APPENDIX A

REVIEW OF OPEN PIT DESIGN
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SECTION 4.0 - OPEN PIT SLOPE DESIGN

4.1 FACTORS WHICH INFLUENCE PIT SLOPE STABILITY AND SLOPE ANGLE

The most important factors which influence rock slope stability of open pit mines are geologic structure, groundwater conditions and seismic acceleration forces due to blasting.

Where geologic discontinuities, (joints, bedding planes, foliation, shears, faults, etc.) singly or in combination dip out of the slope at angles near or in excess of the angle of friction of the discontinuities, a potential for failure exists. It is essential that the geologic model of discontinuities around the pit be determined and the kinematic potential for failure evaluated. Typical failure models are shown in Figure 4.1. They include circular, planar, wedge, block and toppling modes.

Where multiple bench failures can occur along discontinuities, it is normally necessary to obtain samples to perform direct shear tests along these discontinuities. The surface roughness and waviness along the discontinuity must be evaluated in the direction of sliding. This may increase the effective angle of friction by ten degrees or more. Both conditions require assessment in order to evaluate the safety factor for that portion of the slope.

The presence of groundwater in the slopes may influence stability in a number of ways:

a. Reduction in the frictional shear strength due to buoyancy.

b. Reduction in cohesion of clay gouge or clayey rock with increasing moisture content.

c. Development of seepage forces during drainage towards the pit slope.

d. Creation of hydrostatic forces in tension cracks during heavy rainfall or snow melt (this pressure increases as the square of the depth).

e. An increase in hydrodynamic shock due to blasting below the water table.

As a result, it is important that the water table and groundwater pressures be maintained low in the pit slopes. The most effective way to develop this control is with the installation of horizontal drains. Any tension cracks that develop should be monitored and filled in to prevent the build up of water pressures.

Seismic acceleration forces due to blasting must be reduced at the final pit face to allow the steepest practical slopes to be used to minimize the waste to ore ratio. This will usually comprise using controlled blasting for the line holes, angled line holes, buffer holes adjacent to the line holes, numerous delays and blasting to a free face. Photographs of representative conditions are enclosed in Figure 4.2. The relationship between distance from the blast, pounds per delay and particle velocity (related to damage) is given in Figure 4.3.

4.2 STRUCTURAL GEOLOGY

Three boreholes, MP89-152 to MP89-154 were drilled on an inclination and the core was oriented using the clay imprint procedure as discussed in Section 3. Stereographic plots are shown in Figures 3.2 to 3.5. The overall plots indicate the major joint set has an average strike of 167 degrees and an average dip of 75 degrees E. The strike variation is about $\pm$ 20 degrees and the dip variation about $\pm$ 10 degrees. A second set of joints strikes 30 degrees $\pm$ 20 degrees and dips 18 degrees W $\pm$ 10 degrees. These geological structures are the major features which will influence pit slope stability for all walls.

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This structure will lead to certain types of potential instability in each pit face depending on the face direction. The pit slope design must accommodate this potential.

East facing slopes	-	planar failure along shears, faults and contacts
North facing slopes	-	shallow wedges along joint intersections or fault contacts
South facing slopes	-	local wedges adjacent to fault contacts
West facing slopes	-	toppling

Rock joint continuity data (R.Q.D. and Joint Frequency) is shown in Appendix B, together with unconfined compressive strength data. Generally the rock strength is moderate to high so that stability will be controlled by the geologic structure. Several local zones of high fracturing and low strength were noted, i.e. - hole MP89-152, depth 180-350 feet. When the preliminary pit design has been developed, it will be necessary to evaluate rock strength data where drill holes intersect the pit walls to determine if any special design modification would be required. The west facing slopes must be evaluated in particular. If weak zones dip into these slopes, stress relief and high stress could cause subsidence and over stress the overlying rock to initiate toppling movement.

4.3 EVALUATION AND CONTROL OF GROUNDWATER

Groundwater is generally reasonably shallow below the ground surface. In some drill holes it was artesian. Consideration must be given to developing drainage of the pit walls to allow steep slopes and drainage in the pit floor to prevent pit bottom heaving. Wet blast holes will also require the use of more expensive explosives than the standard ANFO.

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The projected open pit dewatering requirements have been estimated from the permeability test results and by a general water balance incorporating climatic data and hydrogeologic information. The volume of recoverable groundwater stored in the bedrock fractures and fissures has been estimated by assuming a specific yield of 0.005. Open pit dewatering systems will be required to remove this water from storage. Additional open pit inflows will include direct precipitation and groundwater recharge from the area of influence. The hydrometeorologic data presented in Section 2.1 has been adjusted for runoff and transpiration losses to enable a prediction of annual recharge to the groundwater system.

The rate of groundwater inflow is controlled by the permeability of the bedrock. The bedrock permeability is expected to be anisotropic due to structural features. Rock structure provides a barrier to groundwater movement across relatively intact formations and low permeability clay gouge but also results in preferential flow parallel to fractured rock zones. The site investigation program outlined in Section 3.2 identified zones with a broad range of permeabilities.

It is anticipated that the principal directions of groundwater inflow will be parallel to the dominant structural features which trend roughly north and south. This groundwater flow trend is advantageous in the later years of operation, when the ultimate depth of the west pit is below the surface elevation for Bootjack Lake. In general, the rock structure provides a natural barrier to flow from Bootjack Lake into the bottom of the ultimate pit. However the approximately east-west trending fault structure in the west zone, as shown on Figure 2.3, may provide a high permeability conduit for groundwater flow. On-going evaluation of pit geology during operations will enable accurate predictions to be made prior to final development.

It is estimated that the average, steady state pumping requirements for open pit groundwater control over the life of the project will be approximately 15 L/s (200

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Igpm). This estimate does not include for removal of direct precipitation which will average approximately 13 L/s (175 Igpm) annually. The actual pit dewatering rate will be higher in the early stages of the operation when groundwater is removed from storage, and during periods of high rainfall.

Generally the most effective ways of controlling seepage into the pit is to install horizontal drain holes. They should be 20 to 30 metres (60 to 100 ft) long, drilled at a +2 to +3 degree slope and spaced about 12 metres (40 ft) apart at the toe of each double bench where seepage is encountered. In order to locate seepage zones, all blast holes which have water in them along the line holes and buffer holes should be plotted on blasting plans. The horizontal drain holes are only installed in areas where seepage is indicated.

If wet holes become a problem, a program to drill every tenth to twelfth blast hole an additional 6 to 9 metres (20 to 30 ft) deep and blast to that lower depth will provide a groundwater sink and help to lower the water table in the blast area. If this procedure is not successful, pumping wells at a number of locations around the pit may be required.

In order to continue to monitor the groundwater level conditions, a percentage of future drill holes should have piezometers installed to monitor the water table.

4.4 BLASTING CONTROL NEAR THE FINAL WALLS

Where the pit slope design angle will not be controlled by the structural geology the use of controlled blasting at the final face will normally allow an increase of 5 to 7 degrees in the slope angle. This involves the use of controlled blasting, with reasonably small diameter blast holes detonated as a pre-shear line in the harder massive rock or as a cushion or post shear line in weak or heavily fractured rock. The line holes should not be larger than 15 cm (6 inches) in diameter and spaced

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1.5 to 2 m (5 to 6 ft) apart. It is preferable if this pre-shear line is drilled at an angle of 70-75 degrees below the horizontal rather than vertical. It is also desirable to drill the line holes double bench depth to do away with a small bench lip which could develop half way up a double benched slope. To minimize wall damage it is preferable that the explosive in the line holes be decoupled laterally and/or air decked to spread the explosive more throughout the length of the hole. Rock excavation should be to the line holes and not beyond.

Two or three lines of buffer holes are recommended, drilled in front of the line holes with the spacing equal to about one-half that between the production holes. The first line of buffer holes should be angled. The others may be vertical. They should use the same diameter as the line holes. In order to drill the angle holes, it will be necessary to purchase or lease a drill which drills this size of hole and can be angled at angles up to about 25 degrees. Tamrock have drills with angled capacity. The amount of explosives per buffer hole will be approximately one-quarter that in the production holes. All buffer holes should be delayed.

Production holes may use larger diameter drills. Every hole of a production blast within one hundred feet of the final wall must be delayed singly.

Blasting should be developed towards a free face with that free face perpendicular to the wall. A typical trial blast pattern is shown in Figure 4.4. Figure 4.2 shows an excellent blast control program at Gortrum Mines in Ireland. Figure 4.3 is a plot of damage to rock, related to the weight of charge per delay, and the blast distance to the final wall. This graph emphasizes the utmost importance of using delays to reduce seismic acceleration forces to minimize wall damage.

The best blast design will result from trial test blast patterns in the field. The most beneficial program will develop where the open pit will be developed initially as a small pit and a subsequent set back will be developed. This initial pit can be

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developed with variable test slopes and test blast patterns to develop the best blast design for the final wall. Test trial blasts will be required wherever the rock conditions change substantially.

It is recommended that double benching be used at the final wall. In order for this procedure to be successful, the upper bench face must be scaled prior to drilling the buffer holes on the next lower bench elevation. It is also important that blast holes be staggered so the bottom of the hole does not intercept the crest of the bench below. Otherwise, very fragmented bench crests will develop leading to considerable ravelling and a greater berm width.

It is recommended that the operators of front-end loaders or shovel loaders who excavate this final face be given a seminar on structural geology, particularly the identification of small planar blocks and small wedges which could fail. By recognizing these in advance, they can dig them out so that ravelling at a later date will be reduced. By minimizing this ravelling the bench width can be narrowed to increase the overall slope angle.

Do not place piles of loose rock at the outer side of berms to catch ravelling rock. This requires an excessive bench width and results in overall flatter slope angles. Berms are to catch ravelled rock. They are not intended for later access.

4.5 MONITORING

The development of slope movement will be indicated by the development of a tension crack or cracks. It is most important that periodic inspections along the crest and bench locations be performed periodically to locate such cracks.

When tension cracks are observed the initial monitoring program requires the installation of surface movement hubs or gages which will allow measurement of

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both vertical and horizontal displacement. This is required to evaluate the type of movement that is occurring - circular, planar, toppling etc.

If larger scale movements appear to be developing, an electronic distance measuring (EDM) prism monitoring system should be developed. Three or four prisms should be installed on a line down the center of the slide with readings taken daily from a fixed station across the pit. Movement records should be plotted daily to determine if there is an acceleration of the movement. Plots should be made of a total movement vs time. At this point in time a decision as to whether the slide will be stabilized or whether it will simply be monitored and allowed to fail will then be made. If it is to be allowed to fail, continued monitoring of the acceleration will allow the date of failure to be predicted several days in advance. Mining would be discontinued at this time.

From a practical standpoint, where failure volumes involve 500,000 cubic yards or more, experience is that a failure will not occur within the next 24 hours if the amount of daily movement is less than about three inches.

Any failures involving one or more benches should be back analyzed. The location, structural geology, face failure geometry, failure surface roughness, seepage and blasting details etc. should be recorded. This allows the most accurate evaluation of the shear strength along the failure surface to be determined, to be used in re-design. Photographs should be taken and described.

4.6 BENCH DESIGN

For final pit wall bench design it is proposed that controlled blasting be used to develop a relatively steep bench face (70 to 75 degrees) which should be scaled with excavating equipment and a drag chain or equivalent from the top. Double benching is recommended. The scaling will reduce subsequent ravelling and reduce

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the catch width required for berms. Where the rock face stands up well, a 6 metre (20 foot) wide bench is suggested. If the rock is very fractured and considerable ravelling occurs the bench will have to be widened to about 8 metres (25 feet).

If areas of heavily fractured rock or faulted rock are encountered in the final slopes, more stable bench faces can frequently be developed using bulldozers and rippers rather than blasting along the final line holes. By utilizing this procedure the slope angles at Bougainville, in Papua New Guinea, for example, were steepened some 8 degrees.

4.7 PRELIMINARY PIT SLOPE DESIGN

East Facing Slope - A revised geological interpretation provided by Imperial Metals Corporation indicates the geologic structure is reasonably uniform over all three pits. Planar failures will occur locally where the structure dip is flatter than the bench face angle.

The slope angle can be developed with bench face angles of 70 degrees. The overall angle will depend on berm width, bench height and whether single or double benching will be used. See Table 4.1 and Figures 4.5 and 4.6.

West Facing Slopes - Based on existing structural geologic data the structure dips into the west facing slopes at about 45 to 75 degrees. Where the structure dip is less than about 60 degrees and does not cross major faults the bench faces can be developed at 75 degrees. The overall angle will depend on berm width, bench height and single or double benching. Where the dip angle is steeper than about 60 degrees the potential for toppling failure exists. The bench faces should be flattened to 70 degrees and berm widths should be increased by 2.0 metres.

The overall slope angles for various conditions are shown in Table 4.1 and Figures 4.5 and 4.6.

It is very important that these slopes be drained.

South Facing Walls - The fault structures generally dip N-S and will not materially influence slope stability. The joints dip into the slope which is a stable condition. Accordingly bench faces can be developed at 75 degrees with minimum berms widths. The overall slope angle will depend on the use of single or double benches. See Table 4.1 and Figures 4.5 and 4.6.

North Facing Walls - The fault structures will have similar influence as in the south facing walls. The two joint sets prevalent at the site will intersect to form shallow wedges. Some local bench size failures can be expected. Bench faces should be developed at 70 degrees with wider catch berms. The overall slope angles for various conditions are given in Table 4.1 and Figures 4.5 and 4.6.

Note that the slope angles recommended do not include haul roads.

Pit Noses - The preliminary design of the west pit and central pit has created a north and a south nose. Such noses usually suffer from excessive ravelling and instability. It is recommended the overall slopes in any such area be flattened 10 degrees by using wider catch benches.

Ramps - It is desirable to locate haul roads on the most stable pit wall - in this instance along the west facing slopes.

Fault Intersections - Where faults intersect the walls at shallow angles (<30 degrees) local wedge failures will occur. Extra scaling will be required.

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