Knight Piésold
CONSULTING

Knight Piésold Ltd.
Suite 1400 - 750 West Pender Street
Vancouver, British Columbia
Canada V6C 2T8

Telephone: 604.685.0543
Facsimile: 604.685.0147
E-mail: vancouver@knightpiesold.com

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March 27, 2008

Mr. Cam Grundstrom
VP Operations
Copper Fox Metals Inc.
650,340 - 12 Ave. S. W.
Calgary, Alberta
Canada, T2R 1L5

Dear Cam,

Re: Schaft Creek – Comparison of Water Management Considerations at TSF Options A and B

Further to our meeting of 14 March, we present herewith, the additional details on the water management aspects of the TSF Option A and B sites, as requested.

1. Introduction

The Knight Piésold (KP) letter report dated March 3, 2007 (Ref. No. VA08-00390) presented a conceptual level, comparative assessment of tailings storage facility (TSF) site options for the Schaft Creek project.

The assessment, which considered three cross valley impoundment sites, referred to as Options A, B, and C, concluded that on the basis of the data currently available, the Option A valley was the preferred site followed by Option B. Within Option A, two other alternative arrangements were proposed, Option Aa and A1, which offered potential cost savings over the originally proposed Option A arrangement. However, for the purposes of the current study and taking account of environmental and permitting issues, Copper Fox Metals (CFM) has determined that these alternatives will not be considered in any further detail at this stage.

Although the initial capital cost estimate for Option B was less than that for Option A, it was noted that the Option B site had a significantly larger catchment area and that water management at the site would pose very significant technical and operating challenges. The report noted that while the design of a water management system for this facility was outside the current scope of work, the costs included in the comparison for this component of the works were considered to be a lower bound estimate which had the potential to increase significantly. The report also highlighted the fact that the maintenance of such a system would entail significant ongoing costs throughout the operational life of the mine. The report concluded that the water management issues associated with Option B together with the higher geo-hazard rating of the site pose a considerable technical risk for the site and render it less favourable than the Option A site.

At a meeting held on 14 March, 2008, CFM requested KP to compare, in more detail, the water management issues associated with each site and to assign some order of magnitude comparative cost estimates to these aspects for each of Options A and B.

2. Average Water Balance and Diversion Efficiency

Under average climatic conditions, the annual water balance at both TSF sites is generally positive (i.e. generating surplus water over the course of a year) assuming that all surface water runoff from the facility external catchment is intercepted and successfully diverted around the impoundment.

A simple monthly water balance model was developed for Options A and B for the final year of facility operation to assess the impact of diversion efficiency on the overall water balance. A comparison of the mean annual surplus volumes for each option with different diversion efficiencies is presented in Table 1.

As can be seen from Table 1, the surplus volumes for Option B are considerably greater than for Option A. This is due to the following reasons:

- The Option B catchment area is more than double that of Option A.
- Option B has a higher mean basin elevation and therefore higher rainfall – estimated to be 15% higher than for Option A.
- The valley side slopes at Option B are considerably steeper than at Option A, resulting in higher runoff coefficients.
- The Option B site has a larger glacier fraction in its contributing catchment.

As facilities at both sites operate with a positive water balance and assuming diversions operate at 100% efficiency, any reduction in diversion efficiency will result in an increased volume of water entering the facility. For the purposes of this study it is assumed that this surplus water will need to be treated and discharged.

Water treatment costs vary greatly but are typically of the order of $\$0.30/m^3$ to $\$1.00/m^3$ (actual costs would be determined after defining a specific treatment process given the predicted supernatant solution chemistry). For the purpose of this comparison it was assumed that the diversion efficiencies would be 75% for Option A and 50% for Option B, reflecting the additional difficulty associated with the steeper valley side slopes at Option B, combined with the increased probability of ditches being filled with snow and difficult maintenance access. Unit rates for water treatment were estimated at $\$0.65/m^3$ and $\$0.50/m^3$ for Options A and B, respectively. The slightly lower cost for Option B was chosen to reflect the likely economy of scale associated with a larger capacity treatment facility.

Using the surplus volumes calculated in the water balance model, water treatment costs at Option B would amount to approximately $20 million per annum in water treatment while those for Option A would be about $3 million per annum. This comparison ignores the capital cost of construction a treatment facility.

3. PMF Inflows and Required Storage or Spillway

The Probable Maximum Flood (PMF) inflows were estimated using a HydroCA® rainfall-runoff model. Precipitation data were obtained from the Rainfall Frequency Atlas of Canada (Environment Canada, 1985) and snowpack and wind speed data were taken from the Schaft Creek 2006 Meteorology Baseline Report (Rescan Tahltan Environmental Consultants 2007). Runoff data from Streamflow in the Skeena Region (Obedkoff 2001), a report specific to the site's hydrologic zone (9A), were used to calibrate the rainfall-runoff model.

The Probable Maximum Precipitation (PMP) was estimated by using data from the Rainfall Frequency Atlas of Canada. The mean and standard deviation of the annual (1-in-1-year) 24-hour maximum rainfall were determined for the project area according to the isohyetal maps in the atlas. Based on these values, a 24-hour PMP of 264 mm was estimated using the Hershfield (1965) method.

Potential snowmelt during the PMP event also has to be taken into consideration when estimating the resulting PMF. A monthly snowpack depth was estimated from site and regional data. The potential snowmelt for each month that could occur in conjunction with the PMP was then estimated by applying a formula that considered monthly mean maximum daily temperature and monthly average wind speeds recorded on site. The highest potential snowmelt was found to be 83 mm in a 24-hour period during the month of May. Onsite snow surveys indicate that this depth of snow water equivalent is generally present in May.

The total potential runoff depth from combined rainfall and snowmelt is estimated to be 347 mm. This value was applied to a HydroCAD® model to estimate the resultant PMF. The total PMF inflow to each option is dependent on basin parameters (area and curve number) while the instantaneous peak flow is related to the time of concentration (t_c). Total inflow volumes for Options A and B are approximately 12.3 Mm^3 and 26.9 Mm^3, respectively. The peak PMF inflow for Options A and B are 1,400 m^3/s and 2,200 m^3/s, respectively.

The TSF must either be sized to store the PMF inflow, or a spillway constructed to facilitate the discharge of excess inflow and prevent overtopping of the dam. The worst-case scenario with respect to inflow floods is during the initial years of operation, when the facility has the smallest impoundment area. As such, the water rise following a PMF event was modelled for the starter layouts of Options A and B.

The modelling indicated that the PMF inflow can be safely stored in the Option A facility, resulting in a moderate rise in water level (nearly to the top of the tailings beach). A spillway is therefore not required for Option A.

The PMF inflow for Option B would result in a significant rise in the water level within the facility. To accommodate this flow the starter embankment would need to be raised by seven to ten metres above the level otherwise required. This requirement would increase the construction cost of the Option B starter facility by around $3.5 million

4. Design Storm Inflows and Diversion Channels

The instantaneous peak inflows associated with a 1-in-10-year storm event were estimated using runoff data presented by Obedkoff. Obedkoff provides a set of graphs relating unit 10-year peak discharge to drainage area for each of the hydrologic zones. The instantaneous peak flows resulting from the 10-year storm event and reporting to the diversions for Options A and B are given in Table 2. Diversion channels were sized to accommodate the peak flow resulting from the 10-year event. For comparison purposes, the channel slopes, dimensions, and maximum flow depths were kept consistent for all diversions. The resulting channel dimensions are shown in Table 2. It can be seen that the channels required for Option B are approximately two times larger than those for Option A.

The valley side slopes at the Option B site are generally 2H:1V, where as those at the Option A site are about 3H:1V. Given the larger diversions at Option B, cut and fill volumes per unit length increase to approximately three to four times those at Option A. This leads to the conclusion that the cost of constructing and maintaining diversion channels at Option B would be in the order of four times greater than at Option A per unit length.

It can be concluded that the water management issues associated with Option B will result in significantly higher capital and operating costs as compared to Option A. The increased costs are primarily due to a larger contributing catchment area, resulting in higher inflows, and subsequently a larger annual water surplus and larger diversion structures. The steeper topography in the Option B valley also poses a major challenge with respect to construction and operation of surface water diversions. Furthermore, the technical and financial risks associated with Option B, particularly with respect to water management cannot be fully defined without more detailed engineering work. We expect that further investigation would bring to light even greater risks and challenges.

We trust that this letter serves to adequately summarise the issues discussed in our 14 March, 2008 meeting, and provides the necessary technical information the support the decision to pursue Option A as the preferred TSF alternative.

Yours truly,

KNIGHT PIÉSOLD LTD.

Daniel Friedman, EIT
Project Engineer

Dermot Claffey, CP.Eng
Specialist Engineer

for Ken Brouwer, P.Eng.
Managing Director

Enclosed:
Table 1 Rev 0 Comparison of Annual Surplus in Final Year
Table 2 Rev 0 Comparison of Diversion Channel Sizes

cc: Shane Uren

/cg

TABLE 1

COPPER FOX METALS INC.
SCHAFT CREEK

TAILINGS STORAGE FACILITIES
COMPARISON OF ANNUAL SURPLUS IN FINAL YEAR

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M:\1\01\00329\03\A\Correspondence\VA08-00645\[Table 1.xls]Table 1-Annual Surpluses Rev'd Mar/19/08

Diversion Efficiency	Annual Surplus in Final Operating Year (m^3)	
	Option A	Option B
25%	15,100,000	59,500,000
50%	10,020,000	39,650,000
75%	4,970,000	19,800,000

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TABLE 2

COPPER FOX METALS INC.
SCHAFT CREEK

TAILINGS STORAGE FACILITIES
COMPARISON OF DIVERSION CHANNEL SIZES

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M:\1\01\00329\03\A\Correspondence\VA08-00645\[Table 2.xls]Table 2 - Diversion Sizing

Rev'd Mar/27/08

	$Q_{10\text{-year}}$[1] (m^3/s)	Channel Top Width[2] (m)	Channel Cross-Sectional Area[2] (m^2)	Cut Volume[3] (m^3/m)	Fill Volume[3] (m^3/m)
Option A - East	26	13	20	72	46
Option A - West	36	15	25	89	51
Option B - West	57	20	36	352	40
Option B - East[4]	90	27	54	543	52

Notes:
1. *Instantaneous peak flow resulting from the 1-in-10-year storm, from Obedkoff (2001).*
2. *Channel side slopes of 2H:1V, constant slope, flow depth, and freeboard.*
3. *Valley side slopes of 2H:1V for Option B and 3H:1V for Option A, cut at 1H:1V, fill at 2H:1V (Option A) and 1.5H:1V (Option B).*
4. *The area to the south of the TSF would be diverted with an intake structure on Hickman Creek and routed into the East Diversion.*

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