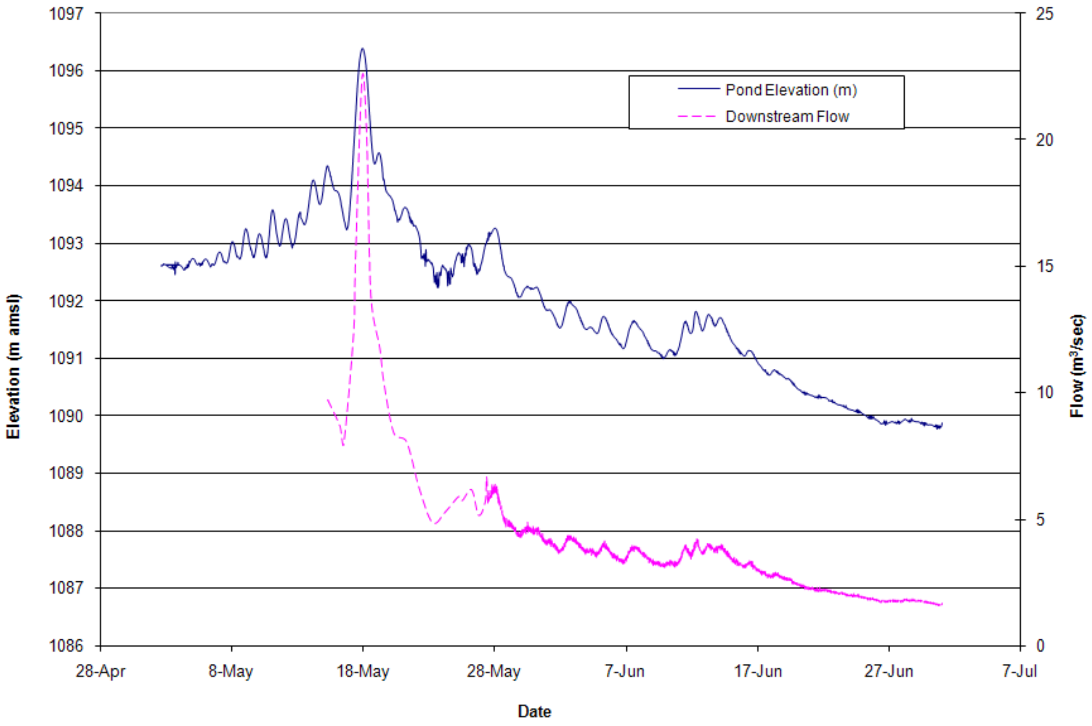

Figure 8 Measured flow and pond water elevation in May – June 2005

7.0 COMPARISON AND CONCLUSIONS

The measured pond elevation and flow from 2005 was compared to the predictions made, Figure 7, using Wilkins' equation. The results are shown in Figure 9. It shows that the measured (diamonds of individual readings) and predicted (dotted lines showing range of predictions based on Figure 7) values compared well, within the accuracy expected.

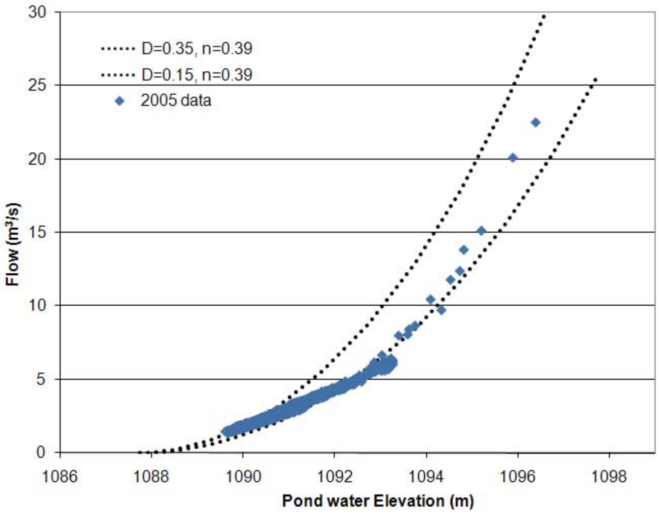

Figure 8 Comparison between predicted and measured, pond elevation and flow

This case study has shown that the method of Wilkins with the refinements of Leps (1973) and of Hansen *et al.* (1995) can be used to successfully predict the flow-through relationship of full scale flow-through rockfill structure.

8.0 ACKNOWLEDGEMENTS

The support of the staff at the Faro mine site during data collection, the permission to publish this particular case study, and the help of personnel from Deloitte and Touche Inc. (the interim receivers for the Faro Mine complex) are all gratefully acknowledged. I would also like to thank David Hansen Ph.D. P.Eng., Dalhousie University, for his willingness discuss the various methods that he has developed to analyze rockfill dams and drains.

9.0 REFERENCES

Beckstead, G.R.F., Slater, J., Gugten, N.V.D. and Slawinski, A. 2000. Embankment Hydrology-Surface Water Controls. Chapter 32 of Slope Stability in Surface Mining. W.A. Hustrid, M.K. McCarter, and D.J.A. Zyl Eds., Society for mining, Metallurgy and Exploration, Littleton, Colorado.

British Columbia Mine Waste Rock Pile Research Committee (BCMWRPRC). 1999. Mined Rock and Overburden Piles, Rock Drain Research Program, Final Report. British Columbia Mine Waste Rock Pile Research Committee.

Canadian Dam Association 2007. Technical Bulletin on Dam Safety. Geotechnical Considerations, sub-section 8.5 “Flow-through Rockfill Dams”.

Claridge, F.B., Nichols, R.S. and Stewart, A.F. 1986. Mine Waste Dumps Constructed in Mountain Valleys. CIM Bulletin, Vol. 892, pp. 79-87.

EBL. 2005. Stability of Downstream Shell and Dam Toe During Large Through Flow. EBL Kompetanse AS, Publication 186-2005.

Golder Associates Ltd., 1987. Letter Report to Curragh Resources Ltd. Re: Inspection of North Fork Rock Drain. Report submitted to Curragh Resources. unpublished

Hansen, D., Garga, V.K. and Townsend, D.R. 1995. Selection and Application of a One-Dimensional Non-Darcy Flow Equation For Two-Dimensional Flow Through Rockfill Embankments. Canadian Geotechnical Journal; 32: pp. 223 – 232.

Hansen, D., Zhao, W.Z., and Han, S.Y. 2005. Hydraulic performance and stability of coarse rockfill desposits. PICE (UK) Journal of Water Management, 158(WM4): 163-174

ICOLD 1993. Reinforced Rockfill and Reinforced Fill For Dams – State of the Art.

Leps, T.M. 1973. Flow Through Rockfill - Embankment Dam Engineering. R.J. Hirschfeld and S.J. Poulos Eds. Wiley, New York, pp. 87 -105.

Nichols, R. 1986. Rock Segregation in Waste Dumps. International symposium on Flow-through Rock Drains, Cranbrook, B.C.

Salmon, G. 2005. Notes for presentation on Embankment Dams – Design for Accidental overtopping and through flow. International Seminar; Stability and Breaching of Embankment Dams, EBL Kompetanse AS

Shank, M. 2003. Automated Detection of Mining Valley Fills from Multi-Date Elevation Data. http://gis.wvdep.org/tagis/projects/fill_paper.doc.

Wilkins, J.K. 1956. Flow of Water Through Rockfill and Its Application to the Design of Dams. Proceedings of the 2nd Australian New Zealand Conference on Soil Mechanics and Foundation Engineering, pp. 141 – 149.

Wilkins, J.K. 1963 The stability of Overtopped Rockfill Dams. Proceedings of the 4th Australian Conference on Soil Mechanics and Foundation Engineering, p.1