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MINING TECHNIQUES FOR URANIUM ORE POD RECOVERY

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A thesis submitted to the Faculty of Graduate Studies and Research in partial fulfillment of the requirements of the degree of M.Eng.

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ABSTRACT

Isolated high grade and low tonnage uranium ore pods are prevalent in Saskatchewan. This thesis reviews mining such a pod with both conventional and unconventional methods. The geomechanical characteristics of the overburden and the bedrock in the area of the pods must be investigated before evaluating the technical feasibility of possible mining methods. Conventional ore recovery techniques appraised include open pit and underground mining. The unconventional mining techniques considered are blind boring and jet boring from surface. These last two techniques can offer a greater degree of selectivity, although ground reinforcement techniques had to be considered due to the poor quality of the ground. Ground reinforcement techniques considered were shaft lining, grouting, and ground freezing. This mining design challenge was found to be feasible with the combination of jet boring and freezing. Recommendations are presented to define future areas of study for blind boring, jet boring, grouting, and freezing.

RESUME

Les amas isolés de minerai d'uranium à forte teneur et faible tonnage sont très répandues en Saskatchewan. Cette thèse examine la possibilité d'exploiter un tel amas à l'aide de méthodes conventionnelles et non conventionnelles. Les caractéristiques géomécaniques du recouvrement et du soubassement dans la région de ces amas doivent être étudiées avant d'évaluer la faisabilité technique des différentes méthodes d'exploitation. Les techniques conventionnelles de récupération du minerai comprennent l'exploitation à ciel ouvert et le forage souterrain. Les techniques non conventionnelles considérées sont celles de la méthode de forage à trou borgne et de la méthode d'abattage hydraulique depuis la surface. Ces deux dernières techniques peuvent offrir une meilleure sélectivité, quoique des moyens de renforcement du sol aient dû être envisagés à cause de la mauvaise qualité du sol. Les moyens de renforcement du sol étudiés comprenaient le cuvelage du puits, l'injection et la congélation du sol. Les difficultés de conception minière furent surmontées en combinant la méthode à abattage hydraulique et la congélation du sol. Cette thèse apporte des recommandations afin de définir de futurs domaines d'études techniques des méthodes de forage à trou borgne, d'abattage hydraulique, d'injection et de congélation des terrains.

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Chapter 1

INTRODUCTION

The province of Saskatchewan in Canada is a major world producer of uranium. Over one quarter of the uranium used for electrical production in the world is mined in that province. Uranium deposits can occur in high-grade/low tonnage pods at shallow depth. Selective mining methods must be considered for uranium ore recovery from such deposits. These mining methods must be chosen so that the exposure of miners to radiation from the ore is minimised. Cogema Resources Inc. has a shallow high-grade uranium deposit in Northern Saskatchewan. This thesis is based on the feasibility of innovative mining methods for the recovery of this deposit which was done in close association with Cogema Resources Inc.

1.1 URANIUM MINING METHODS

The environmental setting and the geomechanical characteristics of a mineral deposit , its overburden, and the host rock mass must be considered before selecting a mining method. The exposure of miners to radiation must be minimised when mining uranium ore.

The characteristics of the environmental setting of a mineral deposit that must be known include information about its complexity, tonnage, grade, depth, and location. The complexity, tonnage and grade are important when deciding on a mining method because they govern the required degree of selectivity for ore recovery. The depth of the deposit is important in determining the tonnage and cut-off grade for cost effective recovery. The deposit location is also important in that the remoteness of a mineral deposit may affect the economic feasibility of any and all mining methods.

Radiation emanating from uranium ore includes radon gases and gamma rays. The exposure of a miner to radon gases can be satisfactorily controlled by ventilation but it does not control gamma radiation. Mining methods that involve non-entry to the confined space of stopes lessen the exposure of the miners to both radon gases and gamma radiation.

The type of ground reinforcement associated with a mining method will vary depending on the geomechanical characteristics of the ground in the neighbourhood of the deposit, the mining method being employed, and operating considerations. Such considerations would include the rate of mining resulting from different methods of reinforcement, and whether or not reinforcing material is recoverable.

1.2 SCOPE AND OBJECTIVES

In this thesis the technical feasibility of mining techniques associated with the recovery of two shallow high grade uranium ore pods in Saskatchewan is considered. Potential options for mining methods will be considered for these pods in light of the need for technical reliability and safety. The two pods are located in the southern periphery of the Dominique-Janine extension which is one of the uranium deposits located in the Cluff Lake mining area in north-western Saskatchewan. The location of Cluff Lake and other notable uranium mining properties within Saskatchewan (Key Lake, Rabbit Lake, and Cigar Lake) are shown in Figure 1-1 (Amok, 1992).

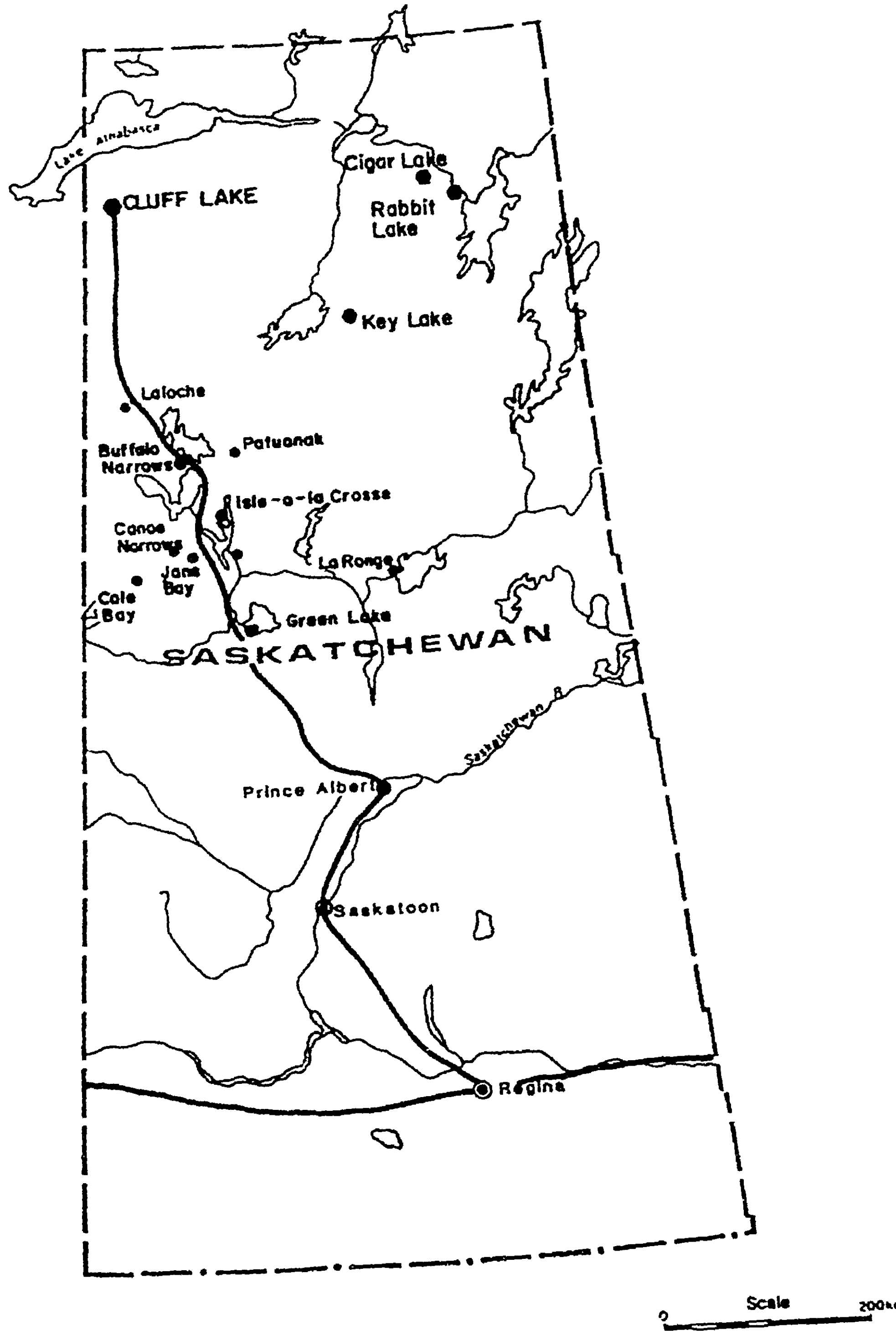

Figure 1-1: *Saskatchewan uranium deposits in Northern Saskatchewan: Cluff Lake, Key Lake, Rabbit Lake, and Cigar Lake, after Amok 1992*

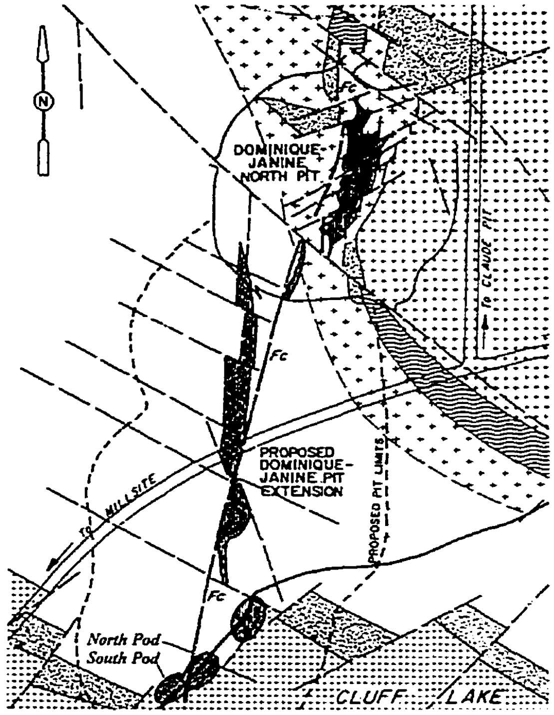

Figure 1-2: *Location of the North and South pods in the Dominique-Janine uranium deposit, after Amok 1992*

The North Dominique-Janine surface uranium mining operation was mined out by December, 1991. The Dominique-Janine extension is to the south of this open pit. It is presently in the process of being recovered by both open pit and underground mining methods. The North and South Pods of uranium ore in the southern extremities of the Dominique-Janine extension are the subject of this thesis, see Figure 1-2 (Amok, 1992). They lie on the shoreline of Cluff Lake and plunge partially under the lake. The North and South Pods contain 13 500t and 60 000t of uranium ore respectively, lying at a depth between 25 and 100m below surface.

Chapter 2

GEOMECHANICAL CHARACTERISTICS OF THE DOMINIQUE-JANINE AREA

The objective of the geomechanical studies was to investigate the competencies of the overburden and the bedrock surrounding the uranium pods in the southern periphery of the Dominique-Janine extension. Sections showing the overburden and bedrock of the North and South pods are shown in Figures 2-1 and 2-2 (Champollion, 1995) respectively.

2.1 OVERBURDEN

The physical properties of the overburden material investigated were: grain size distribution, porosity, and unit weight. Once the unit weight of the material was found, then the in-situ vertical stress within the overburden could be calculated. The in-situ horizontal stress and the shear strength of the overburden material could then be estimated. The overburden permeability was determined through a combination of three different methods.

2.1.1 SAMPLING

An attempt was made to drill the overburden over the North pod in February, 1995 to investigate the competence of the material through geotechnical analysis. This was conducted mainly by checking the grain size distribution of the overburden material. Samples of the overburden material were recovered with a *split spoon sampler* that preceded the drill rod. The split spoon sampler consisted of a small size core barrel

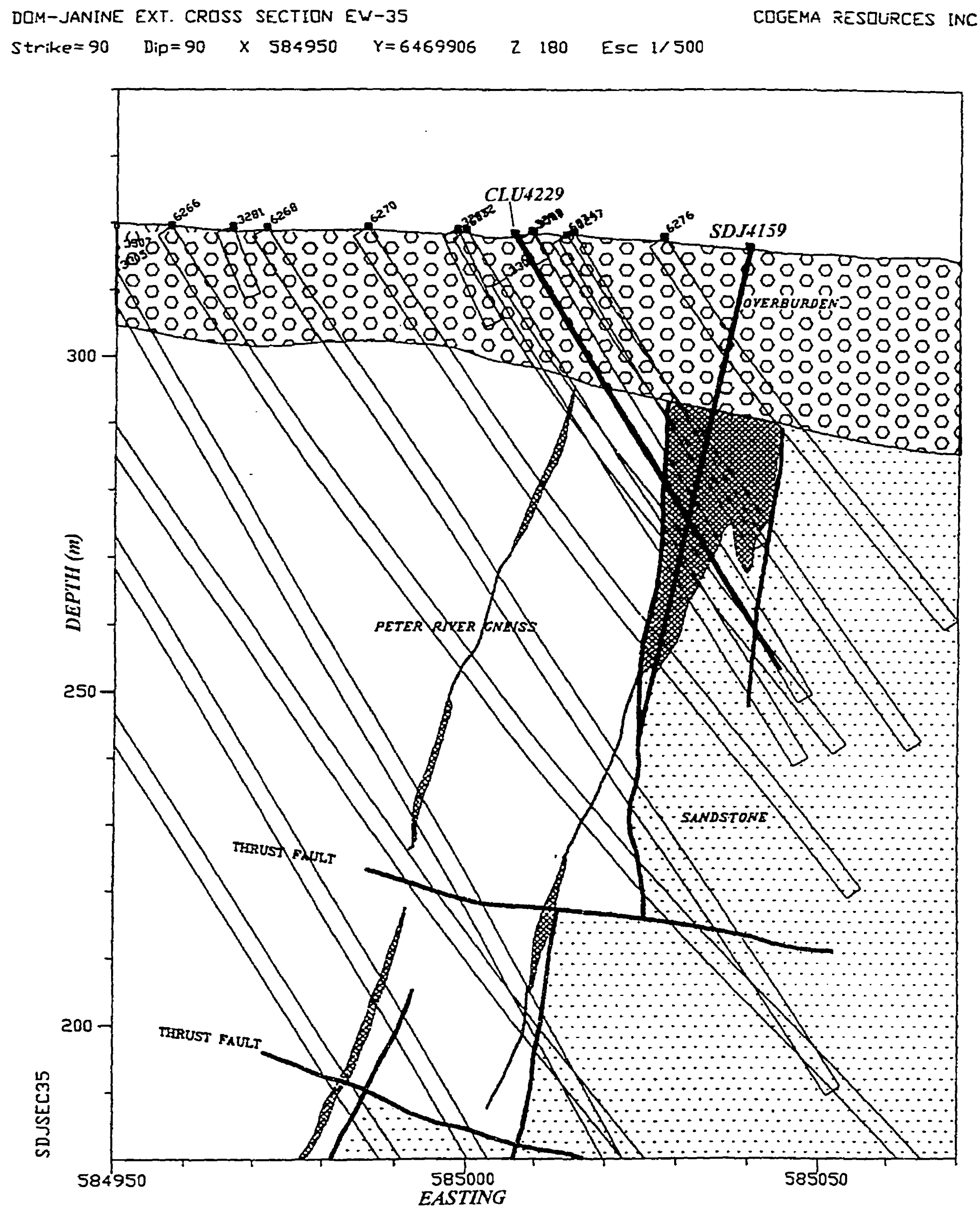

Figure 2-1: *Section through North pod, after Champollion 1995*

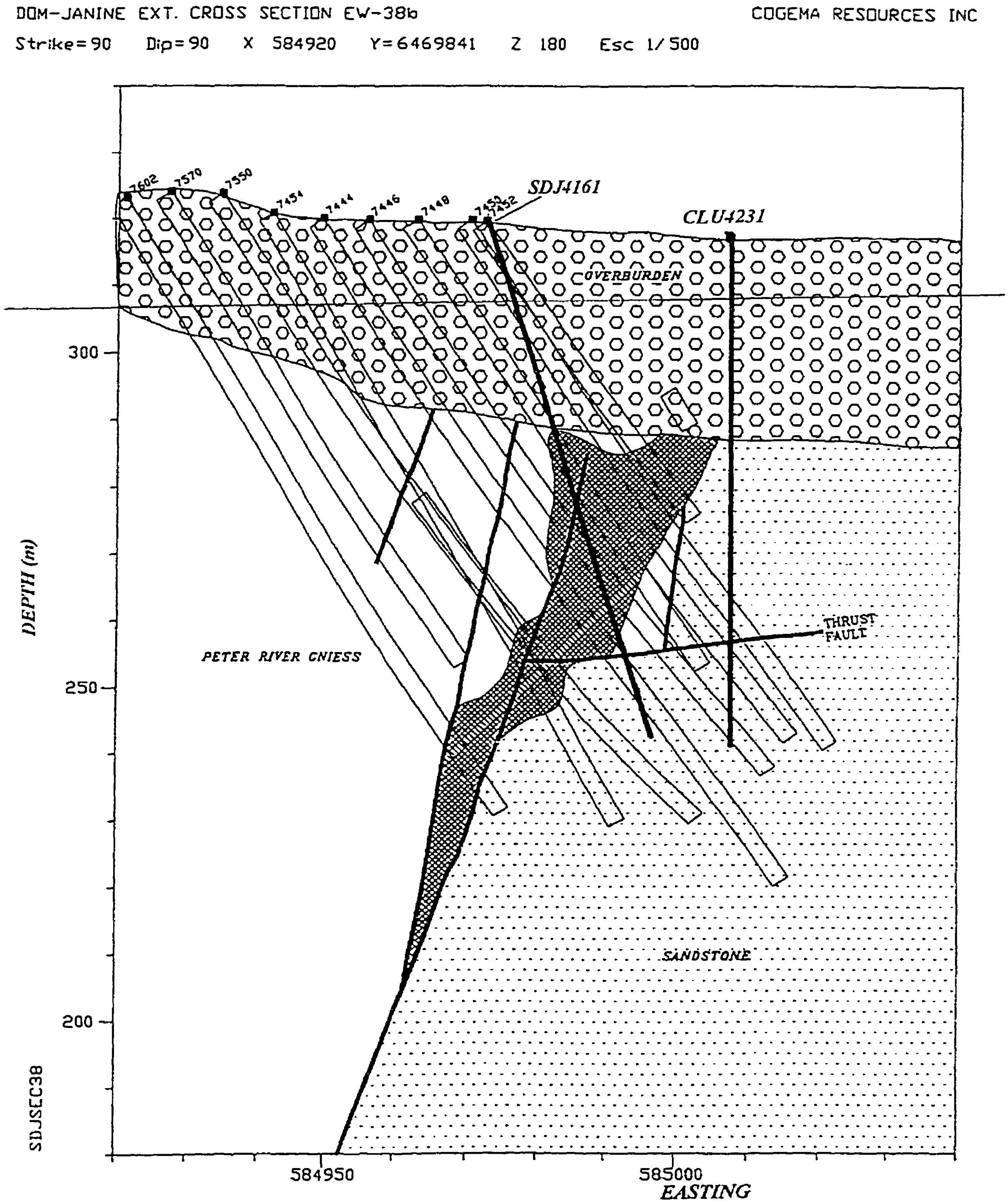

Figure 2-2: *Section through South pod, after Champollion 1995*

(38mm diameter by 610mm length), see Figure 2-3 (McCarthy, 1993).

The sampling procedure conducted for the soil involved a hole being drilled to a certain depth. The rods were then replaced by the split spoon sampler, which was hammered into the bottom of the hole until it recovered the soil 610mm deeper than the hole depth.

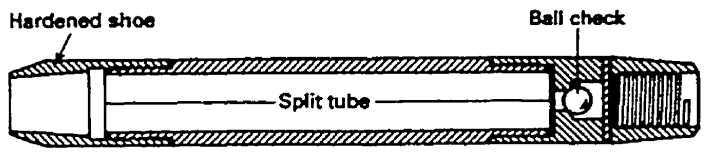

Figure 2-3: *Split spoon sampler, after McCarthy 1993*

Sampling from the overburden began over the North Pod with a split spoon sampler but was halted after drilling 12m downwards in the first hole because it was found that the samples being recovered were non-representative. This was due to the finer particles being washed away in the soil sample and only the larger grains of the soil being retrieved.

Further successful sampling was conducted by Agra E & E Ltd with a procedure which involved drilling within the soil and then retrieving the cuttings for the purposes of analysis.

2.1.2 GRAIN SIZE DISTRIBRUTION

The grain size distribution curves for the overburden material within the Dominique-Janine extension are shown in Figure 2-4 (Champollion, 1994). Each line depicts the soil grain size distribution for each borehole. Table 2.1 (Champollion, 1994) summarises the soil grain size distribution of the overburden. The grain size that only

10% of the soil is smaller than (D_{10}) is 0.2 mm, similar to medium sized sand. The cohesionless properties of the material that was drilled with the split spoon assembly indicates the material's similarity to sand (i.e. cohesionless). Therefore the material is considered to be medium sized sand.

2.1.3 **POROSITY**

A variable that was not analysed in the geotechnical study of the sand over the two pods was its porosity. This value was approximated. The estimated porosity was maximised so that the needs for reinforcement to increase the shear strength of the ground would not be underestimated. The porosity (n) was estimated to be 0.3 (Champollion, 1996).

TABLE 2-1: *Soil grain size distribution of soil, Dominique-Janine extension, after Champollion 1994*

MATERIAL	SIZE (mm)	RANGE (%)	AVERAGE (%)
Gravel	4.75—76.2	0—93	27
Sand	0.075—4.75	6—97	67
Fines	<0.075	0—26	6

2.1.4 **UNIT WEIGHT**

The dry unit weight of the overburden material was found to be 1.8 t/m^3 or $18 kN/m^3$. The saturated unit weight (γ_{SAT}) of the soil is found by combining the dry unit weight with the unit weight of the water (γ_w) in the pores. This is calculated by multiplying the unit weight of water by the porosity of the soil. This calculation is shown in Equation 2.1.

Equation 2.1

$$\gamma_{SAT} = \gamma_D + n\gamma_W$$
$$\gamma_{SAT} = 18.0\, {}^{kN}\!/_{m^3} + 0.3\left(9.8\, {}^{kN}\!/_{m^3}\right)$$
$$\gamma_{SAT} = 20.9\, {}^{kN}\!/_{m^3}$$

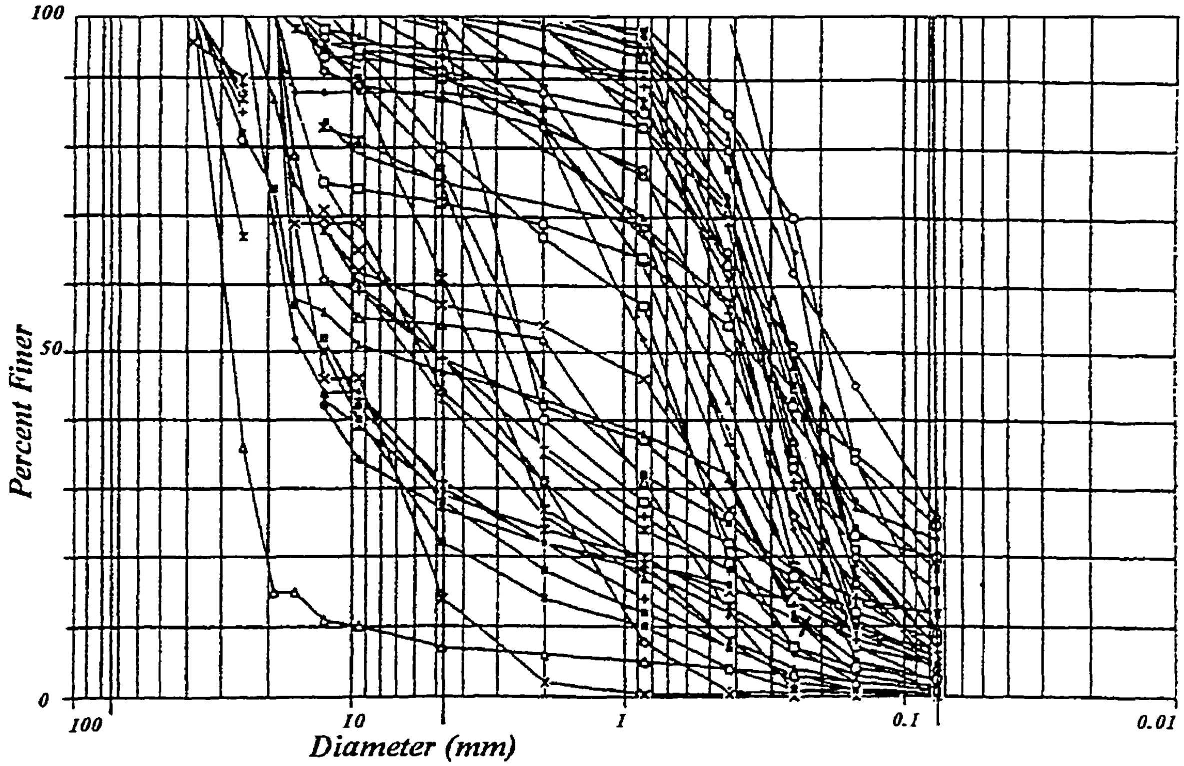

Figure 2-4: *Grain size distribution of overburden material over the two pods, after Champollion 1994*

2.1.5 IN SITU STRESSES

The effective in situ vertical stress (σ'_v) is calculated by subtracting the stress due to the buoyancy effects of water (u) from the vertical stress (σ_v). The vertical stress is calculated by multiplying the saturated unit weight of the soil by the maximum depth. The stress due to the buoyancy effects of water is calculated by multiplying the unit weight of water by the maximum depth of the soil (H). The effective unit stress calculation is shown in Equation 2.2.

Equation 2.2

$$\sigma'_V = \sigma_v - u$$
$$\sigma'_V = (\gamma_{SAT} - \gamma_W)H$$
$$\sigma'_V = (20.9\,{}^{kN}/_{m^3} - 9.8\,{}^{kN}/_{m^3})35m$$
$$\sigma'_V = 390kPa$$

The relation between the effective vertical stress and the effective horizontal stress (σ'_H) is dependent on the degree of mechanical interaction between the saturated sand grains. The degree of mechanical interaction is a function of Poisson's ratio (ν) which equals 0.25 (Das, 1990) for medium sized sand. The equation to find the effective horizontal stress (σ'_H) is shown in Equation 2.3 (Das, 1980).

Equation 2.3

$$\sigma'_H = \frac{\nu}{1-\nu}\sigma'_v$$
$$\sigma'_H = \frac{0.25}{1-0.25}390kPa$$
$$\sigma'_H = 130kPa$$

The total horizontal stress (σ_H) is equal to the effective horizontal stress plus the horizontal pressure due to water. The pore pressure exerted by water is assumed to be hydrostatic. The equation to find the total horizontal pressure is shown in Equation 2.4 (Das, 1990).

Equation 2.4

$$\sigma_H = \sigma'_H + u$$
$$\sigma_H = 130kPa + \left(9.8\,{}^{kN}\!/_{m^3}\right)(35m)$$
$$\sigma_H = 475kPa$$

2.1.6 SHEAR STRENGTH

The shear strength (τ) of material is related to its cohesion (c), angle of internal friction (ϕ), and the normal stress exerted on it (σ_N). The cohesion of sand is zero in dry conditions. The cohesion of sand is not significantly high when saturated. Dry conditions are assumed. The angle of internal friction was assumed to be the minimum value within the range for medium sized dry sand, 29° (McCarthy, 1992). Equation 2.5 shows the shear strength relationship for dry medium sized sand.

Equation 2.5

$$\tau = c + (\sigma_N - \sigma_p)\tan\ \phi$$
$$\tau = 0 + \sigma'_N \tan\ 29^\circ$$
$$\tau = 0.55 \times \sigma'_N$$

2.1.7 PERMEABILITY

The overburden permeability was calculated by two field experimental methods and one empirical relationship. The two experimental methods were the pumping test and the falling head test. The empirical relationship used for analysis was Hazen's equation.

The pumping test is an experimental method that is used to determine the permeability of material over a large area. Regions of the soil can be analysed separately by isolating sections within the borehole with impermeable packers. A constant head is maintained within the borehole by pumping a known quantity of water into it. An observation well a distance (R) away experiences a change in head because of this process. The head in the experimental well eventually stabilises when the water table reaches its steady-state equilibrium. The permeability of the ground could then be calculated from the change in head in the observation well.

Another testing method used to determine the overburden permeability at the

Dominique-Janine extension was the falling head test. This is another hydrological test in which the layer of interest is similarly isolated with impermeable boundaries. The permeability of the isolated layer is determined by finding the time required for a specific amount of water in the borehole to flow into the ground. This process determines the permeability of the ground.

Hazen's equation is an empirical relationship that provides rough but useful estimates of soil permeability. It is based on an empirical relationship between the sand grain size in mm (D_{10}) and its permeability (k) in cm/s, refer to Equation 2.6 (McCarthy, 1993).

Equation 2.6

$$k = D_{10}^2$$

The overburden permeability of the South Dominique-Janine extension area found through the above three methods is shown in Table 2.2 (Champollion, 1994). The weighted average found is 10^{-4} m/s. Because the overburden is shown to have a high permeability and the deposit is located by a lake, the overburden is considered likely to be saturated. In actual fact, the water table was observed in field monitoring as being 1 m below the surface.

2.1.8 **SUMMARY**

A summary of the overburden characteristics surrounding the South Dominique-Janine extension is given in Table 2.3 (Champollion, 1995).

TABLE 2.2: *Overburden permeability results in the Cluff Lake zone, after Champollion 1994*

DEPTH BELOW LAKE BOTTOM (m)	PUMPING TEST (m/s)	FALLING HEAD TESTS (m/s)	HAZEN'S TESTS (m/s)	AVERAGE (m/s)
0—2.5		4.0×10^{-7}		4.0×10^{-7}
2.5—5		9.0×10^{-6}		9.0×10^{-6}
5—10		2.5×10^{-3}	1.5×10^{-3}	2.0×10^{-3}
10—15	1.8×10^{-4}		1.0×10^{-4}	1.4×10^{-4}
15—30			4.0×10^{-4}	4.0×10^{-4}
Range		$4.0 \times 10^{-7} < k < 2.0 \times 10^{-3}$		
Weighted Average			$5.6 \times 10^{-4} \sim 10^{-4}$	

2.2 BEDROCK

The bedrock was analysed from diamond drill core. This core was used to derive a geomechanical classification for the orebody and the country rock surrounding the two pods. Two engineering rock mass classification approaches were utilised: Bieniawski's Rock Mass Rating (RMR) and Barton's Q-system (Bieniawski, 1989). Both these approaches were applied to the different structural regions throughout the rock mass. Generally the limits of these regions are defined by geological discontinuities including faults, dikes, and shear zones. The geomechanical characteristics within these regions were found to be fairly uniform.

The parameters used in Bieniawski's RMR classification method are: (1) groundwater conditions; (2) discontinuity conditions; (3) rock quality designation (RQD); (4) intact rock strength; (5) discontinuity spacing; and (6) discontinuity orientation. The six parameters analysed in Barton's Q-system classification method are: (1) water inflow; (2) degree of alteration or filling along the weakest joints; (3) rock quality designation; (4) stress condition; (5) number of joint sets; and (6) roughness of the most unfavourable joint or discontinuity set. The parallel six variables analysed by Bieniawski's RMR and Barton's Q-system classification methods depend on similar parameters.

TABLE 2.3: *Ground characteristics of overburden area surrounding the Dominique-Janine extension, after Champollion 1995*

D_{10} grain size	0.20 mm
Porosity	0.30
Saturated unit weight of material	20.9 kN/m^3
Poisson's ratio	0.25
Total in-situ horizontal stress	320 kPa
Cohesion	0
Angle of friction	29°
Ground permeability	10^{-4}m/s

The ratings assigned for different geomechanical situations in Bieniawski's *Engineering Rock Mass Classifications* can be referred to in Appendix A (Bieniawski, 1989). A general summary of the ratings used in Bieniawski's RMR geomechanical classification system along with a classic description of their significance can be referred to in Table 2.4 (Bieniawski, 1989).

The parameters for Barton's Q-system can be found in Appendix B (Bieniawski, 1989). A general summary of the ratings used for Barton's Q-system geomechanical classification system along with classic descriptions of their significance appears in Table 2.5.

Measurements of rock density and rock mass permeability were also made. Many of the parameters in the RMR and Q-system classification methods are judgmental. The rock's sensitivities to wetting-drying were also determined by putting them through cycles where they are soaked and dried, refer to Appendix C (Champollion, 1994).

TABLE 2.4: *Values used for Bieniawski's RMR geomechanical classification of the Dominique-Janine extension bedrock, after Bieniawski 1989*

VARIABLES	RATINGS	TYPICAL CHARACTERISTICS
UNIAXIAL COMPRESSIVE STRENGTH OF INTACT ROCK *(R_c)*	0 4 15	• *UCS<1MPa* • *point load strength=1-2MPa; 25MPa≤Strength≤50MPa* • *point load strength>10MPa;UCS>250MPa*
ROCK QUALITY DESIGNATION *(RQD)*	3 20	• *RQD<25%* • *90%<RQD<100%*
DISCONTINUITY SPACING *(Spc)*	5 20	• *<60mm* • *>2m*
DISCONTINUITY CONDITIONS *(JOINT)*	0 10 20	• *soft gouge>5mm thick or separation>5m; continuous* • *slickensided surfaces or gouge<5mm thick or 1<separation<5mm; continuous* • *slightly rough surfaces; separation<1mm; highly weathered wall*
GROUNDWATER CONDITIONS*(HEAD)*	4	• *cylinder excavated by core recovery is dripping*
JOINT ORIENTATION *(Orient)*	-10	• *driving against dip of 20-45 °*
$1 \leq RMR \leq 55$		

2.2.1 ROCK CORE SAMPLING

Rock core was recovered from the two pods mainly for the purpose of collecting geomechanical data. The North and South pods were drilled in February 1994 from which core was recovered (boreholes SDJ4159 and SDJ4161). The sampling program was continued in February 1995 (boreholes were CLU4229 and CLU4231). Details of these drillholes are given in Table 2.6 (Champollion, 1995).

Table 2.6 shows that three of the holes (SDJ4159, SDJ4161, and CLU4231) were of NQ diameter (47.6 mm), specifically for geomechanical classification. The fourth hole (CLU4229) was of HQ diameter (63.5 mm), to test both the ease of cutting this rock with a water jet and for geomechanical classification.

The locations of these boreholes in relation to the ore pods is shown in Figures 2-1 and 2-2. Drillhole CLU4231 was drilled 16 metres further east than intended.

2.2.2 INTACT ROCK STRENGTH

The intact rock strength is a function of many parameters, including the point load strength index (I_S (psi)) and the uniaxial compressive strength (UCS (MPa)). Other parameters examined include the shear strength of the rock and the hardness classification of the rock according to Brown's scale, and the density of the rock mass along the section of core length analysed.

The uniaxial compressive strength test is carried out when the rock is too weak to be tested with a point load test. A rock cylinder is cut such that its length is 2-2.5 times the diameter of the core. The ends are then ground flat and made perpendicular to the cylinder axis. A load is applied axially to the rock cylinder through platens. The stress level is increased until the applied compressive stress causes the sample to fail. The applied stress that causes failure is the uniaxial compressive strength of the rock (ISRM standard).

The hardness classification method assigns a strength grade to the rock. This is

TABLE 2.5: *Values used for Barton's Q-system geomechanical classification of the Dominique-Janine extension bedrock, after Bieniawski 1989*

VARIABLES	RATINGS	TYPICAL CHARACTERISTICS
STRESS CONDITION *(SRF)*	5	• *loose open joints, heavily jointed or "sugar cube," etc. (any depth)*
ROCK QUALITY DESIGNATION *(RQD)*	0-100	• *VALUE/100*
NUMBER OF JOINT SETS *(J_n)*	2 9 20	• *one joint set* • *three joint sets* • *crushed rock, earth like*
DEGREE OF ALTERATION OR FILLING ALONG THE WEAKEST JOINT *(J_A)*	0.8 6 13	• *tightly healed, hard, nonsoftening, impermeable filling* • *strongly overconsolidated, nonsoftening clay mineral fillings(continuous, <5mm in thickness)* • *thick, continuous zones or bands of clay*
WATER INFLOW *(J_w)*	0.5	• *large inflow or high pressure in competent rock with unfilled joints*
ROUGHNESS OF THE MOST UNFAVOURABLE JOINT OR DISCONTINUITY *(J_r)*	0.5 3	• *slickensided, planar* • *rough or irregular, undulating*
0.00 ≤ Q ≤ 0.14		

determined by evaluating the hardness of the rock mass with tools such as a knife or pick in the field. The values obtained when this testing method is used can be referred to in Table 2.7 (ISRM, 1981). The hardness classification was tested for all intervals along the rock length analysed.

TABLE 2-6: *Details of drillholes, after Champollion 1995*

HOLE #	TYPE	LOCATION	AZIMUTH	DIP	DEPTH (m)
CLU4229	HQ	NORTH POD	+90	-60	75
CLU4231	NQ	SOUTH POD	*unknown*	-90	78
SDJ4159	NQ	NORTH POD	+270	-78	74
SDJ4161	NQ	SOUTH POD	+90	-72	80

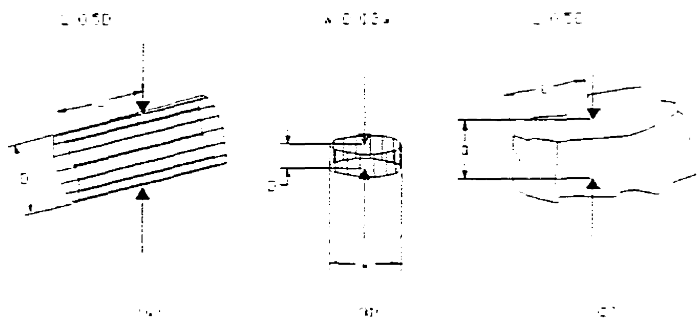

Figure 2-5: *Three types of point-load strength test, showing limitations on the geometry of the specimens: a) Diametral test; b) Axial load test; and c) Irregular lump test*

The RMR classification for the intact rock strength is based on the point load strength index and the uniaxial compressive strength.

Barton's Q-system alternatively takes into account the decreased strength of the rock mass due to the presence of joints, or the stress reduction factor (SRF). The value that was assigned to the SRF was consistently 5.0, corresponding to loose open joints, heavily jointed, or "sugar cube" rock at any depth.

2.2.3 ROCK QUALITY DESIGNATIONS (RQD)

For the boreholes analysed, there is a complete range of RQD's from 0% to 100%. Bieniawski's RMR values ratings for RQD ranged from the minimum value of 0 to the maximum value of 20. The ratings for Barton's Q-system are merely the RQD expressed as a decimal fraction. The orebody tends to be more fractured and have lower RQD values than the surrounding country rock.

2.2.4 JOINT SPACING

The rating for joint spacing is a function of the mean spacing between the discontinuities. The main factor to consider for Barton's Q-system rating for the joint set number (J_N) is the number of joint sets found. The drill core from SDJ4159 and SDJ4161 did not record the number of joint sets within lengths of drill core, although the Q-value was estimated. For CLU4229 and CLU4231 the number of joint sets was measured for the estimation of the Q-value.

2.2.5 DISCONTINUITY CONDITIONS

The variables that describe the geomechanical conditions of the discontinuities for the Dominique-Janine extension are the type of rock, whether or not the rock contains uranium mineralization, and the degree of alteration. The types of bedrock are: sandstone, conglomerates, regolith, clay gouge, gneiss, and granitoid. There is also some clay within the discontinuities. A large quantity of the bedrock is composed of pervious sandstone. The variable concerning alteration is established by: the extent of alteration (highly altered, fairly altered, or slightly altered); the extent of hematization (highly hematized, fairly hematized, or slightly hematized); and if the rock is hard or not.

TABLE 2-7: *Rock Hardness Classification by Field Tests, after ISRM 1981*

GRADE	DESCRIPTION	UCS (Mpa)	POINT LOAD INDEX (Mpa)	FIELD ESTIMATE OF STRENGTH
R6	EXTREMELY STRONG	>250	>10	*rock material only chipped under repeated blows, rings when struck*
R5	VERY STRONG	100-250	4-10	*requires many blows with a geological hammer to break intact rock specimens*
R4	STRONG	50-100	2-4	*hand held specimens broken by a single blow with a geological hammer*
R3	MEDIUM STRONG	25-50	1-2	*firm blow with geological pick indents rock to 5 mm, knife just scrapes surface but material still hard*
R2	WEAK	5-25	N/A	*knife cuts shape into triaxial specimens*
R1	VERY WEAK	1-5	N/A	*material crumbles under firm blows of geological pick, can be shaped with knife*
R0	EXTREMELY WEAK		N/A	*indented by a thumbnail*

2.2.6 GROUNDWATER CONDITIONS

The value given for groundwater conditions from Bieniawski's RMR system throughout all four boreholes is 4.0. The value given for water inflow (J_w) from Barton's Q-system is 0.5. It should be noted that there is a large volume of groundwater in host rock in the neighbourhood of the deposit. The groundwater comes from water flowing to Cluff Lake through the pervious sandstone.

2.2.7 JOINT ORIENTATION

The value consistently assigned to Bieniawski's RMR rating for joint orientation was -10. Referring to Table A-1, it can be seen that this rating is for dips that are orientated in an *unfavourable* manner in tunnelling and in mining projects. Table 2.8 (Bieniawski, 1989) shows examples of separate geomechanical terms used where a tunnel is driven against a discontinuity with a dip of 20-45°.

TABLE 2.8: *Effect of discontinuity strike and dip orientations in tunnelling, after Bieniawski 1989*

STRIKE PERPENDICULAR TO TUNNEL AXIS			
DRIVE WITH DIP		**DRIVE AGAINST DIP**	
DIP 45-90	DIP 20-45	DIP 45-90	DIP 20-45
dip favourable	*Favourable*	*fair*	*unfavourable*
STRIKE PARALLEL TO TUNNEL AXIS		**IRRESPECTIVE OF STRIKE**	
DIP 20-45	DIP 45-90	DIP 0-20	
Fair	*very unfavourable*	*fair*	

In Barton's Q-system the joint roughness (J_r) values range from 0.5 to 3, as determined by visual inspection.

2.2.8 GEOMECHANICAL ROCK MASS CHARACTERISTICS

Bieniawski's RMR system is found by adding all the elements included in geomechanical classification. The elements that are added for Bieniawski's RMR system were: intact rock strength; rock quality designation; discontinuity spacing; discontinuity conditions; groundwater conditions; and joint orientation.

The RMR values range from 1 to 55. This is equivalent to rock that is classified as *very poor to fair*, see Table 2.4.

The Q-system classification is found by grouping six parameters into three groups of two. The parameters are: the stress condition (SRF); the rock quality designation (RQD); the number of joint sets (J_N); the degree of alteration or filling along the weakest joint (J_a); the water inflow (J_W); and the roughness of the most unfavourable joint or discontinuity (J_r).

These parameters are grouped in the following manner (Bieniawski,1989):

Equation 2.7

$$Q=\frac{RQD}{J_N}\times\frac{J_r}{J_a}\times\frac{J_w}{SRF}$$

The Q-values range from 0.00 to 0.14 for the Dominique Janine extension, see Table 2.5. The Q-system rock quality ratings range from 0.001 to 1000 on a logarithmic rock mass quality scale: therefore, the rock quality is considered poor.

2.2.9 **DENSITY AND PERMEABILITY**

Two other factors that were considered were the rock density and permeability. The densities of gneiss and conglomerate samples were established from CLU4229. Granitoid and conglomerate samples were taken from CLU4231 to test for density. The rocks within and surrounding the two pods all have wet densities of approximately 2.65 t/m^3. This corresponds to a dry density for the conglomerates of 2.5 t/m^3 with the assumed porosity being equal to 10%. The dry density for the gneisses and granitoids is 2.6 t/m^3 assuming a porosity of 3%.

The permeabilities of Peter Rock Gneiss and Athabasca Sandstone were also considered. These rock types occurred in competent and tectonitized forms. Sections of Peter River

Gneisses were found within a shear zone and sections of Athabasca Sandstone were found within a gouge zone.

The average permeability for Peter River Gneiss was found to range from 4.8×10^{-8} m/s within the competent zone to 8×10^{-6} m/s within the shear zone. The average permeability for Athabasca Sandstone was found to range from 6×10^{-7} m/s within the competent zone to 2×10^{-5}m/s within the tectonized zone.

2.2.10 **ROCK SENSITIVITY TO WETTING-DRYING CYCLES**

Slake Durability tests were carried out in order to find the rocks' sensitivity to wetting-drying cycles. These tests were carried out by taking 10 samples of rock with mass between 40 and 60 grams, placing them in a sieve mesh drum, oven-drying them, and finding the final mass of the rock and drum. The drum and the samples are then rotated for 10 minutes in a water bath. Rock fragments that have disintegrated through slaking during this cycle leave the drum through the sieve mesh.

After the 10 minute cycle of slaking is completed, the drum and samples are then redried. Then the drum is put through a second slaking cycle and dried again. After the drying action is completed, the drum and samples are weighed again.

The slake durability index (I_{d2}) is the weight of the dry sample remaining in the drum after the two cycles of slaking expressed as a percentage of the initial dry sample weight. I_{d2} values approach zero percent for samples that are highly susceptible and are closer to 100% for the more rock-like materials that are not susceptible to slaking. An example of a slake durability test being carried out is shown in Figure 2-6 (Franklin et al, 1989).

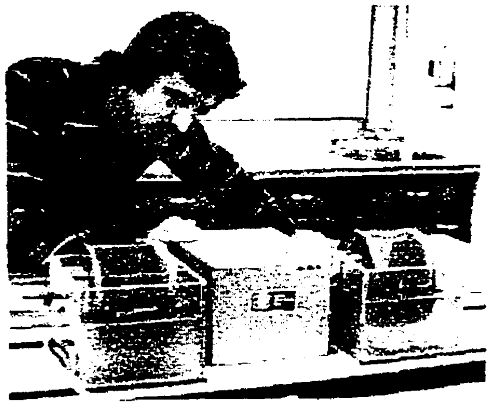

Figure 2-6: *Slake durability test, after Franklin et al. 1989*

Slake durability tests were conducted on CLU4231 samples, totalling a length of 500 mm and 3.5 kg in weight. They were extracted from a depth of about 36.5 m below surface. The data taken from the Slake durability tests for the two pods can be referred to in Appendix C. On average, the slake durability index was 44% (Champollion, 1995).

2.2.11 SUMMARY

Rock core sampling occurred over two separate periods. The first rock core sampling programme took place in February 1994 and both of the cores were NQ types. The second rock core sampling programme took place in February 1995. For this programme, the core recovered from the South pod was an NQ type while the core recovered from the North pod was an HQ type. The rock core recovered from each of these boreholes was analysed for its geomechanical characteristics. The HQ type core was also used to ascertain the susceptibility of failure of the rock when subjected to high pressure water jets.

The intact rock strength was tested by subjecting the core to point load and uniaxial compressive strength tests, and also using the hardness classification method. The strengths of the rock was dependent on only the rock type. The hardest samples of rock were found within the zone of the orebody.

The rock within the ore zone has a poorer RQD value than that of the surrounding country rock.

There seemed to be little difference in the joint spacing within the orebody and the surrounding country rock. The zones within the orebody are more altered than the zones outside the orebody.

The rock in and surrounding the two pods is saturated. This is due to the rock in the neighbourhood of the two pods being permeable and the pods are located right by a lake.

Because the deposition of the ore involved a tectonization process, the rock was mixed up. This led to the discontinuities not being consistently orientated in the same direction. Also, there is clay within the discontinuities.

Bieniawski's RMR of the rock samples found that the rock quality ranged from *very poor* to *fair*. Barton's Q-system rock quality ratings ranged from 0.00 to 0.14. These geomechanical rock mass characterisation systems indicated the rock quality as being poor. A consistent factor noted is that the poorest rock qualities is found within the zone of the orebody. This is largely due to the hydrothermal alteration that the rock went through during the deposition of the uranium ore.

The densities and permeabilities of the rock samples were also found. Granitoids, gneisses, and conglomerates were analysed for their wet and dry densities. The permeabilities of Peter Rock Gneiss and Athabasca Sandstone were also considered.

Slake durability tests were carried out on the rock core that was HQ sized. The average Slake durability index of this sample was 44%. This indicates that the rock would resist poorly when being struck by a water jet.

Chapter 3

ORE RECOVERY

The following factors must be kept in mind when considering mining methods to recover uranium from an orebody:

1. Environmental setting
2. Geomechanical characteristics
3. Radioactive exposure of miners

The mining methods that will be considered for the recovery of the Dominique-Janine uranium deposit while keeping these factors in mind are:

1. Open pit mining
2. Underground mining
3. Blind boring
4. Jet boring

3.1 FACTORS TO CONSIDER

3.1.1 ENVIRONMENTAL SETTING

Environmental information that must be known about an orebody before selecting a mining method includes its tonnage, grade, depth, and location.

The orebody tonnage is important when deciding on the degree of selectivity necessary to recover the orebody. Mining methods can be economically employed that recover large quantities of ore when the corresponding orebody tonnage is high enough. Mining methods that recover large quantities of ore are referred to as bulk mining methods. The cost to recover ore using bulk mining methods is lower than if selective mining methods were used.

Selective mining methods can also be used for ore recovery. These techniques are considered when the grade of an ore deposit is high enough to be economically recovered but the quantity of the ore deposit is low or when the orebody is complex.

The depth of an ore deposit is a decisive factor in determining how high the tonnage and grade of the ore must be for cost effective recovery. The cost of recovering an orebody increases as the depth increases. Conversely as the depth increases in relation to the ore grade, selective mining methods become more important.

The location of mineral deposits is another important consideration when considering the mining method to employ. The deposit may be located in a remote location when transportation costs of material and supplies to and from the deposit become high. The environment where the deposit is located also determines the costs associated with employing different mining methods. The deposit may be surrounded by material that has different geomechanical characteristics than the material within the deposit. This would necessitate changes in design to maintain the stability of the material through which excavation takes place to recover the deposit.

In the North pod in the southern periphery of the Dominique-Janine extension, there is 13 500t of uranium ore. In the South pod, there is 60 000t. These pods are between 25m and 100m below surface.

3.1.2 GEOMECHANICAL CHARACTERISTICS

The geomechanical characteristics of the different rock masses must also be considered. The properties to consider include those of the overburden, of the country rock surrounding the orebody, and of the host rock within which the orebody occurs. It is necessary to know these geotechnical/geomechanical characteristics in order to determine the costs associated with excavating to and through the orebody. These geotechnical/geomechanical characteristics affect:

- the type of mining equipment
- the size of the openings
- the location of the excavations
- any internal/external reinforcement that must be installed

The previous chapter summarised the geomechanical characteristics of the material in the neighbourhood of the orebody. The overburden was found to be composed mostly of sand. The country rock and host rock in the neighbourhood of the orebody is composed of sandstone, conglomerates, regolith, clay gouge, gneiss, and granitoid. The country rock surrounding the orebody is moderately fractured, competent, and practically impermeable. The geomechanical alteration of the host rock of the orebody when it was formed made the host rock less geomechanically competent than the surrounding country rock. The orebody host rock is densely fractured, very weak, and permeable.

3.1.3 RADIOACTIVE EXPOSURE OF MINERS

The exposure of a miner to radon gases and gamma radiation underground is a concern. With the high grade of the uranium in the two ore pods of the Dominique-Janine extension, the radon gases and the gamma fields in the orebody will be high. For this reason the recovery method must minimise the miner's exposure to the orebody. Radon gases can be satisfactorily controlled by ventilation. Ventilation, however, does not control gamma radiation. Mining methods that involve non-entry into the stopes can limit the miners' exposure to both radon gases and gamma radiation: therefore, only these mining methods will be considered.

3.2 MINING METHODS

Conventional open pit and underground mining will be considered. In addition two non-conventional mining methods, blind boring and jet boring, will be considered. How the factors discussed in the previous section affect each mining method considered will be assessed.

3.2.1 CONVENTIONAL OPEN PIT MINING

As depicted in Figure 3-1 (Amok, 1992), most of the Dominique-Janine extension will be recovered with a conventional open pit mining method. The deposit is relatively shallow and the tonnage of the orebody is high enough for conventional open pit mining to be used.

Most of the Dominique-Janine extension orebody occurs in gneisses. Pit walls that are excavated in the gneisses require a bench face angle of 55°. Bench heights would be 18m with an 8m wide berm. This entails that the overall rock slope angle would be decreased to 41°.

If the ore from the North and South pods was recovered with the rest of the Dominique-Janine extension, the southern wall of the open pit would be excavated past the shoreline into Cluff Lake and in sandstone. The sandstone has poorer geomechanical properties than the gneiss. The shear strength of the rock will be reduced further because the rock is saturated. It will be necessary for the south wall to be at a shallower angle of 27° in order to keep the slope stable due to these geomechanical characteristics. It would be necessary to extend the south wall approximately 200m into the lake at this. This would require building a dyke into Cluff Lake to prevent the water from flowing into the pit as shown in Figure 3-1.

The environment surrounding the miners in an open pit is fresh air. The extent of the miner's exposure to radon is therefore minimised. Although the miners are not exposed to a significant amount of radiation when using conventional open pit mining methods, disadvantages behind using this method for ore recovery in the Dominique-Janine extension include the costs associated with flattening the pit slope or reinforcing the weak rock mass surrounding the ore deposit, the cost of constructing a dyke in the lake to prevent water from entering into the pit, and the fact that the environment in the area of the open pit will be disturbed. Therefore, other mining methods should be considered.

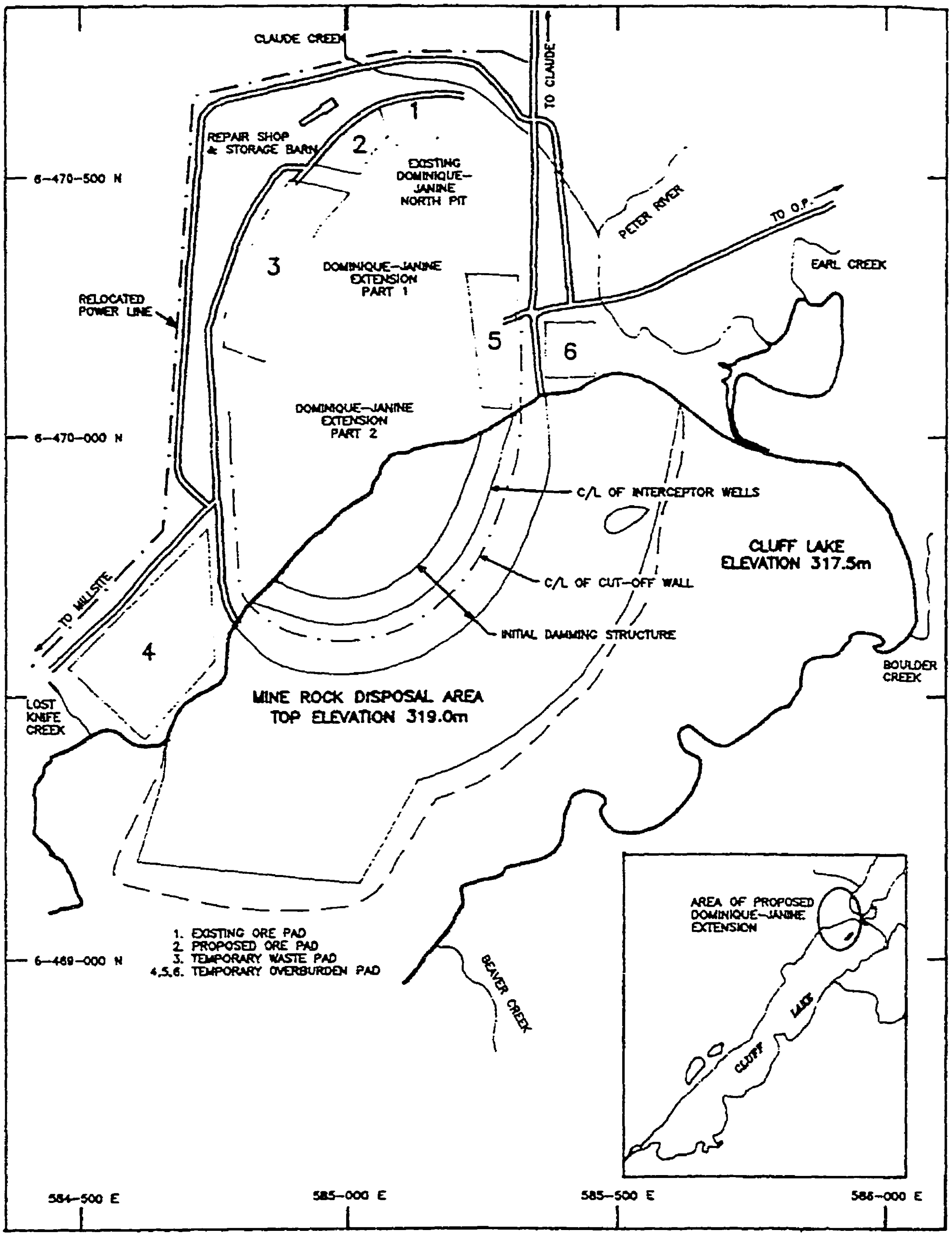

Figure 3-1: *Dominique-Janine pit extension general arrangement, after Amok 1992*

3.2.2 CONVENTIONAL UNDERGROUND MINING

Underground mining was also considered as a possible means to recover the uranium ore from the two pods at the Dominique-Janine extension. This mining method offers a flexibility in terms of the order that the uranium ore is recovered.

The geomechanical characteristics of the overburden and bedrock in the environs of the deposit are of poor quality. This entails that ground support would have to be provided to the excavations as they are mined.

Another factor to consider is the radiation that is emitted by the uranium ore within the two pods. The exposure of the miners to this radiation can be reduced by good ventilation and employing non-entry mining methods where the miner does not work inside the stopes. The heading from where the miner works would contain no radioactive mineralisation. The costs associated with developing headings by using non-entry mining methods would probably not be recouped by the recovery of the ore.

An additional concern when considering underground mining of the two pods is that they occur beside Cluff Lake. The ore was deposited hydrothermally. This process fractured the host rock. As a result, groundwater may flow through the rock into the excavations, and there is an increased risk of rock instability due to the water.

Although conventional underground mining does offer advantages in terms of flexibility, the risks of miners being exposed to radioactivity, the costs and risks of excavating underground, and the cost of providing reinforcement would appear to be too high to represent a suitable method.

3.2.3 BLIND BORING

Another method that Cogema Resources considered for ore recovery was blind boring. Blind boring machines drill holes directly into the ground. The size of the cutting head throughout the overburden and the waste rock is 2.44m. Once the ore deposit is reached, the size of the cutting head is increased to 3.66m.

This section will describe the blind boring drilling rig and its operation, hole stresses, backfilling bored holes when mining is completed, the orebody environment, the radiation exposure of the miners, and the ground stability.

BLIND BORING OPERATION

The blind boring machine considered was developed by the Zeni Drilling Company specifically to recover the ore from the two pods at the Dominique-Janine extension, see Figure 3-2.

Electric motors are used to tram the blind boring rig on surface. These motors are fed by a trailing cable. The machine is designed to be able to move on a grade up to 10%.

The length of the machine is equal to 22.3m. When the tracks are in line, the width is equal to 7.5m. The height of the machine from the ground to the top of the mast is equal to 21.3m.

The drill rig is mounted on four crawler tracks. These independent crawler tracks facilitate the positioning of the drill rig over the holes. Because each track is separately powered, then each pair of tracks can be rotated around the main pivots for steering

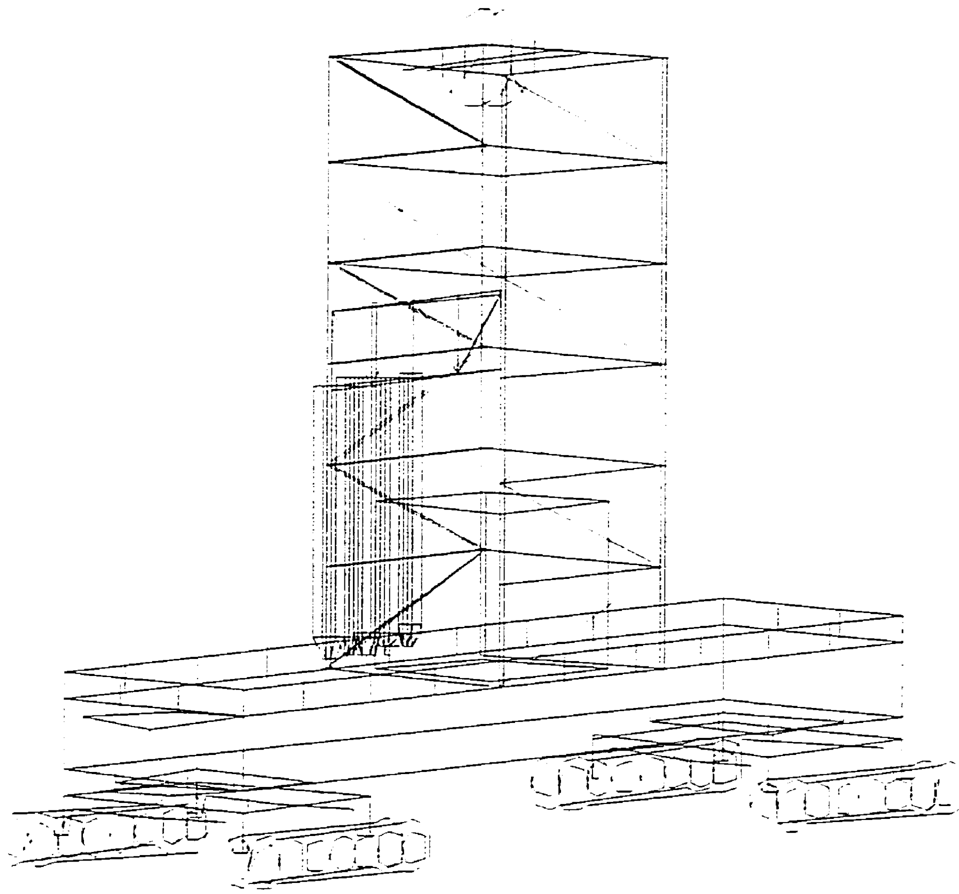

Figure 3-2: *Conceptual view of blind boring drilling rig – not to scale*

purposes. This arrangement allows the entire rig to revolve around its gravitational centre.

The blind drilling machine weighs 250t. The pressure applied by the drilling machine to the ground is equal to 80kPa.

Once the machine is positioned over the borehole, the drill mast is adjusted to be vertical, through the use of levelling jacks that are located at the base of the mast.

After the drill mast is made vertical, the surface casing is lowered to the ground. The total weight of the machine is then transmitted to the casing through casing jacks. This force pushes the cutting edge of the casing some two metres into the ground. The ground material within the casing is then excavated. The drilling tool assembly remains suspended inside the casing during these positioning and excavation cycles. After these cycles are completed, then the drill tool assembly is lowered to the ground. The casing is then filled with water.

At this point, drilling begins. The drilling head is rotated by the rotating table. The speed of the drill tool assembly is varied by partially releasing the hoist brake. The penetration rate is a function of the geomechanical characteristics of the rock (its strength, fracture frequency, hardness, and abrasivity); the characteristics of the drill tool assembly (the cutter type, size and spacing, cutter load, and available torque); and the cutting removal system (Moss et al., 1987).

The cuttings are retrieved as the cutting head advances using air-assisted reverse circulation. Reverse circulation describes drilling fluid flowing through to the bottom of the hole outside the drill string at a rate of 250l/s. This fluid then circulates past the cutting head towards the centre of the drill hole once it reaches the bottom of the hole. The cuttings are collected by the drilling fluid as it flows towards the centre of the string. This mixture of water/cuttings then flows up the centre of the drill string. Air is also blown up the centre of the drill string from the cutting head. This air causes a density imbalance which induces a flow rate to the drilling fluid which is sufficient to carry the cuttings to surface.

The drilling head cuts a hole of diameter 2.44m throughout the overburden and waste rock with disc cutters. The diameter of the cutting head is increased to 3.66m once the orebody is reached by swinging the arms out of close fitting slots 1.175m from the bottom of the cutting head. This is accomplished by the arms from the cutting head being rotated out by a hydraulic jacking operation. This movement is assisted with hydraulic cylinders which also help to keep the arms out. The slots provide lateral strength capable of sustaining the force generated by the transmitted torque. Pick type cutting tools will cut the rock within the increased diameter of 3.66m.

The area excavated by the cutting head increases by 125% when the diameter increases. A section view of the cutting head once it has been expanded is illustrated in Figure 3-3 (Zeni,1994). The increase in diameter of the drill head is accomplished quickly so that the amount of waste rock removal is minimised.

The disc cutter unit can be removed for maintenance from the machine without removing the expanding portion. The expanding portion can be quickly and easily maintained while still in place on the drill rig.

At the completion of the drilling cycle, the cutter head is retracted once again to a diameter of 2.44m before being lifted to surface. As the arms from the cutter head move in, then the drilling fluid flows out beneath the picks and flushes out any accumulated debris that may have migrated to the underside of the slots.

Two cutter heads will be available during operation. This will enable the drilling to continue while one of the heads is receiving maintenance. Once the ore is removed and the drill string is back on surface, then the hole is backfilled with lean concrete. Blind boring is expected to provide nearly total recovery of the ore.

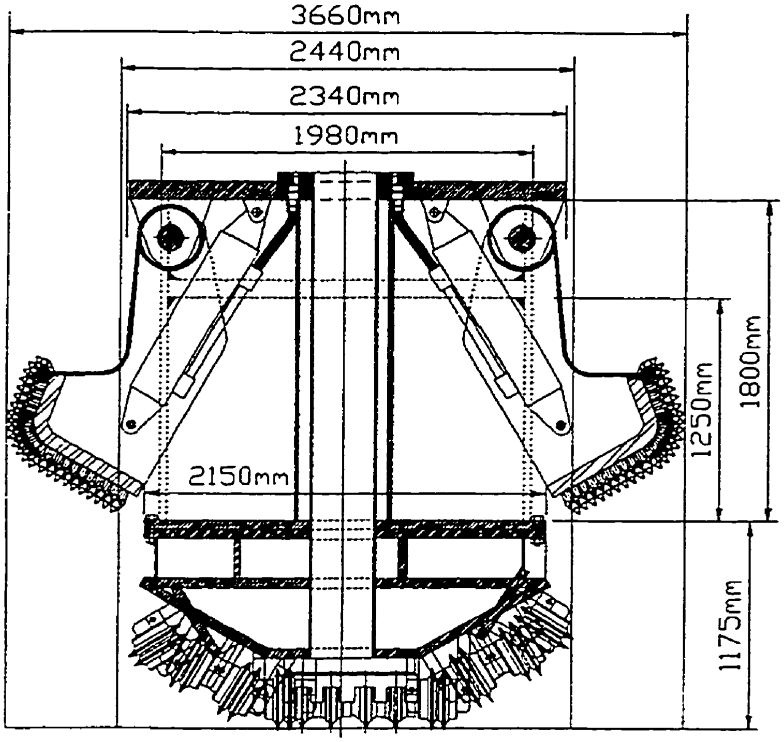

Figure 3-3: *Diagrammatic section view of expanding cutter head, after Zeni 1994*

The following advantages exist when a cutting head that expands from 2.44 to 3.66m is used:

1. the ore to waste ratio is higher than if single diameter cutting heads were used.
2. the backfill costs are reduced because all the material above the orebody is not excavated to the same diameter as it is through the orebody.
3. minimal disturbance will occur near the surface.

Potential disadvantages with using this system include:

1. the design of the reamer is more sophisticated than any that are presently in use and new technology is used to open and close the cutting head. Therefore, more maintenance will be necessary
2. the cutting head would have to be left in the hole if it could not be retracted after its diameter was expanded. This would result in the loss of the reamer, the downhole assembly, and part or all of the drill string.

BACKFILLING

The hole will be completely backfilled once the boring process is completed. The holes bored are filled with a concrete with a UCS equal to 20MPa from its bottom to the overburden contact. This fill will act as a 3.66m diameter pillar when mining an adjacent hole. The remainder of the hole will be filled with waste rock recovered from previously mined open pits. It is not necessary to use consolidated fill because the holes will not be adjacent to each other. Non-adjacent holes can be recovered with the blind boring rig while waiting for the concrete backfill to cure before mining the adjacent section.

ORE BODY ENVIRONMENT

Blind drilling from surface is a suitable method to consider for the recovery of the ore deposit within the constraints posed by the orebody environment if machine reliability concerns are resolved. The orebody environment constraints are principally the geotechnical/geomechanical weaknesses and the high porosity/permeability of the overburden and host rock.

RADIATION EXPOSURE

The boring will be conducted while the operators remain on surface. The cuttings will remain in a pipe throughout the blind boring process. The recovery of the ore by blind boring entails that the miners will not be exposed to as much radioactivity as they would if a conventional mining method was employed. The reason for this is that the mining will be carried out remotely from surface with minimal direct human exposure to the radioactivity of the ore. The ore is transported from the cutting head up to surface through the centre of the drill string. Once it reaches the surface, then the minerals and the gangue are separated. Throughout the separation process, there is little exposure by personnel to radiation.

GROUND STABILITY

Referring to the geomechanical analysis of Chapter 2, it was found that the ground in the neighbourhood of the two pods was weak. Because the overburden over the two pods at the Dominique-Janine extension has similar geotechnical properties to sand, failure is likely to occur around the drill string as it bores vertically downwards if no reinforcement is provided to the overburden. Also, the weak ground would not be able to support the proposed drill rig and other machinery. These geotechnical complications could be circumvented by either applying grout to the ground or freezing it. Another means of providing reinforcement to the ground is by using drilling mud as a drilling fluid. A cursory analysis indicated that ore/mud separation would be both difficult and expensive: water was deemed to be a more suitable drilling fluid.

3.2.4 **JET BORING**

Another non-conventional method that was considered for ore recovery from the southern periphery of the Dominique-Janine extension is jet boring. Jet boring is accomplished by first drilling a 406mm (16 inch) pilot hole down from surface through the overburden and bedrock. The borehole is cased with steel lining throughout the overburden. The casing reduces the diameter of the pilot hole to 381mm (15 inches). At a depth of one metre above the ore deposit, jet boring begins.

The jet boring cutting head is fastened to the end of the drill string. It discharges $70m^3/h$ of water into the orebody at a pressure of 30MPa. An 875HP pump located at surface supplies this pressure. The cutting action of the water jet is intended to create a cavity approximately 4m in diameter. The basic principle of jet boring is shown in Figure 3-4 (Thyssen, 1994).

The slurry of cuttings and water flows up the centre of the drill string at a working pressure of 3.5MPa. The processing of these cuttings is then similar to that for the blind hole bored cuttings. The holes also will be backfilled after the jet boring process is completed for each hole.

The jet boring process would need to be restricted to the warmer months because the hydraulics used are not winterised and would tend to freeze over the winter months. The tonnage and grade of this deposit would appear to be high enough to warrant this mining method. The orebody is located close enough to surface to render this method feasible. Although portions of the orebody are underneath the lake, the jet boring rig could be placed over these portions by building earth platforms above them.

3.3 DISCUSSION

The factors considered in the selection of a mining method are the environmental setting, geomechanical characteristics of the material through which excavation will take place, and radioactive exposure of those operating the mining machinery. The mining methods that were analysed to determine their technical feasibility in terms of these factors were open pit mining, underground mining, jet boring, and blind boring. The most economical methods of mining appear to be the two unconventional mining methods, jet and blind boring. Because the overburden and bedrock are susceptible to failure due to the fact that they are geomechanically weak are saturated, consolidation of the ground would be necessary in order to employ these mining methods.

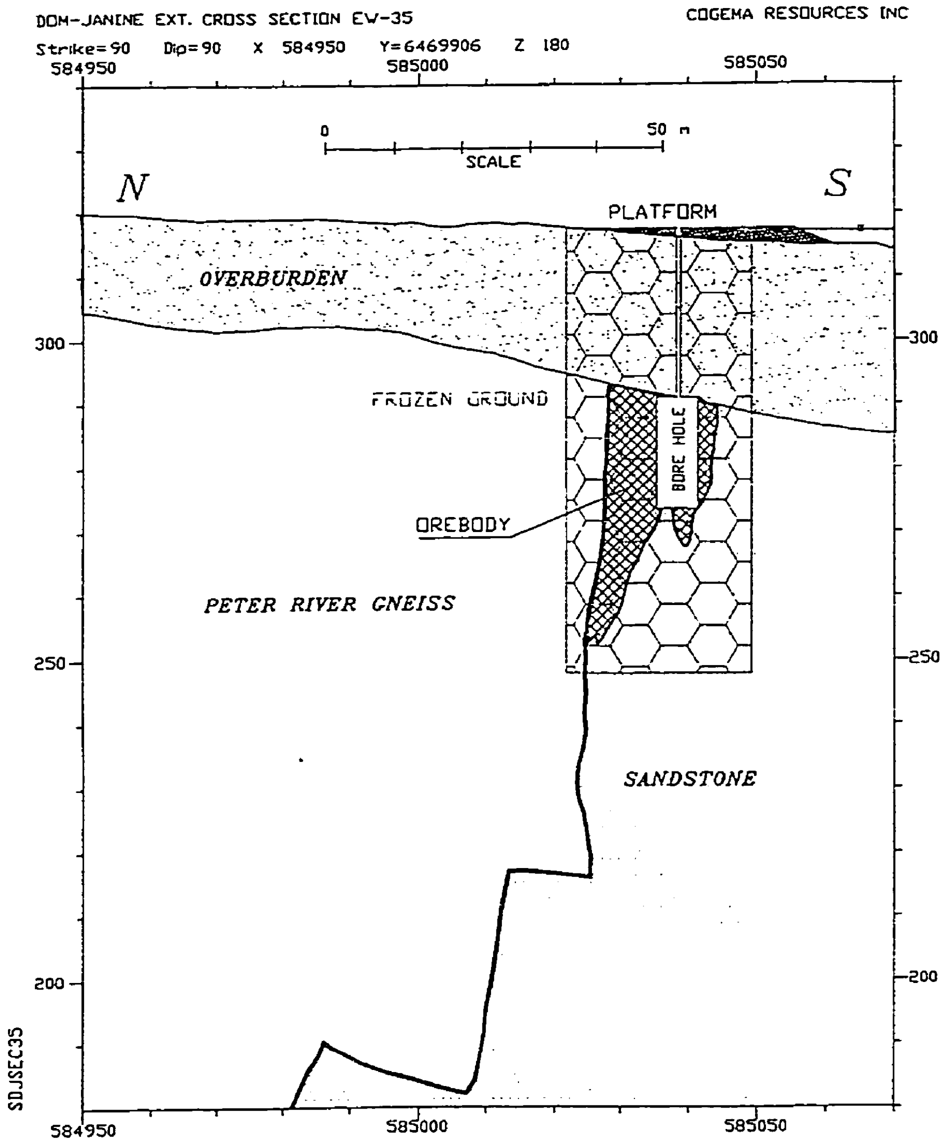

Figure 3-4: *Basic principle of jet boring, after Thyssen 1994*

Chapter 4

GROUND REINFORCEMENT TECHNIQUES

This chapter considers various methods to reinforce the ground to make either blind or jet boring effective. As discussed in Chapter 3, the ground surrounding holes bored at a 2.44m diameter when blind boring through the overburden of the Dominique-Janine extension is prone to failure. It is a cohesionless material with a low shear strength. Failure is also probable around a 406mm diameter hole drilled in the overburden prior to the high pressure emitted by the jet boring machine.

The geomechanical classification studies for the host rock of the two pods at the Dominique-Janine extension indicated that it was geomechanically weak. For this reason, failure is also considered to be likely around the drill string as it blind or jet bores through the host rock surrounding the orebody. Recovery of the drill string trapped due to failure of either the overburden or the bedrock would not necessarily be economically feasible. Under such circumstances, it would not be practical to use a boring method to recover the ore.

In order to prevent failure around the drill string, the application of external and internal reinforcement to the ground surrounding the two pods will now be examined. The external reinforcement considered will be shaft lining. The two internal reinforcement techniques examined will be ground freezing and grouting.

4.1 SHAFT LINING

Shaft lining as a form of external reinforcement consists of cylindrical lining segments. The diameter of these segments would approach that of the hole wall. This method of reinforcement was considered as a means of stabilising the hole walls as boring progressed. The capital cost underlying this method would likely be high due to the need to purchase new lining whenever a hole is drilled/bored. These costs would be lowered if the lining could be recovered and used for subsequent holes.

One of the main objectives in mining ore is to decrease time spent on recovery. Providing reinforcement would have to be done efficiently by minimising the time spent to install it in order to make the blind/jet boring methods feasible. Since the ground to be bored is weakly consolidated, the lining should be installed in stages as the depth increases.

Available types of lining are now considered: concrete lining, cast iron tubbing lining, and steel lining.

4.1.1 CONCRETE LINING

Concrete is frequently used to line shafts. The compressive strength of concrete ranges from 20 to 50 MPa. In order to fully utilise this compressive strength, special attention should be given to its pouring and curing during installation. Inadequate care while pouring can lead to high void ratios. The curing portion of the installation can also lead to volume changes of the voids and the concrete mass as a whole. These volume changes in concrete can cause excessive cracking. In addition, when concrete is used for shaft lining it should have a low water-cement ratio. High ratios can result in a layer of water forming between the lining and the hole wall.

The analytical basis for the calculation of the stresses that act on a concrete lining and its thickness will now be reviewed. Cast-in-place concrete lining, reinforced concrete lining, and concrete pile walls will also be summarised.

STRESSES AND LINING THICKNESS

Stress magnitudes that the concrete lining would likely be required to resist can be calculated using the Lamé relationship (Ostrowski, 1972), where it is assumed that the stress-strain relation is elastic and that the rock has isotropic geomechanical properties.

<u>*Lamé Formulae:*</u>

Equation 4-1

$$\sigma_t = \frac{pb^2}{b^2 - a^2}\left(1 + \frac{a^2}{r^2}\right)$$

Equation 4-2

$$\sigma_r = \frac{pb^2}{b^2 - a^2}\left(1 - \frac{a^2}{r^2}\right)$$

Equation 4-3

$$\sigma_z = \frac{pb^2}{b^2 - a^2}2\nu$$

where,

σ_t = tangential stress
σ_r = radial stress
σ_z = vertical stress
p = radial pressure on the lining
a = inside radius
b = outside radius
$a \leq r \leq b$
ν = Poisson's ratio of the lining material

Lining thickness can then be calculated with either another Lamé formula or the thin-wall ring formula (Ostrowski, 1972). The stress-strain relation is also assumed to be linear elastic in these formulae.

Equation 4-4: *Lamé formula*

$$d = a\sqrt{\frac{\frac{K}{SF}}{\frac{K}{SF} - 2p}} - 1$$

where,

d = lining thickness = b-a
K = uniaxial compressive strength
SF = safety factor

Equation 4-5: *Thin-wall Ring Formula*

$$d = \frac{pa}{0.95\frac{K}{SF} - p}$$

Although concrete is considered an elastic material within working load limits, the strain which the lining undergoes is not recovered instantaneously once the load is removed. This effect is still present at stresses below the yield stress and is known as viscoplastic flow or creep.

CAST-IN-PLACE CONCRETE LINING

This type of concrete lining is installed by pouring the concrete between a form and the hole wall. Because the concrete will be poured, cured, and set in the excavation, a bond will form between the lining and the host rock mass. In addition to acting as a continuum with the rock, this bond would also serve to bind the rock. This will increase the self-supporting ring action of the rock. The concrete can be applied remotely by using a plug at the bottom of the shaft. The plug would be installed up to five metres below the lining. The cutting head would destroy it as it drills/bores downwards.

REINFORCED CONCRETE LINING

One concern is a failure of the concrete lining as it bends under non-uniform outside pressure. To provide resistance to the bending forces in the concrete lining, reinforcing bars can be placed in the form between pours. This reinforcing steel will increase the bending strength of the lining.

The stress which causes the concrete material to bend is shear. For this reason, the steel bars should be orientated horizontally at an angle with respect to the radius as shown in

Figure 4-1. This can be undertaken remotely by installing the bars between poured vertical sections.

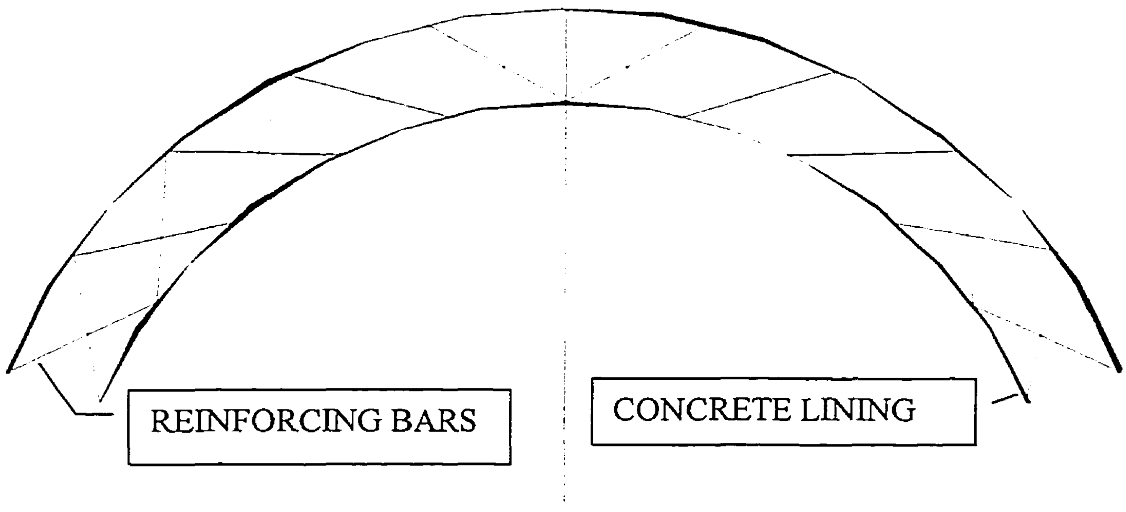

Figure 4-1: *Reinforced concrete lining*

CONCRETE PILE WALL

Concrete piles are installed by driving or drilling them into place. Piles are driven adjacent to each other to form a wall. Steel tubes cut into the concrete exterior adjacent to the pile being installed and form a tight wall, see Figure 4-2. This serves to prevent water and loose rock from penetrating the excavation. When the stability of the ground is particularly poor, slurry is installed with the piles to ensure that passive stress exerted on the wall rises at a consistent rate.

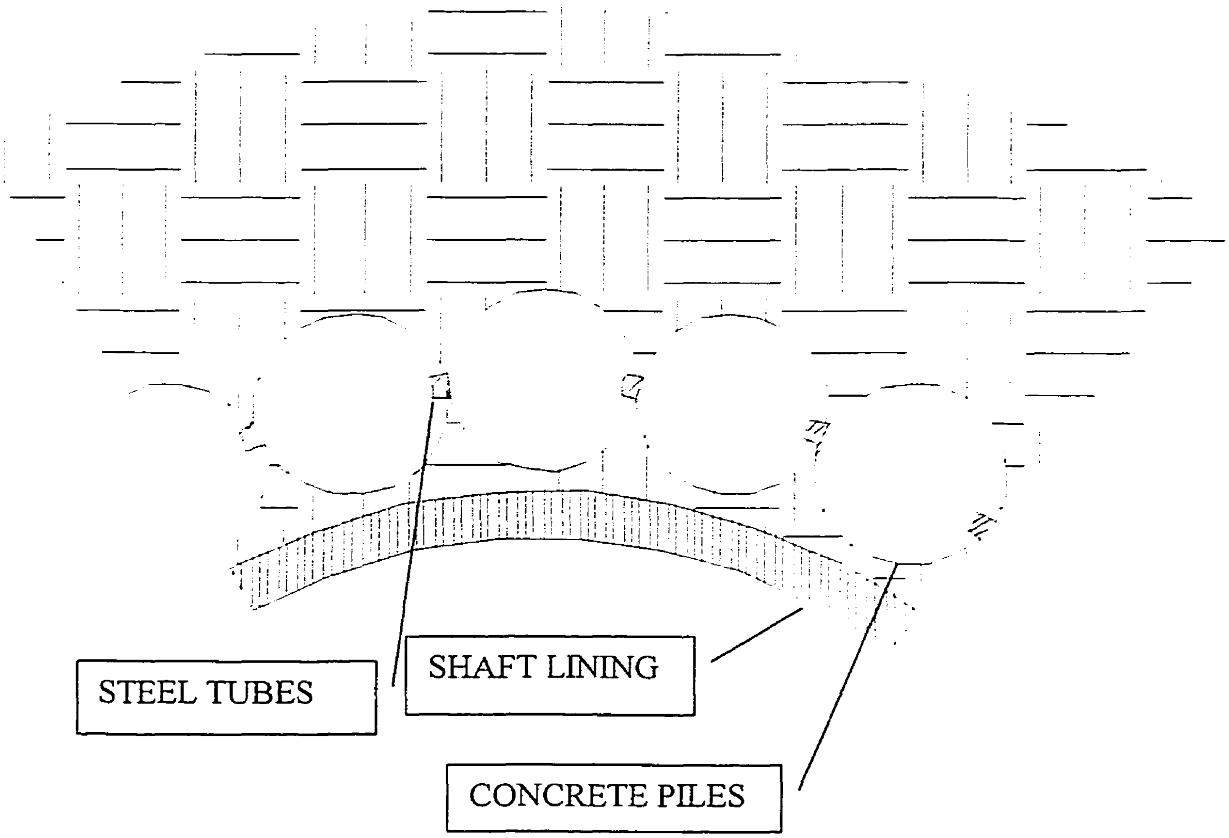

Figure 4-2: *Lateral section through concrete piles*

Concrete should not be used for shaft lining. Concrete is a particularly brittle material and it cannot be economically recovered from the hole. In order to install it in stages, the boring/drilling of the hole must be halted, and the drill string and drill machine must be moved. Then the concrete forms or blocks can be put into the hole. This would entail a lot of time is taken to install the lining.

4.1.2 CAST IRON TUBBING LINING

Cast iron tubbing lining consists of segments of cast iron sheets forming a ring around a shaft. A number of segments are bolted together per section. The number of tubbing segments installed per ring increases as the diameter of the shaft increases.

Concrete is poured between the tubbing and the excavation wall. The water/cement ratio of the concrete must be controlled because the higher the ratio, then the more probable it is that there will be layers of water between the tubbing and the concrete. Layers of water lessen the likelihood of a bond forming between the tubbing and concrete. This entails that groundwater would penetrate through the concrete to the tubbing and exert a higher pressure to the column that will rupture any bond the lining may have with the surrounding concrete, leaving a gap between the two materials.

When a gap exists between the tubbing ring and the concrete, deformation can occur. Because there would be radial and tangential contacts between the tubbing ring and the surrounding concrete in this instance, the ring section will bend. Therefore concavities

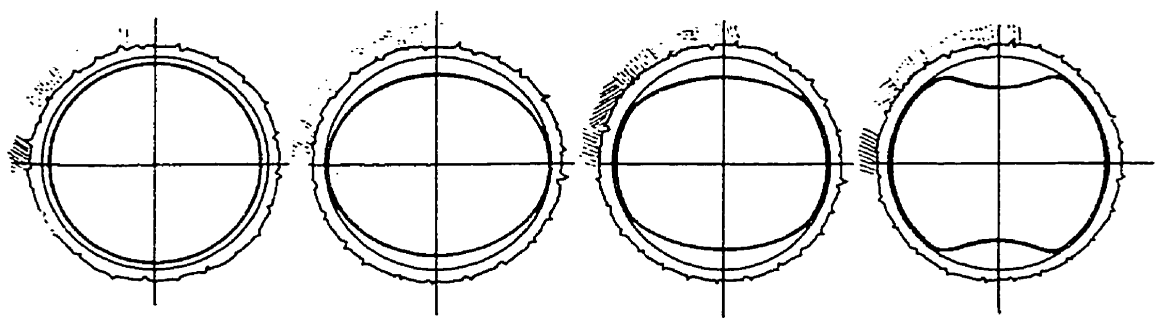

Figure 4-3: *Buckling stages of tubbing lining, after Ostrowski 1972*

will be formed on the circumference as illustrated in Figure 4-3 (Ostrowski, 1972). The progressive buckling of the lining may eventually lead to a collapse of the tubbing ring.

The tubbing segments can be lowered from a crane down the shaft and concrete can be poured between the segments and the hole wall remotely after a plug is installed. This will also be time-consuming because the boring machine must be moved out of the way during this process. The bond formed between the concrete and the lining entails that recovering the lining will also consume time. Alternatives should be considered for shaft lining material.

4.1.3 **STEEL LINING**

The strength of steel entails that it can withstand compressive, tensile, and shear stresses with a lesser thickness than the concrete or concrete and cast iron tubbing lining. Single steel shells would have sufficient strength if the load was uniform; however, a problem may exist with buckling under non-uniform lateral loads. If a concrete core is installed between two concentric steel sheets, then the durability of the concrete fill and the strength of the steel sheets combine to make a stronger lining.

SINGLE STEEL SHELL

The most common material used for a remotely placed lining in a blind bored shaft is a single steel shell. The strength of the steel is increased by stiffener rings on the outside. The strength of the steel shell allows for installation of a slender shell; however if it is too slender, buckling may occur due to the existence of non-uniform loads in the ground. Three guide ropes are employed on the outside of segments to accurately align the subsequent segments.

DOUBLE STEEL LINING WITH CONCRETE CORE

Double steel lining with concrete fill consists of two concentric steel shells, a concrete fill forms a core between these steel sheets. This core is surrounded by a steel shell which makes the concrete lining stronger and diminishes the possibility of buckling. A bituminous envelope is inserted in the gap between the outside steel shell and the primary lining. This bituminous layer acts as a cushion which would absorb rock deformation so it does not affect the main structure. This type of lining is shown in Figure 4-4. The steel lining is subject to compression from the hydrostatic pressure on the bitumen column.

The inside sheet is attached to anchors embedded in the concrete to prevent it from buckling. The risk of buckling can also be diminished by installing bleeder holes in the inside sheet. With these in place, the groundwater will not build up behind the sheet.

The buckling of lining can also arise from the bituminous material applying pressure to the lining. The stresses exerted by the bituminous material will eventually cease provided the rock formation movements do not generate stresses that are so great that they are not absorbable. The hydrostatic bitumen pressure, the non-uniform lateral pressure, and the maximum tolerable radius of curvature for the longitudinal bending should be considered when designing the lining. This radius typically varies between 1000 and 5000 metres.

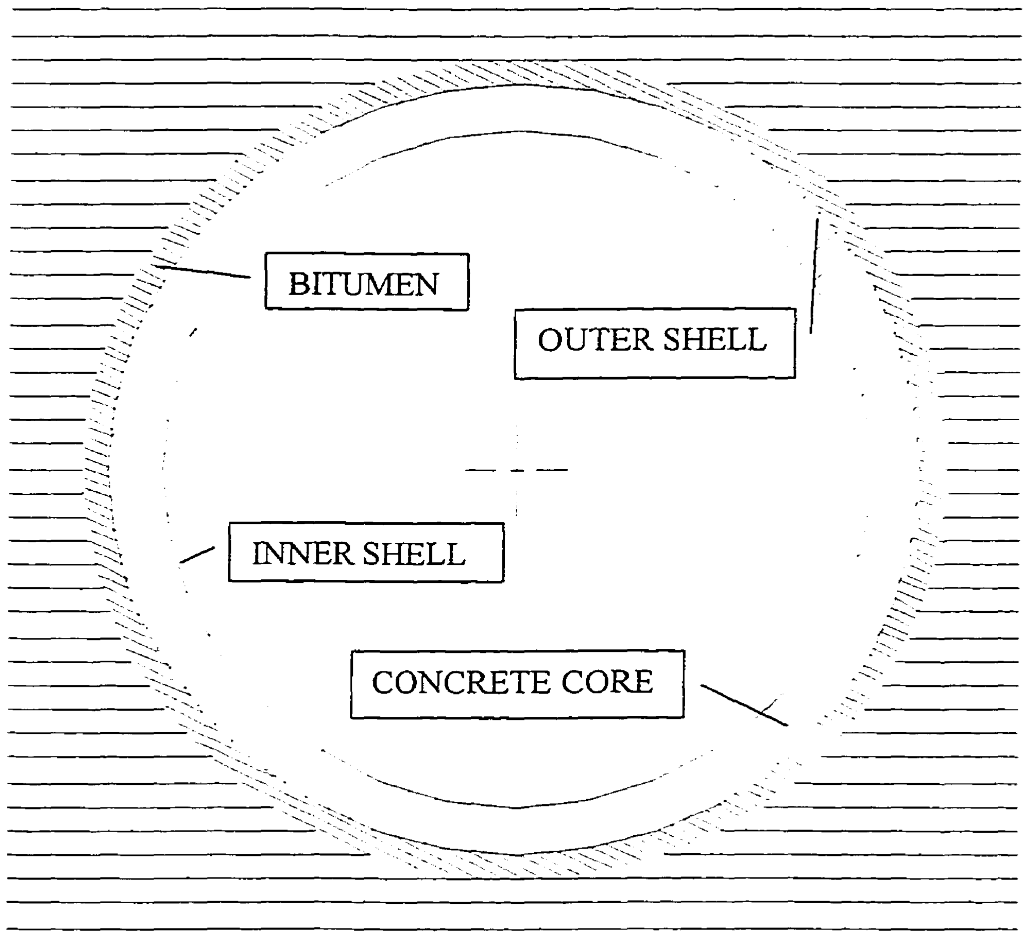

Figure 4-4: *Double steel lining with concrete core*

Installing double steel lining with concrete fill would slow down the sinking process of the holes. Lining recovery would be difficult as well due to the fact that the lining will consist of two steel shells binding with the concrete. This lining method does not appear to be practical for use when recovering the uranium ore from the two pods of the Dominique-Janine extension.

Single steel shell lining deserves consideration. Recovery of this lining is possible. A problem does occur with the fact that the installation of a single steel shell would be achieved in sections. This entails that the drilling/boring machine must be moved away during installation as a crane lowers the sections. The hole walls during the drilling/boring process will be exposed and prone to failure between the times that sections of steel lining are installed.

Effective ground support of the overburden, waste rock, and the orebody is necessary to prevent the ground from sloughing and trapping the drill string. A design of a shaft lining whose diameter would expand when the diameter is increased of the jet bored and blind bored holes when ore is being recovered has not yet been developed. This is another factor that would affect the technical feasibility of this reinforcement method.

4.2 GROUTING

Grout provides internal reinforcement when it is pumped into the ground. The addition of grout to the ground can serve two purposes. It can both decrease the ground permeability and increase the shear strength of the soil.

The permeability of the ground is decreased because the grout occupies voids within the soil and binds the soil particles together. Grout penetrates primarily within the rock discontinuities. In doing so it mixes with material within these discontinuities. This decreases the permeability of the material and therefore the permeability of the rock mass as a whole.

The shear strength of soil is increased by grouting mainly because the grout occupies voids within the soil and increases its cohesion. This is accomplished by the grout binding the soil particles together. Increases in the angle of friction of grouted soil is marginal as is the strength of rock. The aim of grouting the Dominique-Janine extension would be to increase the shear strength characteristics of the soil. When this is accomplished, then the overburden can be drilled without any danger of the soil caving in around the drill string.

Some factors that vary in a grouting program are: the appropriate grout type, the spacing between the holes where the grout is added, and the setting time. In order to ascertain the success of the grouting program then the quantity of grout that flowed into the ground, the extent to which the grout had mixed with the ground, and the strength properties of the grouted soil need to be investigated.

4.2.1 GROUTING PROGRAM

The means to apply the grout to the Dominique-Janine extension that is considered is by injection into the overburden. Grout is injected into the ground through an injection pipe within a borehole at a pressure that is low enough that the ground structure is not disturbed. The vertical section that is grouted is isolated between two internal packers. The grout spreads throughout the ground in an approximately spherical fashion, see Figure 4-5.

The grout is propelled down the injection pipe under pressure and flows out of the end of the pipe between the two packers. After the void between the two packers is filled, then the grout in the pipe is still applying pressure to this filled void. This pressure is dissipated as the grout flows into the surrounding ground.

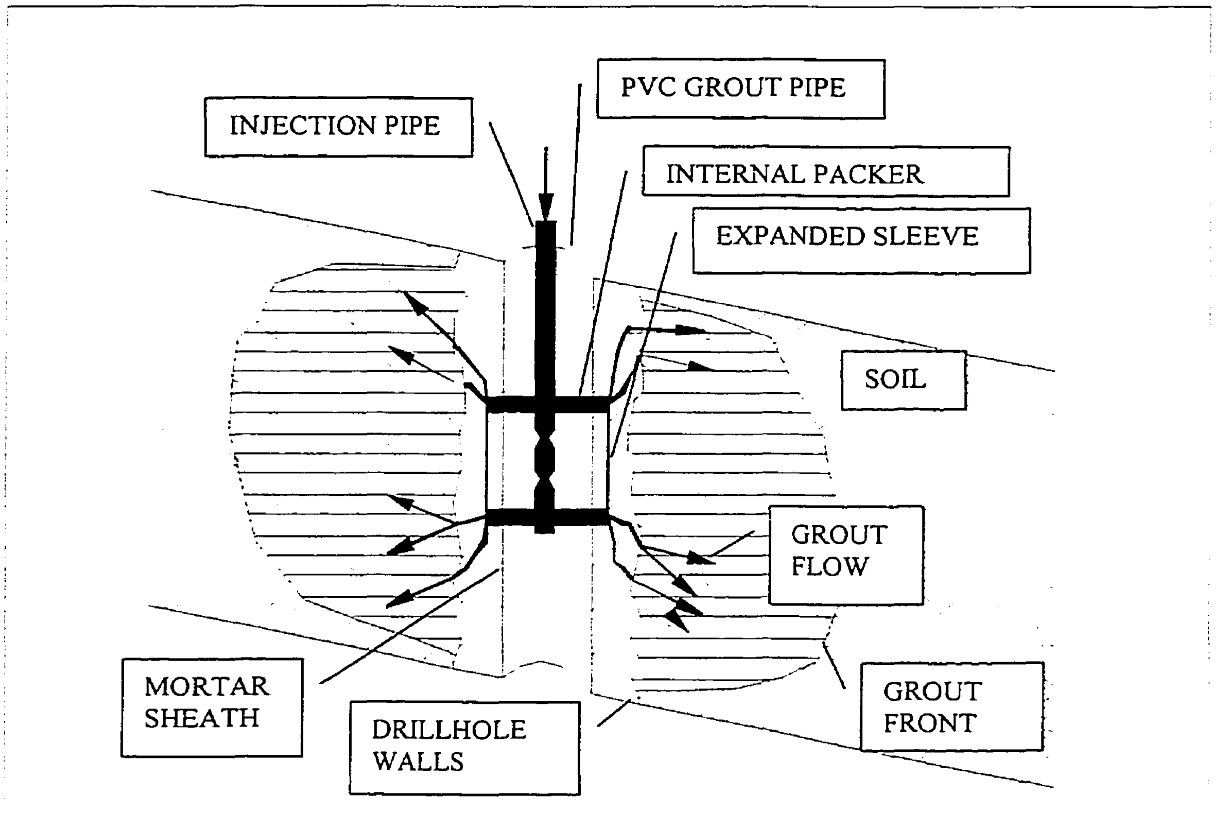

Figure 4-5: *Permeation grouting*

4.2.2 GROUT INJECTION

The pressure with which the grout flows into the ground decreases as the grout flows from the borehole. The grout begins to flow more into the higher permeability zones at the expense of the lower permeability zones as its flow pressure dissipates. The grout permeates in an approximately spherical volume, thus those areas of the ground that lie outside the radial extent of the spheres are not grouted. These voids of ungrouted soil left between the primary injection holes are mixed with grout through secondary injection holes between these holes. This method of split-spacing is shown in Figure 4-6.

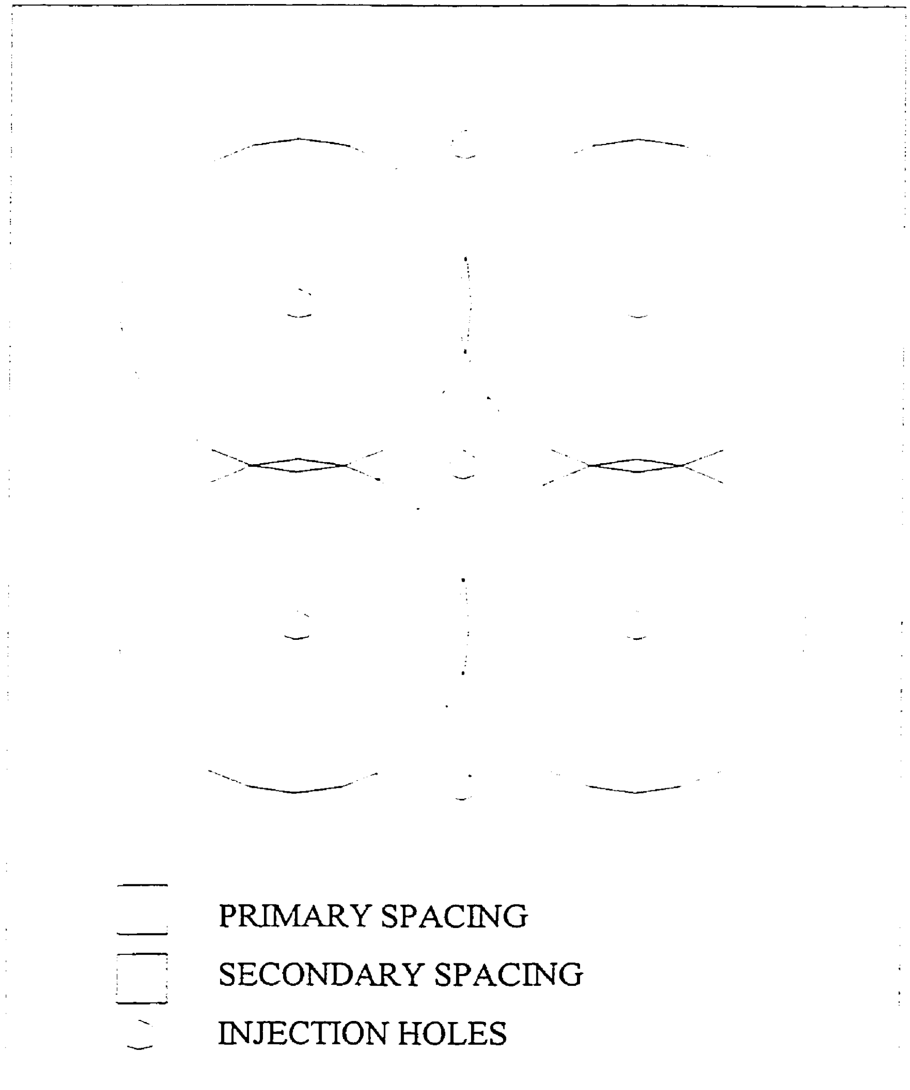

Figure 4-6: *Split spacing*

The pressure which injects the grout into the ground must be limited in order to avoid ground heave. Limiting the spacing between drillholes minimises the injection pressure. The final spacing between holes is usually between 0.5 and 2.5m.

The volume of voids in a specific volume of soil is characterised through the porosity of the soil. The porosity of the overburden at the Dominique-Janine extension was estimated to be 0.3 as shown in Table 2.3. The quantity of grout that must be added to each hole is estimated by considering this value.

The approximate size of the voids into which the grout must flow into the soil is found by conducting an analysis of the grain size distribution. As shown in Table 2.3, the D_{10} grain size of the Dominique-Janine overburden is 0.20mm. The ability of the grout to flow through the voids within the ground is revealed through a permeability analysis of the ground. The permeability of the overburden is 10^{-4}m/s as indicated in Table 2.3.

Table 4.1 provides a summary list of grout types that can be used, depending on the properties of the soils. The principal factors that vary in these soils are the D_{10} grain sizes and permeabilities. The appropriate grout type for the Dominique-Janine Extension would appear to be composed of sodium silicate-mixed esters base.

TABLE 4.1: *Appropriate grouts for different soil grain sizes*

SOIL D_{10} GRAIN SIZE (mm)	SOIL PERMEABILITY (m/s)	APPROPRIATE GROUT TYPE
>0.8	$>10^{-2}$	**Ordinary Portland Cement**
>0.05	$>10^{-3}$	**Ultrafine Cement**
>0.02	$>10^{-5}$	**Sodium Silicate-Mixed Esters**

4.2.3 QUALITY ASSURANCE

Laboratory tests can be conducted to determine the changes in the physical properties of the ground; however, the properties indicated in these tests should be treated with caution.

In order for these tests to be representative, the sample must be in a similar environment as that existing in the field. Tests should be conducted in the field to evaluate if the grout injected into the ground performs adequately in its function of either decreasing the permeability or increasing the ground strength. The effectiveness of a grouting program can be determined also by using geophysical testing methods. These determine the extent to which the grout has filled the voids throughout the volume to which it is applied.

Laboratory tests can also assist in evaluating if a grouting program was effective. One method to recover a sample for tests is through core drilling. The primary means used to recover the core is by rotary drilling with a core barrel; however small gravel particles or broken pieces of the grouted ground can work their way up the core barrel, and abrade the sides of the sample. Such pieces may also break the samples by flexing them. Rough handling of the core after recovery can also cause the sample to be damaged before testing.

Testing soil that remains in the walls of test pits will give a more accurate depiction of the geomechanical properties of the material than testing core samples. Strength tests can be conducted on the undisturbed walls of the test pit. The success behind the permeation of grout can be detected by odour, colour, or chemical analyses of the soil. A disadvantage to this testing method is the costs associated with excavating the pit.

In addition to ensuring the sample is not disturbed for a laboratory test, the in-situ environment where the soil was located must also be represented when sampling. This will ensure a more accurate depiction of the physical properties of the soil.

One means to test if a grouting program has increased the strength of the soil in its in-situ environment is the Standard Penetration Test (SPT). This is a dynamic test. A rod is orientated vertically above the soil and is subjected to a series of blows. The number of blows that must be applied to a rod to have it travel a specific vertical distance is greater when the shear strength of a soil is increased. After a soil is grouted, it is easily shattered: therefore the blow count may not necessarily accurately show the magnitude of strength increase. For this reason, SPT's are only a crude strength testing tool for grouted soil.

Geophysical testing is accomplished on the site without disturbing the samples or the ground. This testing method analyses an entire volume to determine the extent of grout permeation. Borehole radar profiling is one means to accomplish this. It involves the transmission of microwaves from one borehole to another. Silicate grouted sand appears opaque when a microwave is transmitted through it: therefore, the reduction in the intensity of the signal viewed signifies that the grouting program was successful.

Acoustic velocity profiling is another form of geophysical testing which involves transmitting shear waves from one borehole to another. Silicate grouted soil has markedly increased acoustic velocity as compared to natural soil. Increased acoustic velocities of shear waves after grouting attests to successful grouting programs.

Grout would successfully permeate throughout the overburden but not the discontinuities in the host rock due to the presence of clay within them. The permeability of clay is low and requires a grout with a low particle size or a low viscosity. The overburden over the

Dominique-Janine extension can be grouted with sodium silicate-mixed esters grout. This would serve to increase the cohesion of the overburden.

4.3 GROUND FREEZING

Ground freezing prevents water ingress and provides internal reinforcement to the ground. Water ingress is prevented because the frozen groundwater-ground matrix is impervious. This matrix is termed an ice wall. The internal reinforcement of the ground is provided because the ground and ice formed a matrix. This matrix has a greater shear strength than the ground before freezing. This strength prevents the ground from failing when normal stress is applied. The ice wall will normally absorb the load of both soil/rock and hydrological pressure.

4.3.1 FREEZING PROGRAM

Ground investigation must be carried out before a freezing program begins. Boring must be carried out to determine the stratigraphy and the location of permeable zones when investigating the geomechanical properties of the soil/rock. Groundwater may flow within the permeable zones in the soil/rock.

The purpose of a ground investigation program is to alert the engineer designing the freezing program of any potential problems or difficulties. A potential problem that needs to be investigated is the velocity of groundwater flow. This could prevent an ice wall from forming because the groundwater is flowing too quickly through the zone where the freeze pipes are located. A flow rate of 2 m/day has been commonly accepted as the upper bound above which conventional brine coolants cannot be used in the original environment. It may be necessary to use multi-rows of pipes or perform grouting to lower the permeability of the soil/rock by grouting if the groundwater flow exceeds 2 m/day.

The main objective in a freezing program is to extract heat from the ground into a chilled brine, thus lowering the temperature of the ground below the groundwater temperature. The brine is chilled by refrigerating it. It is normally calcium chloride with a specific gravity from 1.24 to 1.28. When the specific gravity is outside this range, it freezes less readily. The primary refrigerant usually used to cool the brine is either ammonia or freon. The refrigeration plant uses the refrigerant to provide a heat transfer medium that dissipates any heat stored in the brine. The chilled brine is pumped through sets of concentric pipes that are connected in parallel. These pipes are sunk into the ground and normally range in diameter from 150 to 200mm. The minimal diameter of the pipes to achieve laminar flow is dictated by normal hydraulic principles.

The brine is pumped into the inner pipe of the freeze hole. Once the brine flows down to the hole bottom, then it flows up the annulus between the inner and outer pipes, see Figure 4-7. The brine is coldest at the bottom of the hole which entails that the ice wall is thickest where the stresses are highest. The material of the freeze pipes must be chosen so that it does not become brittle due to the low temperature of the brine, in order to better resist the loads imposed upon it by the surrounding ground.

The flow of the brine up the annulus is generated by pumping the brine into the central pipe in addition to the kinetic energy when the water flows to the bottom of the pipe.

This provides enough hydraulic head for the brine to flow up the annulus. Brine usually flows to pipes connected in parallel. The heat exchange that occurs when the brine enters the central pipe and then flows up the annulus requires that the brine must be rechilled.

Ground freezing begins the cooling of the ground—groundwater mix from the ambient temperature to the freezing point. At the freezing point, a constant temperature is maintained as the ground goes through a latent heat transition. The temperature then drops below the freezing point. A common source of heat input that will counteract this refrigerative effort is natural or induced groundwater flow.

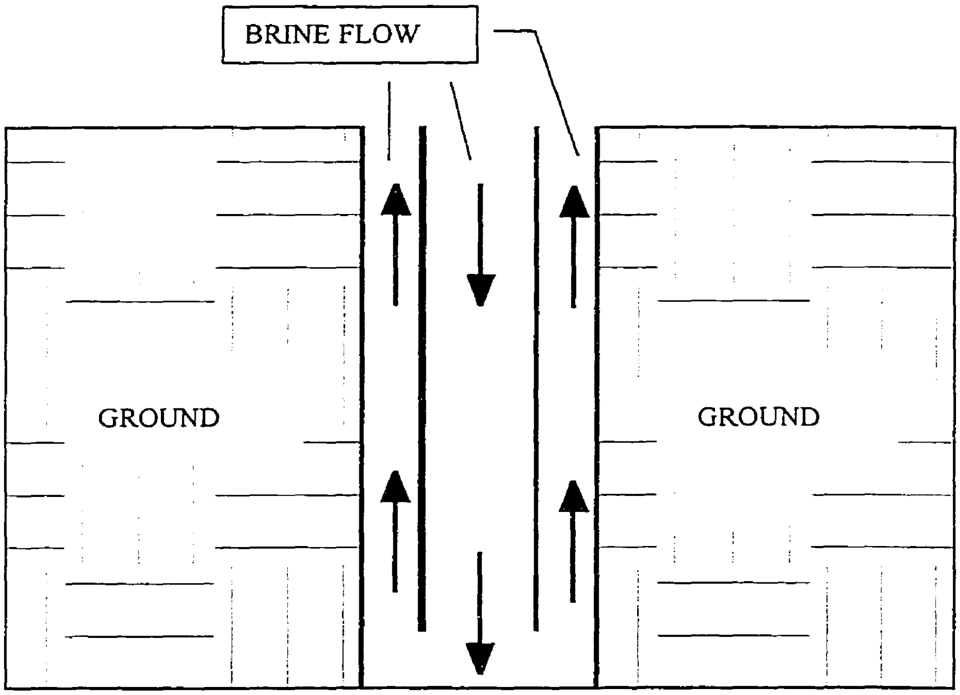

Figure 4-7: *Brine flow through freezing pipes*

The stresses exerted on the frozen ground is shared by both the ice and the ground. The ground must have an adequate moisture content in order to freeze whereupon it forms an ice-ground matrix. The cohesion of the ice-ground matrix is responsible for an increased strength of the frozen ground. Strength increases are more prevalent when soil is frozen as opposed to rock. The hydraulic conductivity of either rock or soil is decreased after freezing due to the fact that ice is relatively impermeable.

The refrigeration capacity of a freezing system is directly proportional to the spacing between freeze tubes, their total length, and the brine flow rate. The selection of the number and spacing of the freeze tubes must keep this in mind.

The brine would likely flow out of a tube into the surrounding ground if the tube would break. When the brine flows out of the broken tube, it would be damaging to the environment. The ground into which the brine flows would not freeze. It is important that fractured pipes are detected soon after they break and the brine flow rate is halted in order to limit environmental damage and to ensure the ground is frozen homogeneously.

4.3.2 QUALITY ASSURANCE

The success of the freezing process is first checked by monitoring the progress in building an ice wall. In order to check the formation, growth and integrity of the ice-wall, then the strata temperature at strategic points can be measured. These points should be located where it is most difficult for the ground to freeze. This situation is prevalent outside the freeze wall and where underground groundwater movement is known or suspected. A decline in the hydraulic conductivity of the ground can be tested by the same testing methods that were reviewed in section 2.1.7.

Geomechanical characteristics of the frozen ground can be tested either in-situ or in a laboratory. Standard equipment used for field investigation are those used for static core penetrometer and pressuremeter tests. Acoustic wave velocities can also be used to monitor the success of ground freezing. In—situ tests have the advantage that they can identify the properties of a frozen soil/rock without mechanical or thermal disturbance. They can be tested in-situ by measuring short term stress-strain relationships, creep parameters, multistage, and a series of long term tests at successive pressure increments.

When extracting a sample and transporting it to a laboratory, care must be taken to ensure that the environment of the sample when tested is kept similar to that of the sample if it were to remain in the ground. Particular attention must be paid to maintain its pre-existing temperature and to preserve its moisture content. This can be accomplished by wrapping the sample in polyethylene bags from which all air is evacuated before it is sealed. The bagged sample is then transferred to a thermostatically controlled freezer. Preparatory work of frozen samples should be carried out in a refrigerated workshop within the laboratory. Specimens should be handled with insulated gloves.

The purpose of freezing the ground within and over the two pods at the Dominique-Janine extension is to make it a solid ground-ice matrix. This is done to prevent the ground from collapsing and trapping the drill strings of the blind or jet boring machines. Groundwater flow is one of the greatest hazards to effective freezing. The ground surrounding the two pods is saturated to one metre below the ground surface surrounding the two pods of the Dominique-Janine extension: ground freezing can be accomplished. A test was conducted with freezing the ground and blind boring. It appears that the drilling fluid thaws the ground: this method was found not to be feasible in conjunction with blind boring. Ground freezing is carried out in conjunction with jet boring at Cigar Lake mine, see Figure 1-1.

4.4 DISCUSSION

The testing methods that were used to geomechanically characterise the ground surrounding the two pods of uranium ore in the southern periphery of the Dominique-Janine extension revealed that both the overburden and the bedrock are geomechanically weak. Blind and jet boring were found to be the most feasible mining methods to employ to recover the uranium ore from these two pods.

Failure would likely occur around the blind or jet bored holes because the ground is weak. Methods of reinforcing the ground surrounding the two pods were therefore examined. An external reinforcement method examined was shaft lining. Concrete, cast iron tubbing, and steel lining were considered. Installation of these lining materials is time consuming. Also, there is no economically feasible technique that can be used to expand the diameter of the lining when the diameter of the holes being bored is expanded: using a lining in this case to provide reinforcement is not feasible.

One of the means explored to provide internal reinforcement was to permeate grout throughout the ground surrounding the two pods. This would increase the geotechnical stability of the overburden. Grouting does not increase the geomechanical stability of the bedrock, although it can decrease its permeability. The discontinuities of the bedrock contain clay inclusions, whose permeability is too low for the grout to flow through effectively. Therefore, it would appear that grout would not provide reinforcement to the overburden and bedrock.

Freezing was another means considered to provide internal reinforcement. This would be accomplished by chilled brine flowing through two concentric pipes that are sunk in the ground from the surface to the bottom of the ore deposit. This action freezes the groundwater in both the soil and the bedrock. The matrix formed between the ice, and the soil or bedrock is consolidated preventing collapse of the ground around the drill string. Ground freezing was found to be possible only in conjunction with jet boring.

Chapter 5

CONCLUSION

The geomechanical characteristics of the overburden, the country rock, and the ore pod of the Dominique-Janine extension have been reviewed in this thesis. The analysis of the overburden revealed that it has a similar grain size distribution as medium sized sand. The tests of the country rock and host rock revealed that the rock quality is poor and that it has been fractured. In addition there are zones of clay within the discontinuities of the rock matrix. The permeability and porosity of the overburden is high enough that water can flow steadily through it. Water can also flow through the bedrock because it is fractured. The Dominique-Janine extension deposit is located on the shore of Cluff Lake and hydraulic continuity could exist between the lake, and the overburden and bedrock matrices.

In designing a mining technique for the uranium pods it was necessary to consider the environmental setting, geomechanical characteristics, and workers' radiation exposures. The mining methods investigated were open pit mining, underground mining, blind boring, and jet boring. Neither open pit nor underground mining methods appeared to be economically feasible to recover the ore. The analyses of blind boring and jet boring revealed that either of these methods could be used to recover the ore.

Because both the overburden and bedrock have geomechanically weak characteristics, failure is likely to occur around the drill string with either blind or jet boring. Therefore, it would be necessary to provide reinforcement to the overburden and bedrock. The use of shaft lining was examined as a means of providing external reinforcement; however, this was found not to be feasible because of the time taken to install shaft lining segments. Two methods investigated to provide internal reinforcement are grouting and ground freezing. The grout could be used to provide reinforcement to the overburden.

The grout would not increase the strength of the rock but rather decrease its permeability. There is clay within the discontinuities of the host rock. Clay has a small void size, a low permeability, and the grout cannot permeate through it. Freezing can be used to consolidate both the overburden and bedrock because of the presence of groundwater in these mediums. Once the groundwater within the overburden and bedrock freezes, the two materials would then be stabilised and rendered impermeable.

At the end of this study a mining method was chosen by Cogema Resources Inc. It decided to use the jet boring method to recover the uranium ore from the two pods in the southern periphery of the Dominique-Janine Extension. Ground freezing was chosen as the appropriate means of providing reinforcement to the overburden and the deposit host rock before drilling/boring to recover the uranium ore. The principal reason for selecting jet boring was due to the higher level of initial investment costs if blind boring was chosen. The initial investment would be higher because the blind boring mining machine with the expandable cutting head has not been previously developed. In contrast, the jet boring mining machine has been previously developed: therefore, the costs incurred to develop jet boring cutting heads will be significantly less.

Also, the cross-sectional area bored in the overburden by the cutting head of blind boring machines is equal to 4.7 m^2. Once the blind boring cutting head diameter expands then the cross-sectional area will increase by 125% to 10.5 m^2. The cross-sectional area of the pilot hole drilled for the jet-boring cutting head to access the deposit is 0.2 m^2. Once the high pressure water jets begin to cut the rock within the orebody, this area should increase by more than 6000% to 12.5 m^2. The stability of the roof over the jet boring cutting head in this situation would therefore be critical.

Ground freezing was considered to be the most practical method to consolidate the overburden and host rock. Freezing will be accomplished by pumping a –25°C brine through holes spaced 6 m apart. The burden between the holes will be 1.5m. The cold from this brine will freeze both the overburden and host rock environments and will provide them with the necessary internal reinforcement. At the time of completing this thesis the performance of the ore recovery and ground reinforcement methods selected was not fully evident.

5.1 FUTURE RESEARCH

Although blind boring was not chosen as the mining method to recover the ore in the two pods at the Dominique-Janine extension, it should be considered if jet boring is found not to be practical. This would be the case when the operating pressure to emit the water jets from the jet boring machine would be too high to cut an adequate excavation volume. When the cutting head expands, the cross-sectional of the cavity cut by the blind boring machine increases by 125%. This results in a significant cost saving because not as much need be removed from the orebody.

Jet boring was chosen as an appropriate mining method. The operating pressure from a cutting head can be compared to the size of the excavation for different rock types. The water pressure eventually reaches a level beyond which no appreciable increase in excavation size is achieved.

Grouting overburden and bedrock may prove worthwhile for future projects. Grout can both increase the strength and decrease the permeability of any soil into which it is injected. In addition once it is injected into the ground, maintenance of the ground-grout matrix is minimal. Grouting rock, however, only decreases the permeability of rock but does not improve its shear strength. After considering grouting the mineral deposit of the Dominique-Janine extension, it was concluded that the grout would not flow through the low permeability clay inclusions within the discontinuities created by the hydrothermal deposition of the ore minerals. If those clay inclusions were absent, grouting would have provided internal reinforcement to the overburden but may have caused excess reagent consumption in the mill when combined with the ore.

Ground freezing was eventually chosen to reinforce both the overburden and the bedrock. It would not have been suitable for reinforcement for blind boring because drilling fluid would have thawed the ground. Future work could investigate temperatures to which the ice-ground matrix could be lowered to prevent thawing of the ground by the drilling fluid.

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Appendix A: *Geomechanical Rock Mass Classification with Bieniawski's RMR System, after Bieniawski 1989*

TABLE A.1: *Bieniawski's Rock Mass Rating System (Geomechanics Classification of Rock Masses)*

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Ranges of Values						
1	Strength of intact rock material	Point-load strength index (MPa)	>10	4 – 10	2 – 4	1 – 2	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength (MPa)	>250	100 – 250	50 – 100	25 – 50	5 – 25	1 – 5	<1
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD (%)		90 – 100	75 – 90	50 – 75	25 – 50	<25		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		>2 m	0.6 – 2 m	200 – 600 mm	60 – 200 mm	<60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered wall	Slickensided surfaces or Gouge < 5 mm thick or Separation 1 – 5 mm Continuous	Soft gouge > 5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10 m tunnel length (L/min)	None	<10	10 – 25	25 – 125	>125		
			or	or	or	or	or		
		Ratio $\frac{\text{Joint water pressure}}{\text{Major principal stress}}$	0	<0.1	0.1 – 0.2	0.2 – 0.5	>0.5		
			or	or	or	or	or		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

TABLE A.1 (cont'd): *Bieniawski's Rock Mass Rating System (Geomechanics Classification of Rock Masses*

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS

Strike and Dip Orientations of Discontinuities		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels and mines	0	−2	−5	−10	−12
	Foundations	0	−2	−7	−15	−25
	Slopes	0	−5	−25	−50	−60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	<20
Class no.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK MASS CLASSES

Class no.	I	II	III	IV	V
Average stand-up time	20 yr for 15-m span	1 yr for 10-m span	1 wk for 5-m span	10 h for 2.5-m span	30 min for 1-m span
Cohesion of the rock mass (kPa)	>400	300−400	200−300	100−200	<100
Friction angle of the rock mass (deg)	>45	35−45	25−35	15−25	<15

[a]After Bieniawski (1979).

CHART E Guidelines for Classification of Discontinuity Conditions[a]

Parameter	Ratings				
Discontinuity length (persistence/continuity)	<1 m	1−3 m	3−10 m	10−20 m	>20 m
	6	4	2	1	0
Separation (aperture)	None	<0.1 mm	0.1−1.0 mm	1−5 mm	>5 mm
	6	5	4	1	0
Roughness	Very rough	Rough	Slightly rough	Smooth	Slickensided
	6	5	3	1	0
Infilling (gouge)		Hard filling		Soft filling	
	None	<5 mm	>5 mm	<5 mm	>5 mm
	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
	6	5	3	1	0

[a]*Note:* Some conditions are mutually exclusive. For example, if infilling is present, it is irrelevant what the roughness may be, since its effect will be overshadowed by the influence of the gouge

TABLE A.2: *Guidelines for excavation and support of rock tunnels in accordance with Bieniawski's Rock Mass Rating System*

Rock Mass Class	Excavation	Support: Rock Bolts (20-mm Dia, Fully Grouted)	Support: Shotcrete	Support: Steel Sets
Very good rock I RMR:81–100	Full face 3-m advance	Generally, no support required except for occasional spot bolting		
Good rock II RMR:61–80	Full face 1.0–1.5-m advance Complete support 20 m from face	Locally, bolts in crown 3 m long, spaced 2.5 m, with occasional wire mesh	50 mm in crown where required	None
Fair rock III RMR: 41–60	Top heading and bench 1.5–3-m advance in top heading Commence support after each blast Complete support 10 m from face	Systematic bolts 4 m long, spaced 1.5–2 m in crown and walls with wire mesh in crown	50–100 mm in crown and 30 mm in sides	None
Poor rock IV RMR: 21–40	Top heading and bench 1.0–1.5-m advance in top heading. Install support concurrently with excavation 10 m from face	Systematic bolts 4–5 m long, spaced 1–1.5 m in crown and wall with wire mesh	100–150 mm in crown and 100 mm in sides	Light to medium ribs spaced 1.5 m where required
Very poor rock V RMR: <20	Multiple drifts 0.5–1.5-m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting	Systematic bolts 5–6 m long, spaced 1–1.5 m in crown and walls with wire mesh. Bolt invert	150–200 mm in crown, 150 mm in sides, and 50 mm on face	Medium to heavy ribs spaced 0.75 m with steel lagging and fore-poling if required. Close invert

[a]Shape: horseshoe; width: 10 m; vertical stress: <25 MPa; construction: drilling and blasting.

Appendix B: *Geomechanical Rock Mass Classification with Barton's Q-System, after Bieniawski 1989*

TABLE B.1: *Q-system descriptions and ratings→RQD, J_n, J_r, J_a, SRF, and J_w*[a]

Rock Quality Designation (RQD)

Very poor	0–25	*Note:*
Poor	25–50	(i) Where RQD is reported or measured as ≤10 (including 0), a nominal value of 10 is used to evaluate Q in equation (5.1).
Fair	50–75	
Good	75–90	
Excellent	90–100	(ii) RQD intervals of 5, i.e., 100, 95, 90, etc., are sufficiently accurate

Joint Set Number J_n

Massive, none or few joints	0.5–1.0	*Note:*
One joint set	2	(i) For intersections, use $(3.0 \times J_n)$
One joint set plus random	3	
Two joint sets	4	(ii) For portals, use $(2.0 \times J_n)$
Two joint sets plus random	6	
Three joint sets	9	
Three joint sets plus random	12	
Four or more joint sets, random, heavily jointed, "sugar cube," etc.	15	
Crushed rock, earthlike	20	

Joint Roughness Number J_r

(a) Rock wall contact and		*Note:*
(b) Rock wall contact before 10-cm shear		(i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m
Discontinuous joint	4	
Rough or irregular, undulating	3	

TABLE B.1 *(continued)*

Smooth, undulating	2.0	*Note:*
Slickensided, undulating	1.5	(ii) J_r = 0.5 can be used for planar slickensided joints having lineation, provided the lineations are favorably oriented
Rough or irregular, planar	1.5	
Smooth, planar	1.0[b]	
Slickensided, planar	0.5	(iii) Descriptions B to G refer to small-scale features and intermediate-scale features, in that order
(c) No rock wall contact when sheared		
Zone containing clay minerals thick enough to prevent rock wall contact	1.0[b]	
Sandy, gravelly, or crushed zone thick enough to prevent rock wall contact	1.0[b]	

Joint Alteration Number J_a

(a) Rock wall contact	J_a	ϕ_r (approx)
A. Tightly healed, hard, nonsoftening, impermeable filling, i.e., quartz or epidote	0.75	
B. Unaltered joint walls, surface staining only	1.0	25–35°
C. Slightly altered joint walls. Nonsoftening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25–30°
D. Silty or sandy clay coatings, small clay fraction (nonsoftening)	3.0	20–25°
E. Softening or low-friction clay mineral coatings, i.e., kaolinite, mica. Also chlorite, talc, gypsum, and graphite, etc., and small quantities of swelling clays (discontinuous coatings, 1–2 mm or less in thickness)	4.0	8–16°
(b) Rock wall contact before 10-cm shear		
F. Sandy particles, clay-free disintegrated rock, etc.	4.0	25–30°

TABLE B.1 *(continued)*

	Joint Alteration Number J_a	
G. Strongly over-consolidated, nonsoftening clay mineral fillings (continuous, <5 mm in thickness)	6.0	16–24°
H. Medium or low over-consolidation, softening, clay mineral fillings. (continuous, <5 mm in thickness)	8.0	12–16°
J. Swelling clay fillings, i.e., montmorillonite (continuous, < mm in thickness). Value of J_a depends on percentage of swelling clay-sized particles, and access to water, etc.	8.0–12.0	6–12°
(c) No rock wall contact when sheared		
K. Zones or bands of disintegrated or crushed rock and clay (see G., H., J. for description of clay condition)	6.0, 8.0 or 8.0–12.0	6–24°
L. Zones or bands of silty or sandy clay, small clay fraction (nonsoftening)	5.0	
M. Thick, continuous zones or bands of clay (see G., H., J. for description of clay condition)	10.0, 13.0 or 13.0–20.0	6–24°

Note:

(i) Values of ϕ_r are intended as an approximate guide to the mineralogical properties of the alteration products, if present

TABLE B.1 *(continued)*

Stress Reduction Factor (SRF)

(a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated				*Note:* (i) Reduce these SRF values by 25–50% if the relevant shear zones only influence but do not intersect the excavation
Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)			10.0	
B. Single-weakness zones containing clay or chemically disintegrated rock (depth of excavation ≤50 m)			5.0	
C. Single-weakness zones containing clay or chemically disintegrated rock (depth of excavation >50 m)			2.5	
D. Multiple-shear zones in competent rock (clay-free), loose surrounding rock (any depth)			7.5	
E. Single-shear zones in competent rock (clay-free) (depth of excavation ≤50 m)			5.0	
F. Single-shear zones in competent rock (clay-free) (depth of excavation >50 m)			2.5	
G. Loose open joints, heavily jointed or "sugar cube," etc. (any depth)			5.0	
(b) Competent rock, rock stress problems				
	σ_c/σ_1	σ_t/σ_1		
H. Low stress, near surface	>200	>13	2.5	(ii) For strongly anisotropic stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c and σ_t to $0.8\ \sigma_c$ and $0.8\ \sigma_t$; when $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to
J. Medium stress	200–10	13–0.66	1.0	

TABLE B.1 *(continued)*

				Stress Reduction Factor (SRF)
K. High-stress, very tight structure (usually favorable to stability, may be unfavorable to wall stability	10–5	0.66–0.33	0.5–2.0	0.6 σ_c and 0.6 σ_t (where σ_c = unconfined compressive strength, σ_t = tensile strength (point load), σ_1 and σ_3 = major and minor principal stresses)
L. Mild rock burst (massive rock)	5–2.5	0.33–0.16	5–10	
M. Heavy rock burst (massive rock)	<2.5	<0.16	10–20	
(c) Squeezing rock; plastic flow of incompetent rock under the influence of high rock pressures				
N. Mild squeezing rock pressure			5–10	
O. Heavy squeezing rock pressure			10–20	(iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H)
(d) Swelling rock; chemical swelling activity depending on presence of water				
P. Mild swelling rock pressure			5–10	
R. Heavy swelling rock pressure			10–15	

TABLE B.1 *(continued)*

Joint Water Reduction Factor J_w

	J_w	Approximate water pressure (kg/cm^2)	
A. Dry excavations or minor inflow, i.e.,			*Note:*
B. 5 L/min locally	1.0	<1	(i) Factors C–F are crude estimates. Increase J_w if drainage measures are installed
Medium inflow or pressure occasional outwash of joint fillings	0.66	1.0–2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5–10.0	(ii) Special problems caused by ice formation are not considered
D. Large inflow or high pressure, considerable outwash of joint fillings	0.33	2.5–10.0	
E. Exceptionally high inflow or water pressure at blasting, decaying with time	0.2–0.1	>10.0	
F. Exceptionally high inflow or water pressure continuing without noticeable decay	0.1–0.05	>10.0	

[a] After Barton et al. (1974).
[b] Nominal.

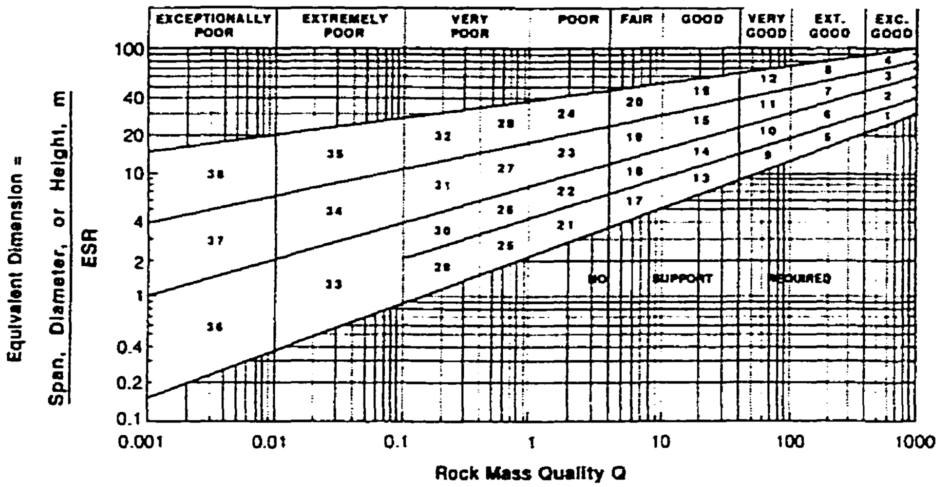

Figure B-1: *Q-system→equivalent dimension versus rock mass quality, after Barton et al., 1974)*

Appendix C: *Slake Durability Tests of Rock Samples*

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Slake-Durability Test **Sample No: 1A**

Sample was tested according to ASTM D 4644 - 87 Standard.

Initial weight	540.45gms	(photo page 1)
After 1st cycle	281.78gms	Water temp. 20deg. C
After 2nd cycle	179.19gms	Water temp. 20deg. C

Slake-Durability Index

After 1st cycle	52.1%
After 2nd cycle	33.2%

1.Water

After 1st cycle
Water color light grey, clarity 0cm, abundant suspended particles, sediment mostly very fine and fine with some particles 1 - 2mm diameter.

After 2nd cycle
Water color light to medium grey, clarity .5cm, sediment mostly very fine and fine with few particles over 1mm in size.

2.Solid material

After 1st cycle
Round and subrounded pieces size from 2mm to 30mm, larger fragments light grey, smaller fragments medium to dark grey (photo. page 2).

After 2nd cycle
Round and subrounded pieces size from 2mm to 30mm, larger fragments light grey, smaller fragments medium to dark grey (photo. page 3).

Figure C-1: *Sample number 1A*

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Slake-Durability Test **Sample No: 1B**

Sample was tested according to ASTM D 4644 - 87 Standard.

Initial weight	539.80gms	(photo page 1)
After 1st cycle	311.68gms	Water temp. 20deg. C
After 2nd cycle	214.07gms	Water temp. 20deg. C

Slake-Durability Index

After 1st cycle	57.7%
After 2nd cycle	39.7%

1.Water

After 1st cycle
Water color light grey, clarity 0cm, abundant suspended particles, sediment mostly very fine and fine with some particles 1 - 2mm diameter.

After 2nd cycle
Water color light to medium grey, clarity .5cm, sediment mostly very fine and fine with few particles over 1mm in size.

2.Solid material

After 1st cycle
Round and subrounded pieces size from 2mm to 30mm, larger fragments light grey, smaller fragments medium to dark grey (photo. page 2).

After 2nd cycle
Round and subrounded pieces size from 2mm to 30mm, larger fragments light grey, smaller fragments medium to dark grey (photo. page 3).

Figure C-2: *Sample number 1B*

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Slake-Durability Test **Sample No: 1C**

Sample was not tested according to ASTM D 4644 - 87 Standard because of insufficient specimen size. Data are only approximate.

Initial weight	394.50gms	(photo page 1)
After 1st cycle	157.41gms	Water temp. 20deg. C
After 2nd cycle	98.20gms	Water temp. 19deg. C

Slake-Durability Index

After 1st cycle	39.9%
After 2nd cycle	24.9%

1.Water

After 1st cycle
Water color light grey, clarity 0cm, abundant suspended particles, sediment mostly very fine and fine with some particles 1 - 2mm diameter.

After 2nd cycle
Water color light to medium grey, clarity .5cm, sediment mostly very fine and fine with few particles over 1mm in size.

2.Solid material

After 1st cycle
Round and subrounded pieces size from 2mm to 30mm, larger fragments light grey, smaller fragments medium to dark grey (photo. page 2).

After 2nd cycle
Round and subrounded pieces size from 2mm to 30mm, larger fragments light grey, smaller fragments medium to dark grey (photo. page 3).

Figure C-3: *Sample number 1C*

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Slake-Durability Test **Sample No: 2**

Sample was not tested according to ASTM D 4644 - 87 Standard because the rock fragments were between 35gms and 65gms in size. Data are only approximate.

Initial weight	463.04gms	(photo page 1)
After 1st cycle	387.25gms	Water temp. 20deg. C
After 2nd cycle	363.51gms	Water temp. 20deg. C

Slake-Durability Index

After 1st cycle	83.6%
After 2nd cycle	78.5%

1.Water

After 1st cycle
Water color medium grey, clarity 1cm, abundant suspended particles, sediment mostly very fine and fine with some particles 1 - 2mm diameter.

After 2nd cycle
Water color light to medium grey, clarity 1cm, sediment mostly very fine and fine with few particles over 1mm in size.

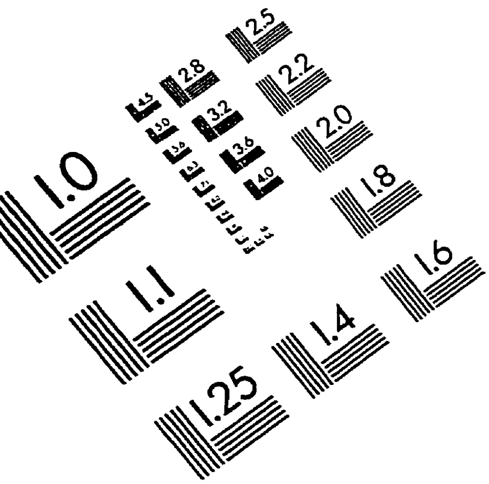
2.Solid material

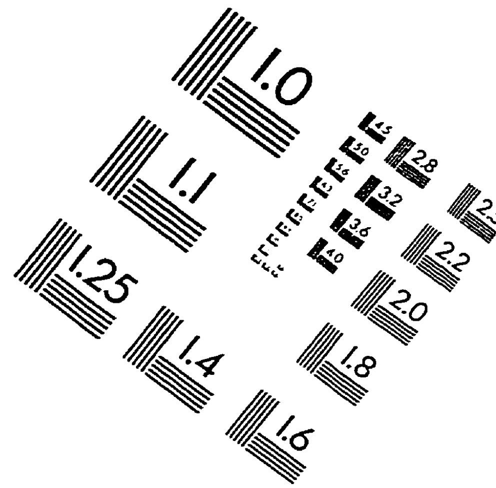
After 1st cycle
Fragments between 2mm and 30mm in size with sharp edges, with dominant one or two dimensions (photo. page 2).

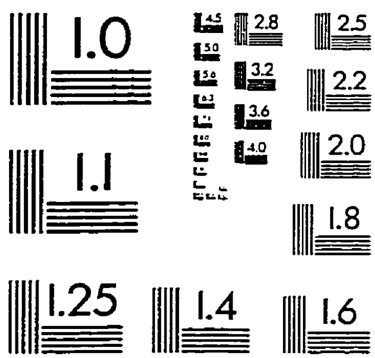
After 2nd cycle
Fragments between 2mm to 30mm in size with sharp edges, with dominant one or two dimensions (photo. page 3).

Figure C-4: *Sample number 2*

IMAGE EVALUATION TEST TARGET (QA-3)







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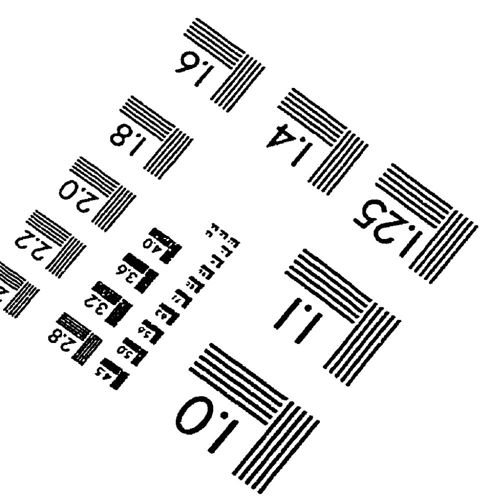

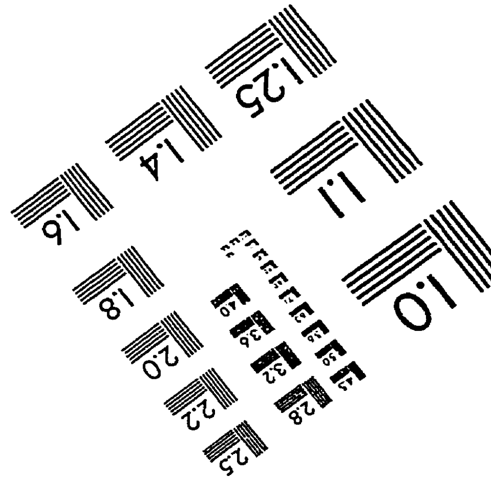

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