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ENVIRONMENTAL SCIENTISTS AND ENGINEERS

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SUBJECT TO REVISION

Environmental Evaluation Report

Copper Flat Project

Prepared for

New Mexico Energy, Minerals and

Natural Resources Department

Mining and Minerals Division

Santa Fe, New Mexico

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1. Introduction

At the request of the New Mexico Energy, Minerals and Natural Resources Department, Mining and Minerals Division (MMD), Daniel B. Stephens & Associates, Inc. (DBS&A) has prepared this report describing its environmental analysis of Alta Gold Company's (Alta) mine permit application (permit application No. SI004RN) for the Copper Flat Mine. DBS&A's analysis was requested by MMD in order to provide an independent, third party review of the extensive technical documentation submitted by Alta during the ongoing permitting process.

The review process, as defined by MMD, was focused on three principal areas of potential environmental impact:

- Impacts to groundwater and surface-water supplies due to pumping from supply wells and from pit dewatering
- Impacts to groundwater and surface-water quality due to operation of and seepage from the proposed tailing impoundment
- The likelihood and magnitude of generation of acid rock drainage from disturbed materials and the concomitant impacts to groundwater and surface-water quality, emphasizing post-closure pit-water chemistry.

1.1 Site Background

The Copper Flat Mine is located approximately 23 miles southwest of Truth or Consequences and about 5 miles northeast of Hillsboro in Sierra County, New Mexico. The Copper Flat mine was operated briefly, from April through July 1982, by the Copper Flat Partnership, Ltd. The mine consisted of the open pit mine, a 15,000 ton-per-day mill and flotation extraction system, and a 515-acre tailing impoundment. The mine was closed due to depressed copper prices.

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As described in the Draft Environmental Impact Statement (BLM, 1996), as well as in the Mine Permit Application prepared for Alta by Steffen Robertson and Kirsten, Inc. (SRK, 1996), Alta has proposed re-opening the Copper Flat mine with a similar design to that used in 1982. That is, Alta intends operate with similar throughput, and further intends to use essentially the same mine layout as was used by the Copper Flat Partnership. Accordingly, Alta's proposal includes use of the existing infrastructure (foundations, pipelines, etc.) to the extent practicable. Additionally, the same tailing ponds (and design) used briefly in 1982 would be used in the new operation.

1.2 Scope of Work

In support of its proposed re-opening, Alta has submitted a large volume of technical information describing proposed operations for the mine and anticipated impacts to the local and regional environment. It is the primary intent of this study to review that information and evaluate it for its reasonableness, completeness, and technical adequacy, and to make recommendations for additional analysis in areas found to be deficient.

As part of the permitting process being conducted by MMD pursuant to the New Mexico Mining Act Rules, a public hearing was held from February 6 through 9, 1997. As summarized in the July 17, 1997, Report of the Hearing Officer (MMD, 1997), several areas of concern were identified and discussed at the hearing by numerous technical experts, advocacy groups, and members of the general public. The areas of concern identified included:

- Adequacy of the baseline data
- Potential water quantity issues
- Potential water quality impacts
- Potential ecosystem impacts
- Reclamation issues
- Economic impacts
- Impacts on existing infrastructure

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In this review, we have attempted to address the significant concerns relating to most of the identified areas of concern. However, evaluation of ecosystem impacts (e.g., impacts to flora and fauna), economic impacts, and impacts to existing infrastructure are beyond the scope of services requested by MMD and, therefore, were not considered.

In its analysis, DBS&A attempted to review all available information germane to the issue being evaluated. In many instances, more than one analysis of a particular issue has been provided by Alta (or its predecessor Gold Express, which initiated the permitting process in 1991) over the course of the permitting process. Examples where this is the case include, but are not limited to, the evaluation of:

- Regional groundwater impacts, including numerical modeling
- Seepage impacts from the tailing ponds
- The potential for the generation of acid rock drainage (ARD)
- Predictions of pit water chemistry

Where more than one analysis of a given issue has been performed, we have assumed that the most recent analysis supersedes all preceding work. Accordingly, we have focused our review effort on those most recent analyses.

1.3 Report Structure

This report is organized into six sections. Sections 2, 3, and 4 detail the analyses of predicted impacts to surface water and groundwater quality from pumping supply wells and pit dewatering, mill tailing disposal, and acid rock drainage from mine stockpiles, respectively. Section 5 discusses the reclamation and closure plan information. Section 6 identifies gaps in the data and areas requiring future investigation.

2. Analysis of Predicted Impacts to Surface Water and Groundwater Supply from Pumping Supply Wells and Pit Dewatering

Adrian Brown Consultants (ABC) conducted much of their water resources impect enelysis using a regional groundwater flow model. The model domain extends form the crest of the Black Range on the west to the Rio Grande and Caballo Reservoir on the east, end from Las Palomas Creek on the north to Berrenda Creek on the south. The Copper Flats pit is located at approximately the center of the model domein (Figure 2-1). The regionel groundwater flow model presented in Appendix D of the Groundwater Impect Evaluation dated March 21, 1997 evolved from several earlier modeling efforts conducted by ABC.

ABC used their proprietary computer code called ABCFEM to simulate groundwater flow within the model domain. ABCFEM is a finite element code that uses triangular elements to discretize the aquifer in the horizontal direction. Three model layers were used to simulate the assumed regional aquifer thickness in the vertical dimension. An additional model layer of limited areal extent was applied to simulate groundwater flow in the Las Animas Creek alluvium (Figure 2-2). Groundwater flow between model layers is simulated using the finite difference approach.

ABC calibrated the regional model to observed hydraulic head data from various sources and to baseflow gain of the Rio Grande as determined by two seepage studies reported in Murray (1959). A steady-state calibration was performed assuming that changes in the hydrologic system from predevelopment conditions to the present day, as well as other variations in time such as seasonal effects, were not substantial. As such, the model calibration does not include aquifer storage es an input parameter.

Using the calibrated model, ABC conducted a base-case transient simulation for a 500-year period to assess the potential impacts of mine-related activities. Potential impacts of concem stem mainly from pumping 2,100 gpm of water from the production well field for 10 years to sustain mining operations, and from groundwater flow into or through the pit once mining ceases. Pumping from the production well field has the potential to adversely affect surface water flow in Las Animas and Percha Creeks end the Rio Grande. In addition to the base-case predictive

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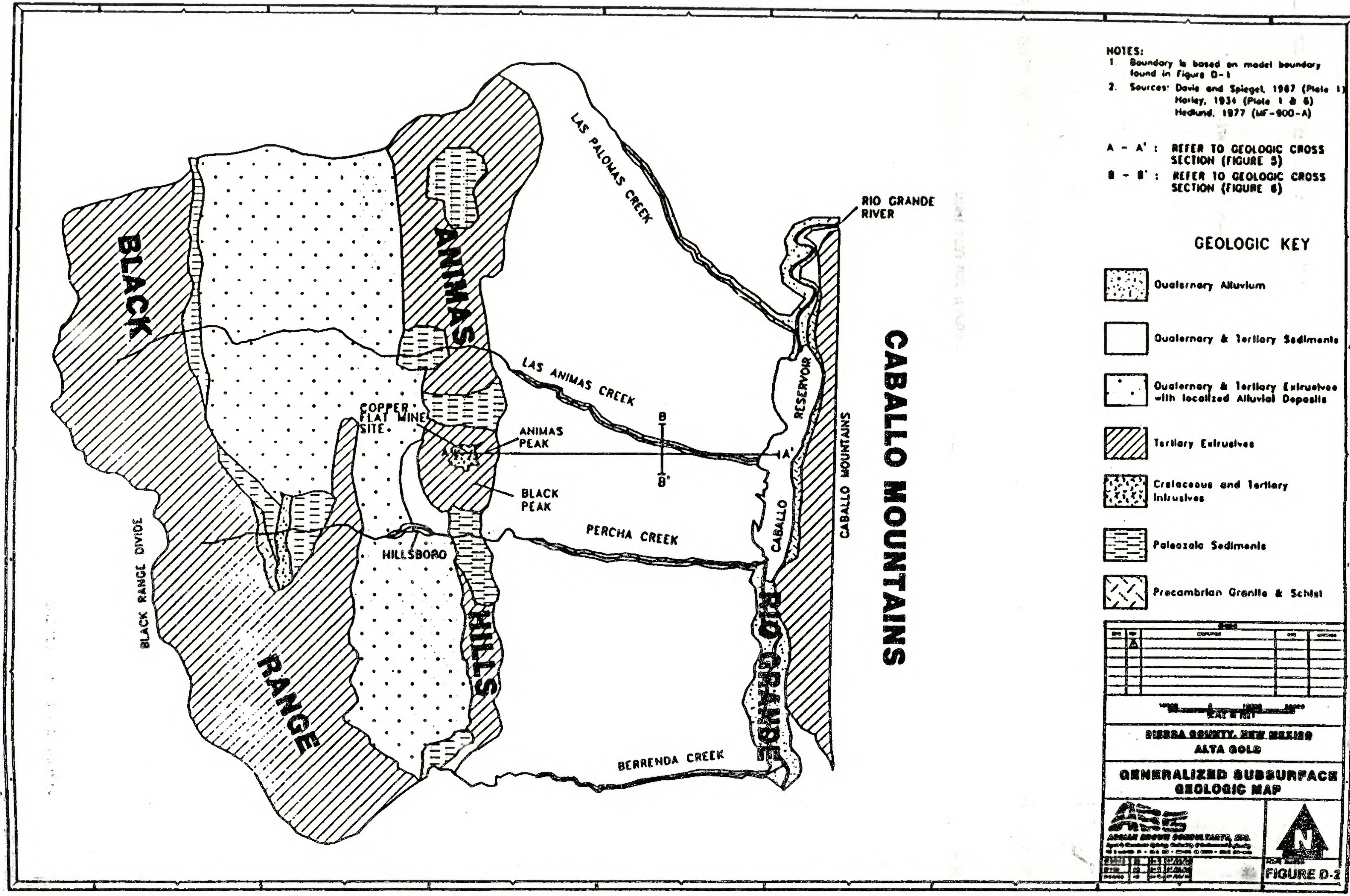
NOTES:
1. Boundary is based on model boundary found in Figure D-1
2. Sources: Davie and Spiegel, 1967 (Plate 1)
Harley, 1934 (Plate 1 & 6)
Hedlund, 1977 (MF-900-A)
A - A' : REFER TO GEOLOGIC CROSS SECTION (FIGURE 5)
B - B' : REFER TO GEOLOGIC CROSS SECTION (FIGURE 6)
GEOLOGIC KEY
Quaternary Alluvium
Quaternary & Tertiary Sediments
Quaternary & Tertiary Extrusives with localized Alluvial Deposits
Tertiary Extrusives
Cretaceous and Tertiary Intrusives
Paleozoic Sediments
Precambrian Granite & Schist
BLACK
RANGE
BLACK RANGE DIVIDE
ANIMAS
HILLS
LAS PALOMAS CREEK
LAS ANIMAS CREEK
PERCHA CREEK
BERRENDA CREEK
COPPER FLAT MINE SITE
ANIMAS PEAK
BLACK PEAK
HILLSBORO
B
B'
A'
RIO GRANDE RIVER
RESERVOIR
CABALLO
CABALLO MOUNTAINS
RIO GRANDE
SIERRA COUNTY, NEW MEXICO
ALTA GOLD
GENERALIZED SUBSURFACE GEOLOGIC MAP
N
FIGURE D-2

Figure 2-1

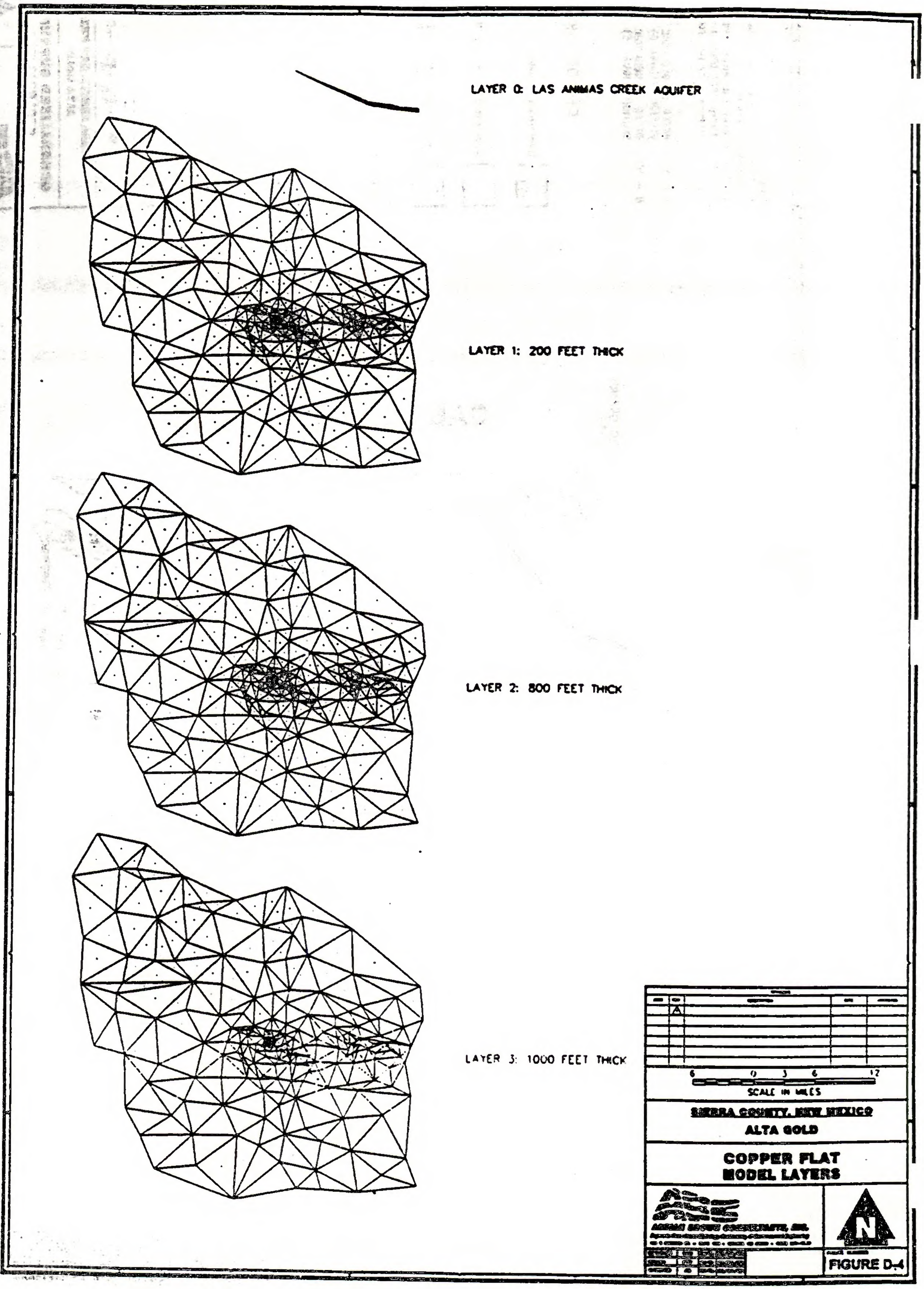
LAYER 0: LAS ANIMAS CREEK AQUIFER
LAYER 1: 200 FEET THICK
LAYER 2: 800 FEET THICK
LAYER 3: 1000 FEET THICK
6 0 3 6 12
SCALE IN MILES
SIERRA COUNTY, NEW MEXICO
ALTA GOLD
COPPER FLAT
MODEL LAYERS
FIGURE D-4

Figure 2-2

simulation, a number of sensitivity analyses were run in which certain input parameters were adjusted either higher or lower than the calibrated values.

The sections below outline DBS&A's comments and observations concerning the ABC regional groundwater flow model. All reference to page numbers, tables and figures, unless expressly referenced otherwise, refer to Appendix D of the Groundwater Impact Evaluation. It is our opinion that there are significant shortcomings with some aspects of the simulation approach and with some of the critical model input parameters applied. As a result, the predictive simulation results provided by ABC are not conservative and are potentially in error.

2.1 Validation of ABCFEM Version 5.1.1 Not Presented

Version 5.1.1 of the ABCFEM model used for the March 21, 1997 Groundwater Impact Evaluation is a recently developed version of the code which has the capability of simulating three-dimensional groundwater flow. Earlier versions of ABCFEM are capable of simulating only two-dimensional groundwater flow. The algorithm implemented into ABCFEM for simulating three-dimensional flow is presented in Appendix E of the ABCFEM User's Guide provided to DBS&A by ABC. However, there is no verification of the model's three-dimensional simulation capability presented in the Appendices or elsewhere in the user's guide.

It is standard practice to test, or verify, numerical models against analytical solutions or other model codes that have already been tested extensively. This apparent omission is particularly important for a three-dimensional simulation model, since it is considerably more complex than a two-dimensional model. There are many potential situations in practice where non-apparent errors can be introduced into the simulation, several of which are noted in Section 2.2. DBS&A does not recommend that the MMD accept any simulation results without written documentation of the three-dimensional model verification.

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2.2 Unrealistic Simulation Results

Examination of the steady-state model calibration results presented in Attachments to Appendix D indicated simulation results at some model nodes that should not be possible according to the model documentation. For example, on p. 12 of the ABCFEM User's Guide, it is stated that for unconfined aquifer analysis "ABCFEM does not allow the (simulated) head to drop below the base of the node." This constraint is also stated on p. E-1 of Appendix E, "The pressure at a node never drops below zero. In ABCFEM's terms, the head at a node is not allowed to drop below the elevation of the base of the node."

However, in Attachment D-2, which among other things lists the nodal base elevations and simulated water levels from the final calibration, the simulated head is below the nodal base elevation at numerous nodes in model layer 1, which is unconfined. This condition also occurs in model layer 2, although less frequently than in model layer 1 (see, for example, nodes 227, 229, 236 and 237).

In addition, this condition occurs not only in the Black Range where significant numerical error would be expected due to the coarseness of the model grid, but it also occurs in eastern regions of the model domain. For example, model layer 1 finite element number 210 is defined by nodes 112, 132 and 139. This element is located south of the production well field and abuts the south side of Percha Creek. At nodes 139 and 112, the simulated head is 7 and 52 feet below the base elevation of each of these nodes, respectively. At node 132, however, the simulated head is 79.5 feet above the node base elevation.

These results are in direct contradiction to the documented ABCFEM solution procedures, and the resulting effects on the simulation results are unknown. It appears that at many nodal locations throughout the model, layer 1 is unsaturated, even though layer 1 supposedly represents the uppermost 200 feet of saturated aquifer thickness. This issue is discussed in greater detail in the following section with regard to the production well field area.

2.3 Model Calibration to Hydraulic Head

The steady-state model calibration results presented in Table D-5 and Figure D-11 (ABC, 1997) are misleading and give a false sense of calibration accuracy. The difference between simulated and observed hydraulic heads at all of the springs (except for Warm Springs South) listed in the table and plotted in the figure is zero feet, which gives the impression that the model simulates the groundwater flow field perfectly at those nodal locations.

What has not been presented is a comparison between observed and simulated spring flows at those nodes that represent springs. Observed spring flow is not available for all springs, but it is available for some, such as Warm Springs North. The observed spring flow is the critical observation to match to, not the hydraulic head at the spring pool, which is typically assumed to be the land surface elevation. Furthermore, given the way that ABCFEM simulates surface water-groundwater interaction, it is very common for the simulated hydraulic head to be set to the ground surface elevation during a model run. In fact, the simulated hydraulic head is set to the ground surface elevation (the top aquifer elevation for a given node) at many locations throughout the model domain, in some cases erroneously. The issue of surface water groundwater interaction is discussed in greater detail in Section 2.5 below.

If the zero difference values reported for spring nodes are removed from Table D-5, as well as several other nodes at which an elevation boundary condition constraint is placed on the node, the average absolute difference between observed and simulated heads listed in the table is 25 feet, and the root mean square difference is 37 feet. These numbers are more accurate representations of the calibration results.

In addition to the above observations, there are other discrepancies between the calibration results presented in Table D-5 and the conceptual model presented by ABC. On p. 7 of Appendix D, ABC states that the effective thickness of the flow system is estimated to be 2,000 feet. This effective thickness is divided conceptually into 3 model layers as follows: model layer 1 represents the uppermost 200 feet of saturated material, model layer 2 represents the

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next 800 feet of saturated material, and model layer 3 represents the remaining 1,000 feet of saturated material (Figure 2-2).

However, at many locations in model layer 1, and at several of the calibration points presented in Table D-5 (i.e., LA-152, PW-2, PW-1, MW-6, MW-1, and MW-8), the simulated hydraulic head is below the prescribed base elevation of model layer 1. In other words, model layer 1 at those locations is not saturated as far as the model is concerned. In fact, analysis of the nodal base elevations and simulated hydraulic heads in the vicinity of the production well field indicates that model layer 1 is consistently dry in the model and the simulated saturated thickness in on the order of 1,600 feet, as opposed to 2,000 feet as documented in the report. It appears that ABC did not simulate the upper 2,000 feet of saturated thickness, but rather simulated groundwater flow within the upper 2,000 feet of aquifer material measured vertically downward from the land surface. If this is indeed the case, many of the computations and analyses based on a 2,000 foot effective aquifer thickness presented by ABC are in error.

2.4 Model Input Parameters

The following sections address specific concerns with model input parameters, such as hydraulic conductivity for andesite and monzonite at the pit, hydraulic conductivity for Santa Fe sediments, and specific storage values.

2.4.1 Hydraulic Conductivity of Andesite and Monzonite at the Pit

ABC presents four types of analyses as indicators of the hydraulic conductivity of the andesite and quartz monzonite in the vicinity of the pit. These analyses are dewatering during mining operations; pit refilling after mining; steady-state conditions in the pit pool; and borehole permeability tests. The first and last of these analyses yielded hydraulic conductivity on the order of 1 foot per year (ft/yr), while the second and third analyses yielded hydraulic conductivities of less than 50 ft/yr and 25 ft/yr respectively. Based on this information, ABC applied what they consider to be a conservative hydraulic conductivity value of 8 ft/yr for the andesite and monzonite in the pit area.

DBS&A believes that while a hydraulic conductivity of 8 ft/yr is potentially appropriate, it may well be on the low side of probable values and is certainly not conservative. Sensitivity analysis with a substantially larger hydraulic conductivity, not a lower one as presented in Section D.6.5.2 of ABC (1997), should be presented.

The reasons behind this conclusion are as follows. First of all, the Theis calculation performed for the pit dewatering analysis by ABC could not be reproduced by DBS&A using the same hydraulic parameters. In addition, ABC assumed a contributing rock thickness of 1,000 feet with no justification when the pit floor was approximately 45 feet below the water table. Furthermore, ABC assumed a storage coefficient of 0.01, representative of near-surface materials, over the entire 1,000 foot thickness.

Using smaller storage coefficients that would be more representative of 1,000 feet of fractured andesite and monzonite, DBS&A could calibrate to the observed drawdown at the pit. For example, 45 feet of drawdown at a radius of 450 feet is computed using transmissivities of 21 and 41 ft^2/d for storage coefficients of 0.001 and 0.0001, respectively. Note that these storage values are still higher than the values that ABC used in their regional model. For example, using a specific storage of 6×10^{-8} (ABC, 1997, p. 15), and a model layer 2 thickness of 800 feet (p. 7), the storage coefficient would be 0.00005, which is one-half that of the lowest value presented above. Note that the alternative transmissivity values calculated above are about 4 and 8 times higher than that presented by ABC.

Regardless even of the above observations, the assumed contributing thickness value of 1,000 feet applied to compute hydraulic conductivity from the transmissivity is unsubstantiated and, in our opinion, inappropriate. A much more reasonable number would be something between 45 feet and perhaps 100 feet, and such a number would be more consistent with the "near-surface material" storage value of 0.01. Even using an estimated effective thickness of 100 feet would yield a hydraulic conductivity 10 times higher than that presented by ABC.

In summary, each of the methods of field-scale hydraulic conductivity estimation for the andesite and monzonite yield hydraulic conductivity values on the order of 10 ft/yr or higher. The borehole tests yield lower values, but we concur with ABC (p. 20) that these tests are not good estimators of large scale (i.e., finite element size) permeabilities due to the very limited volume of the aquifer that they test. A realistic and appropriately conservative sensitivity analysis would apply an andesite and monzonite hydraulic conductivity of at least 25 or 30 ft/yr.

2.4.2 Hydraulic Conductivity of Santa Fe Sediments

ABC apportioned the Santa Fe alluvium in their regional model into 5 distinct zones (Figure 2-3). Each zone has different hydraulic characteristics. Of critical importance is the zone referred to as Santa Fe alluvium east, since the production wells are located in this zone. The hydraulic conductivity applied in this zone has a significant impact on simulation results, and is critical in the determination of impacts to surface water bodies such as the Rio Grande.

DBS&A is concerned about the magnitude of the hydraulic conductivity and the anisotropy factor used in the Santa Fe alluvium east. ABC computed a hydraulic conductivity for this zone of 4,880 ft/yr based on transmissivity values of about 200,000 gal/d/ft calculated for MW-5 at the production well field (Greene and Halpenny, 1977). To obtain the 4,880 ft/yr value, ABC divided the observed transmissivity by an aquifer thickness of 2,000 feet, which is the value supposedly used in their model (as presented above, the simulated aquifer thickness in the production well field area is actually closer to 1,600 feet).

However, the saturated thickness at MW-5 was only about 660 feet when the aquifer tests were conducted. Therefore, due to vertical anisotropy and the short duration of the aquifer tests, a more appropriate computation of hydraulic conductivity from the aquifer test data would use an aquifer thickness value of 660 feet, rather than 2,000 feet. This approach yields a hydraulic conductivity values of approximately 11,000 to 15,000 ft/yr, which are 2 to 3 times higher than those applied by ABC. The effect of using a lower hydraulic conductivity in the model is that simulated drawdowns near the production well field will be increased, while simulated drawdown in regions of concern such as Las Animas Creek and Caballo Reservoir are diminished.

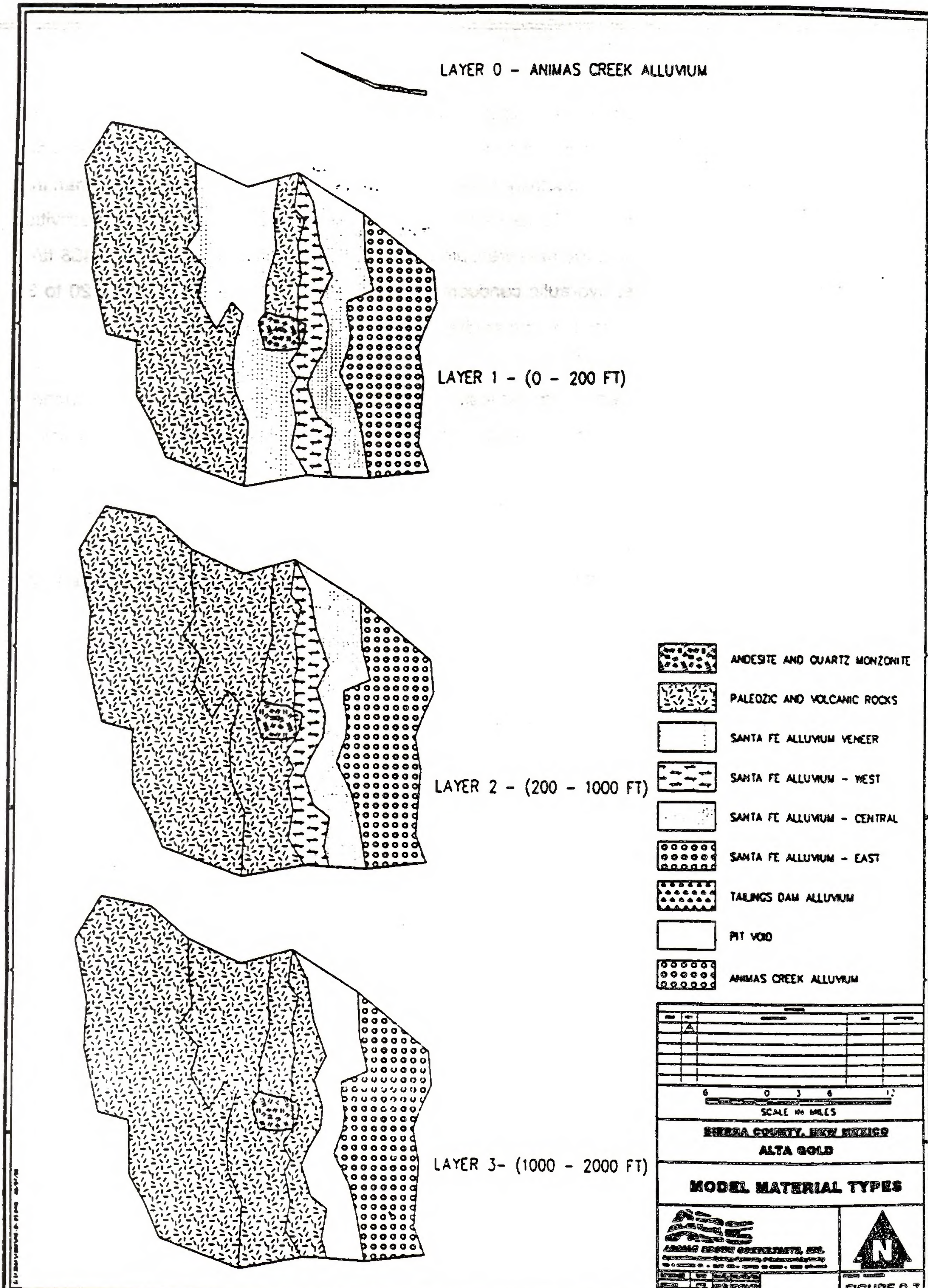
LAYER 0 - ANIMAS CREEK ALLUVIUM
LAYER 1 - (0 - 200 FT)
LAYER 2 - (200 - 1000 FT)
LAYER 3- (1000 - 2000 FT)
ANDESITE AND QUARTZ MONZONITE
PALEOZIC AND VOLCANIC ROCKS
SANTA FE ALLUVIUM VENEER
SANTA FE ALLUVIUM - WEST
SANTA FE ALLUVIUM - CENTRAL
SANTA FE ALLUVIUM - EAST
TAILINGS DAM ALLUVIUM
PIT VOID
ANIMAS CREEK ALLUVIUM
6 0 3 6 12
SCALE IN MILES
SIERRA COUNTY, NEW MEXICO
ALTA GOLD
MODEL MATERIAL TYPES
N
FIGURE D-7

Figure 2-3

DBS&A also does not agree with the horizontal anisotropy factor used for all of the Santa Fe alluvium zones. An anisotropy factor of 10 was applied by ABC, which indicates that at any point within the zone, the hydraulic conductivity in the north-south direction is 10 times greater than that in the east-west direction. For the Santa Fe alluvium east, therefore, the hydraulic conductivities in the north-south direction and the east-west direction used by ABC are 4,880 ft/yr and 488 ft/yr, respectively. The east-west hydraulic conductivity value of 488 ft/yr is approximately 20 to 30 times lower than values indicated by the aquifer test results.

This approach is not supported by aquifer tests conducted at the site or by previous researchers and modeling studies. In addition, since groundwater flow in the Santa Fe alluvium is predominantly from west to east, the hydraulic conductivity in the north-south direction is not a critical factor for the steady-state model calibration. In order to verify the appropriateness of this parameter using model calibration, a calibration that involved significant north-south groundwater flow components (such as a cone of depression formed by pumping wells) would have to be conducted. This type of calibration was not conducted by ABC.

For example, production wells 2 and 3 are each approximately ½ mile from MW-5. However, PW-2 is south of MW-5 and PW-3 is to the west. If the Santa Fe sediments were anisotropic in the horizontal direction, the transmissivity values computed for the PW-2 and PW-3 aquifer tests should be significantly different. This is not the case: the transmissivity values reported by Greene and Halpenny (1977) calculated at MW-5 for the PW-2 and PW-3 tests are nearly identical (i.e., 203,500 gpd/ft during the PW-2 tests and 203,100 gpd/ft during the PW-3 test). Horizontal anisotropy effects were not observed during aquifer tests conducted near Truth or Consequences (Murray, 1959) or GWQ94-17 at the tailing dam (ABC, 1997).

ABC claims in their March 26, 1997 Post-Hearing Submission that the anisotropy ratio applied is appropriate for three main reasons:

- High permeability conduits oriented in a north-south direction exist due to historic reworking of Santa Fe sediments by the Rio Grande

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- Numerous north-south trending faults occur in the region that act as barriers to groundwater flow

- A groundwater flux calculation based on baseflow gain to the Rio Grande reported by Murray (1959) supports a low hydraulic conductivity number in the east-west direction

DBS&A does not believe that these observations are sufficient to support the horizontal anisotropy ratio of 1:10 applied in the modeling for the following reasons. First of all, it is inappropriate to simulate high permeability north-south oriented conduits (if they exist) using anisotropy. The appropriate approach would be to use heterogenous aquifer parameters. In addition, if the conduits do exist, they are apparently undetectable at the scale of aquifer tests conducted in this and similar basins along the Rio Grande Rift. Therefore, the application of horizontal anisotropy in the ABC model is based merely on hydrologic speculation rather than on direct field data or scientific precedent for simulating similar hydrogeologic systems in New Mexico.

Secondly, ABC assumes that faults within the model domain act as barriers to groundwater flow. Although this is possible, it is not evidenced by field data or the hydraulic head contour maps developed by ABC. Furthermore, the groundwater flow in many other basins along the Rio Grande in New Mexico has been successfully simulated without resorting to the unconventional type of parameterization used by ABC.

Finally, the baseflow calculation presented by ABC to support the low east-west hydraulic conductivity used in the model is potentially flawed, and ABC apparently does not consider the ramifications of the uncertainty in this calculation. ABC assumed that 2,000 feet of saturated Santa Fe alluvium contributes a baseflow of 1 cfs per mile of river reach. Based on this information and the fact that a hydraulic gradient of 0.007 is observed in the field, ABC back-calculated a Santa Fe alluvium hydraulic conductivity of 400 ft/yr, which is similar to the value applied in their model. Murray (1959) conducted a similar computation using transmissivity estimates from aquifer tests conducted in the Truth or Consequences area (at that time named Hot Springs). He applied an average hydraulic gradient of 0.01 and a transmissivity of 11,000 gpd/ft to calculate an approximate baseflow gain of 1 cfs per 1 mile reach of the Rio

Grande. However, Murray's calculation implicitly assumes an aquifer thickness similar to that tested by the wells he considered. The effective thickness for which Murray obtained transmissivity values is at most about 400 feet, and is likely closer to 200 feet. These ranges of effective aquifer thickness were determined based on the depths and screened intervals of the pumping and observation wells that Murray used in his analysis. Murray illustrated, therefore, that the 1 cfs baseflow gain to the Rio Grande could be supplied by the upper 200 to 400 feet of the saturated Santa Fe alluvium.

The discrepancy between Murray's calculations, ABC's calculations, and observed baseflow gain as measured by seepage studies is the 2,000-foot "effective thickness" assumed by ABC, and ABC's conceptualization where water within the aquifer flows to. ABC assumes that all water passing through the estimated aquifer thickness of 2,000 feet discharges to the Rio Grande, whereas Murray (1959) illustrated that the upper 200 to 400 feet of aquifer is capable of supplying the observed volume of baseflow as determined from seepage studies. Because ABC applied the observed hydraulic gradient of about 0.007 (note that no wells penetrate into the lower 1,000 feet of ABC's assumed 2,000-foot-thick aquifer), the observed baseflow gains of about 1 cfs per mile, and an aquifer thickness of 2,000 feet, ABC is forced to reduce their model value of hydraulic conductivity of the Santa Fe sediments in the east-west direction by a factor of at least 10 below observed values indicated by the aquifer tests.

DBS&A does not agree with this approach because there are other conceptual models that do not require that the observed aquifer parameters be adjusted in a nonconservative, and probably unrealistic, manner. The conceptual model developed by ABC intrinsically causes them to substantially modify observed aquifer parameters in the area of potential impacts. ABC estimated a low hydraulic conductivity from the production well field aquifer tests to begin with assuming a 2,000-foot aquifer thickness, and then ABC decreased the east-west hydraulic conductivity even more by assuming horizontal anisotropy. Although there is a large degree of uncertainty associated with the 2,000-foot thickness number, this number constrains the entire modeling effort. The effective aquifer thickness could be 3,000 feet, 4,000 feet or more, yet ABC's conceptual model does not allow for this possibility.

An alternative conceptual model is one where the entire saturated thickness of the aquifer system does not discharge to the Rio Grande, but rather there is a different direction (most likely to the south) of groundwater flow at depth. This conceptual model would be more consistent with those developed for other major groundwater basins along the Rio Grande rift in New Mexico, such as the Albuquerque Basin to the north, and the Mesilla Basin to the south (see, for example, Kernodle et al., 1995). Furthermore, this conceptual model would not require that observed hydraulic parameters be as substantially modified as was done by ABC.

The effect of the anisotropy ratio used by ABC is to exaggerate simulated drawdowns in the north-south direction and underpredict them in the east-west direction. **Therefore, drawdown beneath Las Animas Creek is increased, while drawdown at Caballo Reservoir is decreased** relative to **expected values.**

2.4.3 Specific Storage Values

The reasoning behind the selection of some of the specific storage values applied in the model is not clear, and it appears that some of the values applied in the model are inappropriate. For example, ABC computed a specific storage of 5×10^{-7}/ft based on the aquifer test results of Greene and Halpenny (1977). However, they only applied this value in the model for the sensitivity run that had a "high" specific yield value (presumably 0.4). For the base case model runs ABC used a specific storage of 5×10^{-6}/ft for the Santa Fe alluvium, an order of magnitude higher than that observed in the field. As a justification for this, ABC cites other modeling studies.

It also appears that ABC applied a specific storage of 1.45×10^{-4}/ft for the upper 200 feet (model layer 1) of andesite and monzonite (Table D-2). This value corresponds to a specific yield of 0.03, which is three times higher than the quoted reasonable value of 0.01 on p. 10 of Appendix D.

It is inappropriate to ignore field data at the site in favor of values reported in other studies. This is particularly so when an input parameter is adjusted in a nonconservative direction (i.e., larger specific storage values cause smaller simulated impacts). This approach is particularly

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inappropriate since ABC only performed a steady-state model calibration that does not rely on storage. Therefore, there is no verification of the validity of the storage terms obtained during the model calibration process. **The MMD should require that ABC apply the lowest observed or reasonably calculated specific storage values during their base case predictive (transient) simulation.**

2.5 Simulation of Spring Flow and Surface Water-Groundwater Interaction

The interaction between groundwater and the Rio Grande as simulated by ABC is covered in detail in Section 2.4.2. In this section, comments concerning simulated spring flow and other surface water flow, mainly as simulated for Percha and Las Animas Creeks, are presented.

2.5.1 ABCFEM Methodology for Simulating Surface Water-Groundwater Interaction

ABCFEM applies a unique algorithm for the simulation of surface water-groundwater interaction. Basically, series of nodes are "linked" together along stream courses or drainages. Each node in the model can have one link node, which represents a downstream node that surface water will flow to. If, at a given node, surface water is available for flow, that surface water is routed to the downstream (link) node. In the routing process there is no loss of water by evapotranspiration or seepage.

A key step in the surface water-groundwater simulation algorithm is the determination of the availability of surface water at a node point. If the simulated water level in the aquifer rises above the prescribed top elevation of what is called a "swamping" node, the water level is set to the nodal top elevation and the excess water is assigned to that nodal location as surface water. The volume of water assigned as surface water is then available for routing to the assigned downstream link node. At the link node, the incoming water is evaluated in terms of the groundwater flow conditions at that node. If the link node is also "swamped," additional water will exit the link node, be added to the incoming surface water, and the summed volume will be routed downstream to the next link node. If the link node is not swamped, the incoming surface water will be injected into the groundwater system until it is entirely used up or until the link node becomes swamped, whichever condition occurs first.

DBS&A notes the following limitations of this algorithm. The applicability of the algorithm itself is not contended, but the below points must be considered when evaluating the reasonableness of the simulation results.

- The algorithm does not allow for losses of water by evapotranspiration while it is routed between node points.

- Flow of water out of the groundwater system to surface water, or vice versa, is controlled by the simulated groundwater conditions at the nodal point considered. Other physical considerations, such as a stream bed conductance term that encompasses the effects of channel geometry and the vertical hydraulic conductivity of the sediments beneath the stream, are neglected. These assumptions lead to greater inaccuracies at the finite element size increases.

- When there are numerous links between model nodes and the water level lies close to the land surface, it is very easy using this algorithm to get surface water flow where it should not exist, even if the simulated head values appear to match observed values.

2.5.2 Percha Creek

Simulated baseflow of Percha Creek for the model calibration was examined since Percha Creek is an important surface water resource fed by groundwater. Percha Creek is perennial in the Percha Box (the Box) area where groundwater in the Paleozoic rocks is forced to the surface by a rise in the underlying Precambrian rocks. Percha Creek baseflow observations are limited. Appendix F of the Post-Hearing Submittal prepared by ABC dated March 25, 1997, contains observed Percha Creek discharge in the Box area collected at 13 locations. Nine of the locations were dry at the time of measurement. A flow of 119 gpm was observed at the entry to the Box (there was no flow 400 feet upstream of this point), and the flow increased to 456 gpm at the exit from the Box. Percha Creek was dry at the next measurement location 2,400 feet downstream. About 1 mile downstream from the downstream end of the Box, a flow of 177 gpm was observed. Weber and Cole (1996) observed average flows of 233 and 777 gpm at the upper and lower reaches of the Box, respectively.

The ABC Copper Flat model simulates substantially greater baseflow in the Percha Box area than is observed in the field, and it simulates the occurrence of baseflow over a channel distance substantially longer than the Box reach. In the vicinity of Hillsboro and the Box, baseflow to Percha Creek is simulated at model nodes 49, 222, 84, 108, 118, and 132 (listed in downstream order beginning at Hillsboro). The Box exists primarily between nodes 222 and 84. Simulated baseflow in Percha Creek (called the outflow to link in ABCFEM terminology) is 953 gpm at Hillsboro (node 49), 1,079 gpm upstream of the entrance to the Box (node 222), and 1,441 gpm near the exit of the Box (node 84). In addition, simulated baseflow does not cease at the downstream end of the Box, but continues at a rate of 1,600 gpm or more (nodes 108, 118 and 132) for about 5 miles downstream. These discrepancies are partially due to the methodology that ABCFEM uses to route surface water (Section 2.5.1), but they are primarily due to the fact that the model significantly overestimates baseflow along Percha Creek, at least according to the existing data.

The significance of the simulated Percha Creek baseflows is twofold. First, the fact that the model significantly overestimates baseflows throughout the entire reach from Hillsboro downstream to some 5 miles below the exit of the Box indicates that the model is not well calibrated in this region, with respect to baseflow or hydraulic head. Therefore, the model is not appropriate or conservative for performing predictive simulations of potential impacts to the Percha Box area. The model is not conservative because as simulated drawdown caused by the production well field propagates towards the Box area, there is an artificial source of water available for infiltration at affected nodes. In essence, there is about 1,600 gpm of water available for infiltration that could be "used up" prior to drawdown reaching the Box area.

Second, the overprediction in simulated baseflow artificially routes water from the Percha Box area eastward toward Caballo Lake and the production well field area. Approximately 1,600 gpm

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of excess water that exists in Percha Creek in the simulation (but not in reality) is infiltrated back into the groundwater system at node 132, which is southwest of the production well field. This is of note, especially when one considers that 1,600 gpm is 76 percent of the estimated production well field pumpage, and node 132 lies within the cone of depression at the end of 10 years as simulated by ABC (Figure D-32).

2.5.3 Las Animas Creek

Las Animas Creek lies between ½ and 1 mile north of the production well field, and due to its close proximity to the project potential impacts to Las Animas Creek are of particular concern. Measurements of surface water flow along Las Animas Creek are limited. The best study was probably performed by Davie and Spiegel (1967). In that report, a discharge at Warm Springs North of about 363 gpm (0.8 cfs) was observed. Along Las Animas Creek north of the project area flows ranging from 180 gpm to 1,248 gpm were measured. Appendix F of the post-hearing submittal presents a single measurement of Las Animas Creek flow (245 gpm) made north of the production well field area. DBS&A believes that additional measurements should have been made along the creek (similar to what was done for Percha Creek), particularly in the Warm Springs North area.

Since the intent of the modeling study was only to simulate baseflow to Las Animas Creek, the model nodes used by ABC to simulate the Las Animas Creek alluvium begin at Warm Springs North and continue downstream to Caballo Reservoir. The model nodes that represent the Las Animas Creek alluvium are, in upstream (Warm Springs North) to downstream (Caballo Reservoir) order, 697, 677 680, 683, 686, 689, 692, and 695. Each of these nodes are linked in sequential order to route surface water flow, and each of these nodes are linked in the vertical direction to a node in model layer one that represents the Santa Fe alluvium beneath the Las Animas Creek alluvium. Each of these nodes has two other nodes attached to it to complete the triangular finite element, but the nodes not listed do not appear to have vertical connections with layer one of the ABC Copper Flats model.

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The simulated discharge at Warm Springs North, which is routed along the Las Animas Creek nodes as surface-water flow, is 1,819 gpm. This simulated discharge is about 5 times greater than that observed by Davie and Spiegel (1967). In addition, the adjacent downstream node (node 677) receives 118 gpm of surface water from Animas Gulch, which is simulated by model layer 1 nodes 225, 89 and 99. In other words, the model simulates 118 gpm of baseflow in Animas Gulch which does not exist according to available reports, and it then routes this flow into the Las Animas Creek alluvium.

An additional observation is that the simulation results do not appear to match the conceptual model of Las Animas Creek as presented in Appendix F of the post-hearing submittal. In that appendix (p. 11), the conclusion is presented that "The results of the surface water study for Las Animas Creek indicate that this stream is ephemeral with flows occurring as a result of snow melt and spring and summer thunderstorms." How ABC views Las Animas Creek conceptually is unclear, since it is simulated as a stream that receives a substantial amount of perennial baseflow beginning at Warm Springs North.

DBS&A's main concern with respect to Las Animas Creek is that if the simulated groundwater flow field does not match the observed conditions fairly closely, the accuracy of predictive simulations cannot be evaluated. A very important item missing from ABC's analysis is a water balance developed specifically for the Las Animas Creek alluvial aquifer. DBS&A also has not yet received the analysis of the second Las Animas Creek aquifer test.

In one respect, the simulation results may be conservative concerning Las Animas Creek because of the horizontal anisotropy applied to the Santa Fe alluvium. The larger hydraulic conductivity in the north-south direction, relative to the east-west direction, causes greater drawdowns beneath Las Animas Creek than would occur otherwise (i.e., a greater proportion of the pumped water is being derived from the north-south direction), all other model inputs being equal. However, additional aspects of the modeling approach seem nonconservative. For example, it appears that only the Las Animas Creek alluvium nodes used to simulate the creek itself are connected vertically to underlying model nodes that represent the Santa Fe Group aquifer. If this is true, the

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simulation results are nonconservative because the area across which downward leakage from the Las Animas Creek alluvium could occur is artificially constrained.

In addition to these concerns, the match between the simulated and observed water balance for the Las Animas Creek alluvial aquifer could not be determined. Near the western end of the alluvial aquifer simulated by ABC, between model nodes 677 and 680, the groundwater discharge for the valley was calculated by DBS&A from the simulation results to be about 770 gpm. ABC (p. 27) estimates that the "groundwater flow capacity of the shallow alluvium is . . . approximately 1,400 gpm." Simulated groundwater flow in the alluvium is not presented in the modeling study. The hydraulic conductivity of the alluvium applied by ABC in the model is 110,000 ft/yr (301 ft/d), which appears to be low relative to aquifer tests conducted at the Saladone Well and Shipping Pen Well on the Ladder Ranch where hydraulic conductivities as high as 476,000 ft/yr (1,300 ft/d) were calculated (Atkins Engineering Associates, Inc., 1992).

In addition, important groundwater flow components, such as groundwater inflow from the alluvium upstream of Warm Springs North and groundwater discharge by phreatophytes along the creek, are not presented or, apparently, considered in the analysis. An additional discrepancy is that, although ABC states that Las Animas Creek is ephemeral (Appendix F), the model simulates surface water flow at every Las Animas Creek model node in the model calibration run.

In summary, ABC has not provided sufficient information for DBS&A to make a sound determination concerning simulated impacts to Las Animas Creek. The aquifer test results provide very useful information, but the short duration of the tests relative to the proposed operational period eliminates the possibility of using the test results as a direct indicator of potential impacts. Potential impacts should be analyzed using modeling or other quantitative analysis, but the existing model appears inadequate for impact determinations at this time.

3. Analysis of Predicted Impacts to Surface Water and Groundwater Quality from Mill Tailing Disposal

Alta plans to expand the existing tailing impoundment, which was constructed during previous mining operations (DEIS, 1996). Potential water quality impacts to groundwater from the proposed tailing impoundments are a major concern to residents located downgradient of the facility. DBS&A has completed a review of the Alta submittal on the proposed tailing disposal operation. The following sections describe the major issues identified by DBS&A concerning (1) site characterization, (2) tailing disposal, and (3) seepage control. The layout of the impoundment structure and the location of nearby monitor wells are shown on Figure 3-1.

3.1 Site Characterization

Site characterization activities were completed in order to evaluate geotechnical properties of foundation soils (SHB, 1980) and potential seepage impacts to underlying groundwater from operation of the facility (SRK, 1995). The site was evaluated primarily through soil excavation, monitor well installation, and physical and chemical analysis of collected soil and groundwater samples. This section provides an evaluation of the hydrogeologic characterization.

3.1.1 Conceptual Model

Based on the available data, SRK developed a conceptual model for groundwater flow beneath the impoundment, which consists of two distinct groundwater zones that discharge toward the east (Figure 3-2). The first zone, referred to as the lower zone, consists of clayey and silty sands, and gravels of the Santa Fe Group. Groundwater within the lower zone is confined under a clay unit which, from its western edge, dips and thickens toward the east. The western edge of the lower zone is located approximately 200 feet west of the starter dam. The second zone, referred to as the perched zone, consists of recent alluvial materials located above the confining clay. The perched zone of groundwater flow was likely created during operation of the north cell in the early 1980s by Quintana Minerals (SRK, 1995).

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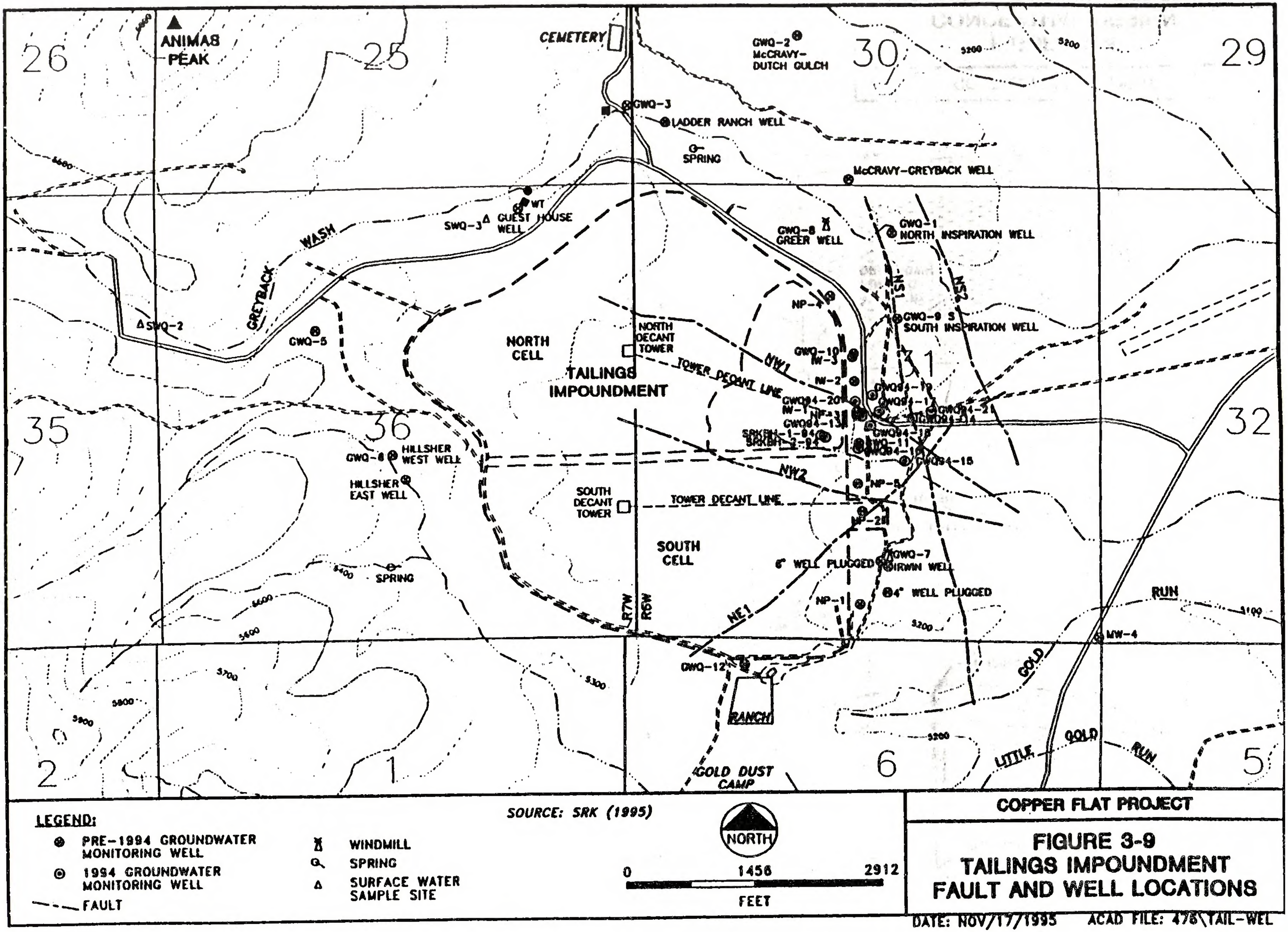
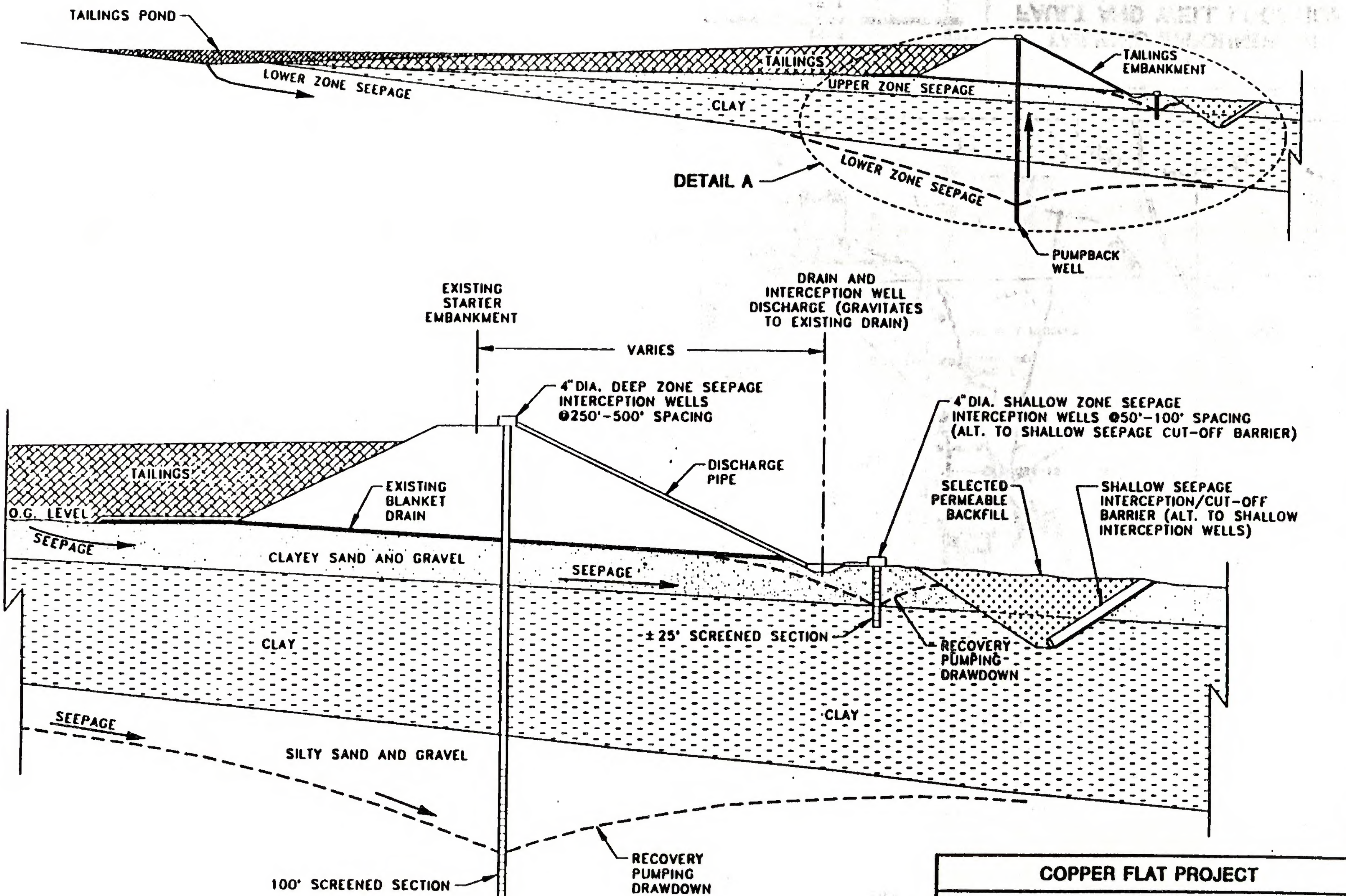


ANIMAS PEAK
CEMETERY
GWQ-2 McCRAVY-DUTCH GULCH
GWQ-3
LADDER RANCH WELL
SPRING
McCRAVY-GREYBACK WELL
SWQ-3
GUEST HOUSE WELL
WT
GREYBACK WASH
GWQ-8 GREER WELL
GWQ-1 NORTH INSPIRATION WELL
NS1
NS2
NP-4
GWQ-9
SOUTH INSPIRATION WELL
SWQ-2
GWQ-5
NORTH CELL
NORTH DECANT TOWER
TAILINGS IMPOUNDMENT
TOWER DECANT LINE
NW1
GWQ-10
IW-3
IW-2
GWQ94-20
GWQ94-13
GWQ94-19
GWQ94-17
GWQ94-21
GWQ94-14
GWQ94-18
GWQ94-15
NP-5
NW2
GWQ-6
HILLSHER WEST WELL
HILLSHER EAST WELL
SOUTH DECANT TOWER
TOWER DECANT LINE
NP-2
SOUTH CELL
GWQ-7
IRWIN WELL
6" WELL PLUGGED
4" WELL PLUGGED
NP-1
NE1
SPRING
R7W
R6W
RUN
MW-4
GOLD
GWQ-12
RANCH
GOLD DUST CAMP
LITTLE GOLD RUN
26
25
30
29
35
36
31
32
2
1
6
5
5200
5300
5400
5600
5700
5800
5900
SOURCE: SRK (1995)
LEGEND:
PRE-1994 GROUNDWATER MONITORING WELL
1994 GROUNDWATER MONITORING WELL
FAULT
WINDMILL
SPRING
SURFACE WATER SAMPLE SITE
NORTH
0
1456
2912
FEET
COPPER FLAT PROJECT
FIGURE 3-9
TAILINGS IMPOUNDMENT
FAULT AND WELL LOCATIONS
DATE: NOV/17/1995
ACAD FILE: 478\TAIL-WEL

Figure 3-1



SOURCE: SRK (1995)

DETAIL A

COPPER FLAT PROJECT

FIGURE 2-4
CONCEPTUAL DESIGN
TAILINGS SEEPAGE CONTROL

DATE: NOV/1/1995 ACAD FIL \SEEP-CON

Figure 3-2

DBS&A concurs with the conceptual hydrogeologic model developed for the tailing impoundment area. However, DBS&A believes that SRK has over simplified the isolating properties of the clay layer that separates the two groundwater systems. Based on lithologic descriptions, it is evident that the clay unit has a highly variable thickness and composition. Further, significant heterogeneity exists below the impoundment, as evidenced by lithologic variations, the presence of a paleochannel filled with gravels and basalt, and the presence of numerous faults in the area. On a regional scale, zones of contrasting permeability, such as those caused by faulting or paleochannels, may have little effect on the flow system. However, on a local scale these heterogeneities can allow preferential groundwater flow beneath the impoundment that could result in widespread groundwater contamination, if not properly addressed by a seepage control system. Thus, the importance of understanding the hydraulics in these zones is critical to capturing seepage.

3.1.2 Aquifer Characteristics

In 1994, SRK performed a pumping test downgradient of the starter dam to determine aquifer characteristics for the lower zone (SRK, 1995, Appendix G). The test consisted of pumping the lower zone monitor well GWQ94-17 for approximately three days and measuring the aquifer response in 13 monitor wells located along the eastern edge of the starter dam. Based on the test, SRK determine representative hydraulic parameters for the lower zone and concluded that the upper and lower zones are not hydraulically connected.

The pumped well and most of the monitor wells are completed in the upper 25 percent of the lower zone aquifer as currently defined (i.e., screened from roughly 100 to 150 feet below ground surface [bgs]). Drawdowns on the order of 5 to 10 feet were measured in several of the observation wells completed within the lower zone. The deep nested well containing monitoring well GWQ94-21A (screened from 213 to 263 feet bgs) and GWQ94-21B (screened from 285 to 315 feet bgs) both experienced drawdowns on the order of 6.5 feet. The pump test demonstrated that the lower zone aquifer is well connected within the 200-foot section of aquifer currently monitored.

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The test data were analyzed using the standard Theis solution for homogenous and isotropic conditions within a confined aquifer (Theis, 1935). Transmissivity (1,400 gpd/ft) and storativity (1.1×10^{-4}) values calculated from the test appear to provide an adequate characterization of the hydraulics near the toe of the starter dam. However, the analysis could have been improved by correcting for the effects of partial penetration.

The lower boundary of the flow system has yet to be determined through either hydraulic response during the pumping tests or observed lithologic changes. DBS&A believes that the vertical extent of the lower zone has been poorly defined. The vertical extent is an important issue because it affects subsequent calculations for seepage containment and water quality protection presented in the application materials.

The problem arises from how hydraulic conductivity (K) is determined and used throughout the SRK analysis. It appears that unsaturated hydraulic conductivity was determined by dividing the average transmissivity value determined from the pumping test by the average screen portion of the aquifer (40 feet); this resulted in an unsaturated hydraulic conductivity value of 1,700 ft/yr (SRK, 1995; Table 4-2). At first glance, this procedure appears to provide an unsaturated hydraulic conductivity value that is conservatively high for groundwater flux calculations within the upper portion of the confined aquifer. However, throughout SRK's analysis the average unsaturated hydraulic conductivity is multiplied by inconsistent aquifer thicknesses such as when performing groundwater flux or chemical mixing calculations. For example, as discussed in Section 3.3, a 200-foot-thick aquifer was used for post-closure mixing calculations and a 40-foot-thick aquifer was used for pump-back system design.

SRK's interpretation that the perched upper and the lower confined groundwater systems are hydraulically disconnected is not well supported by existing data. Wells completed within the perched zone are located at least 250 feet away from the pumping well. Additionally, the rate of pumping was insufficient to reverse the hydraulic gradient across the clay unit in order to allow seepage from the perched zone into the lower aquifer. Under static conditions the hydraulic head within the lower zone is 10 to 15 feet higher than the hydraulic head within the perched zone, and much more would be required to effect greater than 10 feet of drawdown at the perched observation wells located at least 250 feet from the pumped well. Thus, at least 10 feet of drawdown in the lower zone would be required to allow vertical seepage of water from the perched zone. This condition was not established during the pumping test.

Further, monitor well NP-3, completed at a total depth of 79.3 feet bgs, would be considered a shallow well as defined by SRK (1995, Section G.3.4). However, this well maintains a static head representative of lower-zone head conditions. During the test, over 14 feet of drawdown were measured in NP-3, indicating that the hydraulic separation provided by the clay unit is much less near the center portion of the starter dam (Figure 3-1).

3.2 Management of Tailing Operation

The tailing impoundment design details and proposed operation are described primarily in the DEIS (1996; Chapter 2), SRK (1995, Section 7, and Appendix G) and Volume I of the permit application (199_). This section provides a review of Alta's proposed operating procedures and the integrity of the tailing impoundment.

3.2.1 Tailing Impoundment Operation

Information provided in the permit application materials describes operation of the tailing impoundment in rather general terms. This section summarizes the proposed operation and discusses (1) design and operational issues not addressed fully in the application materials and (2) past operational problems and potential solutions.

The proposed design has the following specifications:

- Approximately 69 million tons of tailing are proposed to be impounded within a 454-acre area.

- The starter dam is about 6,600 feet long and 50 feet high, and will be raised in five additional 30-foot raises. The tailing will be raised by upstream construction. Cycloning is proposed to separate the coarser particles from the whole tailing along the upstream beach of the impoundment.

- Drainage will be away from the dam, and surface runoff in and around the tailing will be contained by the impoundment. Surface water pooled in the impoundment will be removed using a pump barge and pumped back to the plant as process water.

- Tailing will be deposited at approximately 16,300 dry tons per day at a density factor of approximately 50 percent solids by weight. During tailing settlement, water will be decanted from the tailing impoundment and returned as process water.

3.2.1.1 Design and Operational Issues

The upstream method of tailing placement has been successful at mines throughout the world. Nonetheless, DBS&A believes several design issues, including grain-size separation, tailing compaction, and water management, must be addressed in greater detail prior to proceeding with the project. Proper placement of the tailing within the impoundment requires details on how the tailing will be distributed and managed during the operation of the facility. Poor depositional practices can result in structural weaknesses if the dam walls are built on partially unconsolidated slimes.

The application should provide a management plan that includes (1) an estimate of the grain-size distribution expected to be delivered from the mill, (2) a discussion on how tailing will be placed in order to maximize the segregation of material required for dam stability, (3) the anticipated width of the zones for sands, transition sands to slimes, and slimes for each lift on the impoundment, (4) how deposition will control drainage and collection of seepage to avoid excess pore pressures within the impoundment, and (5) how tailing deposition will be managed to force the pool away from the embankment. Lateral variations in permeability are controlled by the range of particle size, the pulp density, the spacing of spigots, and the location of the decant pond (Vick, 1983). These details are lacking in the permit application materials.

Grain-size distribution data obtained from previously deposited tailing indicate that significant fines are present within 150 feet of the starter dam (SRK, 1995, Appendix O). High percentages of fines can prevent the beaches from draining freely. More data should be collected to determine the existing distribution of grain sizes within the north cell in order to determine if previous placement practices were sufficient.

Compaction is proposed to occur naturally by the placement of additional tailing on each lift. DBS&A believes that more analysis may be required to evaluate compaction requirements for the impoundment. Little information is given to support the statements by SRK that the tailing used to form beaches will have an saturated hydraulic conductivity of 10^{-6} cm/sec after consolidation. In fact, the limited data collected to date do not support this conclusion (SRK, 1995, Appendix O). The saturated hydraulic conductivity values for the beaches are more on the order of 5×10^{-5} cm/sec, although only partial consolidation may have occurred because of the minimal volume of existing tailing. Also, the anticipated frequency for cycling deposition between the north and south cells to consolidate tailing should be stated in the operational plan.

3.2.1.2 Past Operational Problems and Potential Solutions

Several operational problems were identified during the initial operation of the north cell by Quintana (Section 7 of the Hydrogeologic Study). The primary problem involved poor drainage of water through the drain pipe located at the downstream toe of the impoundment. The poor drainage resulted in flow through the toe of the structure and a significant increase in water levels in downgradient monitor wells. In conjunction with the water level rise, an increase in total dissolved solids (TDS) and sulfate concentrations downgradient of the dam was observed, particularly in monitor well NP-3.

Once tailing are deposited in the south cell, SRK seepage flows through the toe of the dam unless measures are taken to intercept the flow. SRK offers little information on potential methods to intercept the seepage through the toe. This is a critical issue for dam stability and should be addressed in detail prior to placement of tailing within the impoundment. SRK states that planned measures to prevent the problem include routing and intercepting flow; however, detailed measures are not described in the permit application.

SRK has not seriously considered the use of liner and drainage systems to prevent drainage through the toe of the structure and subsequent degradation of groundwater quality. DBS&A agrees that lining the north cell would be difficult where tailing is already deposited. However, compaction and consolidation of the existing tailing could possibly reduce the threat of differential settling. SRK has presented a potentially biased evaluation of seepage control technology by favoring the low cost, unlined option.

3.2.2 Tailing Stability Considerations

At the proposed rate of tailing placement, the anticipated elevation rise is expected to be approximately 15 to 20 feet per year. At this rate, the slimes can potentially exist in an unconsolidated state with high excess pore pressures. The application documents provide minimal details on how the phreatic surface within the impoundment will be controlled during the course of operation. DBS&A believes that a documented analysis of the phreatic surface control must be provided as an integral part of the permit application. As documented at other tailing operations, poor phreatic surface control can result in dire consequences.

The areas within the impoundment underlain by poorly sorted sediments appear to be suitable as foundation material. However, the SHB investigation (SHB, 1980) offers no discussion on the suitability of the underlying clays for foundation material. Results of the in situ permeability test conducted by SHB indicate that the unsaturated hydraulic conductivity values for the foundation materials range from roughly 10^{-5} to 10^{-4} cm/sec. The unsaturated hydraulic conductivity values for the underlying foundation material indicate that seepage through the foundation material could potentially result in unsafe pore pressures within the tailing impoundment. Again, measures for control of the phreatic surface within the tailing impoundment should be provided in greater detail.

Since the blanket drain appears to be undersized (Section 3.2.1), it is critical to know the permeability characteristics of the starter dam. A low permeability starter dam, as described in the SHB report (SRK, 1995, Appendix G), can result in seepage through the slope above the starter dam once the tailing elevation is raised. More attention should be given to potential seepage through the downstream slope of the starter dam and the potential effects on the stability of the structure.

Based on the available data, the proposed design appears to meet acceptable design criteria with the exceptions noted in this section. For successful upstream construction, sufficient volumes of sand must be available to maintain the dike ahead of the rising pond level. Spigotting must be managed to allow free drainage of the sands to maintain a low phreatic surface for dam stability. The tailing management plan should specify how the permeability ratio between the sands and slimes can be maintained at 100:1 (Vick, 1983) in order to ensure a depressed phreatic surface at the starter dikes.

3.3 Control of Seepage from the Tailing Impoundment

DBS&A has reviewed the materials submitted by Alta for containment of seepage. It is apparent from this review that Alta has biased its analysis toward containment of seepage through the use of a pumpback system. The proposed approach to seepage control has been criticized by the NMED GWB, environmental groups, and local residences. Never the less, DBS&A believes that a well designed and maintained pumpback system can contain the seepage of poor quality water within the aquifer; however, there are certainly risks of not achieving complete containment.

The level of analysis should be commensurate with the potential impacts to the high quality groundwater resource. DBS&A believes that a more detailed analysis of containment options should be provided by Alta. The analysis should include a more detailed evaluation of the range of potential seepage rates that may occur throughout the impoundment area, a cost benefit analysis for the use of liner alternatives, and more design details for the proposed pumpback system. In addition, SRK has stated that abatement plans would be implemented should leakage be detected; however, the abatement plans are not provided. Prior to initiation of facility operation, these plans should be described at least in general terms.

The following sections provide a summary of Alta's proposed plan for water management and seepage control, and DBS&A's comments concerning areas that require further clarification within the submitted permit materials.

3.3.1 *Seepage Rate Estimates*

The anticipated volume of water used at the mine roughly reveal the amount of seepage from the tailing piles. The overall water balance (i.e., mill discharge, decanted water, evapotranspiration, seepage, etc.) defines the ultimate potential impacts to groundwater. Over the life of the mine, mill water discharge can be calculated based on the anticipated mill tailing tonnage and the pulp density. A detailed water balance for the proposed operation has not been completed.

DBS&A attempted to reconcile the water balance, based on available information, as follows:

- Given on the anticipated discharge from the tailing thickener, approximately 2,800 gallons per minute (gpm) of water will be required to slurry the tailing to the impoundment. Concerning natural inputs, SRK reports that the impoundment is designed to contain the "normal" operating volume of water and the surface water runoff from the 24-hour, 100-year precipitation event.

- SRK has estimated that seepage into the underlying groundwater system will account for approximately 50 percent or 1,400 gpm of the water output. SRK has proposed a design to recover 1,400 gpm of water through a pumpback system. Rather than basing the seepage estimate on hydraulic properties the tailing or underlying materials, SRK used a typical saturated hydraulic conductivity value for a clay/silt mixture of 5×10^{-6} cm/sec, a vertical hydraulic gradient of unity, and a cross-sectional area equal to the final tailing impoundment (SRK, 1995, Appendix L).

- The draft EIS stated that roughly 70 percent of the total water input to the impoundment would be recovered. Since 50 percent will be recovered by the pumpback system, the remaining 20 percent would probably be recovered through the surface pond decant system.

- DBS&A assumes that the remaining 30 percent would be lost through evaporation and/or storage of water within the tailing.

3.3.2 Seepage Pumpback Design

Water from the tailing is designed to drain at a maximum rate of 1,400 gpm to the underlying materials in order to enhance tailing consolidation, ensure stability of facility, and expedite reclamation of the facility (SRK, 1996). The proposed seepage collection system consists of a series of collection wells and trenches designed to address seepage (1) beneath the starter dam, (2) perched above a clay layer present at roughly 20 feet bgs, and (3) within deep groundwater confined beneath the clay layer. Figure 3-2 shows SRK's conceptual design for seepage control for the tailing impoundment.

Several sections of the hydrogeologic report provide supporting calculations for the proposed pumpback system for the deep groundwater zone (SRK, 1995). DBS&A has identified several problems with both the analytical and numerical analyses use to design the proposed collection system. Within the application materials, seepage collection for the upper perched zone is not discussed in no more detail than what is given above. A discussion of the problems and potential ramifications of the analyses for the deep zone collection system are provided below.

3.3.2.1 Analytical Solution

The basic design criteria for the deep zone collection system were based on the use of Darcy's law to evaluate seepage flux

$$Q = TIW$$

Where T = Transmissivity, 1,400 gpd/ft (based on GWQ94-17 pump test results)

I = Hydraulic gradient, 0.15 (based on final elevation of pond and current water level elevation in monitor wells east of impoundment)

W = Aquifer width, taken as length of starter dam

Based on the above parameters, SRK calculated that the maximum groundwater flux through the lower zone at 1,500 acre-feet per year (950 gpm). SRK then estimated that the operation of 20 extraction wells spaced at approximately 300 feet and pumping at an average rate of 50 gpm satisfies the seepage containment requirements. The preliminary specifications for well design are as follows:

Well spacing:	250 to 500 feet
Well diameter:	4 inches
Well depth:	150 to 200 feet
Screened section:	From bottom of clay to total depth
Flow rates:	10 to 50 gpm

DBS&A's comments concerning the well specifications proposed by SRK include selecting well screens based on anticipated yields to avoid the excessive well inefficiencies observed during the GWQ94-17 pumping test, and better defining vertical mixing of seepage within the lower zone. Estimation of resulting water quality should be modeled in order to determine more realistic completion depths for the pumpback wells.

In addition, the seepage collection system was sized without conducting a sensitivity analysis that could be used to anticipate the potential range of groundwater fluxes through the deep system. During the period of operation, the groundwater flux calculations result in less water moving through tne system than the assumed rate of seepage recharge to the lower zone. The calculations and well design specifications assume that the seepage will completely mix within the upper 50 to 100 feet of the lower zone system. While this is probably true over the short distances assumed here, no justification was presented. DBS&A believes that the pumpback system may need to pump significantly more than the rate projected by SRK given the high degree of uncertainty regarding seepage rates, particularly early in the operation before the natural media are plated with tailing.

3.3.2.2 Numerical solutions

The revised Appendix L (ABC, 1997) presents an evaluation of the performance of the proposed pumpback system. The model is based on a sub-domain of the regional flow model developed for evaluation of the Copper Flat groundwater flow system (ABC, 1996, 1997). The following comments address the modeling effort.

The hydraulic conductivity values used in the numerical modeling are poorly supported. For example, it is unclear on what basis the unsaturated hydraulic conductivity values were divided into the three zones shown on Figure 3-1 (Figure 2 in Appendix L). Local hydraulic conductivity data do not support this zonation. For example, based on SHB's geotechnical evaluation (SHB, 1980), the borings advanced within the impoundment footprint had similar lithologic character as the sediments east of the starter dam to the investigated depth of 75 feet bgs, suggesting there is little basis for varying hydraulic conductivity in the lower zone. Since there is no obvious difference in the grain-size distribution throughout the lower zone based on available well logs, it seems the model would be best run with actual measured local values of hydraulic conductivity determined from the pumping test conducted immediately downgradient of the starter dam. However, the area west of the impoundment should be assigned a lower unsaturated hydraulic conductivity based on the andesite present at the GWQ94-6 location.

Since the saturated hydraulic conductivity value calculated for the pump test area is the highest value, the outcome of using the higher hydraulic conductivity will show that a greater flux of water is moving through the lower zone. It is not clear why the local model was constructed with the regional saturated hydraulic conductivity value of 1,000 ft/yr instead of the calculated average saturated hydraulic conductivity from the pumping test of 1,700 ft/yr (SRK, 1995, Table 4-2). In the eastern zone of the model, saturated hydraulic conductivity is set at a value of 1.5 ft/yr, which corresponds with the regional model, but there are no local data to support this abrupt reduction in hydraulic conductivity east of the tailing impoundment. The overall effect of the use of regional saturated hydraulic conductivity values in the local tailing-dam-area model is to underestimate the total flux of groundwater underneath the impoundment.

There appears to be a discrepancy between the proposed depth of wells to be used in the pumpback system and the modeled depth of the aquifer. A definitive bottom of the aquifer has yet to be determined; therefore, it is unclear why the modeled aquifer thickness was set at 200 feet.

Concerning model boundary conditions, the west boundary was fixed at 5,260 ft msl in order to create a head of 5,180 ft msl at the toe of the dam, as represented by GWQ94-17, the calibration point. Measured heads are roughly 100 feet higher in the western portion of the modeled area (see Figure 3-11 in the DEIS). In addition, measured heads in monitor well GWQ-5, located northwest of the impoundment, are higher than the head along the western edge of the model domain. For the eastern boundary, ABC set the head lower than in the regional model because the material underestimates the transmissivity in Zone 3. These boundary conditions should be adjusted to more accurately reflect the measured heads in the area.

During the operational period, the pumpback system maintains a depressed groundwater elevation of 100 feet below the static water level, or 5,080 ft msl immediately downgradient of the impoundment. Based on the assumed head field, the model determined that 1,411 gpm or 6 gpm more than the assumed seepage from the tailing would be recovered. This figure was provided as verification that complete capture would be realized. DBS&A believes this is an optimistic simplification of seepage capture within the lower zone, although the collection wells will certainly pump more water than what is naturally flowing through the groundwater system.

Since the estimated maximum seepage rate was based on a rather optimistic estimation of the average hydraulic conductivity of the tailing (5×10^{-6} cm/sec), a more conservative approach would be to design the system so that all of the water applied to the tailing impoundment and a percentage of the natural groundwater flux would be captured by the system such as might occur during early time before the surface expression of the lower aquifer in the western portion of the

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tailing pond is plated with lower permeability tailing materials. Using this criterion, the system should be designed to pump up to roughly 3,000 gpm.

3.3.3 Post-Closure Seepage Control

3.3.3.1 Transient Drainage

Tailing placement will likely result in interlayered sand lenses and slime zones between the embankments and the final decant pond location. Within the impoundment, the process of moving upstream after each starter embankment will result in less sand and more fines with depth. Once the tailing slurry is no longer supplied, a period of transient drainage will occur in which fluid stored in the tailing will drain primarily by gravity, and to a lesser extent as a result of consolidation. During the transient phase, the rate of seepage depends on the moisture retention characteristics and the layering of the tailing. Eventually, tailing seepage will reach steady-state conditions, in which flow from the base of the impoundment will be equal to the net infiltration of precipitation on the surface of the impoundment.

For the post-closure period, ABC assumed a 5 percent drainable porosity for the tailing material without any supporting rational or documentation on how transient drainage was simulated in the model (Appendix L). The drainable porosity assumption likely results in an overprediction of the final volume of water stored permanently within the tailing. Estimates of drainable porosity, or field capacity, can be easily determined by a laboratory for the material present within the tailing. Based on typical moisture retention characteristics for soils, DBS&A believes that the average drainable porosity could easily be within the range of 10 to 15 percent. Thus, the predicted transient seepage through the tailing may be significantly underestimated.

DBS&A believes that the model prediction of achieving a steady-state seepage rate of 13 gpm within a 10-year period may be overly optimistic (Appendix L, Figure 7). Typically, transient drainage from tailing impoundments requires anywhere from the 10 years (predicted by ABC) to 50 years or more (Vick, 1983). The model should be used to simulate when the pumpback system could cease operation based on a range of predicted drainable porosities and permeability characteristics for the tailing material. In conjunction with mixing calculations, the possible periods required to operate the pumpback system can be more conservatively estimated. The operational costs for various post-closure scenarios should be factored into any performance bond requirements.

The DEIS (p. 3-28) states that the existing tailing have completely drained since operation in 1982 by Quintana. However, water level measurements in shallow monitor wells downgradient of the dam continued to rise from 1985 to the present. The fact that water levels have not lowered from the original placement of tailing in the north cell suggests that transient drainage has been occurring since the bried period of operation in 1982, or approximately 15 years. In addition, soil samples collected from the tailing suggest that near saturated conditions still exist at the base of the tailing (Parshley, 1995, DEIS p. 3-28). If steady-state drainage conditions had been achieved, the mixing within the aquifer should by now have resulted in a decrease in sulfate and TDS concentrations below the New Mexico Water Quality Control Commission (WQCC) standards in the downgradient monitor wells: this has not occurred.

In a related matter, the application documents do not effectively address disposal of pumpback water during the post-closure period. The current plan involves a combination of routing the extracted water into the tailing impoundment and/or the pit lake for evaporation. Supporting analysis is required to determine if the application of water to the surface of the impoundment will prolong operation of the pumpback system and/or increase the likelihood of leaching sulfides into the underlying groundwater. DBS&A is not aware of any attempt to determine if the pit lake water disposal alternative is practical with respect to the estimated pit lake water balance. DBS&A recommends that the disposal of post-closure water be addressed in greater detail in the application documents.

3.3.3.2 Water Quality Impacts

As stated by SRK, groundwater impacts will depend on post-closure seepage rates, concentration of inorganic constituents in naturally occurring groundwater, and rate of groundwater flow. SRK has identified elevated sulfate and TDS concentrations as the greatest potential threat to groundwater quality. Water within the tailing is expected to have an essentially neutral pH and sulfate concentrations in the range of 600 to 2,000 mg/L. Seepage of tailing water is expected to enter the regional flow system primarily through the zone approximately 200 feet west of the starter embankment, where the underlying clay unit is absent (SRK, 1995, hydro study p. 4-21).

For the impoundment watershed, SRK predicts that the average recharge rate will likely be 1.55 inches per year. Based on the recharge rate, SRK believes that sufficient groundwater flow is available to dilute any long-term impacts related to steady-state seepage from the impoundment (SRK, 1995, p. 4-23, hydrogeo study). Post-closure seepage was estimated by SRK at 32 acre-feet per year (20 gpm) based on an ultimate tailing area of 380 acres and an average recharge rate of 1 inch per year. DBS&A feels that the application materials may portray an overly short duration that the pumpback system must be operated during the post-closure period and an optimistic projection for the reduction of seepage impacts within the regional groundwater flow system.

Based on the hydraulic characteristics of the groundwater system, the sulfate concentrations in GWQ94-17, located 250 feet downgradient of the impoundment, may represent the leading edge of a growing sulfate plume. In order to evaluate the plume, a better understanding of the background concentration of sulfate within the regional groundwater system should be determined. SRK's assertion that sulfate concentrations decrease rapidly away from the dam, while true, are overstated because several of the wells used to establish background concentrations are immediately downgradient of the sulfate plume and may have been impacted to some, albeit minor, degree. Sulfate concentrations less than 100 mg/L have been measured in several groundwater monitoring locations nearby.

DBS&A believes that the extent of vertical mixing of seepage within the regional groundwater system should be evaluated more rigorously. If seepage immediately mixes throughout the 200-foot aquifer thickness, as described by SRK, DBS&A agrees that water quality impacts will be negligible (i.e., WQCC standards for pH, sulfate, and TDS will not be exceeded). However, DBS&A believes that the more likely scenario is that complete assimilation of the poor quality water within the regional flow system will not occur before the water flows downgradient of the impoundment. Slow mixing will likely continue to result in exceedance of WQCC standards within the upper portions of the regional flow system, as evidenced by the degradation of water quality observed in monitor wells immediately downgradient of the impoundment.

At a minimum, post-closure seepage impacts must be calculated consistently with respect to the groundwater containment calculations used to size the pumpback system. Whereas a 200-foot-thick aquifer was used for post-closure mixing calculations (SRK, 1995), a 40-foot-thick aquifer was assumed for the purpose of collection system design. Using the same 40-foot saturated thickness for the lower groundwater zone as used in the seepage capture calculations the sulfate concentrations will exceed WQCC standards. Consequently, the dilution of sulfate may well be overestimated in the application materials.

DBS&A believes that a properly designed and maintained pumpback system can control the migration of poor quality water within the regional aquifer. However, the post-closure period for pumpback system operation has been unrealistically estimated, given the available data and level of analysis. An analysis of sulfate plume stability downgradient of the starter dam should be incorporated into the estimates of how long the pumpback system will be operated during the post-closure period.

3.4 Data Gaps in the Assessment of Tailing Impoundment Performance

Based on DBS&A's review of relevant documents, we have identified the following areas where additional data and analysis are needed for the assessment of tailing impoundment performance:

- Baseline (background) conditions for inorganic constituents appear to be based on newly installed (1994) wells located downgradient of the impoundment. These wells may already be impacted by seepage through the tailing.

- The extent and hydraulic characteristics of the clay that separates the perched and lower aquifers have not been fully determined, adding uncertainty to seepage estimates.

- The hydraulic conductivity of native material beneath the proposed location for tailing should be determined in greater detail.

- It may be appropriate to rerun the numerical model using saturated hydraulic conductivity values determined from local aquifer tests.

- Design issues, including grain-size separation during deposition, tailing compaction, and water management, must be addressed in greater detail.

- Methods for intercepting seepage through the toe of the starter dam should be addressed.

- Documented analysis of phreatic surface control within the impoundment is needed.

- A sensitivity analysis of the range of potential seepage rates and potential impacts on containment should be completed.

- Vertical mixing of seepage within the lower zone should be modeled in order to determine more realistic completion depths for pumpback wells.

- The period of transient drainage should be evaluated using a range of drainable porosities and permeability characteristics.

- A more detailed analysis should be made to determine how pumpback water will be disposed of during the post-closure period.

3.5 Tailing Impoundment Stability

According to Section 3.2.4.1 of Volume 1 of the permit application, the existing starter dam for the tailing impoundment is 6,600 feet long and 50 feet high. Also, Section 3.2.4.1 states that the starter dam will be raised in five raises of 30 feet each. However, on p. 4-22 of Volume 1, the starter dam is described as being 60 feet high and being raised by four additional raises of

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30 feet each, with 30 feet wide setbacks. The tailing stability analysis performed by SHB in Appendix G, states that the tailing dam will be raised at a rate of about 15 feet per year during the initial stages of deposition. Also, the total height of the dam as computed based on information in different sections of the report, varies. For example, according to Section 3.2.4.1, the full height of the dam is 200 feet; according to p. 4-22, the full height is 180 feet; and Appendix G explicitly states that the full height is 225 feet. These contradictions in dam height need to be resolved.

A review of Appendix G (Stability Analysis of the Tailing Impoundment) yielded no information regarding the procedure for the evaluation of the tailing dam stability. Most of the report by SHB pertains to the possible locations of the phreatic surface and their effects on the factor of safety with respect to stability. It mentions that stability analyses was performed on the tailing dam. However, no details are provided regarding what computer program was used, if any, to analyze stability. Hence, it provides no information on the specific type of procedure used for the analyses, whether it was the Spencer's procedure, Bishop's, or some other procedure that was used.

Appendix G also does not provide any information on the input parameters used (unit weight, cohesion, and friction) and how those input parameters were obtained. Furthermore, no information is given on how the phreatic surface is modeled in the program and whether drained or undrained analyses were performed. Assuming undrained analyses were performed, how were the saturation conditions during construction simulated? Also, was the stability of the tailing dam evaluated under seismic, or at least under "pseudo-static" conditions? Finally, how representative are the input parameters used and what is the level of confidence in using these parameters? Was a sensitivity analysis performed by varying the strength parameters in case there are any uncertainties in the parameters chosen? These are all critical factors impacting the results of a stability analysis and they need to be addressed. In addition, there are several operational considerations that may impact the stability of a tailing impoundment dam constructed by the upstream method. These considerations are discussed as follows.

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According to Vick (1983), a reasonably competent beach for support of the perimeter dikes is central to the application of the upstream embankment construction method. Generally, the sand content in the discharged whole tailing must not be less than 40 to 60 percent. Also, the location of the phreatic surface is a critical element in determining embankment stability. It should be noted that for upstream embankments constructed by tailing spigotting, there are few structural measures for control of the phreatic surface within the embankment.

The most important factors influencing the location of the phreatic surface are the permeability of the foundation relative to the tailing, the degree of grain-size segregation and lateral permeability variation within the deposit, and the location of the ponded water relative to the embankment crest. It appears from a review of Section 3.2.4 (Tailing Impoundment) of the permit application (SRK, 1996) and Appendix D that the surface water pooled in the impoundment would be removed rapidly by using barge-pumps, and the edge of the pond will be maintained at a distance of 714 feet from the dam crest in order to meet the requirement for a minimum of 5 feet of freeboard. Also, the combined use of the coarser particles from the spigotting process in the construction of the embankment raises and the gravel-filter-fabric underdrain will probably be adequate to maintain a low phreatic surface and ensure stability.

However, a permeability ratio of at least 100 is recommended between the coarser grained materials used for the embankments and the beach, and the fines and slimes further upstream. In addition, proper spigotting and pumping operations are recommended for maintaining an adequate separation distance between the pond and the embankment crest. Most failures of upstream embankments can be attributed to inadequate separation distance between the decant pond and the embankment crest (Vick, 1983).

3.6 Waste Rock Disposal Area Stability Analysis

It appears from a review of Appendix F-1 (Waste Rock Disposal Area Stability Analysis) that the procedure utilized for the stability analysis of the waste rock stockpiles is adequate. However, there appear to be some contradictions regarding the input parameters mentioned in the Technical Memorandum dated February 7, 1996, (from Mathew Culpo to Pete Kowalewski of

SRK) and the values used in the input for the PCSTABL5M program. For example, the moist unit weight for both unoxidized waste rock and natural ground is cited to be 120 pcf. However, in the PCSTABL5M output for file FLAT5E.OUT, the unit weights for unoxidized waste rock and natural ground are given as both 120 pcf and 140 pcf. In addition, the source(s) for the input parameters for the three soil types used needs to be referenced or explained. The input parameters include moist unit weight, saturated unit weight, friction angle, and cohesion. In addition, the basis for estimating the phreatic surfaces for the waste rock stockpiles needs to be explained.

Nevertheless, aside from the aforementioned uncertainties, it appears that the waste rock stockpiles are stable for both the static and the pseudo-static analyses.

3.7 Design of Run-On Diversion Structures

The analysis and design of the run-on diversion structures (as address in Volume 2, Technical Design Documents Part 1, Appendix D) seems to be adequate. All diversion ditches are designed to handle runoff from 100-year, 24-hour storm while maintaining a minimum of 6 inches of freeboard. However, one potential concern is that a design using 100-year, 24-hour storm may not provide the most conservative peak flows for designing the surface water ditches. More frequent, shorter duration, and more intense storms could have peak flows higher than the more infrequent, longer duration, and less intense storms. For example, a 10-year, 6-hour storm event could cause higher peak runoff. The ability of diversion ditches to handle a variety of shorter duration, more intense storms should be verified. This is especially significant since approximately 40 to 50 percent of the total precipitation of 13 inches per year is in the form of short but intense summer storms (Section 2.3 of Volume 1).

The other issue, though relatively minor, is that no references are cited in the Technical Memorandum included in Appendix D for the SCS curve number of 72 that is used for all the catchment and subcatchment areas. It would be helpful if the source or the basis for using this curve number were explained. It should be noted that it is assumed in the design of the diversion channels that the peak runoff from all the subcatchment areas are simultaneous, when, in reality, they should differ by the travel time from each of the subcatchment areas. However, this assumption is a conservative assumption with respect to determining peak flow in the ditches.

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4. Analysis of Predicted Impacts to Surface Water and Groundwater from Acid Rock Drainage from Mine Stockpiles

The areas addressed in this section include the potential for acid rock drainage, post-mining pit water chemistry, and geochemical modeling using MINTEQA2 and PHREEQE. The final section presents general and specific comments from the technical review by the SRK documents.

4.1 Potential for Acid Rock Drainage

Acid rock drainage (ARD) is a common problem that results when sulfide ore bodies are mined, such as the Copper Flat deposit. Acid can be generated either directly through oxidation of primary sulfide minerals, such as pyrite, or during re-dissolution of secondary acid sulfate minerals, such as chalcanthite. Both classes of minerals are present at Copper Flat. Primary sulfide minerals present in the quartz monzonite stock include pyrite, chalcopyrite, molybenite, galena, and sphalerite (Dunn, 1982), and all of these, except galena and perhaps molybenite, have the potential to produce acid during the weathering process (Jennings and Dollhopf, 1995).

Secondary precipitates of blue-green copper sulfate have formed around the south side of the pit during periods of dry weather as a result of evaporation (Photographs 1 and 2). According to SRK (Bowell, 1997), these salts consist primarily of the mineral chalcanthite, some of which may be the result of past in situ leaching attempts to recover copper (Photograph 3; SRK, 1995, Section 5.2.6). However, copper sulfate salts may also occur naturally in the vicinity of porphyry copper deposits, and it is unclear how much of the salts at Copper Flat may be natural.

While the total volume of chalcanthite and other sulfate salts is probably small compared with the primary sulfide minerals present, these salts are quite soluble and tend to be concentrated along exposed soil and rock surfaces where they are susceptible to dissolution by incident precipitation and surface runoff. The equilibrium pH of a solution of rainwater and chalcanthite is in the range of pH 4 to 5. Therefore, re-dissolution of sulfate salts during storm events can probably account for the low pH observed in shallow pools that periodically form around the perimeter of the pit (Bowell, 1997), and some of this water ultimately finds its way into the pit lake.

In order to evaluate the ARD potential, SRK has performed a series of laboratory tests and computer modeling exercises, as described in the following sections. The static and kinetic test methods employed by SRK are standard for the analysis of ARD potential at mine sites and are generally accepted as the current state of the art. The following comments will focus on information presented in SRK (1995; 1996), and SRK's interpretation of the test results.

4.1.1 Paste pH and Static Tests

A total of 19 waste rock samples and two tailing samples were collected by SRK in June 1994 for analysis of paste pH and acid generation potential (SRK, 1995, Table 5-1). Twelve of the 19 waste rock samples were tested for paste pH, and three of these were in the range of pH 2 to 4, indicating that acid generation was occurring at the locations where these samples were collected, including the west dump area and the north pit wall (Figure 4-1). This is not surprising given that the observed sulfate salts (e.g., chalcanthite) and acidic seeps in the vicinity of the pit are strong indicators of sulfide mineral oxidization. Subsequent paste pH testing by SRK (1996, Figure 5.1) has confirmed that low pH values (less than 5) were observed in the north pit wall and west waste rock dump and that these areas are currently generating acid (Figure 4-2).

Two tailing samples collected from the tailing impoundment had paste pH values between 7 and 8 (SRK, 1995, Table 5-1). However, these two samples were collected from between 5 and 12 feet below the surface. Numerous studies have shown that oxygen concentrations drop off rapidly within a few feet of the tailing surface, and that acid generation is most significant above this depth (e.g., Elberling and Nicholson, 1996). Therefore, near-surface samples of the tailing should also have been collected to properly assess acid generating status. Although tailing samples were subsequently obtained from five test pits in the tailing impoundment during July 1996 (SRK, 1996, Section 6.1), no static test data were available for review to indicate whether the shallow tailing are generating acid.

Static tests were performed on 19 waste rock samples and two tailing samples. Total sulfur content of the waste rock samples ranged from 0.37 to 4.34 percent and averaged 2.15 percent. For most samples, pyritic sulfur concentrations were approximately half as great as total sulfur,

Table 4-1: Summary of Static Test Results

Sample ID	Sample Description	Paste pH	Total Sulfur (%)	Pyritic Sulfur (%)	Sulfate Sulfur (%)	Neutralizing Potential (t/kt)	Sulfide Sulfur (%)	Undefined Sulfur (%)	Pyrite AP (t/kt)	Pyrite NNP (t/kt)	Pyrite NP/AP (t/kt)	Sulfide AP (t/kt)	Sulfide NNP (t/kt)	Sulfide NP/AP (t/kt)
Tailings														
T-10-12	Tailings from borehole SRKBH-1-94	7.8	1.26	0.68	0.03	24	1.23	0.55	21.25	2.75	1.13	38.44	-14.44	0.62
T-5-7	Tailings from borehole SRKBH-1-94	7.5	1.10	0.53	0.18	31	0.92	0.39	16.56	14.44	1.87	28.75	2.25	1.08
	Average		1.18	0.61	0.11	28	1.08	0.47	18.91	8.58	1.50	33.58	-6.10	0.85
Waste Rock														
WD-1	West Dump Area, QM Waste Rock	2.7	4.34	2.12	0.005	0.1	4.34	2.22	66.25	-66.15	0.00	135.47	-135.37	0.00
PW-3	Pit Wall, Northwest of Pit Lake	2.8	2.20	0.84	0.005	0.1	2.20	1.38	26.25	-26.15	0.00	68.59	-68.49	0.00
SW-1	*Sulfide Waste Pile, QM Waste Rock*		*1.36*	*0.47*	*0.005*	*36*	*1.36*	*0.88*	*24.66*	*21.31*	*2.45*	*42.34*	*-6.34*	*0.85*
PW-2	*Pit Wall, Oxidized Cap Rock*		*0.37*	*0.04*	*0.005*	*11*	*0.37*	*0.23*	*1.25*	*9.78*	*8.80*	*11.41*	*-0.41*	*0.96*
PW-4	Pit Wall, Northeast of Pit Lake	3.9	1.89	0.78	0.005	16	1.89	1.11	24.38	-8.38	0.66	58.91	-42.91	0.27
SWP-1	Sulfide Waste Pile, QM Rock	8.8	3.08	1.46	0.005	40	3.08	1.62	45.63	-5.62	0.88	96.09	-56.09	0.42
LGSSP-1	Sulfide Waste Pile, QM Rock	6.6	1.52	0.61	0.005	47	1.52	0.91	19.06	27.94	2.47	47.34	-0.34	0.99
LGSSP-2	*Sulfide Waste Pile, QM Rock*	*8.9*	*0.61*	*0.20*	*0.005*	*39*	*0.61*	*0.41*	*6.25*	*32.75*	*6.24*	*18.91*	*20.08*	*2.06*
WD-2	West Dump Area, QM Waste Rock		1.98	0.87	0.005	60	1.98	1.11	27.19	32.81	2.21	61.72	-1.72	0.97
IDC-24-222 241	*QM From IDC Drillhole 24, 222-241 Feet*		*1.74*	*0.75*	*0.005*	*31*	*1.74*	*0.88*	*23.44*	*7.56*	*1.32*	*54.22*	*-23.22*	*0.57*
CF10-177.8-190	Andesite From Drillhole CF10, 177.8-190		2.86	1.77	0.06	52	2.80	1.03	55.31	-3.31	0.94	87.50	-35.50	0.59
CF10-190-199	*QM From Drillhole CF10, 190-199*		*3.89*	*1.09*	*0.07*	*44*	*3.82*	*2.43*	*24.06*	*8.84*	*1.28*	*110.00*	*-66.00*	*0.40*
CF10-214-220	QM From Drillhole CF10, 214-220		3.92	2.05	0.005	85	3.92	1.87	64.06	0.94	1.01	122.34	-57.34	0.53
H75-53-42	QM, Reverse Circulation Cuttings	8.2	1.77	0.88	0.005	36	1.77	0.89	27.50	8.50	1.31	55.18	-19.16	0.65
H75-64-44	QM, Reverse Circulation Cuttings	7.2	1.69	0.69	0.005	39	1.69	1.00	21.56	17.44	1.81	52.66	-13.66	0.74
H75-51-34	QM, Reverse Circulation Cuttings	8.6	2.02	0.72	0.005	49	2.02	1.30	22.50	26.50	2.18	62.97	-13.97	0.78
H75-48-58	QM, Reverse Circulation Cuttings	7.2	1.18	0.38	0.005	16	1.18	0.80	11.88	4.13	1.35	36.72	-20.72	0.44
H75-48-44	QM, Reverse Circulation Cuttings	7.4	1.06	0.15	0.005	9	1.06	0.91	4.69	4.31	1.92	32.97	-23.97	0.27
PW-1	Pit Wall, SW of Pit, Transition Zone, QM	6.1	3.61	2.00	0.14	32	3.47	1.47	62.50	-30.50	0.51	108.44	-76.44	0.30
	Average		2.15	0.94	0.02	33	2.13	1.19	29.39	3.38	1.97	66.51	-33.77	0.62

Notes: Sulfate sulfur non-detect reported as 1/2 of the detection limit
Neutralization potential non-detect reported as 1/10 of the detection limit
Sulfide Sulfur = Total Sulfur - Sulfate Sulfur
Samples in *italics* selected for kinetic testing

AP pyrite) = Pyritic Sulfur x 31.25
AP (sulfide) = (Total Sulfur - Sulfate Sulfur) x 31.25

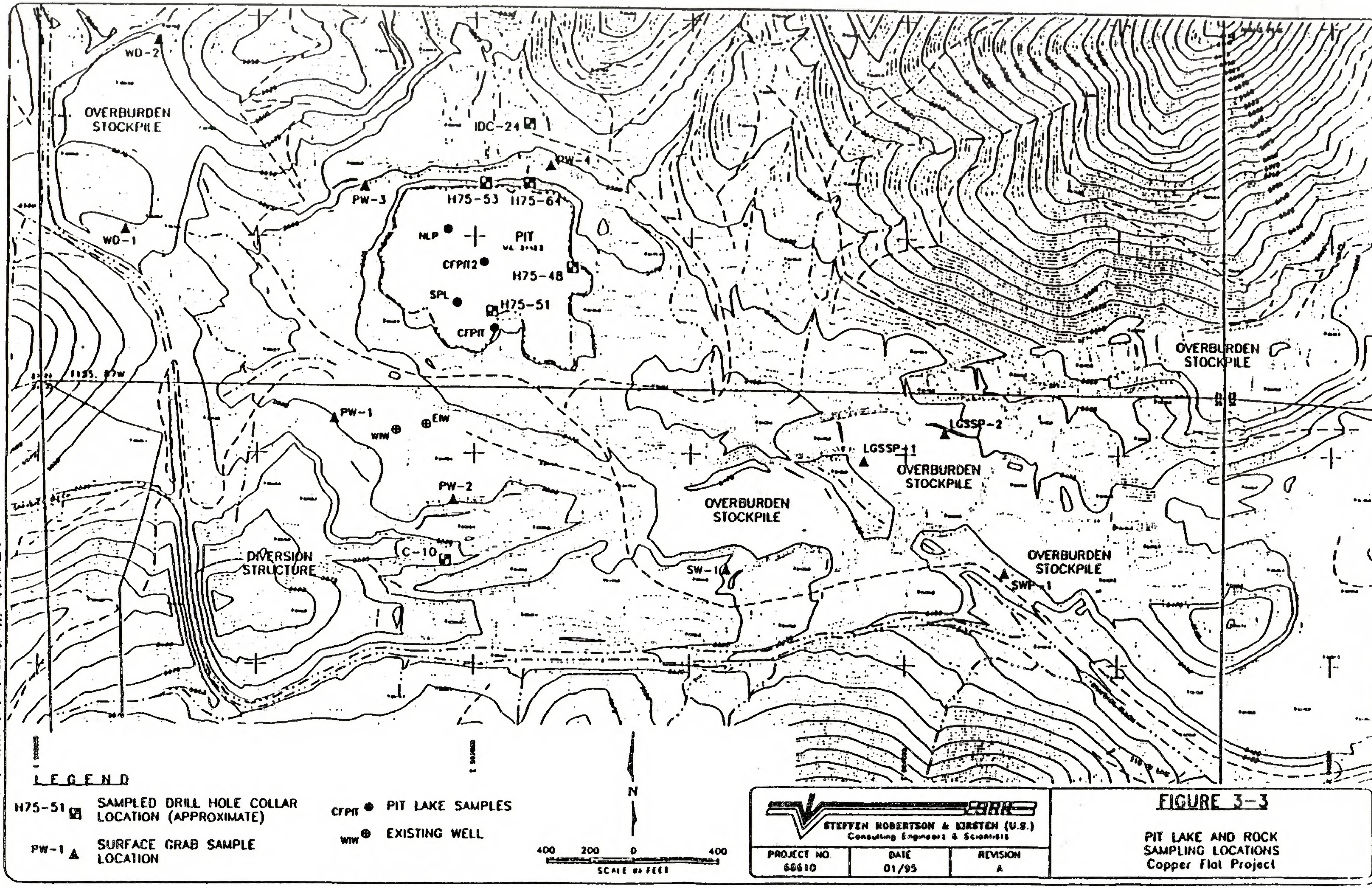
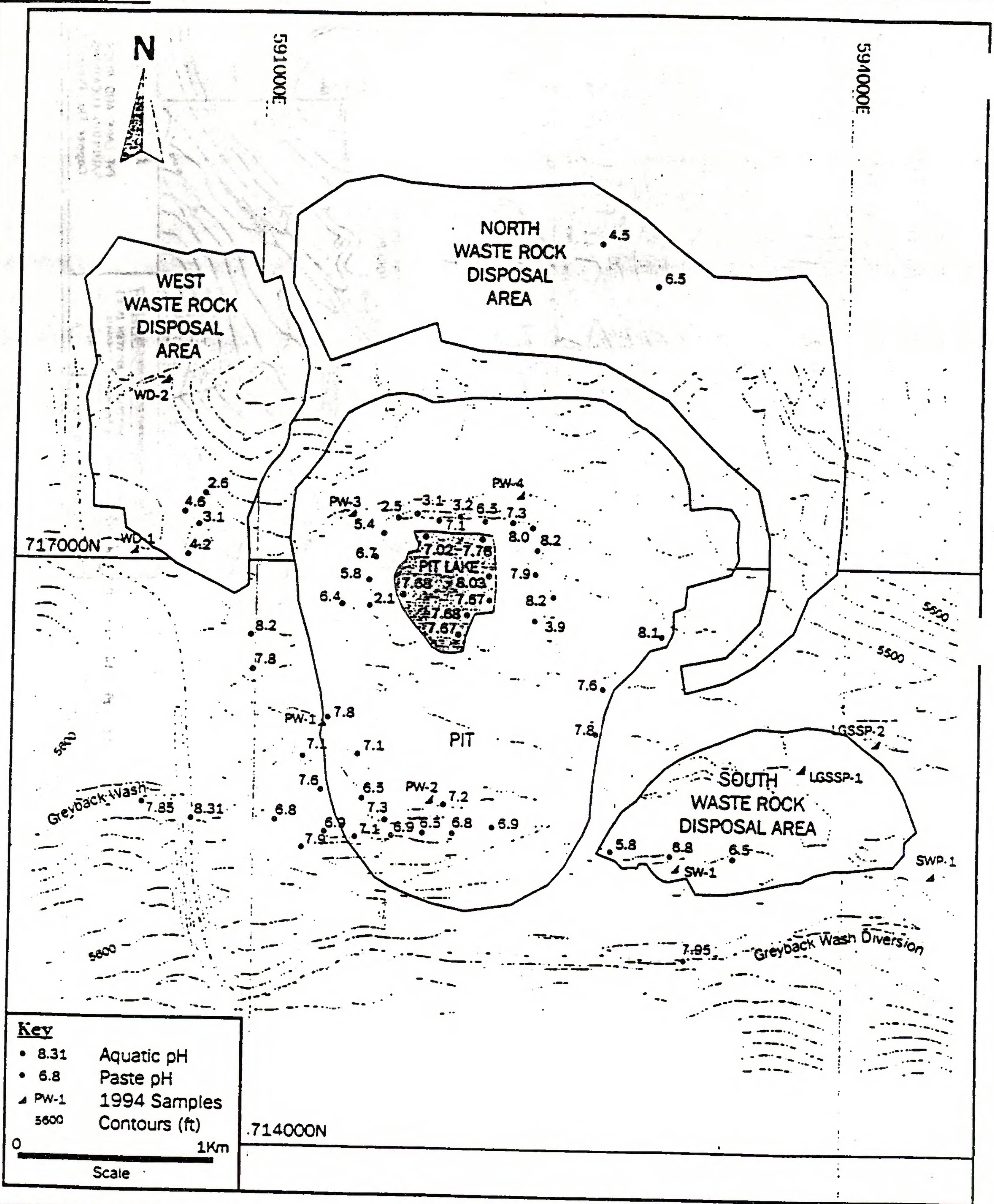

WD-2
OVERBURDEN STOCKPILE
IDC-24
PW-4
PW-3
H75-53
H75-64
WD-1
NLP
PIT
CFPIT2
H75-48
SPL
H75-51
CFPIT
OVERBURDEN STOCKPILE
PW-1
WIW
EIW
LGSSP-2
LGSSP-1
OVERBURDEN STOCKPILE
PW-2
OVERBURDEN STOCKPILE
DIVERSION STRUCTURE
C-10
SW-1
OVERBURDEN STOCKPILE
SWP-1
N
LEGEND
H75-51 SAMPLED DRILL HOLE COLLAR LOCATION (APPROXIMATE)
PW-1 SURFACE GRAB SAMPLE LOCATION
CFPIT PIT LAKE SAMPLES
WIW EXISTING WELL
400 200 0 400
SCALE IN FEET
STEFFEN ROBERTSON & KIRSTEN (U.S.)
Consulting Engineers & Scientists
PROJECT NO. 68610
DATE 01/95
REVISION A
FIGURE 3-3
PIT LAKE AND ROCK SAMPLING LOCATIONS
Copper Flat Project

Figure 4-1



DATE: 3/12/96 PROJ. No. U857

COPPER FLAT

SRK

PASTE AND MEASURED AQUATIC pH MAPS OF WASTE ROCK, PIT LAKEN AND SURFACE WATERS

Figure 5.1

and the difference probably results from sulfur in the minerals chalcopyrite and molybenite, which do not appear to be effectively dissolved by the nitric acid pyritic sulfur extraction procedure (Jennings, 1993; Jennings and Dollhopf, 1995).

Both chalcopyrite and molybenite are important minerals in the Copper Flat ore body (Dunn, 1982). However, compared with pyrite, chalcopyrite has been found to oxidizes very slowly in the environment (Rimstidt et al., 1994), and molybenite oxidizes even more slowly. Therefore, these two minerals probably do not contribute appreciably to acid generation, and hence pyritic sulfur content is believed to be more appropriate than total sulfur for calculating the acid-base account.

Except for the three samples with low pH values, acid neutralization potential (NP) values were uniformly between 10 and 70 tons per kiloton (tons/kton) $CaCO_3$ equivalent (SRK, 1995, Table 5-1). No information is provided regarding the laboratory methods used to determine the acid-base account. Assuming that the Sobek (1978) method was used, much of the reported NP may represent dissolution of silicate minerals, as opposed to carbonate minerals. Lawrence and Wang (1997) have shown that this method can significantly overestimate the NP because of dissolution of relatively unreactive silicate minerals (e.g., sodium plagioclase) under the aggressive acid dissolution conditions of the test. Therefore, the NP values probably represent maximum values, and the NP of the materials under field conditions could be considerably less.

We agree with SRK (1995, Section 5.2.2.5.2) that nearly all of the samples tested exhibit acid generating potential, with NP/acid potential (AP) ratios (calculated on the basis of total nonsulfate sulfur) that are generally less than 1.0. We also agree with SRK that "if these materials are left exposed indefinitely, then the neutralizing potential will be consumed before the acid potential" (SRK, 1996, Section 5.3). This situation is typical for a sulfide ore body in noncarbonate igneous host rocks.

4.1.2 Kinetic Tests

SRK performed kinetic (humidity cell) tests on five samples believed to represent a wide range of materials present at Copper Flat. Each sample consisted of approximately 4 kg of less than 2-inch material placed in a 6-inch-diameter polyvinyl chloride (PVC) column, and subjected to multiple weekly cycles of 3 days of aeration with humid air, 3 days of aeration with dry air, followed by flushing (irrigation) with deionized water. An attempt was made to match the water flush volume to rainfall intensities expected at the mine site to avoid artificially maintaining high pH conditions in the columns (SRK, 1995, Section 5.2.4.1). The solution flushed from the columns was analyzed for pH, redox potential, electrical conductivity, and the concentrations of a suite of dissolved constituents. The kinetic tests were run for a total of 28 weeks, with the columns left undisturbed (no flushing) between weeks 20 and 27.

Of the five samples tested, one sample (PW-2) had a low initial pH (5.8), and the remaining four had initial pH values of 7 to 9 (SRK, 1995, Figure 5-3). Figures 5-5 through 5-9 generally show declining concentrations of dissolved constituents in the column flush solutions with time. The one exception to this trend was that, following an initial decline, the total acidity of all samples increased during the latter part of the tests (Figure 5-10). One wonders if the rates of acidity generation would have continued to rise had the tests been continued beyond 28 weeks.

Based on the results of the kinetic tests, SRK concluded that "the rates of production of acidity and sulfate are very low" (Section 5.2.5). We agree that the rates of acid production were low during the kinetic tests; however we do not agree that these tests are necessarily representative of what is occurring in the field. Acid generation is actively occurring today, at least in the vicinity of the pit and the West Dump (see below), and we do not believe that this generation can be attributed entirely to prior solution mining activities (see SRK, 1995, Section 5.2.6). For reasons that aren't entirely clear, laboratory kinetic tests are notoriously inadequate at predicting field weathering behavior of sulfide minerals. Rapid acid generation rates are often not observed in short-duration laboratory kinetic tests, even though the materials may ultimately prove to be acid generating in the field (White and Jeffers, 1992). Therefore, we suggest that the favorable results of the kinetic tests be interpreted cautiously, and that the results of the static tests be kept in however evaporative losses of surface water precipitation from the pit walls will also occur" (SRK, 1995, Section 6.2.1, p. 6-3).

Other pit water balance studies have estimated much lower runoff coefficient values. For example, a study of another pit lake in New Mexico calculated a runoff coefficient of 5 percent based on comparison of average annual precipitation with streamflow measured at a U.S. Geological Survey (USGS) gauging station (Warren et al., 1997). While the 5 percent runoff coefficient appears to apply to a larger basin, SRK's value of 50 percent seems too high. Because the runoff coefficient is critical to the predictive capability of the water balance, the choice of the wrong value could result in incorrect model output.

SRK assumes that groundwater outflow is negligible (SRK, 1995, Figure 6-1). This assumption is based on a rather confusing argument (see Section 6.2.2.1), whereby groundwater inflow to the pit prior to 1988 is estimated to be 64 gpm, but after 1988 is believed to be approximately equal to the net evaporative loss of about 8 gpm. This, coupled with the lower hydraulic head in the existing pit lake when compared with groundwater levels in the monitor wells, leads SRK to the conclusion that groundwater outflow from the pit is zero.

While groundwater outflow may be small compared with other flows, it almost certainly occurs as a result of the superposition of the pit lake on the regional groundwater hydraulic gradient. If groundwater outflow from the pit lake is truly negligible, and evaporation is the only means of water loss, then the concentrations of conservative solutes, such as chloride, should continually build up in the pit lake over time. On the other hand, if groundwater outflow occurs, then the chloride concentration would be expected to climb at a slower rate as chloride is lost through the pit lake bottom. Review of the rate at which chloride concentrations increase in the pit lake could provide information on the rate of pit leakage.

Following mine closure, SRK has recommended re-diversion of Greyback Wash into the pit (SRK, 1995, Section 8.2.1). The rationale for this is to (1) reduce the time required to refill the pit, thereby reducing the time available for sulfide mineral oxidation, (2) reduce the concentrations of dissolved constituents in the pit lake, and presumably (3) maintain near-neutral pH conditions in the pit lake through the continuous addition of alkaline surface water. According to Newcomer et al. (1993), a single sample of surface water flowing in Greyback Wash (SWQ-1) contained an appreciable concentration of bicarbonate (430 mg/L), suggesting that this water has substantial acid neutralizing capacity. However, this sample was collected at low flow (1 to 2 gpm), and therefore is not representative of average stormwater quality, which would undoubtedly possess much lower alkalinity.

DBS&A believes that the concept of rapidly re-filling the pit lake to prevent prolonged exposure of sulfide minerals to oxygen is a good one. Studies at other mine sites have confirmed that ARD problems are minimized if the sulfide-bearing rocks are kept below the water table. However, the Greyback drainage should be diverted in a manner that prevents the water from flowing over or through waste rock dumps or other areas containing high concentrations of acid-forming minerals. Otherwise, appreciable acid will be added to the pit, which could trigger acidification of the pit over time.

4.2.2 Bench-Scale Testing of Rock Samples

A total of 13 rock samples were collected for paste pH measurement (SRK, 1995, Table 6-1). Although Figure 3-3 of the SRK report shows the locations where samples were collected, comparison with Table 6-1 indicates that not all samples collected were analyzed for paste pH (e.g., no data for samples WD-2 and SW-1). Furthermore, it is unclear whether all of the rock samples were collected from the surface, or whether some of the samples were taken from a drill core at depth (e.g., H75-51-34).

The data gaps above make it difficult to draw conclusions based on the rock sample testing. Nevertheless, the results of the paste pH tests clearly show that ARD has progressed to an advanced degree in 3 of the 13 samples, yielding free acid upon dissolution in deionized water. The very limited information contained in Appendix M (Bench Scale Test Data) indicates that aluminum, copper, iron, manganese, and sulfate were released from two samples (PW-3 and PW-4) collected immediately north of the pit.

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mind. The static test results indicate that nearly all of the waste rock material has the potential to generate acid, sulfate, and dissolved metals, given sufficient time following its exposure to oxygen.

4.2 Post-Mining Pit Water Chemistry

The primary sources of information pertaining to pit lake chemistry are Chapter 6 of the Copper Flat Mine Hydrogeological Studies (SRK, 1995) and a subsequent study by SRK (1996, Section 7.2). The purpose of the Post-Closure Pit Water Study was "to characterize the potential impact of the post-closure pit lake on the quantity and quality of groundwater resources near the proposed open pit."

During the 1995 study, SRK performed three tasks in an effort to predict the quantity and quality of water that will ultimately fill the pit:

- Pit water balance calculations to estimate final pit lake elevation
- Bench-scale testing of rock samples for paste pH and soluble metals
- Geochemical modeling to predict ultimate pit lake chemistry

Sections 2.2.1, 2.2.2, and 2.2.3 provide review comments for each of these tasks.

4.2.1 Pit Water Balance

Water balance calculations were performed to quantify water flows to and from the pit and to estimate the final pit lake elevation. The pit water balance is based on the simple concept that, at some time in the future when pit lake levels have stabilized, the sum of direct precipitation and inflows of surface water and groundwater to the pit will be equal to the combined water lost from the pit as a result of evaporation and the combined surface water and groundwater outflows.

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SRK used the following values as input to the pit water balance:

- Precipitation: 12.9 inches per year (Hillsboro gauge at 5,270 feet above mean sea level [ft msl])

- Evaporation: 65 inches per year (NOAA, 1982)

- Surface inflow: assumed 50 percent of precipitation over 104-acre catchment

Based on the model, SRK then calculated the following values for the existing 12.8-acre pit lake:

- Groundwater inflow to pit: 8 gallons per minute (gpm)
- Groundwater outflow from pit: none
- Surface water outflow from pit: none

There are several problems with the pit water balance as presented by SRK (1995). First, SRK claims that the precipitation at Copper Flat (elevation 5,440 feet msl) would likely be less than that at the Hillsboro gaging station (elevation 5,270 feet msl) "because of the elevation change and the rain-shadow effect of neighboring mountains" (SRK, 1995, Section 6.2.1, p. 6-4). Although the rain-shadow effect is very difficult to quantify without establishing an on-site gauging station, we would generally expect more precipitation with increasing elevation, suggesting that Copper Flat should receive more precipitation than Hillsboro. Nevertheless, the use of the Hillsboro average annual precipitation in the pit water balance is reasonable, and does not introduce appreciable uncertainty compared with other input parameters. Likewise, the use of a regional evaporation rate of 65 inches per year is reasonable and is not a source of great uncertainty.

SRK's water balance model is weakest in assuming that (1) 50 percent of the precipitation runs off into the pit, and (2) groundwater outflow from the pit is negligible. The first assumption appears to be completely arbitrary and not based on any site-specific data. The rationale for the 50 percent runoff coefficient is explained in the following statement: "Surface water runoff within the pit boundary will be promoted by the steep slopes and low permeability of the pit walls;

Coupled with observed paste pH values as low as 2.9, it is evident that the ARD is presently occurring in the vicinity of the northern edge of the pit and at the west dump (Figure 2). The results support SRK's statement that "surface water which contacts exposed acid generating material will solubilize dissolved iron, aluminum and sulfate." The acid carried into the pit by such "contact water" is gradually consuming the residual alkalinity in the pit lake over time.

4.3 Geochemical Modeling Using MINTEQA2

Geochemical computer modeling was performed by SRK (1995, Section 6.4) to predict the ultimate post-closure pit lake chemistry using the computer code MINTEQA2. The approach taken was to use the upgradient groundwater quality as input to the computer model, then allow the water to come to chemical equilibrium with a suite of minerals believed to be present at the site. The upgradient groundwater quality was assumed equal to that of a groundwater sample collected from monitor well GWQ-4 located about a half mile southwest of the pit in Greyback Wash (Figure 4-3).

Furthermore, SRK's model assumes that any surface water entering the pit lake will have the same chemistry as the upgradient groundwater. DBS&A believes that this assumption is unrealistic because surface water would almost certainly have a much lower alkalinity (bicarbonate concentration) than the groundwater, as a result of the limited contact time available for dissolution of carbonate minerals by surface water runoff. In fact, not only is surface water likely to have little or no alkalinity, but may possess appreciable free acidity in response to contact with acid-forming minerals exposed at the surface (SRK, 1995, Section 6.3; Photograph 2). Surface water quality is of critical importance in determining the ultimate pit water chemistry, especially given that, according to SRK's pit water balance (SRK, 1995, Appendix L), future surface water flows will far exceed groundwater flows. We recommend that the quality of surface water inflows to the pit be measured directly, rather than making an arbitrary and probably incorrect assumption that this water is chemically similar to the upgradient groundwater.

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When running the MINTEQA2 computer model, SRK assumed the following:

- The minerals pyrite, chalcopyrite, and calcite are present in infinite supply.

- The amorphous "minerals" ferrihydrite, $Fe(OH_3)$, and amorphous aluminum hydroxide, $Al(OH)_3$, were allowed to precipitate from the pit lake if the model predicted supersaturation with respect to these phases.

DBS&A believes that there are several problems with these assumptions. First, given that there are no carbonate sedimentary rocks in the immediate vicinity of the pit, the assumption of unlimited calcite is unrealistic. In fact, the average calcite content of the igneous rocks within the ore body is probably a few percent at most. Neutralization potential tests reported by SRK (1995, Table 5-1) confirm this, as the average NP for all samples (32 tons $CaCO_3$ per kton) is equivalent to a maximum of 3.2 percent calcite, assuming that all neutralization results from calcite dissolution. Therefore, it is possible that the calcite present and exposed to the pit water could be entirely consumed by reaction with acid generated by sulfide mineral oxidation. Therefore, it is inappropriate to assume an infinite supply of calcite.

In addition, DBS&A believes that the failure to allow for gypsum precipitation is a major oversight in SRK's MINTEQA2 modeling effort. Many gypsum crystals up to a centimeter long are visible around the shore of the present pit lake (Photograph 4), and this gypsum clearly precipitated within the last few years when the lake level was somewhat higher than at present. In addition, the laboratory results for a pit water sample collected in 1994 (SRK, 1995, Table 6-2) indicate that the water is saturated with respect to gypsum. These observations confirm that gypsum can and does precipitate from the pit water, and any geochemical modeling should take this into account. SRK's failure to include gypsum precipitation casts doubt on the validity of the pit water geochemical model.

DBS&A also disagrees with the statement that "[d]issolved metal concentrations in the existing pit water are at or near method detection limits" (SRK, 1995, p. 6-7). While this is true for many metals, the concentrations of manganese, copper, and zinc are elevated well above their respective detection limits. We also disagree with the statement that "[t]he occurrence of poor quality (e.g., low pH and elevated dissolved metals concentrations) surface water runoff is minimal at present due to the arid conditions at the site and due to the lack of oxidized acid generating material at the site" (SRK, 1995, p. 6-8). There is clearly no lack of acid-generating materials surrounding the pit, as shown in Section 6.3 of SRK's report.

Evaluation of SRK's geochemical modeling is complicated by the fact that the MINTEQA2 model output was not included in their report. Instead, we must rely on a brief description of the model results (SRK, 1995, Section 6.4.2), and details of the modeling effort (e.g., open versus closed system conditions and fixed versus variable Eh) were not available. From the MINTEQA2 modeling results, SRK concluded that the "existing pit water quality is an approximation of post-closure pit water quality" (p. 6-10). However, DBS&A does not believe it is possible to conclude this with any certainty based on the information reviewed.

Subsequent geochemical modeling by SRK (1996, Section 7.2) included more realistic assumptions than those in the 1995 report. For example, SRK acknowledged that it is unrealistic to assume the presence of calcite (SRK, 1996, Section 7.2.1.2). The later work incorporated three water types that enter the pit lake: (1) groundwater inflow, (2) uncontaminated surface water inflow, and (3) contaminated "contact" water inflow. Each water type was assigned a representative chemistry (SRK, 1996, Table 7.3). By varying the annual precipitation and evaporation rates, as well as the runoff coefficient, the proportions of each water entering the pit lake change.

Various scenarios were then considered using the geochemical code PHREEQE to model the evolution of the pit water chemistry over time, and to determine which scenario best matched the existing pit lake water quality. The results indicated that ARD "constitutes an important source of sulfate to the existing pit water, but a high evaporation rate is necessary to concentrate the sulfate to observed levels without lowering solution pH" (SRK, 1996, Section 7.2.1.2).

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The model that best fit the existing pit lake chemistry included the following conditions:

- Total annual precipitation = 10 inches per year
- Lake evaporation rate = 88 inches per year
- Runoff coefficient = 0.3
- No pH buffering minerals initially present

SRK then used the model to predict post-closure water quality in the pit lake (1996, Section 7.2.2), and concluded that "the water quality in the proposed pit should be better than existing pit-water quality for about 120 years after mine closure." Although not explicitly stated by SRK, this prediction appears to be valid only for the scenario including post-closure diversion of Greyback Wash into the pit. The scenario that does not include diversion of Greyback Wash results in a final pH of 3.9 following 120 years after closure (Table 7.11). While the more recent modeling by SRK is admirable, it is important to remember that no meteorologic data exist for the mine site to support the model assumptions. These data will have to be collected before the model can be considered reliable.

The pit lake currently has a pH of about 7.8 (SRK, 1995, Table 6-2); however, the pH of the lake has reportedly been significantly lower in the past, with pH values of 6.6 and 6.7 measured by Newcomer et al. (1993, p. 20) at depths of 10 and 40 feet, respectively. Even lower values in the pH 4 to 5 range have been measured in the past from the shore. The reason for the trend toward higher pH values is unknown, but could be the result of (1) an influx of alkaline surface water, or (2) questionable pH measurements during past sampling events. It is important to understand whether the pit lake chemistry has really fluctuated significantly in the past, and if so, what events led to such rises in lake water pH. We understand that SRK is currently preparing a report summarizing historical sampling and analysis of the existing pit lake (Parshley, 1997). When available, this report will be helpful in clarifying the evolution of the pit lake water chemistry.

4.4 Geochemical Modeling using PHREEQE

Geochemical computer modeling was performed by SRK (1997, Section 6) to predict the pit water quality at the present and in the proposed pit. The geochemical model PHREEQE (Parkhurst et al., 1980) was selected because it allows titration of one solution by another in specified reaction steps. Evaporation can also be simulated by titrating the solution with negative volume which represents the water that evaporates.

SRK states that the geochemical model presented in this report is essentially the same model used that was run using MINTEQA2 and reported in SRK (1995) with a few minor refinements. The refinements included the following:

1. "[B]etter definitions of water quality and inflow volumes of each fraction"

2. "[F]ield data is input to the PHREEQE directly with no prior equilibration steps"

3. "[C]harge balance errors from averaging of several sample concentrations and/or from laboratory errors were overcome by adjusting the most uncertain ions"

4. "[S]olutions are equilibrated at the end of mixing reactions"

DBS&A found several problems with the input used in the PHREEQE models. Most of the concentrations input to the PHREEQE models (Input files in Appendix E) are not completely consistent with the values reported in Tables 6.2 (groundwater), 6.3 (surface water runoff), and 6.4 (contact water) in SRK (1997). In many cases the values in the tables approximate the values used in the model, but some values differ. It is not clear how or if multiple analyses were averaged. In the case of the surface water runoff sample (Table 6.3) the chloride concentration was input as 80 mg/L, while the concentration in the table is shown as 180 mg/L. In addition the sample identification for this sample is missing from Table 6.3. Inconsistencies such as these raise questions about the reliability of the model output.

SRK states on p. 6-11 that "The final pit solution following mixing was supersaturated with respect to calcite and amorphous iron hydroxide phases, which are kinetically feasible to precipitate. As a result, these minerals are equilibrated to precipitate in the pit lake." In reviewing the output from the calibration run, calcite is actually undersaturated (SI = -0.85) in the water prior to it being equilibrated with calcite. By equilibration with calcite, SRK is assuming that there is an infinite source of calcite in contact with the pit lake, which is not realistic. DBS&A believes that the assumption of infinite calcite available to neutralize acid waters is among the most significant problems with SRK's geochemical model, as it results in unrealistically optimistic (i.e., high) predictions of the pit lake pH.

SRK uses the NEUTRAL species option of PHREEQE to create charge balance between the dissolved constituents used as input in their model. Depending on whether the imbalance resulted from insufficient cations or anions, they typically added magnesium (cation) or sulfate (anion) to create the charge balance. Their justification is that the process of averaging several analyses may cause a charge imbalance in the resulting model solution. DBS&A believes this was unnecessary, as all the analyses have some charge imbalance and that by averaging the concentrations of several water analyses the resulting imbalance of the average solution may increase or decrease. The imbalance in charge in an individual analysis may result from analytical error and/or incomplete analysis. Chemical analyses that have a charge imbalance of greater than about 10 percent probably should not be included in the calculations.

Mass balance analysis is a method typically used to critically evaluate the reasonableness of a modeling effort of this type. DBS&A believes that this opportunity was lost when SRK, in their use of the PHREEQE model, allowed the addition of mass (Mg and SO_4) to achieve both charge balance and equilibrium with specific minerals.

DBS&A also has the following additional concerns with the model:

1. The model calibration using the titration option in the geochemical model PHREEQE oversimplifies the problem of interpreting the changes in water quality in a developing pit lake. It assumes instantaneous and complete mixing of all of the inflow components to

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the pit. It does not address the potential for stratification of the pit lake, and any changes in water quality of the inflow components over time.

2. The groundwater inflows at early times will likely mobilize some of the acidity that has developed along fractures in the unsaturated wall rocks around the pit. Following extensive dewatering and deepening of the pit, an extensive unsaturated zone will have developed. Sulfides in the unsaturated zone will likely oxidize to varying degrees, probably faster in fractures close to the pit walls. The dissolved salts associated with these early inflows, along with the runoff from the pit walls, will strongly affect the initial water quality in the pit lake. Therefore, water quality of the inflows from the groundwater and contact water components are likely to be poorer at early times, but may improve as the pit lake fills. This was not represented in the model.

3. The equilibration of the modeled solutions with minerals (i.e., calcite) at the end of mixing reactions and prior to evaporation is probably not realistic. This assumes an infinite source of calcite, which buffers the pH in the circum-neutral range, and limits any decrease in the pH of the mixture. A more conservative approach would be to run the model without allowing calcite to control the calcium and alkalinity concentrations of the pit lake. Although calcite is present in some of the rocks at the mine site, its distribution and concentration are not known well enough to justify this assumption. Even if calcite is widespread and present in significant concentrations in the rocks, its ability to contribute alkalinity to the groundwater system over time would be finite. Available calcite along fractures will likely be consumed over time, and/or armored with secondary minerals or reaction products. Regardless, not all of the calcite would be available for reaction. The uncertainty associated with this assumption seems too great, and the potential risks of not predicting the development of an acidic pit lake are too high, to warrant this simplification.

4. Based on laboratory analysis of some samples collected in Greyback Wash at low flow, SRK assumed that all surface runoff is supersaturated with respect to calcite. This condition is unrealistic, as surface runoff at higher flows would have little time to react with and dissolve calcite. The low flow samples were collected from pools that are believed

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to form as a result of spring flow in this area. The dissolved constituents in these pools would be derived primarily from groundwater, and become further concentrated by evaporation. Therefore, DBS&A believes that the water samples from this stretch of Greyback Wash are not representative of the average water quality of surface runoff. A stormwater with lower alkalinity and pH-buffering capacity would be more representative.

4.5 Copper Flat Technical Review of SRK Document "Compilation of Pit Lake Studies"

SRK issued a new document in October 1997 entitled "Copper Flat Mine: Compilation of Pit Lake Studies." The document was prepared in response to comments from Shepherd Miller Inc. requesting a single document summarizing the geochemistry and predicted post-closure conditions of the Copper Flat pit lake. DBS&A has reviewed the new SRK document and offers the following comments.

4.5.1 General Comments

SRK's pit lake report was apparently prepared quickly, as there are numerous typographical errors and missing words. More importantly, there is a general lack of documentation concerning sampling locations and laboratory methods. For example, Table 2.2 shows whole rock chemical analyses; however, we cannot determine where and when the sample or samples were collected or what laboratory methods were employed. The units are not correctly specified, and should presumably be ppm, not mg/L. Such lack of documentation recurs throughout the report, and results in confusion for the reader.

With regard to the geochemistry of the existing pit lake, SRK states that the lake has been sampled 40 times between 1989 and 1997 (Sect. 3.1, p. 3-1). One would expect that such prior sampling would enable us to develop a good understanding of how concentrations of dissolved constituents have changed over time in the pit lake. The SRK report contains trend plots for pH and sulfate over time, however it does not contain time-series plots for alkalinity, calcium, and other important species. These plots are essential to predicting future pit lake water quality.

SRK states that "All water chemistry data for the pit area is included in Appendix C" (p 3-2). However, we were unable to locate the pit lake chemistry data in Appendix C. Without access to the raw pit lake chemistry data, it is impossible to construct water quality plots for the missing constituents (e.g., alkalinity). Furthermore, it is impossible to determine the source of the data in Appendix C (e.g., who collected the samples, etc.). Again, sufficient documentation is lacking to allow the reader to make sense of the available data.

SRK has not adequately explained the four low pH values recorded in the pit lake during 1992 (Fig. 3-1). The reader is not told where these measurements were taken in the lake, or by whom. Lacking information to the contrary, we can only guess that future low pH excursions are possible in the lake, and that predictions of future pH trends in the ultimate pit lake (e.g., Fig. 6.2) are speculative. Furthermore, SRK's assertion (p. 7-1) that "net acid generation is not an issue at Copper Flat" is obviously not supported by the existing data.

Regarding the pit lake water balance, SRK has revised its estimate of the surface runoff coefficient downward from 50% in an earlier report (SRK, 1995) to 30% in the present report (SRK, 1997b). The 30% value is more reasonable, but may still be too high, especially given that studies of other basins in New Mexico have shown coefficients of approximately 5% (Warren et al., 1997), and SRK's own estimate of surface runoff to Greyback Gulch was only 5.8% of precipitation (SRK, 1997b, p. 4-7). Given the uncertainty in the quantity and quality of surface runoff that will flow to the pit, SRK's predictions of future pit lake elevation and water quality are speculative at best.

4.5.2 Specific Comments

The following specific comments pertain to the technical content only; numerous typographical errors or omissions are ignored, except where the meaning of the text is unclear.

- Figure 2.2, p. 2-4: The term "quartz majarte" is used by SRK in Figure 2.2. This term is not in common usage among geologists in the U.S., and is not used elsewhere in SRK's report. Presumably, this is synonymous with "quartz monzonite."

- Section 2.4.1, p. 2-9: SRK states, "As the sulfides are disseminated there is little contact between individual grains, consequently electrochemical transfer cannot occur...." No data or test results are provided to support this assertion.

  SRK maintains that oxidation of sulfide minerals would be very slow on account of the low humidity at the site. While water is indeed required for oxidation of sulfide minerals, DBS&A believes that the relative humidity of the air has little effect on the rates of sulfide oxidation within a waste rock dump or tailing pile. In fact, with the exception of the uppermost few centimeters, the relative humidity of soil gas is generally in excess of 99%. Therefore, the low relative humidity at the site would only be important with respect to sulfide minerals exposed at the surface.

- Section 2.4.3 p. 2-10: The phrase "oxyanions such as arsenate, molybdenite, and selenite" should probably read "oxyanions such as arsenate, molybdate, and selenate."

- Section 2.5, p. 2-10: SRK states, "A small seep was observed in August 1997 in the north west of the pit wall." DBS&A believes that several seeps were present in this area.

- Table 2.2, p. 2-12: This table of whole rock analyses does not indicate where or when each sample was collected and whether the values are for single samples or averages of multiple samples. The units should presumably be ppm, not mg/L. Additionally, if the whole rock analysis represents the quartz monzonite, then the concentrations of aluminum and sodium seem far too low to be believable.

- Section 2.5, p. 2-12: SRK states, "Low pH areas are observed in the south and west walls of the pit..." According to Figure 2-3, the low pH areas are located <u>north</u> and west of the pit. Also, the last sentence of the paragraph reads "...but the presence of fresh sulfide under the coating suggests that oxidation is limited and has not occurred but at a rapid rate." The meaning of this sentence is unclear.

- Table 2.4, p. 2-14: No documentation is given as to where or when the sample of quartz monzonite was collected.

- Table 3.2, p. 3-5: The electrical conductivity value of 70 µmhos/cm for well GWQ-22B is presumably in error, as it is far too low for a water with this chemistry.

- Section 3.2, p. 3-6: SRK states that monitor well GWQ96-23A "intersects partially oxidized sulfides (up to 15% oxidation) ....." What is meant by 15% oxidation and how was this determined?

- Figure 3.3, p. 3-7: The data shown at the lower right should be for well GW23, not GW22 as shown. Presumably this figure portrays the shallow monitor well of the cluster (GW22A and GW23A), although this is unclear.

- Table 3.3, p. 3-8: No information is given regarding the locations of water samples collected in Greyback Wash.

- Figure 3-4, p. 3-10: There is no way to correlate most of the points on the isotope plot with specific groundwater sampling locations.

- Section 4.2.2.4, p. 4-8: The report states that "The ABC model includes less than 10% surface water runoff while the revised SRK pit water balance includes 30% surface water runoff to the pit. This difference exists because of a different emphasis in model rationale." This explanation does not clarify the discrepancy in the assumed value of the runoff coefficient, which is critical to correctly modeling the pit water balance. DBS&A suspects that the correct value is closer to 10% than 30%.

- Section 5.2.2, p. 5-2: The meaning of this confusing paragraph is unclear.

- Section 5.3.1, p. 5-3: "radical groundwater flow" should presumably read "radial groundwater flow."

- Section 5.4.2, p. 5-4: SRK asserts that "At the base of the pit an anoxic environment will be created...." This will not necessarily occur, especially if the pit lake experiences seasonal turnover. In addition, DBS&A believes that SRK's proposed "introduction of anoxic bacteria" to reduce sulfate levels in the pit lake is unnecessary. Sulfate reducing bacteria (SRB) are nearly ubiquitous in the environment. If conditions are suitable for their growth at the bottom of the pit lake, SRB will begin reducing sulfate to sulfide. Addition of bacteria to the lake will in no way accelerate this process.

- Table 6.4, p. 6-8: It is significant that the acidic seep (ARD) contained far higher concentrations of dissolved solutes than any of the three contact test solutions. This demonstrates the inability of laboratory contact tests to adequately simulate the ARD processes that so readily develop in the field.

- Section 6.3, p. 6-11: The phrase "adjusted to P_{CO2} = -2.6" should presumably read "adjusted to log P_{CO2} = -2.6." It is unclear how SRK arrived at this value for the partial pressure of carbon dioxide.

- Figure 6.2, p. 6-13: Predictions of future lake chemistry, and especially lake pH, should be regarded as highly uncertain, particularly over the long time spans (centuries) modeled here.

- Section 6.6, p. 6-15: The sensitivity analysis involved varying model parameters by ±20%. While this is probably adequate to encompass the uncertainty in evaporation rate at the site, the uncertainty in the remaining model parameters (fraction of ARD component, percent surface water runoff) could be considerably more than ±20%. Varying these by ±100 percent during sensitivity analysis would have been more reasonable.

- Section 6.6, p. 6-16: SRK maintains that "the diversion of Greyback Wash into the pit will measurably improve water quality in the pit." DBS&A generally agrees with this statement, however it should be pointed out that we do not know the average water quality of Greyback Wash floodwaters. Such floodwaters, which will probably account for the majority of flow to the pit, could be considerably more dilute (lower alkalinity) than surface water samples collected to date from the wash under low flow conditions. Therefore, SRK's assumptions regarding the alkalinity contribution of surface water to the pit lake may be overestimated.

5. Reclamation and Closure Plan

The proposed reclamation and closure plan (Section 4, SRK, 1996, Copper Flat Mine Permit Application) appears to be fairly complete and generally conforms to the environmental standards for mines with relatively benign wastes. The plan has conceptually addressed many of the important issues for site stabilization and reclamation, including surface water control, optimized slope configuration, and revegetation practices and it is probably adequate based on the data and assumptions presented in the permit application and supporting documents. However, the baseline data and materials characterization that form the basis for development of the plan are lacking. Thus, it is difficult to evaluate the adequacy of the reclamation plan.

The primary issues of concern that were identified include:

- Cover thickness and availability
- The suitability of the waste rock and tailing as soil substitutes (plant growth media)
- The suitability of the alluvial materials as topdressing.

In addition, minor issues relating the post-mining land use were noted.

5.1 Topdressing Cover Thickness and Availability

The material characteristics and thickness of the cover are critical to protection of the environment because the cover design affects the ability to establish a self-sustaining ecosystem, withstand wind and water erosion, and provide a barrier to prevent water entry and movement through the acid-forming waste materials. Determination of the optimum cover thickness should consider plant growth requirements, local climatic regime, and the relative toxicity of the underlying materials.

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Topdressings (native soils or soil substitutes) used in mine reclamation and waste containment applications typically range in thickness from 6 inches to more than 8 feet. The thinner covers are generally used over nontoxic geologic materials that are considered soil substitutes, and thicker covers are generally required over materials that are acid and/or toxic forming. Frequently, on a national basis that includes coal mines, a nontoxic root zone 40 to 48 inches thick has been used. The 48-inch root zone requirement probably originated from the prime farmland reconstruction efforts in the Midwestern coal fields. This thickness was used to ensure that the reclaimed land was suitable to return to crop production. The scientific basis for requiring a 40- to 48-inch nontoxic root zone under southwestern conditions is not well supported in the scientific literature. Similarly, the scientific basis for the use of thinner covers over toxic materials is also poorly documented. Thus, in the absence of strong scientific evidence otherwise, the 6- to 12-inch-thick covers proposed in the permit application should be considered adequate only if the underlying materials are suitable soil substitutes; that is, they are nontoxic to the proposed vegetative cover.

The cover thicknesses proposed in the permit application range from 6 to 12 inches depending on the facility. Soil substitutes are proposed for use as a topdressing because sufficient quantities of suitable upper solum soil materials do not occur in the areas to be disturbed. The topdressing deficit is predicted by SRK (1996) to be about 600,000 yd^3, based on the proposed 6- to 12-inch cover design. The topdressing will be excavated from the tailing impoundment area prior to tailing deposition and stockpiled for later use.

The proposed design is commendable because, by obtaining borrow material from the tailing pond area, it limits the amount of disturbed area. However, the permit application also implies that the tailing impoundment is the only significant source of additional topdressing in the mine permit area, and that it will be unavailable for use after the initiation of mining and tailing deposition. The apparent lack of access to additional borrow materials after tailing deposition requires a high degree of confidence in the competency of the cover design. In other words, if 6 to 12 inches of cover do not provide adequate environmental protection, there is apparently no alternative on-site source of material. It would seem prudent, therefore, for Alta to develop a contingency plan to ensure that sufficient topdressing resources are available if the proposed cover thicknesses prove to be inadequate.

5.2 Materials Characterization

Fundamental to any reclamation design is an adequate analysis of the materials to be reclaimed as well as the borrow materials to be used as topdressing. Considerations in the characterization are the hydraulic properties of the materials, as well as their suitability as a soil substitute. The following sections describe DBS&A's analysis of the materials characterization.

5.2.1 Waste Rock

The materials from the tailing impoundment and north and west waste dumps are proposed for use as substitute soil materials. The use of the tailing and waste rock as soil substitutes is critical to the proposed conceptual design because the prevention of groundwater impacts depends on the cover and underlying substrates intercepting water. For instance, the HELP model (SRK, 1996) uses a 24-inch evaporative root zone regardless of the cover thickness. This implies that the 12- to 18-inch layer of materials below the cover is considered suitable as a root medium.

In a supplementary report, Johnson (1996) indicated that the tailing and waste rock are a suitable, albeit somewhat infertile, growth media. However, his assessment lacks consideration of the potential acid-forming and toxic character of the materials. Many of the waste rock samples had extremely acid pH values (i.e., less than 3.0) and negative acid base accounts indicating the potential for acid formation.

Additionally, limited data from waste rock samples suggest that total selenium concentrations may be relatively high (4.0 to 9.0 mg/kg) when compared with normal soil concentrations (total selenium concentration of 0.1 to 20 ppm; average 0.5 ppm [Mayland, 1989]). Elevated soil selenium concentrations are not a plant toxicity issue, but can be a concern for livestock under a grazing post-mining land use.

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5.2.2 Tailing

Apparently, the tailing characterization included the analysis of only two samples, thus it is difficult to evaluate the suitability of the tailing as a soil substitute. Additional tailing characterization should include samples from the near surface (e.g., upper 1 to 2 feet) and from a range of particle sizes.

5.2.3 Borrow Area

The borrow area characterization included the analyses of only four samples. The borrow area samples were chemically benign, but their use would be restricted by the coarse texture of the materials (i.e., sands and loamy sands). The use of materials in the sand textural class is limited by the low water-holding capacity and susceptibility to wind and water erosion (noncohesive nature). It is important to note that the 12-inch cover layer was justified based on a HELP model analyses (SRK, 1996, Section 5.5.2), which used a sandy loam rather than sand-textured materials characterized from the borrow area.

The occurrence of coarse fragments (rock fragments greater than 2 mm in diameter) in the topdressing would further restrict the volume of suitable soil substitutes available from the borrow. Soil data in Appendix M (SRK, 1996) suggest that the proposed topdressing may contain more than 50 percent rock fragments on a mass basis. Thus, the total volume of available borrow materials may be limited by texture and rock fragments because the cover was designed (e.g., HELP modeling) to limit percolation into the waste rock. Accordingly, additional physical and chemical data for the proposed topdressing and underlying wastes are needed in order to adequately evaluate the permit application.

5.2.4 Post-Mining Land Uses

The specific post-mining land use (PMLU) is not clearly stated in Section 4.3 (SRK, 1996). The permit application indicates that mining, grazing, wildlife habitat, watershed, and recreation will constitute the PMLUs. However, the primary PMLU for each facility (e.g., tailing impoundment,

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stockpiles, and plant site) was not identified. The identification of a PMLU for specific areas is important since different concerns and success criteria are associated with different PMLUs. For instance, vegetative productivity is a success criteria for the grazing PMLU, but not for wildlife. Thus, if grazing is the primary PMLU, but the facility will also accommodate wildlife, the more stringent success standard would apply. In addition, acceptable heavy metal levels in the plant root zone (e.g., selenium, molybdenum, copper) vary depending on whether the area is grazed by domestic livestock or used by wildlife. The "mining" PMLU is not currently defined in the MMD guidelines and the proposal needs more complete development.

The open pit is proposed as a source of irrigation water, but the permit application lacks an agricultural water quality assessment to determine if the pit lake water will contain acceptable contaminant concentrations for its intended usage. For example, the pit lake water is predicted to have elevated heavy metal concentrations, including cadmium (SRK, 1996, Section 4.7.4.1). The use of cadmium-enriched waters for irrigation is a questionable practice since the bioavailability (and toxicity) of cadmium is relatively high with respect to other heavy metals. Irrigation will result in the evapoconcentration of cadmium and other heavy metals in the soils; thus, the proposal to use the pit lake water as irrigation needs careful consideration and more detailed analysis to determine its suitability.

5.3 Compaction, Thin Covers, and Groundwater Protection

The permit application proposes to contain the transitional (acid-forming) waste rock by regrading and compacting the waste rock, and covering the surface with a maximum of 12 inches of native soil. The cover is meant to provide water storage capacity that will reduce percolation of water into the waste rock, thereby inhibiting potential acid generation, heavy metal mobilization, and contaminant transport in general. The textural data in Appendix H (SRK, 1996) indicate that the proposed cover materials are coarse textured. The available water holding capacity of the proposed cover materials was not specified, but is expected to be quite low. That is, a 12-inch thickness can likely hold 0.5 to 1.4 inches (roughly 4 to 12 volume percent) of water depending on the coarse fragment content (i.e., the higher the coarse fraction, the lower the water holding content).

SRK proposes to compact the waste rock (SRK, 1996) to reduce infiltration (or percolation) of water into the potentially acid-forming materials. The apparent mechanism assumed for reduced percolation is that compaction of the waste rock reduces the saturated hydraulic conductivity (limiting flux), and in response, infiltrating water will accumulate at the waste rock-soil cover contact because of the relatively low hydraulic conductivity of the underlying compacted layer when compared with the overlying cover material. Arresting infiltration at that contact then allows accumulated water to be removed by evapotranspiration.

SRK (1996, Appendix O, p. 19, paragraph 2) claims that compaction will result in a local decrease in conductivity, which, in turn, will lead to a reduced annual percolation rate. In the context of this unsaturated system, the reasoning is incorrect. Compacting a soil reduces saturated hydraulic conductivity because larger soil pores are reduced to smaller pores. With more small pores per unit volume, unsaturated hydraulic conductivity typically is higher in compacted soils than in uncompacted soils. Thus, conductivity is likely increased in this unsaturated system.

While the conductivity argument is likely invalid, it is also probably irrelevant. Seepage in this system is likely controlled by the water flux that passes the plant root systems (if present). Seepage in this system is highly unlikely to be controlled by the conductivity of the compacted waste rock. For example, the total annual precipitation of 13 inches is equivalent to a flux of 10^{-6} cm/s. Thus, even using the lower end of the waste rock conductivities assumed on p. 4 (i.e., 10^{-6} cm/s) and the upper end of seepage rates calculated in the modeling runs (approximately 2 inches per year), the system is not likely to become saturated and conductivity-limited. Of course, conductivity of the material does become relevant when designing a cover where the annual dynamics of water movement may result in acid redistribution into the topsoil from underlying waste rock.

Similarly, the compaction of the waste underlying a thin (6 to 12 inches) soil cover will reduce the amount of water that enters the waste only if the saturated hydraulic conductivity of the compacted material in contact with the cover is less than the supply rate of water. The supply rate of water through the cover materials is a function of soil water redistribution in relation to precipitation and the evaporative demand of the climate-cover-plant system. The permit application lacks sufficient detail to determine whether these conditions will be met at the Copper Flat mine site. Nevertheless, given the climatic regime of Copper Flat, the infiltration capacity of the covered waste rock is probably supply-limited, rather than flux-limited in most years, and the storage capacity of the proposed cover materials is likely to be exceeded at least during winter months.

Compaction of the waste materials may also actually increase rather than decrease the quantity of water that enters and ultimately moves through the wastes. Compaction will reduce the potential for root penetration and water removal by plant transpiration. This is a concern only if the upper surface of the waste is nontoxic to plants and can potentially support plant growth. If the wastes are toxic to plants, compaction will reduce the total volume of pore space, and a given volume of water may move deeper into the compacted wastes than uncompacted wastes. The depth of water penetration is important because evaporative rate and the depth to water in soils are inversely related. Thus, compaction of the wastes may even increase the potential for acid formation and metal migration. Therefore, the efficacy of waste compaction and thin cover placement should be evaluated more carefully.

5.4 Soil Cover Performance Evaluation

The following section addresses the methods and theories used to evaluate soil cover performance.

5.4.1 Capillary Barrier

In Section 2.1, the document incorrectly states that water will not flow into a capillary barrier until the air-entry pressure is reached. Capillary barriers do not prevent vertical flow. Capillary barriers are often described as preventing vertical flow until water potential increases to the water entry value (misstated as the air-entry value in the document) of the larger pores of the underlying coarse layer. The permeability on the coarse layer depends on water potential and must transmit some water vertically. The transmission rate must increase when subjected to a higher flux and

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potential. In short, the capillary barrier has some conductivity at all potentials and some water will always move through the capillary barrier if a potential gradient, such as gravity, is present.

Support for an assumed air entry value for ⅜-inch material of −10 cm is not given. Nor is the water entry value or its approximate inverse, van Genuchten ***a***, listed on p. 6 or elsewhere in this section. The efficacy of a capillary barrier cannot be accurately modeled without the unsaturated parameters for materials of both layers. A constant lower boundary condition equated to the air-entry value (as stated on p. 2) has dubious physical validity. Water-entry values are typically lower than air-entry values, which would increase flux through a capillary barrier. More importantly, the estimated ***a*** for the gravel is likely to exceed the 'real' ***a*** value in the field. For example, if 12 inches of local topsoil are placed over the gravel and the topsoil dries out, it is probable that the topsoil will sift down through the gravel. Water can also accumulate on the lower boundary of the capillary barrier, approach saturation, and move colloidal (clay) material down into the gravel layer. This is particularly likely if the soil is noncohesive and placed in thin and coarser textured covers. Burrowing animals and roots can also degrade capillary barrier performance. A combination of local variations in soil texture and drying can result in loss of soil cohesiveness in the field.

Small variations in the height of the capillary barrier-soil interface are also important in thin cover systems. For example, the thinnest engineered lift practical is on the order of 6 inches. Thinning the soil over a thick gravel layer will reduce local water storage and percolation performance. Making a low spot in the gravel layer will result in local pooling of water and the potential for preferential flow. The consequence of the preceding arguments is that flux through a capillary barrier is likely to be underestimated in the field.

The importance of some of these processes on percolation could be simply modeled using the Brakensiek et al. (1986) method, where the soil and gravel are partially recombined by sifting and elluviation over time. The suitability of the soil texture for a capillary barrier application could be tested by wetting-drying column studies using the proposed cover soils.

While these observations are not necessarily fatal flaws, an adequate capillary barrier design would require more study. In an area where annual rainfall is 13 inches with summer monsoons and the free water surface evaporation is approximately 65 inches, a simpler cover may be adequate. Testing a cover where cheaper topsoil is substituted for the gravel layer and the performance is compared within a single model (instead of using HYDRUS and HELP) and where vegetation and rooting parameters are more accurately specified may lead to better performance and reduced cost.

How soil samples were taken and processed is unclear. The assumed bulk density used in the model affects saturated hydraulic conductivity, field capacity, wilting point, ***a***, and ***b***. In turn, these parameters also affect capillary barrier performance. Incorrectly defined soil testing procedures would result in incorrect parameters for both the HELP and HYDRUS models and concomitant bias in percolation predictions. In addition, it is unclear that this single sample is representative of the hundreds of acre-feet of material contemplated as cover soil.

Finally, the Brakensiek reference on p. 2 is not given. The presumed reference is Brakensiek, D.L., W.J. Rawls, and F.R. Stephenson. 1986. Determining the saturated hydraulic conductivity of a soil containing rock fragments. Soil Sci. Soc. Am. J. 50:834-835.

5.4.2 HELP Modeling for Soil Cover

The conclusion on p. 19 (SRK, 1996, Appendix O) that a 12-inch cover will result in less infiltration than an 18- or 24-inch cover is based on a physical and botanical misinterpretation of the cover system as represented by the model. The HELP model was run with a fixed "zone of evapotranspiration" at 18 inches, irrespective of soil thickness or the nature of the underlying waste rock (e.g., the potential for acid rock drainage as discussed on pp. 21-22). In a shallower soil (i.e., less than 18 inches in this system), few roots would be expected to grow in acidic rock waste. In a deeper soil system (i.e., greater than 18 inches) we have observed rooting depths in areas near this mine covered with vegetation typical of that discussed in the draft Environmental Impact Statement that greatly exceed 18 inches.

The physical reality in a typical waste rock cover is that the soil depth controls rooting depth, not that a model-specified "zone of evapotranspiration" will result in a constant rooting depth of 18 inches. Thus, the rooting depth and "zone of evapotranspiration" are likely limited to the depth of soil added. If the model were rerun with the more reasonable assumption that the "zone of evapotranspiration" is equivalent to soil thickness, we would likely get the intuitive and more physically correct answer that a thicker topsoil layer is likely to result in less infiltration.

5.4.3 HYDRUS Modeling for Soil Cover

HYDRUS simply assumes that transpiration is limited by soil water potential. This is a reasonable simplification where agronomic crop growth is limited by soil water and not soil chemistry or climate. This is a less reasonable assumption in a thin cover over acid-containing materials in an arid environment where vegetation is often not easy to establish. The consequences of this assumption are that the percolation values obtained from HYDRUS are likely to be low.

In terms of quantitative results from the present modeling effort, where were the root parameters obtained? Accurate parameters are important because root distribution, particularly in the lower part of the profile, will likely make a significant difference in percolation.

5.4.4 Conclusions

1. The main objective of the soil cover performance evaluation appears to be to support a cover thickness of 12 inches. This value may ultimately be acceptable, but the reasoning supporting this conclusion and the associated modeling are flawed.

2. The capillary barrier modeling and reasoning are incorrect and incomplete. In addition, the same model should be used to compare the performance of a simple soil cover versus a capillary barrier.

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3. Insufficient detail is presented on how soil samples were selected, collected, and processed. All the modeling results hinge on numbers from a single core processed with uncertain methodology.

5.5 Miscellaneous Observations and Comments

5.5.1 General Deficiencies in Materials Characterization

Johnson (1996) incorrectly cites Glover (1977) when he indicates the texture of the native soils are silt loams and silty clay loams, rather than the sandy loams and sandy clay loams reported in the 1977 environmental assessment (Glover, 1977). Both the Johnson and Glover reports lack any reference to laboratory methods; thus, it is difficult to either evaluate or compare the laboratory data. For instance, it unlikely that the waste dumps were "high" in nitrogen in 1977, but were deficient in 1996. Differences in methods or the basis for reporting the data are more likely to account for the differences in laboratory results.

Johnson's ratings of the fertility status of the materials are difficult to evaluate because they are not qualified with respect to their source or application (e.g., agronomic or native vegetation). Methods of sample collection, preparation, digestion, extraction, and analyses are not clearly stated. The size of the materials analyzed for acid balance analysis is important to the results, as are the methods for sample preparation for soil pH measurements (e.g., 1:1 or saturated paste pH).

5.5.2 Vegetation Data Deficiencies

The baseline vegetation data lack a map of the vegetation types at an appropriate scale and a complete listing of the plant species that occur at the mine site. Additionally, the vegetation is not referred to by scientific names in either the Johnson (1996) report or supporting Natural Resource Conservation Service (NRCS) data forms. The use of NRCS range site data to establish baseline is questionable because the data were probably not identified in the vicinity of the mine site. Discussions with NRCS personnel revealed that the source of the range site data was unknown, but could have come from anywhere in southern New Mexico. No methods were included for description of the vegetation, and therefore the vegetation data collected probably do not meet the statistical adequacy requirements of the MMD guidelines.

5.5.3 Vegetation Data and Erosion Model Contradictions

Erosion was predicted using USLE with plant cover parameters ranging from 30 percent to 40 percent. It should be noted that the existing plant cover estimates made by NRCS (SRK, 1996, Appendix H) in the vicinity of the mine are much lower than those used by SRK and range from 3 to 15 percent.

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6. Summary and Recommendations

DBS&A has performed an extensive review of the technical information submitted by Alta as part of its Mine Permit Application. Overall, we have found the submitted materials to be of good quality, although in many instances lacking in site-specific supporting information. Too few data exist in many areas to allow conclusions to be drawn with confidence, and thus, there is considerable uncertainty surrounding the potential impacts of the mine. Additionally, some design information (e.g., the tailing pumpback system), while based on a valid conceptual model, lacks sufficient detail to allow a rigorous analysis of its efficacy.

Of course, there is expected to be some degree of uncertainty surrounding the environmental impacts of any new mining operation, and in many ways Copper Flat is advanced having some data from the brief mining activity there 15 years ago. For example, we know that there is some acid generation in the pit area, and that for a brief period acid conditions existed in the pit lake. We also know that there is currently enough buffering capacity to be maintaining the pit lake at neutral to slightly alkaline conditions. It has also been established that the proposed tailing operations will impact groundwater quality, resulting in contaminant concentrations above WQCC standards (since standards are locally exceeded at present from the brief period of tailing deposition in 1982).

The following sections summarize our findings as they pertain to each of the section of the report.

6.1 Predicted Impacts to Surface Water and Groundwater Supply

The potential impacts to surface water and groundwater were assessed using the three-dimensional, numerical groundwater flow model ABCFEM. A regional model domain was established extending from the crest of the Black Range on the west to the Rio Grande and Caballo Reservoir on the east, and from Las Palomas Creek on the north to Berrenda Creek on the south. The Copper Flat pit is located at approximately the center of the model domain.

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DBS&A's analysis focused on the regional groundwater flow model presented in Appendix D of the Groundwater Impact Evaluation dated March 21, 1997, which evolved from several earlier modeling efforts conducted by ABC. In general, it is our opinion that there are significant shortcomings with some aspects of the simulation approach and with some of the critical model input parameters applied. As a result, we do not believe the predictive simulation results to be conservative, and in some areas we believe the simulated results are potentially in error.

The principal concerns of our investigation include:

- There is no verification of the model's three-dimensional simulation capability presented in the Mine Permit Application or in the ABCFEM user's guide, suggesting it has not been verified. Therefore, DBS&A recommends that MMD not accept any simulation results without written documentation of the three-dimensional ABCFEM model verification.

- Steady-state model calibration simulations show results at some nodes that should not be possible according to the model documentation. Consequently, the resulting effects on the simulation results (and the reliability of those results) are unknown.

- Steady-state model calibration results overstate calibration accuracy at springs because the comparison between observed and simulated spring flow is not presented. The observed spring flow is the critical calibration criterion, not the presented hydraulic head at the spring pool.

- It appears that at many nodal locations throughout the model, layer 1 is unsaturated, even though layer 1 theoretically represents the uppermost 200 feet of saturated thickness. The result of this is a reduction in the thickness of the aquifer in some areas.

- The simulated hydraulic conductivity of the quartz monzonite of 8 ft/yr is likely on the low side of probable values and is not conservative. A realistic and appropriately conservative sensitivity analysis would apply an andesite and monzonite hydraulic conductivity of at least 25 or 30 ft/yr.

- A storage coefficient of 0.01, representative of near-surface materials, was used over the entire 1,000-foot thickness of quartz monzonite in calculating a hydraulic conductivity based on pit dewatering.

- The assumed contributing thickness value of 1,000 feet applied to compute hydraulic conductivity from the transmissivity calculated from pit dewatering is unsubstantiated and, in our opinion, inappropriate. A much more reasonable number would be something between 45 feet and perhaps 100 feet, and such a number would be more consistent with the "near-surface material" storage value of 0.01.

- The utilized hydraulic conductivity of 4,880 ft/yr in the Santa Fe alluvium east may be inappropriately low. It was computed from transmissivity values obtained from MW-5 at the production well field divided by an aquifer thickness of 2,000 feet, although the saturated thickness at MW-5 was only about 660 feet when the aquifer tests were conducted. Use of the 660-foot thickness yields a hydraulic conductivity value in the range of approximately 11,000 to 15,000 ft/yr.

- The effect of using a lower hydraulic conductivity in the model is that simulated drawdowns near the production well field will be increased, while simulated drawdown in regions of concern such as Las Animas Creek and Caballo Reservoir are diminished.

- DBS&A does not agree with the horizontal anisotropy factor of 10 used for all of the Santa Fe alluvium zones. It appears to based on hydrologic speculation rather than on direct field data or scientific precedent for simulating similar hydrogeologic systems in New Mexico.

- The effect of the anisotropy ratio used by ABC is to exaggerate simulated drawdowns in the north-south direction and underpredict them in the east-west direction. Therefore, drawdown beneath Las Animas Creek is increased, while drawdown at Caballo Reservoir is decreased relative to expected values.

- There is no verification of the validity of the storage terms obtained during the model calibration process. DBS&A recommends applying the lowest observed or reasonably calculated specific storage values during the base case predictive (transient) simulation.

- The model significantly overestimates baseflows throughout the entire reach of Percha Creek, from Hillsboro downstream to some 5 miles below Percha Box. This overestimation indicates that the model is not well calibrated in this region, with respect to baseflow or hydraulic head. Therefore, the model is not appropriate or conservative for performing predictive simulations of potential impacts to the Percha Box area.

- The overprediction in simulated baseflow artificially routes water from the Percha Box area eastward toward Caballo Lake and the production well field area. Approximately 1,600 gpm of excess water that exists in Percha Creek in the simulation is thus infiltrated back into the groundwater system southwest of the production well field.

- DBS&A's main concern with respect to simulation of Las Animas Creek is that if the simulated groundwater flow field does not match the observed conditions fairly closely, the accuracy of predictive simulations cannot be evaluated. A very important item missing from ABC's analysis is a water balance developed specifically for the Las Animas Creek alluvial aquifer.

- DBS&A also has not yet received the analysis of the second Las Animas Creek aquifer test.

- Important groundwater flow components in the Las Animas Creek alluvial system, such as groundwater inflow from the alluvium upstream of Warm Springs North and groundwater discharge by phreatophytes along the creek, are not presented or, apparently, considered in the analysis.

- Although ABC states that Las Animas Creek is ephemeral, the model simulates surface water flow at every Las Animas Creek model node in the model calibration run.

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- The match between the simulated and observed water balance for the Las Animas Creek alluvial aquifer could not be determined. Near the western end of the simulated alluvial aquifer, DBS&A calculated the groundwater discharge for the valley from the simulation results to be about 770 gpm, yet ABC estimates that the groundwater flow capacity of the shallow alluvium is approximately 1,400 gpm.

- The simulated hydraulic conductivity of the alluvium is 110,000 ft/yr (301 ft/d), which appears low relative to aquifer tests conducted at the Saladone Well and Shipping Pen Well on the Ladder Ranch, where hydraulic conductivities as high as 476,000 ft/yr (1,300 ft/d) were calculated.

In summary, ABC has not provided sufficient information for DBS&A to make a sound determination concerning simulated impacts to Las Animas Creek.

6.2 Data Gaps in the Assessment of Tailing Impoundment Performance

Based on DBS&A's review of relevant documents, we have identified the following areas where additional data and analysis are needed for the assessment of tailing impoundment performance:

- Baseline (background) conditions for inorganic constituents appear to be based on newly installed (1994) wells located downgradient of the impoundment. These wells may already be impacted by seepage through the tailing.

- The extent and hydraulic characteristics of the clay that separates the perched and lower aquifers have not been fully determined, adding uncertainty to seepage estimates.

- The hydraulic conductivity of native material beneath the proposed location for tailing should be determined in greater detail.

- It may be appropriate to rerun the numerical model using saturated hydraulic conductivity values determined from local aquifer tests.

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- Design issues, including grain-size separation during deposition, tailing compaction, and water management, must be addressed in greater detail.

- Methods for intercepting seepage through the toe of the starter dam should be addressed.

- Documented analysis of phreatic surface control within the impoundment is needed.

- A sensitivity analysis of the range of potential seepage rates and potential impacts on containment should be completed.

- Vertical mixing of seepage within the lower zone should be modeled in order to determine more realistic completion depths for pumpback wells.

- The period of transient drainage should be evaluated using a range of drainable porosities and permeability characteristics.

- A more detailed analysis should be made to determine how pumpback water will be disposed of during the post-closure period.

6.3 Predicted Impacts from Acid Rock Drainage

The following discussion summarizes our concerns with respect to predicted impacts from acid rock drainage.

- DBS&A disagrees with SRK (1995, Section 5.4.4) that the unoxidized portions of the ore body do not pose a threat to water quality. Although the sulfide minerals in this rock have not oxidized appreciably over millions of years, we believe they could do so rapidly upon exposure to oxygen, low pH waters containing appreciable dissolved ferric iron, or both. Waste rock containing unoxidized sulfide minerals can and has created significant ARD problems at numerous other sites. It has been shown that even completely unoxidized

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sulfide minerals can be rapidly oxidized when the water table is lowered and the materials are exposed to air in the vadose zone (Earley et al., 1995).

- We disagree that acid currently being generated in the vicinity of the existing pit is entirely attributable to past solution mining activities that used sulfuric acid (SRK, 1995, Section 5.2.6). While solution mining may have accelerated the ARD process in some areas, we believe that, given the mineralogy of the rocks, ARD generation would be evident regardless of such activities.

- Although laboratory kinetic tests appear to show that acid production rates are very low, the validity of the 28-week tests in simulating field conditions is questionable. Based on the increasing rates of acidity production (SRK, 1995, Figure 5-10), some of these materials could eventually begin yielding appreciable acid and sulfate if the tests were run longer (White and Jeffers, 1992). While the rates of ARD generation will likely be less at Copper Flat than at mine sites with higher total sulfide concentrations, the existing acid seeps and acid sulfate salts at Copper Flat are ample evidence that, once initiated, ARD can and does occur in rocks at the site.

- SRK's recommendations for waste rock management and tailing management (SRK, 1995, Section 5.5.3 and 5.5.4) should be clarified. For example, if cover material is to be applied (as mentioned in Sections 5.5.3.4 and 5.5.4), the type and depth of the material should be specified.

- We disagree with SRK's assertion that "infiltration flux and subsequent leaching of (waste rock) sulfate or metals within the proposed period of operations is not likely to occur" (Section 5.5.3.2). To the contrary, evidence of recent seepage from the toe of the west dump was observed by DBS&A during a site visit on August 27, 1997. This seepage appeared to result from surface runoff in the former drainage in which the waste rock was deposited, which confirms that acid generated in the waste rock can be transported away from the dump during wet weather.

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- At present the water quality data are piecemeal, conflicting, and questionable. Discrepancies and gaps in the data set make it much more difficult to predict future water quality. Therefore, a routine program for monitoring the quality of pit water, groundwater, and surface water and surface water flows should be implemented, as detailed by SRK (1997, Table 3.1). Surface water monitoring should include storm event sampling, as the long-term pit-water quality will largely depend on the chemistry and quantity of infrequent stormwater pulses into the pit.

- Sampling of the pit lake at two depths by Newcomer et al. (1993, p. 20) suggested a lake with uniform composition that is not stratified; however, this observation is not certain from the limited data. Therefore, pit water quality should be profiled against depth before the pit is drained. This information will be used later to predict future pit-water quality. Temperature, electrical conductivity, and pH can be continuously profiled from a small boat by lowering a long weighted plastic tube into the lake. A small battery-powered peristaltic pump in the boat can continuously pump water through the tube. Water samples can be collected for laboratory analysis at depths corresponding to significant variations in water quality based on the electrical conductivity measurements.

- An on-site weather station should be established to record site-specific precipitation and other important meteorological variables, as described by SRK (1997, Section 7). These data will be invaluable in refining and updating the previous water balance calculations, as well as in verifying the validity of SRK's model for geochemical evolution of the pit lake.

- DBS&A agrees with SRK's recommendation to divert Greyback Wash into the pit following mine closure. This action will drastically reduce the rate of sulfide mineral oxidation by submerging remaining portions of the ore body below the water table and should also neutralize acid generated in the vicinity of the pit lake by alkaline calcium-bicarbonate-type surface water.

- Before the tailing pond is reactivated, the paste pH of the upper few feet of the existing tailing should be profiled in detail along several transects. This information can be used to estimate oxidation rates in the tailing over the past 15 years. The two tailing samples that were collected by SRK are insufficient for this purpose.

6.4 Reclamation and Closure Plan

Based on DBS&A's review of relevant documents, we have identified the following deficiencies in reclamation and soil cover design performance evaluation:

- The proposed topdressing cover thicknesses of 6 to 12 inches may be inadequate to provide sufficient water storage capacity to limit percolation into the waste materials and to support viable plant communities.

- The availability of borrow materials is predicated on the assumption that 6 to 12 inches of cover will provide adequate long-term environmental protection. Additional borrow sources should be identified.

- Alta's conceptual design hinges on the use of waste rock and tailing as soil substitutes. However, the suitability of the waste rock and tailing as soil substitutes is poorly supported. The issues of acid formation and toxicity were not addressed in a substantive manner.

- The capillary barriers and compaction of the wastes were proposed to complement the use of relatively thin covers. The theoretical basis for the selection of this design is physically flawed and poorly supported by their materials characterization. The capillary barrier design is flawed by the insufficient water storage capacity associated with a thin cover design.

- The limited materials characterization data do not support the parameters used in models that form the basis for the conceptual design.

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