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001958

Mr. Dick Arndt, P. Eng.
Chief Engineer
Anvil Range Mining Corporation
P.O. Bag 1000
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Dear Mr. Arndt:

Re: Geotechnical Site Review

As per your request, Piteau Associates Engineering Ltd. has completed a brief geotechnical review of a number of aspects that relate primarily to mining of the Grum Pit. The field review was carried out between May 24 and 26, 1995, when Mr. A. Stewart visited the mine, met with yourself and other mine personnel, and conducted a reconnaissance and assessment of the various aspects under consideration. At the conclusion of the site visit, a brief meeting was held with yourself and Mr. D. Hindy to outline preliminary observations and comments resulting from the review. These observations and comments are summarized below under the general topics outlined during the site visit. References to direction are generally relative to the local mine grid.

PIT SLOPE DESIGN

At the time of the site visit, mining in the Grum Pit had reached the 1240m elevation over about half of the pit area. On the east and south sides of the pit, the pit wall is up to about 60m high and is being developed to its ultimate limits as defined by the present mine plans. A large portion of the east and south walls have been excavated in overburden. Much of the overburden is comprised of glacial till, described as a very dense, cohesive, well graded gravelly sandy silt with some clay. This material is underlain by a complex mixture of ice-contact sediments (some of which are permeable) and glacial till which overlie bedrock. Along the east side of the pit, the benches that have been excavated in the overburden appear to be performing well, with only a minor amount of seepage observed on the slope. This seepage is interpreted to originate primarily in the near surface veneer of colluvium and alluvium. As anticipated, along the southern side of the pit, and particularly in the southwest corner, seepage was more pronounced and a minor amount of erosion and sloughing was occurring within silty sand lenses. A sump had been established to intercept and collect overburden seepage in this area.

The only bedrock that has been exposed along the east or south walls is along the northern portion of the east slope. In this area, the S2 foliation in the phyllite has a dip/dip direction of about 20-30/250. At about Section 6400N, a bench scale planar failure occurred from just above the 1252m safety berm (and just below the overburden/bedrock contact) about one week after the bench was mined. Rather than sliding along S2 foliation, which in this specific location has a 30° to 40° dip, it would appear that the basal failure plane was a "slip" with an orientation of about 45/225 (i.e. a fault or shear-like structure that strikes slightly oblique to and dips steeper than S2 foliation). Lateral release for the failure appears to have been along a set of steep dipping faults that strike approximately perpendicular to the trend of the wall. It is understood that a similar failure appears to be occurring just north of this area at about Section 6500N. In this regard, cracking and some movement has been observed about 10 to 15m behind the bench crest in the area of the sump on the 1272m safety berm. It is noteworthy that similar features were encountered in the eastern wall of the Vangorda Pit and are likely to be encountered at depth in this area of the Grum Pit.

The north and west walls of the pit are up to about 50m high and excavated almost entirely in bedrock. These walls are interim slopes that will be pushed back in later mining phases. Based on a brief inspection of the walls, and discussions with mine personnel, it would appear that S2 foliation dips essentially into the pit wall in most areas. While a large amount of ravelling debris is present on some berms, the north and west walls appear to be performing reasonably well and no significant geotechnical concerns were noted.

Based on the above, it would appear that the present interramp slope angle in the overburden is suitable and that no slope design changes are required. Other than in the southwest corner of the pit, where seepage has caused minor erosion and sloughing, seepage and groundwater pressures appear to be under control and do not appear to be a significant concern at this time; however, continued seepage and sloughing within the overburden in this area is anticipated. On the east side of the pit, where little bedrock has been exposed to date, and two bench scale failures along "slips" have been reported, it is suggested that the design 40° interramp slope angle should not be exceeded, and may have to be reduced slightly. As noted above and in our report to Curragh Resources dated April 23, 1992, there are structural and lithologic similarities between this wall and the eastern walls in the Vangorda and mined out Faro Pit, where there was considerable difficulty in establishing final wall slopes. In the Faro Pit, interramp slope angles for the eastern wall of 35° to 43° were recommended, depending on such factors as groundwater conditions within the wall, use of control blasting, etc. Portions of the eastern wall of the Faro Pit eventually ravelled and degraded to an overall slope angle of about 36°. With regard to the remaining walls of the pit, where S2 foliation dips along or into the slope, it would appear that the present interramp design slope angle of 45° is reasonable.

Considering the height of the slopes that have now been developed, particularly on the eastern side of the pit, it is recommended that a program of pit slope monitoring be instituted. In this regard, it is suggested that a general array of prisms (i.e. about ten prisms on various benches and at the crest) be initially established on the eastern wall of the pit to measure the response of the walls to mining and to monitor any areas of potential instability. Additional prisms should be installed on this and other walls as the pit is deepened and/or if areas of instability are identified. In addition to prism monitoring, regular visual inspections of all pit walls should be conducted.

To better rationalize and optimize slope design parameters for bedrock slopes in the Grum Pit, it is recommended that an initial program of data collection and analysis, followed by an ongoing assessment of overall slope performance, be carried out. This approach is particularly suited to pits that will be mined in a number of phases or pushbacks, where the interim pit walls can be used as trial slopes (such as on the northern and western sides of the Grum Pit) from which final slope designs can be determined. Although this approach should work well for much of the pit, the present mine plan does not include interim slopes on the southern portion of the east wall, where the flattest slopes are planned. While the use of core orientation has been considered as a means of obtaining additional geologic structural data on which to base a final slope design, it is felt that the incompetent nature of much of the rock mass, particularly along the eastern wall where such information would be critical, would considerably limit the amount of useable data that would be obtained by such a technique. Thus, it is recommended that geologic structural mapping of the interim and ultimate pit walls as they are developed be the prime source of information for any slope stability studies, as it is felt that this information would provide the best source of structural data. In conjunction with the structural mapping, it is recommended that documentation of the slopes (i.e. including noting actual bench face angles and their controlling mechanisms, berm widths, bench breakback, etc.) be carried out. It is likely that slope documentation information from the northern end of the east wall of the Vangorda Pit would also be helpful in evaluating the likely behaviour of the east wall of the Grum Pit.

With regard to the presence of "major structures" (i.e. faults) that could individually or in combination influence the stability of a large portion of a pit wall, it suggested that an update of a review conducted by Piteau Associates in 1992 be carried out. As it is understood that additional level plans and cross-sections are now available, it is suggested that geologic structural contour plans of each of the major faults that have been identified (e.g. Doal Fault, UPP Fault, etc.) be updated and that kinematically possible failures involving these structures be assessed with respect to the interim and ultimate pit walls.

To assess the overall competency of the rock mass that will be exposed on the final walls, particularly on the eastern side of the pit where poor competency rock along with adversely oriented geologic structure could significantly effect slope stability, it is suggested that the core from the upcoming exploration program (i.e. understood to be planned for this Fall) be geotechnically logged. Such information can be assessed with regard to the potential for rock mass failures and to evaluate variations in rock mass competency with depth and between rock types.

It is suggested that much of the data collection for the above work can be conducted by mine personnel, with Piteau Associates conducting a sufficient amount of data collection to become familiar with relevant geotechnical aspects of the rock mass, and to provide initial direction and guidance with respect to data collection format, procedures, etc. If mine personnel do not have the time for such work, Piteau Associates could provide as much assistance as necessary. Piteau Associates could then compile and process the mapping, slope documentation and rock mass data and carry out appropriate stability assessments.

During Piteau Associates' involvement with previous operations in the Faro and Vangorda Pits, it became apparent that blasting had a significant influence on the stability and general condition of the walls. In this regard, it is the writer's opinion that the use of an engineer whose primary function was to design all production and final wall blasts yielded definite benefits with respect to the stability of the slopes, and the ability of the mine to maintain design slopes. It is recommended that consideration be given to the use of such an individual.

SUMPS IN VICINITY OF EAST PIT WALL

Two sumps were inspected during the site visit. The first, located toward the north end of the 1276m bench, has been excavated in bedrock. This sump is intended to act as a temporary holding pond or staging pond for water that is being pumped out of the pit to the treatment plant. Notwithstanding that there is an apparent bench scale failure occurring just west of and on the same bench as the sump, the use of sumps on pit benches is not considered to be normal practice unless it has been carefully lined such that there is no leakage from the sump that could recharge the pit wall and possibly destabilize the slope. The sump that was discussed in our report of April 23, 1992 was intended as a possible means of locally depressing the water table and collecting seepage from the vicinity of the overburden/bedrock contact, and not as a holding pond for water from the pit.

A second sump, which is not in operation, has been excavated north of the water treatment plant about 200m east of the pit. This sump is also intended to be a temporary holding pond for water being pumped from the pit to the treatment plant. The sump is in the order of 4m deep and built primarily in cut, with the western side of the pond appearing to be comprised of up to about 2.5m of fill. It is understood that the pond, which has inner slopes of about 20°, has been lined with an approximately 1m thick layer of glacial till that has been spread and compacted by bulldozer. Foundation conditions under the pond are uncertain; however, based on subsurface conditions in the surrounding area, the pond is likely underlain by compact colluvium, dense glacial till and/or bedrock. Apparently the pond will be operated at a depth of only about 1 to 2m. It has been proposed by Anvil Range that before pumping water to the pond from the pit, the pond would be tested for leaks by pumping clean water into the pond and monitoring the water level and surrounding area over a period of time. Based on this plan, no significant geotechnical concerns are apparent. The pond is sufficiently removed from the pit that in the event of instability of the pit wall, it is highly unlikely that the integrity of the pond would be jeopardized. The greatest concern related to this pond and the pit is the potential for any leakage to seep on top of the underlying dense glacial till towards the pit. This could cause minor erosion of the exposed near surface organic and colluvial soils at the pit crest, and would also result in recirculation of pit water back into the pit. Icing of the pit crest would likely occur in the winter. Both of these occurrences are felt to be primarily of operational rather than geotechnical concern.

INTERCEPTION DITCH ON EAST SIDE OF GRUM PIT

The interception ditch that has been excavated about 100m behind the east wall of the Grum Pit at about a 1% gradient, is an unlined ditch that serves to intercept surface runoff and groundwater before it enters the pit and to act as a conduit for water from the pumping wells on the east side of the pit. While the ditch was not inspected in detail, it is anticipated that it has been excavated into surficial soils that are comprised of compact mixed grained colluvium overlying a very dense, cohesive, well graded gravelly sandy silt with some clay, which in turn overlies bedrock.

It would appear that under present conditions, any seepage from the ditch is not recharging the deeper soils, but is flowing along the top of the dense glacial till toward the pit, eventually surfacing on the pit wall near the slope crest. At the time of the site visit, seepage on the pit face was minimal and not causing any significant concerns. To limit seepage from the ditch, it is suggested that wherever practical, the ditch be excavated such that any flow is within a trench in the dense glacial till. In areas where founding the ditch in the dense till is impractical, it may be possible to line the ditch with a compacted layer of till. As the till is expected to be self armouring, it is felt that lining the ditch with rip-rap will not likely be necessary. An alternative to ensuring that the ditch is excavated into relatively impervious material, would be to line the ditch with some form of artificial liner, such as a half culvert, or to install a pipeline along the ditch. While these solutions would be best for transmitting concentrated flow sources (such as from the wells or water pumped from a stream diversion), any groundwater seepage or general surface runoff along the ditch alignment would still have to be carried in the open ditch. Pipelines have the potential disadvantage of having freezing problems should pumps malfunction in sub-freezing weather.

With regard to the area behind the northeast corner of the pit, which is just north of the north end of the present ditch, it is understood that considerable icing and seepage problems were encountered during the recent spring thaw. Apparently runoff flowed down a draw, saturating a surficial peat/muskeg layer that is at least 2m thick. While there appears to be a dense glacial till horizon underlying the organic material, the elevation of the dense material is apparently lower than the invert of the ditch to the south, thus precluding gravity flow from the wet area to the ditch without lowering the ditch invert along the entire east side of the pit. Based on the above, two possible solutions are suggested. The first would be to excavate a ditch and sump across the draw to a sufficient depth such that any seepage from the draw would be intercepted and prevented from flowing toward the pit. The sump could then be pumped on an as needed basis. A second possible solution would be to construct a cutoff berm across the draw by excavating a trench down to dense glacial till and filling the trench with compacted backfill. Seepage and surface runoff would backup behind the berm and, depending on grades, could then either flow by gravity into the existing ditch or be pumped into the ditch. In order to determine the optimum solution, it will be necessary to more accurately determine elevations of the ground surface, top of the dense till layer, ditch elevations and grade, etc.

WASTE DUMPS

Three waste dump areas are being utilized for the Grum Pit. It is understood that final configurations for these dumps are still being prepared. The first dump, which is located south of the pit, is referred to as the Till Dump or Southeast Overburden dump. At the time of the site visit, this dump appeared to be performing well and no geotechnical concerns were apparent. It is understood that an extension to the Till Dump is being planned to accommodate an increased volume of overburden soils. In this regard, Piteau Associates conducted an assessment for an extended dump that was to be built to the east of the present Till Dump and powerline and to the south of the water treatment plant. Results of this assessment are summarized in our report to Curragh Resources dated October 30, 1992, and include design criteria for the extended dump. Assuming that the dump extension being planned by Anvil Range is in the same area as that investigated in 1992, and that the size and configuration of the dump are essentially the same, it is unlikely that additional geotechnical investigations are required or that the design criteria need to be altered. However, it is suggested that once the dump plans have been prepared, they be reviewed for conformance with the design criteria.

The primary waste dump for the Grum Pit is referred to as the Main Dump, located just south of the Grum Pit on the west side of the haulroad to the Vangorda Pit. At the time of the site visit, waste rock was being dumped on the northern (i.e. upslope) portion of the dump and no geotechnical concerns were apparent. Two preliminary geotechnical assessments have been carried out in the past for this waste dump; however, neither assessment involved conducting detailed stability analyses. The first, which was general in nature and summarized in our report dated August 14, 1992, was conducted without the benefit of subsurface investigations and was based on broad assumptions regarding foundation materials, material strengths, etc. The second study, the results of which are summarized in our report of November 10, 1992, was a general engineering geology assessment of the proposed dump area. While it was concluded that foundation conditions were favourable for most of the waste dump area and the dump configuration under consideration at that time, some areas of the dump (i.e. particularly in the Grum Creek area) required further investigation. The scope of work that would be required to finalize dump designs depends to a large degree on the proposed configuration of the dump. For instance, if the dump will not interfere with Grum Creek and will be confined to the area where subsurface geotechnical information is already available, it is likely that little, if any, field investigation will be required. In this case, it may only be necessary to carry out stability analyses of the proposed dump configuration. If, on the other hand, the dump will extend further to the west or will fill in the Grum Creek valley, additional test pitting, sampling and laboratory testing will likely be necessary. While filling in the Grum Creek valley could be accomplished, a number of additional factors, including the extent of the groundwater discharge area, the competency of waste rock placed in the valley bottom, the ability of the dump material to act as a rock drain, etc., would have to be considered. It is noteworthy that while it is likely that the discharge area in the Grum Creek valley will dry up in the short term, it must be assumed that Grum Creek valley will have to carry surface and groundwater on abandonment after the pit fills with water.

The third waste dump, referred to as the Southwest Dump, is located to the west of the Grum Pit. No field investigations or stability analyses have been conducted for this dump. Thus, after an initial dump configuration has been prepared, it is suggested that it be reviewed to estimate the level of effort required to finalize the design. Based on the limited amount of information that has been reviewed concerning this site, it is likely that field investigations could be limited to the excavation and logging of backhoe test pits, followed by a limited amount of laboratory index testing and stability analyses.

PELLY POND

Based on discussions carried out with mine personnel and Mr. Gregg Jilson of Access Mining Consultants Ltd., it is understood that consideration is being given to using what is referred to as the Pelly Pond as a temporary treatment pond (i.e. to be used over a maximum period of about four months, until approximately September) for water being pumped from the Vangorda Pit. The embankment forming the pond would be removed prior to abandonment of the mine. The background for the planned use of the temporary impoundment is discussed in a document prepared by Mr. Jilson in late May 1995 and will not be repeated in detail. In summary, however, it is understood that the Pelly Pond embankment was constructed in 1990 by Curragh Resources as a temporary holding pond for contaminated water from the Vangorda area while the Little Creek water containment system was being constructed, and as an access to the Vangorda Waste Dumps for haul trucks. Apparently the embankment was constructed of local glacial till material, with the fill material being spread and compacted by scrapers, bulldozers and haul trucks. It is also understood that the embankment has retained water from time to time since being constructed. Two, twenty inch diameter decant pipes have been installed in the embankment about 3m below the crest and a third pipe is located at the base of the pond, all of which can serve to control the water elevation within the pond. It is noteworthy that with the exception of precipitation that may fall into the area immediately surrounding the pond, virtually all of the inflow to the pond can be directly controlled by regulating the pumping from the Vangorda Pit. Any inadvertent release from the Pelly Pond would flow a short distance into the Little Creek impoundment, which apparently has sufficient capacity to store over twice the volume from the Pelly Pond, while still maintaining about 1.4m of freeboard.

At the time of the site visit, the water level in the pond was about 2m below the decant pipes. The crest of the embankment was approximately 11m wide, with the upstream and downstream embankment slopes being about 32° and 35°, respectively. Based on a temporary trench in the crest, the fill material appeared to be comprised primarily of a sandy silt with cobbles and boulders up to about 0.4m in diameter. No seepage was observed on the downstream face of the embankment and no signs of distress were seen in the entire structure.

While the Pelly Pond embankment is not an "engineered structure" and the stability of the embankment has not been assessed in detail, it apparently has retained water on a number of occasions to the operating level that is being considered without any significant consequences. Thus, considering the almost absolute control that Anvil Range has on the inflow to the pond, the history of the embankment, the limited time period over which the pond would be used and the ability of the Little Creek Pond to store over twice the volume of water retained by the Pelly Pond, it is our opinion that the embankment in question would likely perform as anticipated without experiencing instability. Even if instability did occur, there appears to be little potential for a catastrophic failure that would destabilize the Little Creek Pond.

LITTLE CREEK POND

The Little Creek Pond is a permanent facility that will be retained after abandonment of the mine. The embankment for this pond apparently is an engineered structure that was constructed under engineering supervision. While the present structure does not include a spillway, and design drawings for the structure have not been reviewed, it is understood that a spillway may have been planned for abandonment. At the present time, the only engineered means of discharging water from the pond is a pipeline that returns water to the water treatment plant.

With regard to the need for a spillway, such facilities are normally mandatory for ponds that will remain in permanent service after abandonment. The use of decant lines is not favoured as an alternative as they are usually considered to require ongoing maintenance. Notwithstanding this, if conditions are such that there is little or no catchment area above the pond, it may be possible to use an alternative to a conventional spillway. For example, it may be acceptable to install a "fuse plug" into a shoulder of the embankment. In the event of the embankment being overtopped, the fuse plug, which is a designed low point in the shoulder, would erode slightly without jeopardizing the integrity of the main embankment. Such a solution can be relatively inexpensive. In any event, specific requirements for the Little Creek Pond can only be determined following a hydrologic assessment of the Little Creek Pond area.

I trust the above is sufficient for your needs at this time. If you have any questions concerning this report, please contact me.

Yours very truly,

PITEAU ASSOCIATES ENGINEERING LTD.

Alan F. Stewart, P.Eng.

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