2.11 Overburden Management

This section addresses the management of overburden material, which includes on-site clays and peat/muskeg. The management of dolomitic overburden will be presented in the Waste Rock Disposal Section (Section 2.12).

Overburden will be managed in several ways. The vast majority of peat and clay overburden that needs to be removed to gain access to the ore reserves and to built infrastructure will be stored in an Overburden Disposal Facility (ODF). Low permeability clays will be salvaged and stockpiled in sufficient quantities to enable the construction of low permeability liners where required. For example, a low permeability liner will be installed on the upstream side of the Tailings and Ultramafic Waste Rock Management Facility (TWRMF).

Dredging was selected as an overburden management option for the Minago Open Pit, because of logistical challenges, tight scheduling issues, and capital and operational costs related to safe disposal of mechanically excavated overburden (Wardrop, 2009b). Dredged material will be deposited in the ODF. Victory Nickel is also considering using mechanical equipment to remove the overburden material from the pit area. The mechanical removal option of the overburden will be undertaken during the winter months.

The ODF capacity will be approximately 15 Mm^3. The ODF will be capable of retaining a total of 11.2 Mt (~ 13.4 Mm^3) of overburden that will be discharged into the facility during an 8 months dredging period, scheduled to run from April to November “2011” (Year -3). A further 1.6 Mm^3 of swelled peat and soft clay will be added in “2012” (Year -2). This material will originate from the downstream side of the dam foundation of the TWRMF and from runoff and seepage collection ditches.

The ODF will be located immediately south and east of the open pit as shown in Figure 2.1-2.

2.11.1 Overburden Disposal Facility (ODF) Design Criteria and Design Basis

The in situ material quantities that were used as the design basis for the ODF are detailed in Table 2.11-1. The design basis for the ODF assumes that the overburden materials will be comprised of 50% solids and 50% water by weight. The change in solids mass from 70% prior to the dredging to 50% at the point of disposal will be a result of the mixing and pumping of the slurry. After deposition, a certain portion of the initial water content will be released to bring the longer term ratio to 65% solids and 35% water (Wardrop, 2009b). The estimated total mass and volume of solids and water upon deposition in the ODF are presented in Table 2.11-2.

The engineering design criteria used for the development of the ODF are presented in Table 2.11-3.

Table 2.11-1 In-situ Overburden Material Quantities

Item	Value
Effective Unit Weight	1.86 t/m³
Effective Moisture Content	52 %
Total Overburden Weight	11,200,000 t
Total Overburden Volume	6,021,000 m³
Effective Solids Content (By Weight)	66%
Effective Water Content (By Weight)	34%

Source: Wardrop, 2009b

Table 2.11-2 Design Basis Criteria for the ODF

Item	Value
In situ Solids Weight	
In situ Water Weight	
Solids Weight	7,347,000 t
Water Weight (at 50% water to 50% solids by weight)	7,347,000 t
Total Weight	14,694,000 t
Solids Volume	6,022,000 m³
Water Volume	7,347,000 m³
Total Volume	13,369,000 m³

Source: adapted from Wardrop, 2009b

Table 2.11-3 Basic Engineering Design Parameters for ODF

Item	Target	Comments
1. Geotechnical Slope Stability		
• Construction (in stages)	• Static F.O.S. 1.3, pseudo static F.O.S 1.05.	
• Normal Operating	• Same as above.	
• Closure	• Static F.O.S. 1.3, pseudo static F.O.S 1.05.	
2. Seismicity		
• Operating Design Basis Earthquake	• 1: 475 year return	
• Seismicity induced by pit blasting	•	• Input will be required for the detailed design.
• Closure Earthquake	1:2,475 year return	

Source: Wardrop, 2009b

2.11.2 ODF Design

The layout of the ODF is shown in Figures 2.11-1 and 2.11-2. The ODF will be surrounded by a perimeter dyke that will be approximately 4.5 m above the local topography and the dyke crest will be 12 m wide to accommodate construction traffic and facilitate feeder and discharge pipes (Wardrop, 2009b). Peat will be left in place in the dyke foundation.

The discharge of dredged peat and clay slurry will be through a number of discharge pipes spaced out along the ODF dyke crest. Carriage water that was used to transport the solids will be released from the ODF through a series of stop log weirs constructed in the perimeter dyke at the central apex of the ODF. The weirs will pass the water into a triangular collection pond contained by another dyke. The collected carriage water will then be reused for dredging operations. In addition, a 0.3 m perforated HDPE or ADS pipe will be installed in the ODF apex to enhance carriage water collection efficiency during and post the dredging operations (Wardrop, 2009b).

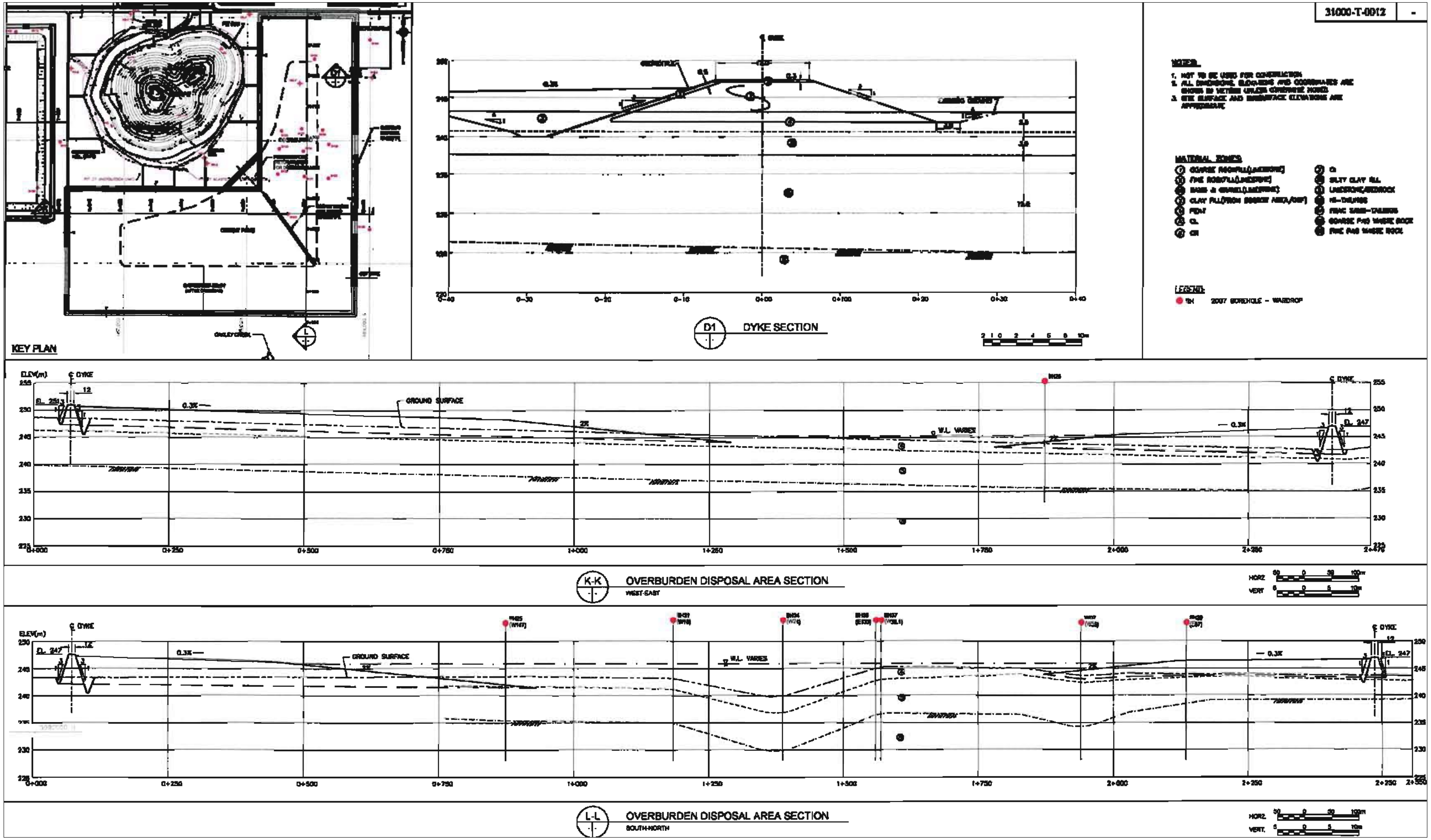
2.11.2.1 Dredging Operations

The peat and clay soils will be removed using a hydraulic dredging process utilizing a boom mounted rotating cutter attached to barge. The boom will have sufficient length and flexibility to cut the overburden material to vertically and horizontally control the cutter to accurately remove the overburden materials to the desired plan and profile (Wardrop, 2009b).

The selection of the cutter head size and number of dredge units will be identified in the detailed engineering design with input from dredging contactors. Preliminary discussions with a dredging contractor suggest that two 1 m diameter cutter units may be required for the Minago Project. Water will be added at the cutter head to facilitate the conveyance of the solids to the ODF. The water and solids slurry will be pumped through a pipeline system by booster pumps to the ODF and discharged within the operating cell of the ODF (Wardrop, 2009b).

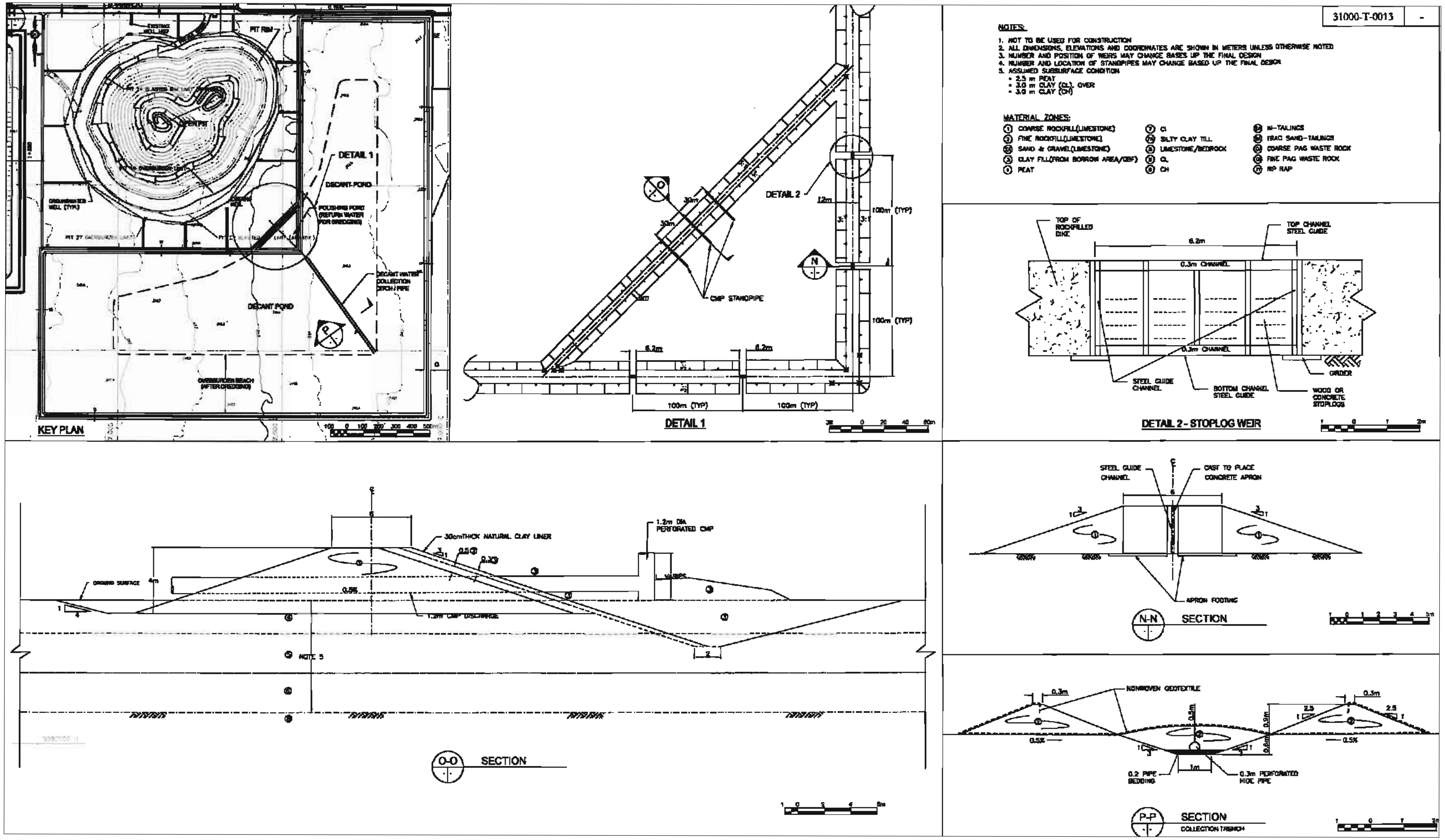
During the dredging operation, the slurry is expected to be comprised of 20% solids and 80% water by weight. For the planned 8-month dredging period, the estimated dredging production will be approximately 25,000 m^3/day (46,500 tonnes/day) of *in situ* overburden (Wardrop, 2009b).

The disposal strategy will involve perimeter discharge of a peat and clay slurry starting along the western side of the southern leg of the ODF and continuing in parallel along the northern and southern sides (Figure 2.11-1). The same strategy will apply to the eastern ODF leg where the deposition will start at the northern side and will continue along the western and easterm sides. The dredged material is expected to form a beach at a 0.3 % slope and a 2 % subaqueous slope (Wardrop, 2009b). The beach will divert decant water towards the pervious dyke section. Decant water from the dredging operations will be collected in the decant water collection pond, shown in Figure 2.11-2.



Source: adapted from Wardrop's drawing 0951330400-T0012 (Wardrop, 2009b)

Figure 2.11-1 Overburden Management Facility Plan and Sections



Source: adapted from Wardrop's drawing 0951330400-T0013 (Wardrop, 2009b)

Figure 2.11-2 ODF Water Management Structure Plan and Sections

The outboard pond dyke will be constructed out of coarse limestone rock fill that will be 4 m high, a 0.5 m of fine limestone rock fill on the upstream side, and a 0.3 m thick inboard clay liner to increase the dyke's water holding capacity.

To effectively manage water release and to support continued dredging operations, a total of three 1.3 m in diameter corrugated metal pipe (CMP) will run through the dyke (laterals) and these will be connected to perforated standpipes installed within the pond (Wardrop, 2009b). Collected water will be returned to the dredging operations for continued dredge water demand. It is estimated that over the eight months dredging period, approximately 7.4 Mm^3 of make-up water will be required. To support the dredging operations and assuming a 15 percent water loss, the estimated make-up water demand will be approximately 35,000 m^3/day (Wardrop, 2009b).

Water pumped from the pit dewatering wells will be used for the dredging operations. The cone of depression created by the groundwater dewatering wells will provide under drainage for the overburden clays. This will be considered in geotechnical analyses for major site earth/rock fill structures.

The water level in the dredging pit will be drawn down at the end of the dredging period to assist in the de-watering of the dolomite (Wardrop, 2009b).

On closure, the ODF will be reshaped and revegetated and overflow will be directed to the ditch near Highway #6 that reports to Oakley Creek (Wardrop, 2009b).

2.11.3 ODF Dyke

Plan and section views of the ODF dyke are shown in Figure 2.11-1 and Figure 2.11-2 illustrates a plan view, a dyke design section, a stop log structure section, and details for the ODF Polishing Pond.

The ODF dyke will be constructed out of coarse rockfill (Zone 1 material) that will be comprised of 800 mm minus dolomite waste rock originating from the limestone outcrop located approximately 3 km northwest of the facility (Figure 2.1-2). The upstream side of this zone will support a 0.5 m thick zone of fine rockfill (Zone 2 material) comprised of minus 75 mm dolomite waste rock. A geotextile layer will be placed on the upstream side over the top of Zone 2. The dyke crest will be 12 m wide and both upstream and downstream slopes will be 3H:1V (Wardrop, 2009b).

The ODF Polishing Pond dyke will also be constructed out of coarse rockfill (Zone 1 material) and a 0.5 m thick fine rockfill (Zone 2 material) on the upstream side. Both upstream and downstream slopes will be 3H:1V. A 0.5 m clay liner will be provided on the upstream side of Zone 2. A total of three DMP pipes, 1.3 m in diameter and sloped at 0.5%, will be installed within the dyke. These pipes will have vertical perforated intakes immediately upstream of the dyke (Wardrop, 2009b).

2.11.3.1 ODF Dyke Stability and Seepage Analyses

Seepage and slope stability analyses were performed on the ODF dyke Sections D1 and D2. Section D1 assumes that there will be 2.5 m of peat, over 3.0 m upper clay (CL), on top of 12.5 m of lower clay (CH) (Figure 2.11-1). Section D2 assumes that there will be 2.5 m of peat, over 3.0 m upper clay (CL), on top of 3.0 m of lower clay (CH) (Wardrop, 2009b).

Coupled analyses using Sigma/W and Slope/W, components of GeoStudio 2007, were used in the Seepage and slope stability analyses. Sigma/W uses finite element methods to solve both stress-deformation and seepage dissipation equations simultaneously. Pore water pressures generated during lift placement were calculated with Sigma/W and then incorporated into Slope/W for stability analysis. Slope/W was used to locate failures with the least factor of safety within defined search limits (Wardrop, 2009b).

Sections D1 and D2 were modeled assuming that the embankment was placed in a single lift on the first day, and then 20 days were allowed for consolidation. Slope stability analyses were conducted by assuming that 4 days had passed after the embankment had been placed and at the end of 20 days (Wardrop, 2009b).

Slope stability analyses were performed on the upstream and downstream sides of the ODF dyke. Another analysis was performed 30 days after the completion of the facility, assuming that the disposed peat and clay material were placed at once on the upstream side. After that, a seepage analysis was performed under steady state conditions to calculate the seepage through the ODF dyke (Wardrop, 2009b).

A pseudo static analysis was also performed to simulate earthquake conditions using an acceleration of 0.03 g (Wardrop, 2009b).

Material Properties

Assumed foundation material properties (CL, CH and bedrock) were based on field and laboratory data. Assumed properties for peat, coarse and fine rockfill, and dredged peat and clay were based on previous experience and professional judgement. Table 2.11-4 and Table 2.11-5 show material properties used in Sigma/W, Seep/W and Slope/W for the ODF dyke.

2.11.3.1.1 ODF Dyke Stability Results

Table 2.11-6 presents slope stability results assuming that 4 days and 20 days had passed after the placement of the facility, and 10 days after the ODF was filled with dredged peat and clay material. The slope stability results show that the ODF satisfies the minimum requirements for static and pseudo static conditions. Detailed slope stability results are given elsewhere (Wardrop, 2009b).

Table 2.11-4 Sigma/W Input Material Properties

Materials	Material Category	Material Model	Poisson's Ratio	Young's Modulus (kPa)	Hydraulic Conductivity (cm/s)*
Disposed Peat and Clay	Effective Parameters w/PWP Change	Linear Elastic	0.33	2,000	8.64E-03
Coarse Rockfill	Effective Parameters w/PWP Change	Linear Elastic	0.33	50,000	8.64E-01
Fine Rockfill	Effective Parameters w/PWP Change	Linear Elastic	0.33	7,000	8.64E-03
Sand and Gravel	Effective Drained Parameters	Linear Elastic	0.35	8,000	
Peat	Effective Parameters w/PWP Change	Linear Elastic	0.35	2,000	1.00E-01
Soft Clay (CL)	Effective Parameters w/PWP Change	Soft Clay (MCC)	0.36		1.36E-08
Soft Clay (CH)	Effective Parameters w/PWP Change	Soft Clay (MCC)	0.37		4.97E-09
Bedrock	Effective Parameters w/PWP Change	Linear Elastic	0.49	100,000	6.89E-04

Source: Wardrop, 2009b

Note: *Used in Seep/W.
w/PWP Change with porewater change

Table 2.11-5 Slope/W Input Material Properties

Materials	Model	Unit Weight (kN/m^3)	Cohesion (kPa)	Phi (º)
Disposed Peat and Clay	Mohr-Coulomb	16	18	0
Coarse Rockfill	Mohr-Coulomb	19	0	40
Fine Rockfill	Mohr-Coulomb	22	0	38
Sand and Gravel	Mohr-Coulomb	22	0	35
Peat	Mohr-Coulomb	13	18	0
Soft Clay (CL)	Mohr-Coulomb	21	20	29
Soft Clay (CH)	Mohr-Coulomb	18	10	25
Bedrock	Bedrock (Impenetrable)			

Source: Wardrop, 2009b

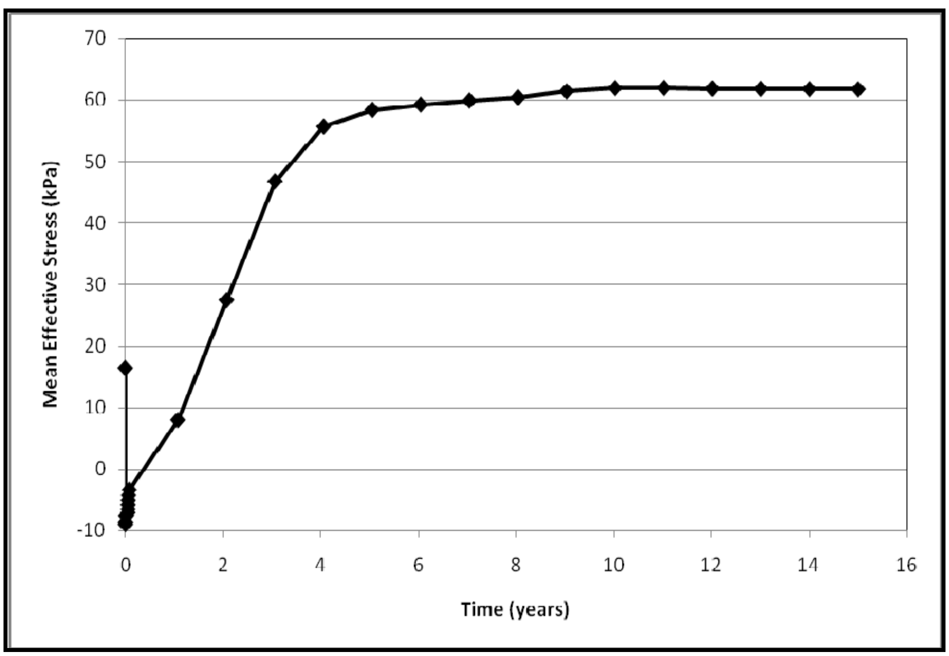
Table 2.11-6 Slope Stability Results for the ODF Dyke

Section	Elapsed Time (days)	Upstream		Downstream	
		Static F.O.S	Pseudo static F.O.S	Static F.O.S.	Pseudo static F.O.S.
D1	4	1.3/1.32		1.30/1.23	
	20	1.3/1.47	1.05/1.32	1.30/1.45	1.05/1.32
	30*			1.30/1.51	1.05/1.36
D2	4	1.30		1.32	
	20	1.39	1.25	1.36	1.25
	30*			1.48	1.34

Source: Wardrop, 2009b

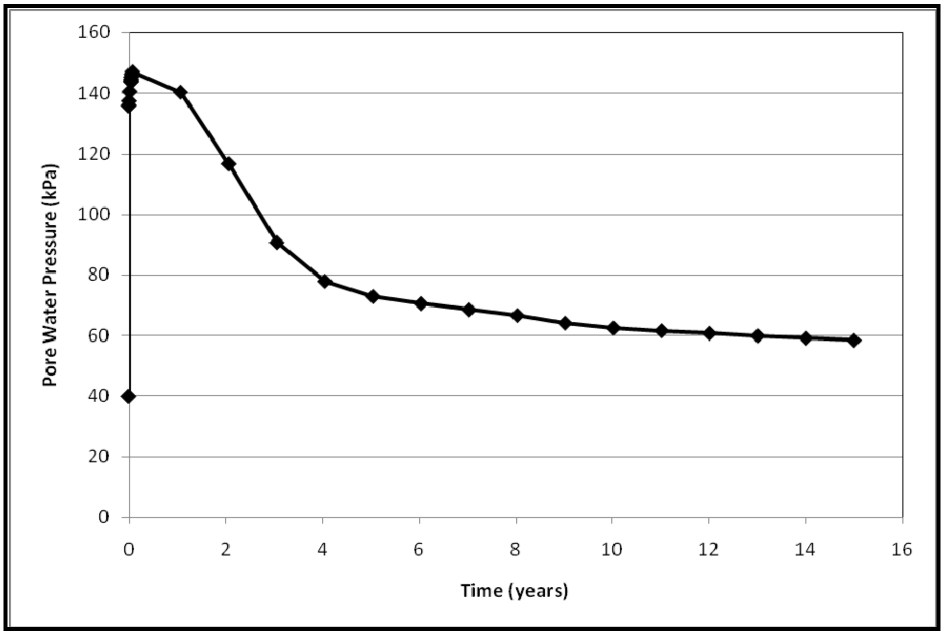
*Assumed disposed peat and clay material was placed on the upstream side of the embankment.

Figures 2.11-3 and 2.11-4 show modelling results for effective stress versus time, and pore water pressure versus time predicted for the foundation soils below the centerline of the dam. Figure 2.11-3 illustrates how the effective stress increases after placing the embankment, and then stabilizes over time. Figure 2.11-4 shows the pore water pressure increase upon the dyke construction and its dissipation over time. Based on these computations, full pore water pressure dissipation will occur in approximately 15 years.



Source: Wardrop, 2009b

Figure 2.11-3 Mean Effective Stress versus Time for the ODF Dyke



Source: Wardrop, 2009b

Figure 2.11-4 Pore Water Pressure versus Time for the ODF Dyke

2.11.3.1.2 ODF Seepage Results

Seepage through the embankment was estimated using Seep/W for a one meter wide slice or rockfill material against the upstream perimeter of the dam. The computed seepage quantities for sections D1 and D2 were in the order of 50 m^3/day. The initial seepage rate is expected to be much higher until a seal is created by the discharged peat and clay (Wardrop, 2009b).

2.11.4 Construction Considerations

2.11.4.1 Peat Overburden

The in-situ peat is unsuitable for construction purposes, but it may have potential for use in site reclamation. If pre-loaded, the peat may be used as foundation material for structures that are not sensitive to settlements, such as waste rock dumps (Wardrop, 2009b). Pre-loading tests on the peat were not carried out for determination of consolidation characteristics. These tests will be conducted during the detailed engineering design phase.

2.11.4.2 Clay Overburden

The construction of water containment structures and dykes across the site will require low permeability materials. Site clays were assessed during the pre-feasibility and feasibility geotechnical investigations and the results of laboratory tests on selected clay samples may be summarized as follows (Wardrop, 2009b):

- The optimum moisture content ranged from 16.3% to 18.6% at standard Proctor maximum dry densities (SPMDD) ranging between 1,600 and 1,752 kg/m^3.
- Clay with natural moisture contents reasonably close to the optimum for compaction may be found within the uppermost 5 m of the deposit. The moisture content of the tested clays was typically well above the optimum at depths greater than 5 m. The natural moisture content of tested clay was generally higher than 20% (Figure 7.3-7).
- It was found that site areas with shallow thickness of overburden contained stiff clays that exhibited natural moisture contents close to the optimum for compaction.
- Recovery of clays from perennially flooded terrain will pose formidable logistical challenges as the muskeg/peat is water logged. More specifically, these areas will require that the muskeg/peat are bermed off so that the upper stiff clay may be excavated in a “dry” condition. Also, clays may experience moisture uptake during excavation even if the borrow areas are bermed off (Wardrop, 2009b).

2.11.5 Overburden Removal using Mechanical Equipment

Victory Nickel is evaluating alternative options to hydraulic methods as the removal of the material using conventional methods (excavator, load, haul) are generally feasible during the winter months. There will be additional impacts should VNI decide to use mechanical methods.

2.12 Waste Rock Disposal

During the operation of the open pit, a total of 268.695 Mt of waste rock will be mined out of which 111.03 Mt will be limestone and 157.67 Mt will be basement rock. Basement rock will consist of two types: 122.01 Mt of granite (non-acid generating) and 35.66 Mt of ultramafic (potentially acid-generating and selenium containing). A summary of projected material quantities that will be mined from the Open Pit until closure is given in Table 2.9-6 and the yearly waste rock placement schedule is detailed in Table 2.12-1.

Waste rock will be deposited in three areas (Figure 2.1-2). Dolomitic waste rock will be deposited in the 191 ha Dolomite Waste Rock Dump, granitic waste rock will be deposited in the 301.4 ha Country Rock Waste Rock Dump, and ultramafic waste rock will be co-disposed with the tailings in the 219.7 ha Tailings and Ultramafic Waste Rock Management Facility (TWRMF). All of the waste rock disposal areas will be located close to the open pit to minimize haulage costs and to optimize utilization of the site.

Limestone will be used in the construction of roads, containment berms, the basement layer for the ultramafic waste rock and causeways inside the Tailings and Ultramafic Waste Rock Management Facility (TWRMF), and for the site preparation of a Crusher Pad and a Ore Stockpile Pad; excess limestone will be deposited in the Dolomite Waste Rock Dump (Dolomite WRD).

2.12.1 Design Criteria and Considerations for the Waste Rock Dumps

The key design objective is to construct non-reactive waste rock dumps in the proximity of the open pit within compact footprints to the maximum heights governed by geotechnical analyses to minimize operational costs. As the dolomitic and Country Rock waste rock is inert, no special environmental protection measures are necessary (Wardrop, 2009b).

Tables 2.12-2 and 2.12-3 summarize the basic design criteria and parameters adopted for the waste rock dumps.

2.12.2 Waste Rock Dump Designs

The design of the waste rock dumps focusses on minimizing dump footprints and maximizing their heights through staged construction and in accordance with the results of engineering analyses and the waste production schedule. With both dumps containing non-acid generating (NAG) waste rock, there will not be a need for a seepage collection system and the storm water can report directly to the natural environment.

The locations of Country Rock Waste Rock Dump (CRWRD) and Dolomite Waste Rock Dump (DWRD) were selected to be on muskeg/peat covered weak overburden clay characterized by average thicknesses of 15 m and 10 m, respectively.

Table 2.12-1 Yearly Waste Rock Placement Schedule

Product		2012	2013	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023	TOTAL kt
		Year												
Dolomite (Limestone)	kt	42,655	43,179	15,183	10,015	0	0	0	0	0	0	0	0	**111,032**
Granite	kt	0	1,744	20,890	20,440	35,711	24,459	9,784	4,944	3,832	199	0	0	**122,005**
Ultramafic	kt	0	861	7,941	5,524	5,667	5,732	4,382	3,026	2,297	229	0	0	**35,659**
TOTAL	**kt**	**42,655**	**45,784**	**44,014**	**35,979**	**41,379**	**30,192**	**14,166**	**7,970**	**6,128**	**428**	**0**	**0**	**268,695**

Source: adapted from Wardrop, 2009b

Table 2.12-2 Design Basis for Rock Dumps

Item	Value
Life of the Open Pit mine	10 years
Total Waste Rock	268,696,000 t
Total Dolomite Waste Rock	111,032,000 t
Total Country Rock Waste Rock	122,005,000 t
Country Rock Waste Rock Specific Gravity	2.07 t/m³
Dolomite Waste Rock Specific Gravity	2.79 t/m³
Swelling	30%
Total Required Volume for Country Rock Waste Rock Dump	~ 59,000,000 m³
Total Required Dolomite for Construction of Mine Infrastructure (TWRMF, roads, dykes, etc.)	10,743,600 m³
Total Required Volume for Dolomite Waste Rock Dump	41,000,000m³

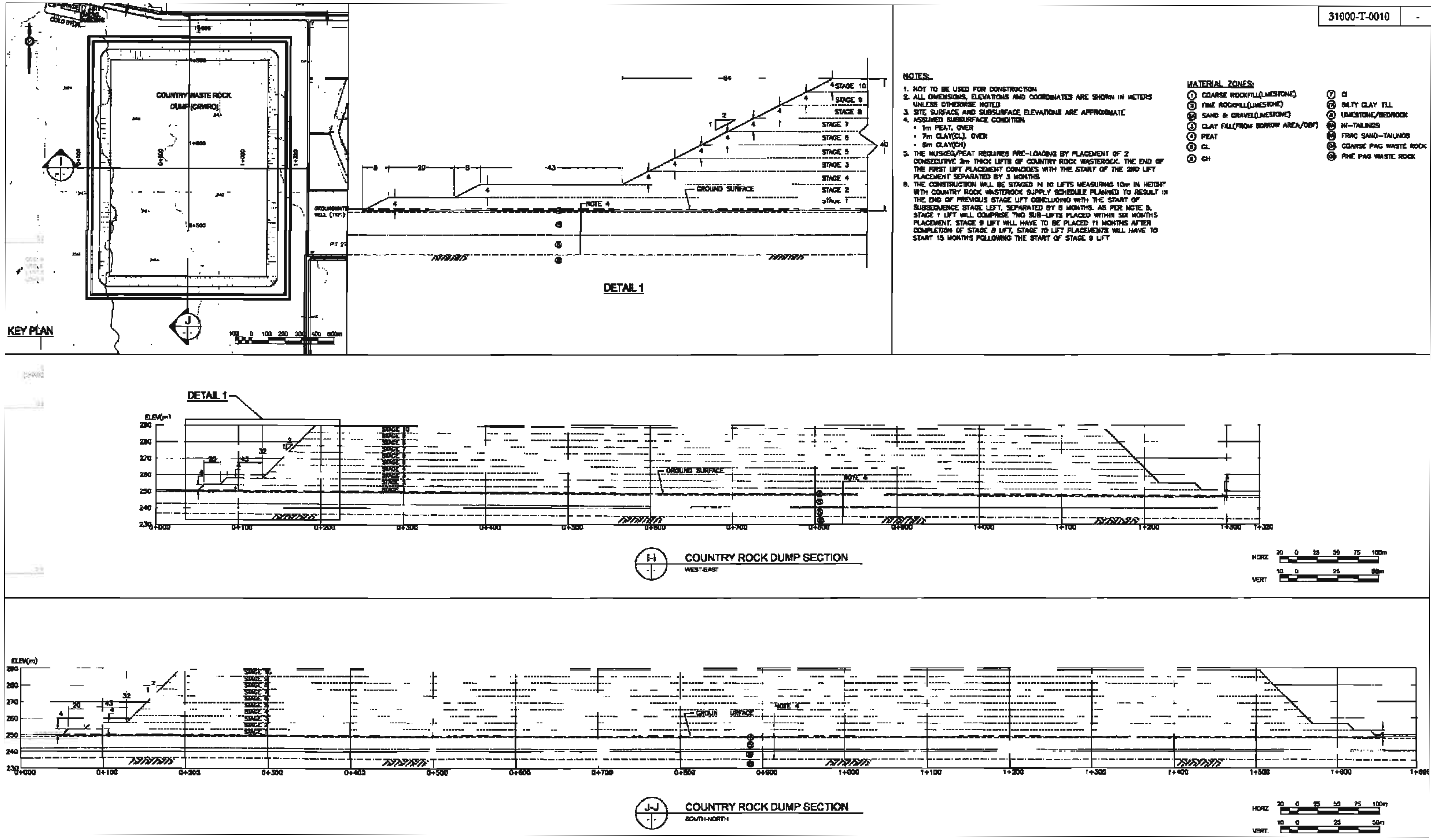
Source: Wardrop, 2009b

Table 2.12-3 Basic Engineering Design Parameters for Rock Dumps

Item	Target
1. Geotechnical Slope Stability: • Waste Dump	
• Construction (in stages)	• Static F.O.S 1.3, pseudo static F.O.S 1.05
• Normal Operation	• Same as above
• Closure	• Static F.O.S. 1.3, pseudo static F.O.S 1.05
2. Seismicity: • Operating Design Basis Earthquake	• 1: 475 year return
• Closure Earthquake	• 1: 2,475 year return
3. Max Dump Height	• Dependent on the results of engineering analyses in support of staged construction.

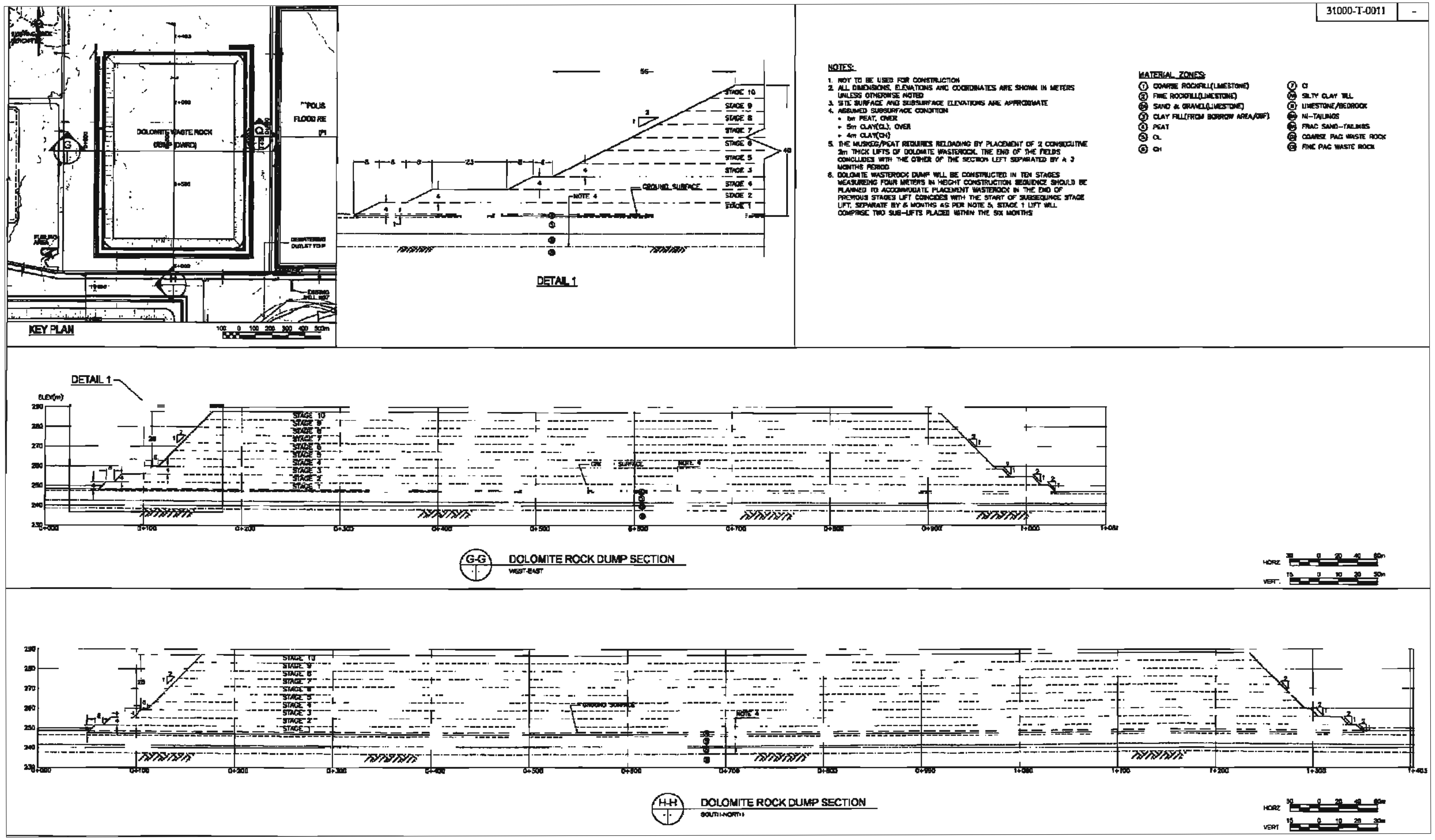
Source: Wardrop, 2009b

Plan and sectional details of the waste rock dumps are shown in Figures 2.12-1 and 2.12-2.



Source: adapted from Wardrop's drawing 0951330400-T0010 (Wardrop, 2009b)

Figure 2.12-1 Country Rock Waste Rock Dump Plan and Sections



Source: adapted from Wardrop's drawing 0951330400-T0011 (Wardrop, 2009b)

Figure 2.12-2 Dolomite Waste Rock Dump (DWRD) Plan and Sections

2.12.2.1 Country Rock Waste Rock Dump (CRWRD)

The Country Rock Waste Rock Dump (CRWRD) is designed for storing 59 Mm^3 of inert granitic waste rock. The dump will be founded on existing overburden comprised of muskeg/peat and clay averaging approximately 15 m in thickness. This dump will measure 1,596 m by 1,240 m in plan and will be staged in ten (10) lifts of 4 m for an ultimate dump height of 40 m. The dump configuration includes a 20 m and a 43 m setback for the toes of the Stage 2 and Stage 3 lifts with subsequent lifts set-back to give a 2H:1V slope (Wardrop, 2009b).

To allow for sufficient time for consolidation of the soft clay layer, successive lifts of this waste rock dumps will be sequenced with sufficient time for consolidation. Assuming 4 m lifts and a repetitive placement operation, any subsequent lift may only be started after the current lift has been in place for sufficient time for consolidation to be effective. Stages 2 to 8 may be sequenced 6 months after the previous stage, Stage 9, 11 months after that and Stage 10 after 15 months.

Construction of the Country Rock WRD will commence with the grubbing of all trees.

2.12.2.2 Dolomite Waste Rock Dump (DWRD)

The Dolomitic Waste Rock Dump is designed for storing 41 Mm^3 of inert dolomite rock. This dump will be founded on existing overburden comprised of muskeg/peat and clay averaging approximately 10 m in thickness. The dump will measure 1,303 m by 974 m in plan and will be staged in ten (10) lifts for a maximum height of 40 m. The dump configuration will be formed with overall slopes of 2H:1V and setbacks of 8 m, 23 m and 6 m for the toes of Stage 2, Stage 3 and Stage 4 lifts, respectively (Wardrop, 2009b).
Successive lifts of this dump will be sequenced with a set period of time (as will be done for the Country Rock WRD) to allow for sufficient time for consolidation of the soft clay layer underlying the dump. Assuming 4 m lifts and a repetitive placement operation, all subsequent lifts may only be started after a consolidation period of 6 months (Wardrop, 2009b).

Construction of the Dolomite WRD will commence with the grubbing of all trees.

2.12.2.3 Stability Analyses for the Waste Rock Dumps

Stability and settlement analyses were carried out in support of developing dump design sections that satisfy the design criteria (Table 2.12-2). Coupled analyses using Sigma/W and Slope/W, components of GeoStudio 2007, were used in the dam stability and settlement analyses. Sigma/W uses finite element methods to solve both stress-deformation and seepage dissipation equations simultaneously. Pore water pressures generated during lift placement were calculated with Sigma/W and then incorporated into Slope/W for stability analysis. Slope/W was used to locate failures with the least factor of safety within defined search limits (Wardrop, 2009b).

The Country Rock WRD and Dolomite WRD were modelled as underlain by 15 and 10 m of overburden, respectively. In the modelling, the overburden was divided into peat, and, upper (CI) and lower (CH) clay horizons. Both clay horizons were modeled using the non-linear Modified Cam-Clay (MCC) constitutive relationship (Wardrop, 2009b).

Initial pore pressure conditions were defined with an initial water table at the ground surface in the peat material. Zero pressure boundary conditions were applied to the bottom of the bedrock to model dewatering wells pumping water out of the bedrock layer. The duration between placement of each lift was assumed to be 6 months (Wardrop, 2009b). However, the Stage 9 and Stage 10 lifts of the Dolomite WRD were assumed to have a longer time interval between the placement of successive lifts. The time interval was assumed to be 11 and 15 months for the Stage 9 and the Stage 10 lifts, respectively. In the modelling for lifts 1 through 8, each lift was assumed to be placed on the first day, and then 182 days were allowed for consolidation prior to the placement of the next lift.

The stability analyses are representative of conditions immediately after placement of each lift (Wardrop, 2009b).

Pseudo static analysis was performed to simulate an earthquake condition of 0.03 g (Wardrop, 2009b).

Material Properties

Material properties for soft clays (CL and CH) and bedrock properties were based on laboratory data; whereas peat and waste rock material properties were based on professional judgement and previous experience (Wardrop, 2009b). Table 2.12-4 and Table 2.12-5 present the material properties used for the waste rock dump stability analyses in Sigma/W and Slope/W models, respectively.

2.12.2.3.1 Results of Stability Analyses for the Waste Rock Dumps

Table 2.12-6 presents results of the stability analyses. These results satisfy the minimum factor of safety requirements for static and pseudo static conditions, except for the short times following completion of some lifts in the Country Rock WRD, shown bolded numbers in Table 2.12-6. For these cases, the lower factors of safety are considered acceptable, because of their very short duration and their relatively fast increase beyond the specified factor of safety (Wardrop, 2009b). For the Country Rock WRD, lifts 9 and 10 will reach a factor of safety of 1.3 after 11 and 15 months of placement of the last lift, respectively. Detailed slope stability results for Country Rock WRD and Dolomite WRD are presented elsewhere (Wardrop, 2009b).

Table 2.12-4 Assumed Sigma/W Material Properties for the Waste Rock Dump Stability Analyses

Materials	Material Category	Material Model	Poisson's Ratio	Young's Modulus (kPa)	Hydraulic Conductivity (cm/s)
Waste Rock	Effective Drained Parameters	Linear Elastic	0.35	70,000	-
Peat	Effective Parameters w/PWP Change	Linear Elastic	0.35	2,000	1.00E-01
Soft Clay (CL)	Effective Parameters w/PWP Change	Soft Clay (MCC)	0.36	-	1.36E-08
Soft Clay (CH)	Effective Parameters w/PWP Change	Soft Clay (MCC)	0.37	-	4.97E-09
Bedrock	Effective Parameters w/PWP Change	Linear Elastic	0.49	100,000	6.89E-04

Source, Wardrop, 2009b

Note: PWP Porewater pressure.

Table 2.12-5 Assumed Slope/W Material Properties for the Waste Rock Dump Stability Analyses

Materials	Model	Unit Weight (kN/m^3)	Cohesion (kPa)	Phi (º)
Waste Rock	Mohr-Coulomb	20	0	40
Peat	Mohr-Coulomb	13	18	0
Soft Clay (CL)	Mohr-Coulomb	21	20	29
Soft Clay (CH)	Mohr-Coulomb	18	10	25
Bedrock	Bedrock (Impenetrable)			

Source, Wardrop, 2009b

Table 2.12-6 Slope Stability Results

Lift No.	Country Rock Waste Rock Dump (CRWRD)			Dolomite Waste Rock Dump (DWRD)		
	Static (10 day) Required/ Computed	Static (6 months) Required/ Computed	Pseudo static (6 months) Required/ Computed	Static (10 day) Required/ Computed	Static (6 months) Required/ Computed	Pseudo static (6 months) Required/ Computed
1	1.30/**1.15**	1.30/1.69	1.05/1.53	1.30/1.90	1.30/2.04	1.05/1.87
2	1.30/**1.28**	1.30/1.46	1.05/1.20	1.30/1.34	1.30/1.33	1.05/1.18
3	1.30/1.67	1.30/1.93	1.05/1.45	1.30/1.37	1.30/1.31	1.05/1.20
4	1.30/1.75	1.30/1.89	1.05/1.47	1.30/1.37	1.30/1.46	1.05/1.23
5	1.30/1.77	1.30/1.75	1.05/1.46	1.30/1.36	1.30/1.45	1.05/1.24
6	1.30/1.53	1.30/1.58	1.05/1.36	1.30/1.37	1.30/1.46	1.05/1.26
7	1.30/1.35	1.30/1.38	1.05/1.31	1.30/1.38	1.30/1.44	1.05/1.27
8	1.30/**1.26**	1.30/1.32	1.05/1.22	1.30/1.39	1.30/1.44	1.05/1.28
9	1.30/**1.22**	1.30/1.30*	1.051.20*	1.30/1.40	1.30/1.45	1.05/1.29
10	1.30/**1.23**	1.30/1.30**	1.05/1.18**	1.30/1.40	1.30/1.44	1.05/1.29

Source: adapted from Wardrop, 2009b

Notes: * 11 months after lift placement.
** 15 months after lift placement.

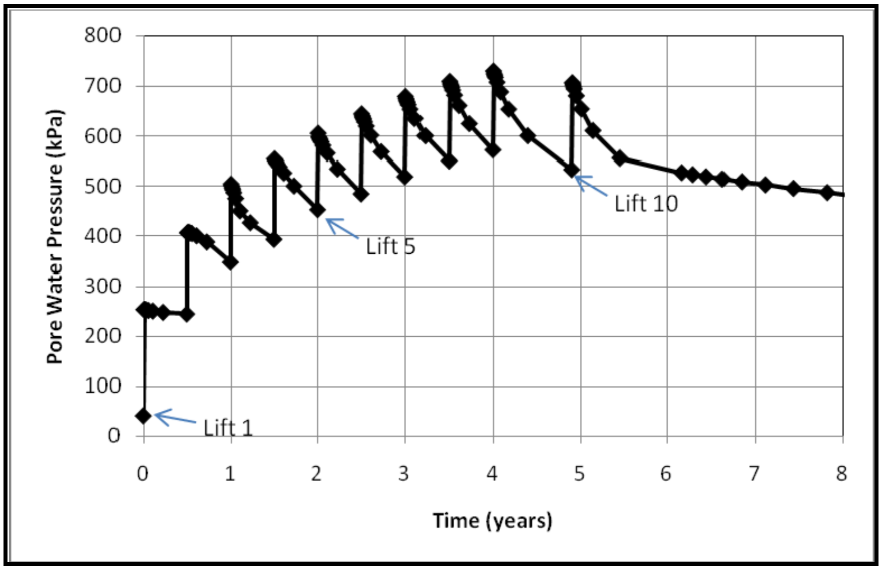
In order to achieve design heights of 40 m, the configuration of the dumps must include setbacks as summarized in Table 2.12-7 (Wasrdrop, 2009b).

Table 2.12-7 Required Setbacks for the Waste Rock Dumps

Lift No.	Country Rock Waste Rock Dump Setback (m)	Dolomite Waste Rock Dump Setback (m)
Stage 1	20	8
Stage 2	43	23
Stage 3	0	6

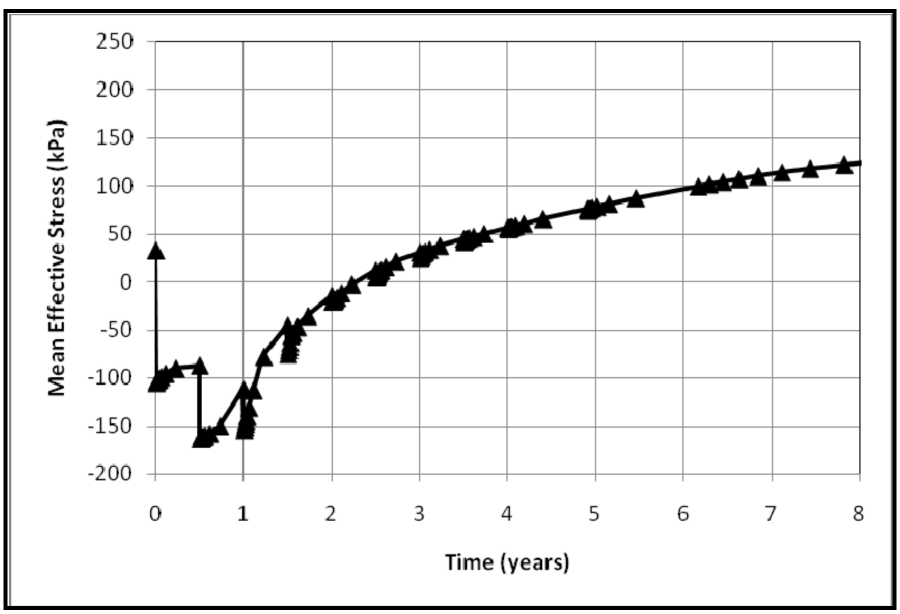
Source: Wardrop, 2009b

Figure 2.12-3 through Figure 2.12-10 show the effective stress versus time, and pore water pressure versus time for the short- and long-term conditions as computed in the foundation soils underneath the Dolomite WRD and Country Rock WRD. Figures 2.12-3, 2.12-5, 2.12-7 and 2.12-9 illustrate the effective stress increases after placement of each lift and their stabilization over time. Figures 2.12-4, 2.12-6, 2.12-8 and 2.12-10 show the pore water pressure generation after placing each lift and its dissipation over time. The estimated period for the pore water pressures to dissipate are 31 years for the Country Rock WRD and 16 years for the Dolomite WRD (Wardrop, 2009b).



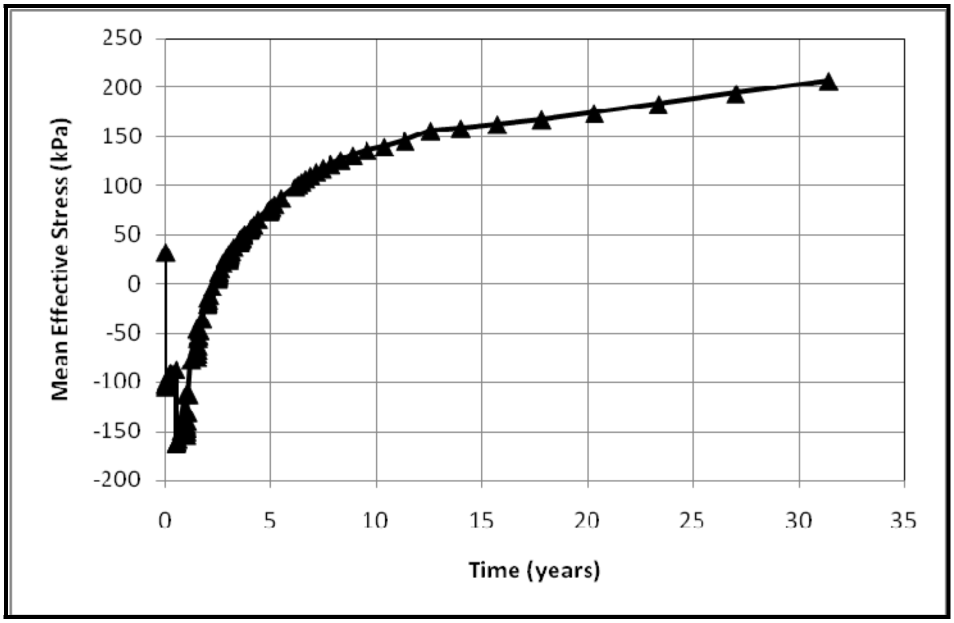
Source: Wardrop, 2009b

Figure 2.12-3 Short-term Mean Effective Stress versus Time for the Country Rock WRD



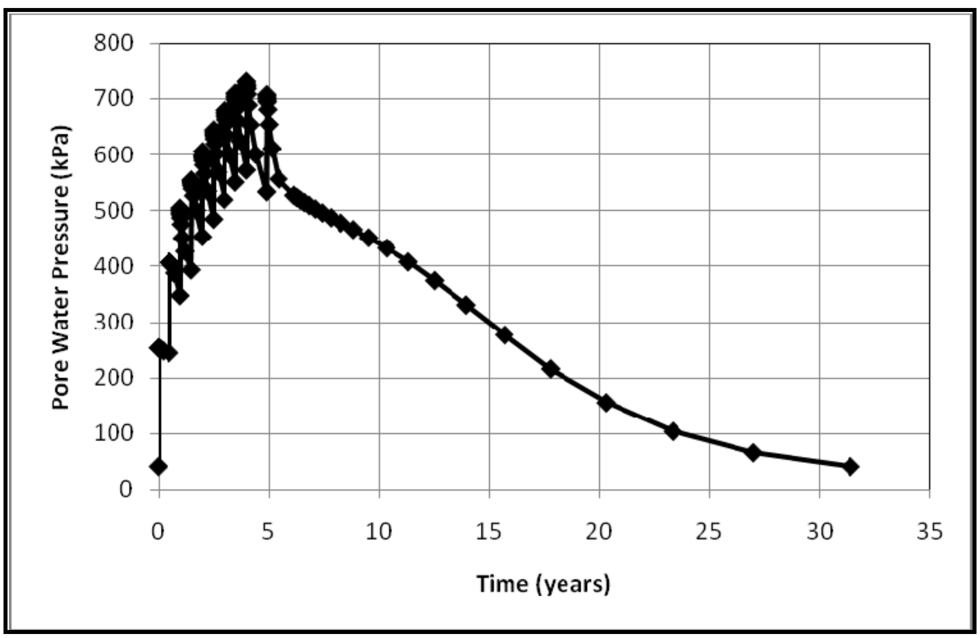
Source: Wardrop, 2009b

Figure 2.12-4 Short-term Pore Water Pressure versus Time for the Country Rock WRD



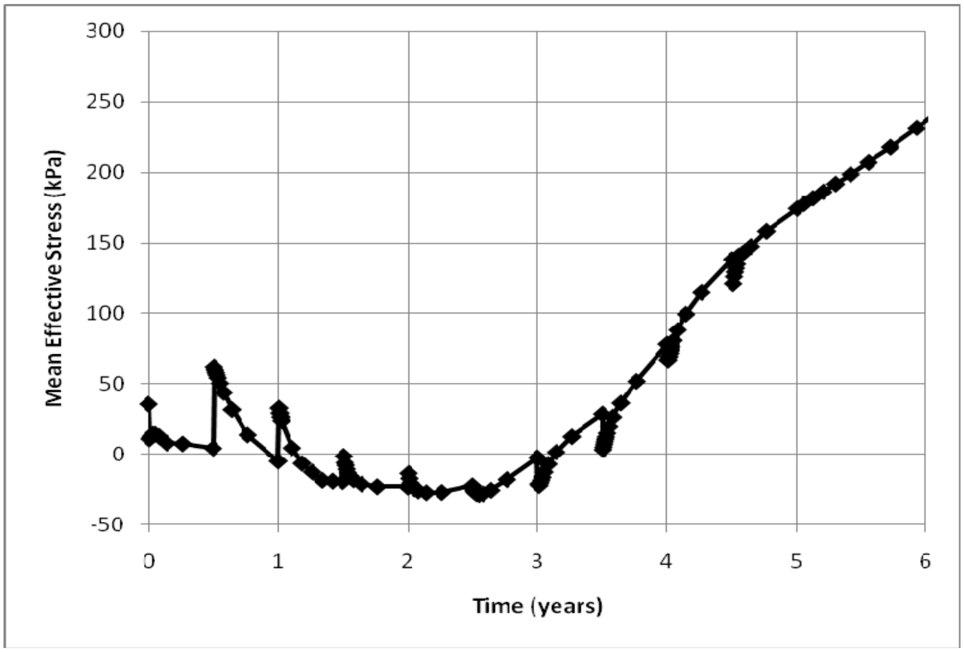
Source: Wardrop, 2009b

Figure 2.12-5 Long-term Mean Effective Stress versus Time for the Country Rock WRD



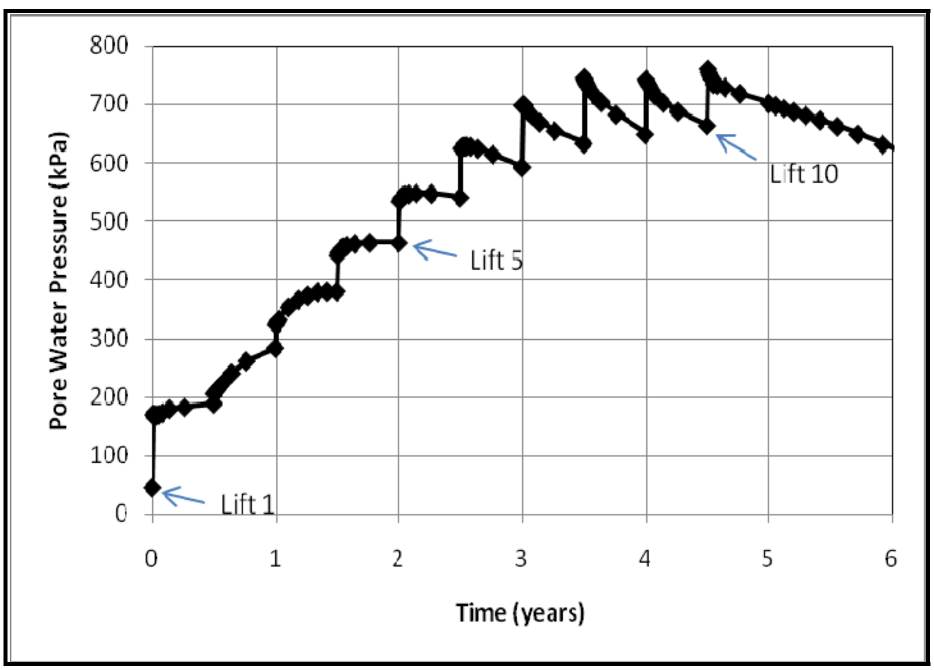
Source: Wardrop, 2009b

Figure 2.12-6 Long-term Pre Water Pressure versus Time for the Country Rock WRD



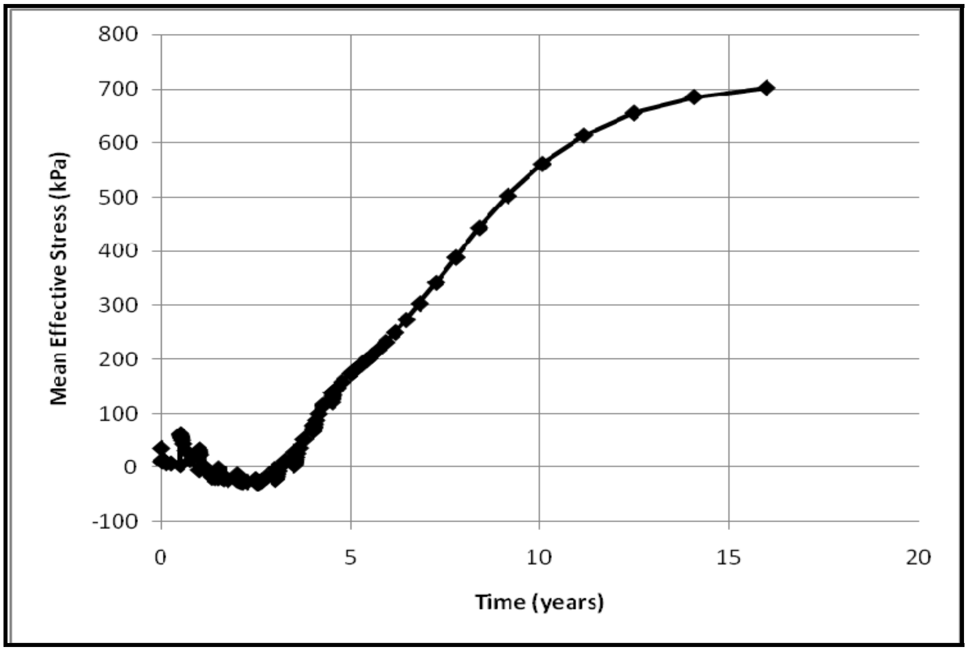
Source: Wardrop, 2009b

Figure 2.12-7 Short-term Mean Effective Stress versus Time for the Dolomite WRD



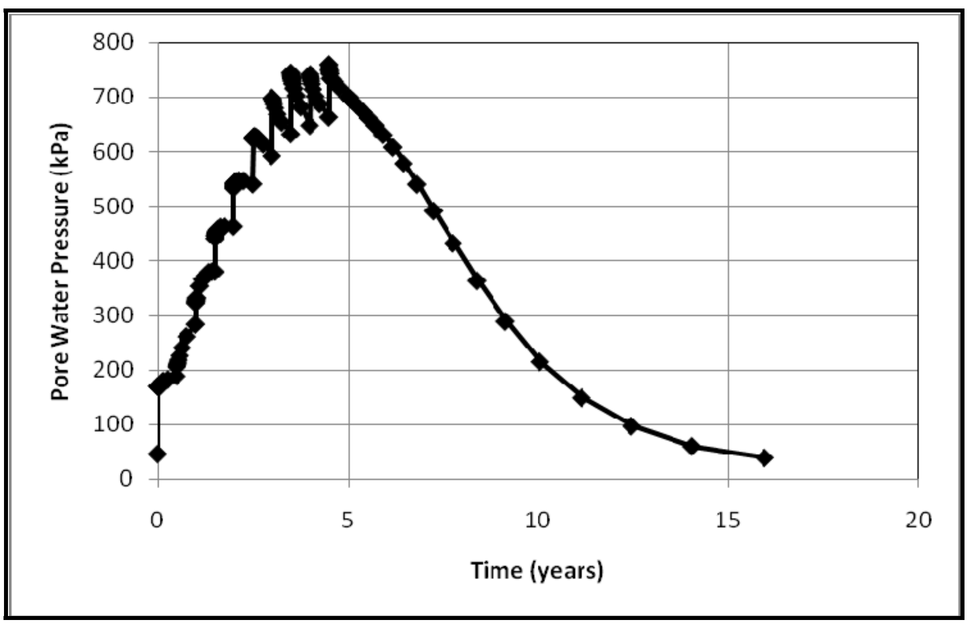
Source: Wardrop, 2009b

Figure 2.12-8 Short-term Pore Water Pressure versus Time for the Dolomite WRD



Source: Wardrop, 2009b

Figure 2.12-9 Long-term Mean Effective Stress versus Time for the Dolomite WRD



Source: Wardrop, 2009b

Figure 2.12-10 Long-term Pre Water Pressure versus Time for the Dolomite WRD

2.13 Tailings and Ultramafic Waste Rock Management Facility and Polishing Pond

The Tailings and Ultramafic Waste Rock Management Facility (TWRMF) is a key component of the water and waste management system at Minago for tailings, liquid waste and ultramafic waste rock. The disposal of tailings and waste rock has been studied from a number of different perspectives. The selected alternative is tailings co-disposal with ultramafic waste rock behind a lined rockfill embankment dam. Muskeg and/or clay will be forming the base of the embanked repository. The remaining waste rock will be disposed of in the Dolomite Waste Rock Dump, if it is dolomite/limestone, or in the Country Rock Waste Rock Dump otherwise (Figure 2.1-2).

The TWRMF location within the project area (Figure 2.1-2) was selected to take into account factors such as the exclusion zones, the distance from the open pit and the favourable subsurface conditions, including shallow soft clay overburden (Wardrop, 2009b).

One key objective for the co-disposal is to initially induce invasion of tailings into the voids of end-dumped PAG/ML waste rock to encapsulate the PAG waste rock in tailings for the ultimate goal of providing acceptable seepage water quality from the facility. Other key objectives are to facilitate closure without long-term water treatment and to significantly lower CAPEX/OPEX and closure cost (Wardrop, 2009b).

Material in the TWRMF will be stored subaqueously whenever possible. Subaqueous disposal is practiced at many metal mines to keep oxidative rates at a minimum and to minimize metal leaching. Based on geochemical work done to date, Minago's mill tailings contain low sulphide levels and were deemed to be non acid generating (NAG) (URS, 2009i). Sulphide levels were less than or equal to 0.07 % in the Master tailings samples tested. However, ultramafic waste rock has been found to be potentially acid generating (PAG) (URS, 2009i).

The TWRMF will remain in place after all operations have ceased at the site. The TWRMF inflow will consist of:

1) mill tailings;
2) tailings and liquid waste from the Frac Sand Plant;
3) outflow from the sewage treatment system;
4) sludge from the potable water treatment plant; and
5) precipitation.

Outflows from the TWRMF include the TWRMF Decant, losses due to evaporation and sublimation, and seepage. Seepage will be captured by interceptor ditches surrounding the TWRMF and will be pumped back to the TWRMF. The seepage design criteria has tentatively been set at 250 m^3/day to satisfy walk-away requirements (Wardrop, 2009b). The TWRMF Decant will be discharged to the Polishing Pond (Figure 2.1-2) and will be regulated automatically by a control system.

2.13.1 TWRMF Design Criteria

The TWRMF design requires compliance with permitting requirements as well as dam design and water quality guidelines. The TWRMF dam design is controlled to a significant extent by the presence of weak peat and clay foundation soils and a sufficient separation of the dam from Highway 6. The TWRMF must accommodate a total of 27.4 Mt of nickel and frac sand tailings and 36 Mt PAG-waste rock over the course of 9 years and provide secure storage for the long-term.

The Design Basis and Basic Engineering Design Parameters are summarized in Tables 2.13-1 and 2.13-2, respectively. Additional Design Criteria for the TWRMF are as follows (Wardrop, 2009b):

- The rate for the construction of successive stages of the TWRMF Dam should be governed by foundation strength and consolidation characteristics as well as the mine waste production schedule.
- The cone of depression created by pit dewatering is predicted to extend laterally in the dolomite to a distance of approximately 5,000 m to 6,000 m from the proposed open pit. The cone of depression will provide under drainage for the overburden clays and should be considered in geotechnical analyses for the TWRMF dam.
- A designated decant pond should be located between the causeways.
- The tailings deposition plan should ensure minimal exposure of PAG waste rock to atmospheric conditions during operations, closure and post closure.
- The configuration of PAG waste rock within the facility should allow for 2 m tailings cover at the end of the tailings deposition.
- Based on experience, tailings deposition slopes of 0.5% sub-aerial and 2% subaqueous should be assumed in the design.

2.13.2 Deposition Plan for the TWRMF

Construction of the TWRMF dam will take place in 2011 and 2012. Concurrently disposed tailings and ultramafic waste rock will be fully contained behind a perimeter dam to be constructed as a part of a robust operation. Key elements of the concurrent disposal of tailings and ultramafic waste rock in the TWRMF are illustrated in Figure 2.13-1 and the deposition strategy is briefly described in the following paragraphs (Wardrop, 2009b):

- In order for the frac sand deposition to start and subsequently to support the initial phase of Ni-tailings deposition in 2014, a dolomite waste rock base will be constructed where the coarse PAG-waste rock rind will be placed and underneath the north and south causeways. The construction of the dolomite waste rock base will be completed during the last stages of the TWRMF dam construction in 2012.

Table 2.13-1 Design Basis for the TWRMF

Item	Value
Life of TWRMF	9 years
Total Nickel Tailings (tonnes)	24,847,889
Total Sand Tailings (tonnes)	2,571,804
Total Combined Tailings to TWRMF (tonnes)	27,419,693
Total PAG Waste Rock (tonnes)	35,660,000
Tailings Specific Gravity (Nickel)	2.6
Initial Tailings Void Ratio (Nickel)	1.0
Initial Tailings Density (Nickel)	1.3 t/m^3
Average Final Tailings Density (Nickel)	1.5 t/m^3
Tailings Pulp Density (solid weight) (Nickel)[1]	45%
Water in Tailings Voids (Nickel)	22%
Average Initial Tailings Density (Sand)	1.4 t/m^3
Average Final Tailings Density (Sand)	1.6 t/m^3
Tailings Pulp Density (solid weight) (Sand)	20%
Ultramafic Waste Specific Gravity	2.59
Ultramafic Waste Swelling	30%
Void Space in PAG Waste Rock	4,130,502 m^3
Void Space in Coarse PAG Waste Rock	3,304,402 m^3
Void Space in Fine PAG Waste Rock	826,100 m^3
Total Volume of Ni Tailings	16,565,259 m^3
Total Volume of Sand Tailings	1,607,378 m^3
Total Combined Tailings Volume	18,172,637 m^3
Total PAG Waste Rock (solids and voids)	17,898,842 m^3
Total Ni-Tailings Ingress into Voids of Coarse Ultramafic Waste Rock (at initial tailings density)[2]	2,478,301 m^3
Total Ni- and Frac-Sand Tailings ingress into Voids of Fine Ultramafic Waste Rock (at initial tailings density)[3]	413,050 m^3
Total Ni-Tailings Between the Ultramafic Waste Rock Rind and Central Causeway (at final tailings density)	15,376,725 m^3
Required TWRMF Storage	33,275,567 m^3
Required TWRMF Storage (with 15% contingency included)	38,300,000 m^3

Source: adapted from Wardrop, 2009b

NOTES:

1. A 45% solids density is used in the feasibility study water balance. However, higher water-to-solids ratios to enhance transport into and through the rock fill may be considered in the detailed engineering.

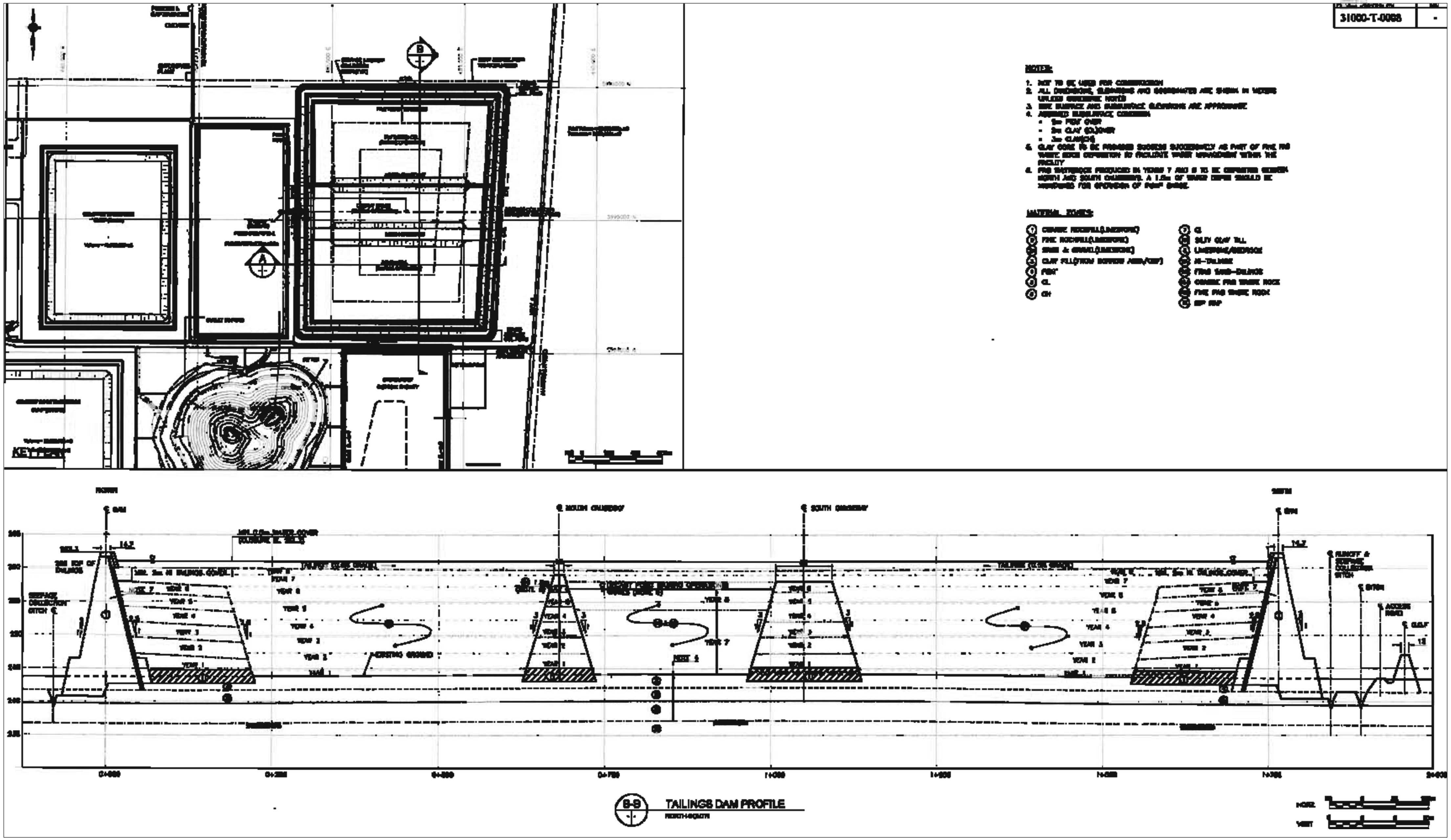
2. Coarse ultramafic waste rock, represented by fractions larger than 0.2 m, is estimated to be 80% of total ultramafic waste rock. Infilling of voids within coarse ultramafic waste rock with tailings is estimated to be 75%. Ingressed tailings were assumed to remain at their initial density due to the relative incompressibility of the waste rock matrix.

3. Fine ultramafic waste, represented by fractions finer than 0.2 m, is estimated at 20% of total ultramafic waste. Infilling of voids within fine ultramafic waste rock with tailings is estimated to be on the order of 50%. Ingressed tailings are assumed to remain at their initial density due to the relative incompressibility of the waste rock matrix.

Table 2.13-2 Basic Engineering Design Parameters for the TWRMF

Item	Target	Comments
1. Geotechnical Slope Stability		
• Construction (in stages)	• Static F.O.S. 1.3, pseudo static F.O.S 1.05.	
• Normal Operating	• Same as above.	
• Closure	• Static F.O.S. 1.5, pseudo static F.O.S 1.05.	
2. Seepage	• Limit on Contaminants of Concern (CoC) concentrations	• Analyses using SEEP/W targeting a total estimated seepage volume less than 250 m^3/day. • Low permeability barrier to be provided on the upstream face of the containmant structure to reduce seepage through the ultramafic waste rock – tailing composite. • Seepage from the TWRMF to be collected via collection ditches and ponds.
3. Hydrotechnical • Construction Diversion Peak Flow	• 1:20 yr - 24 hr rainfall	• All peak flows are estimated from catchment times of concentration and storm. Seepage to be collected via collection ditches reporting to the overall water management system.
• Operation peak flow	• 1:200 yr – 24 hr rainfall	
• Closure Spillway and Diversion peak flow	• 1:1,000 yr – 24 hr rainfall	• Determine wave run-up in the freeboard.
• Freeboard	• 1.0 m on the top of Closure Spillway wet section for 1:200 year runoff	
• Closure Flood	• 1:1,000 yr – 24 hr rainfall	
• Runoff Coefficient	• 1	
4. Decant System (if applicable) • Water Storage	• Minimum five days retention or 1.5 m of water level at all times, whichever is higher	
5. Closure Cover	• A minimum of 0.5 m of water on the top of final tailings at the containment structure.at all times.	• Runoff (dry year), seepage, infiltration and evaporation to ensure a minimum thickness water cover.
6. Seismicity • Operating Design Basis Earthquake	• 1: 475 year return	
• Closure Earthquake	• 1:2,475 year return	

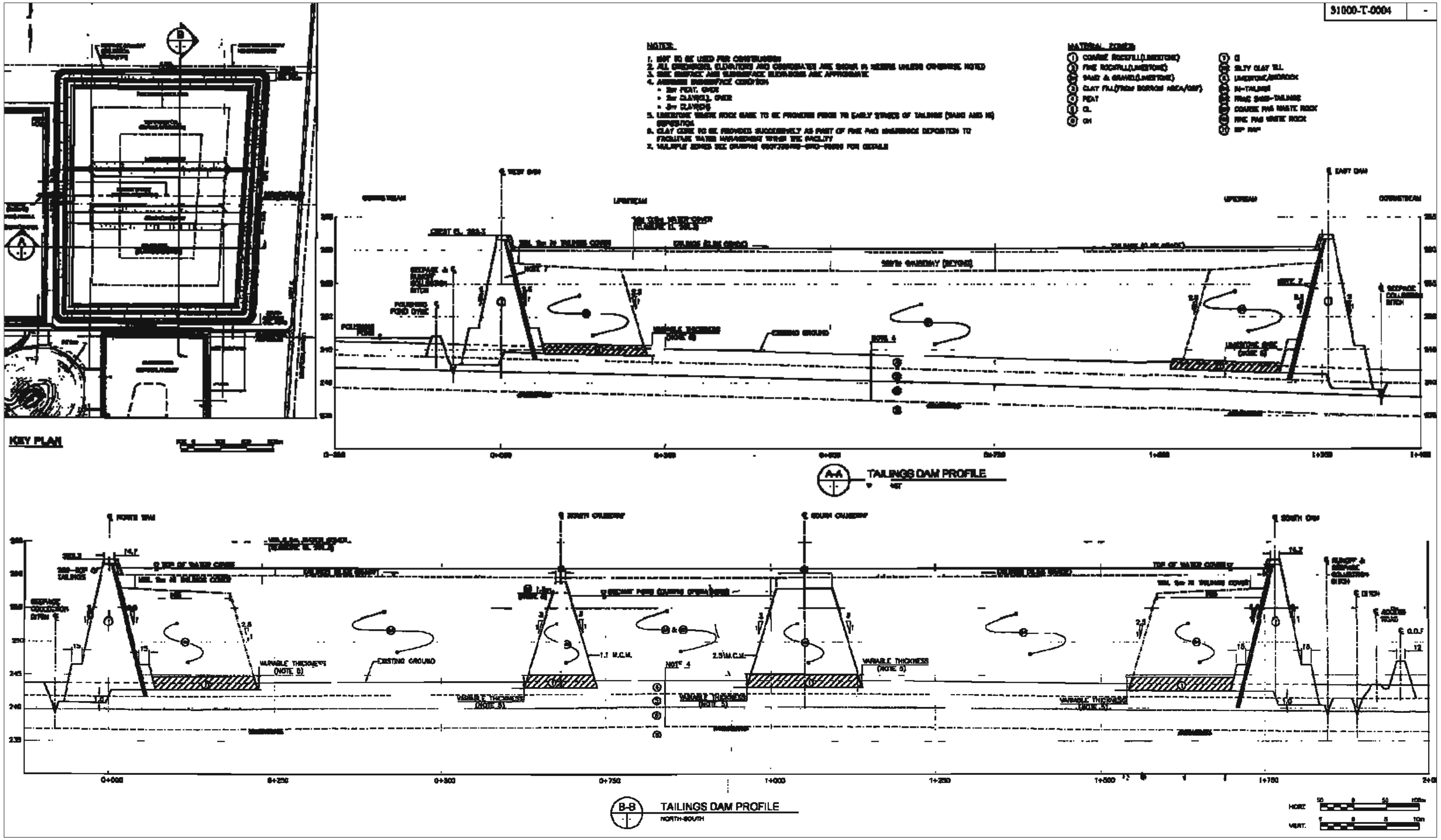
Source: Wardrop, 2009b



Source: adapted from Wardrop's drawing 0951330400-T0008 (Wardrop, 2009b)

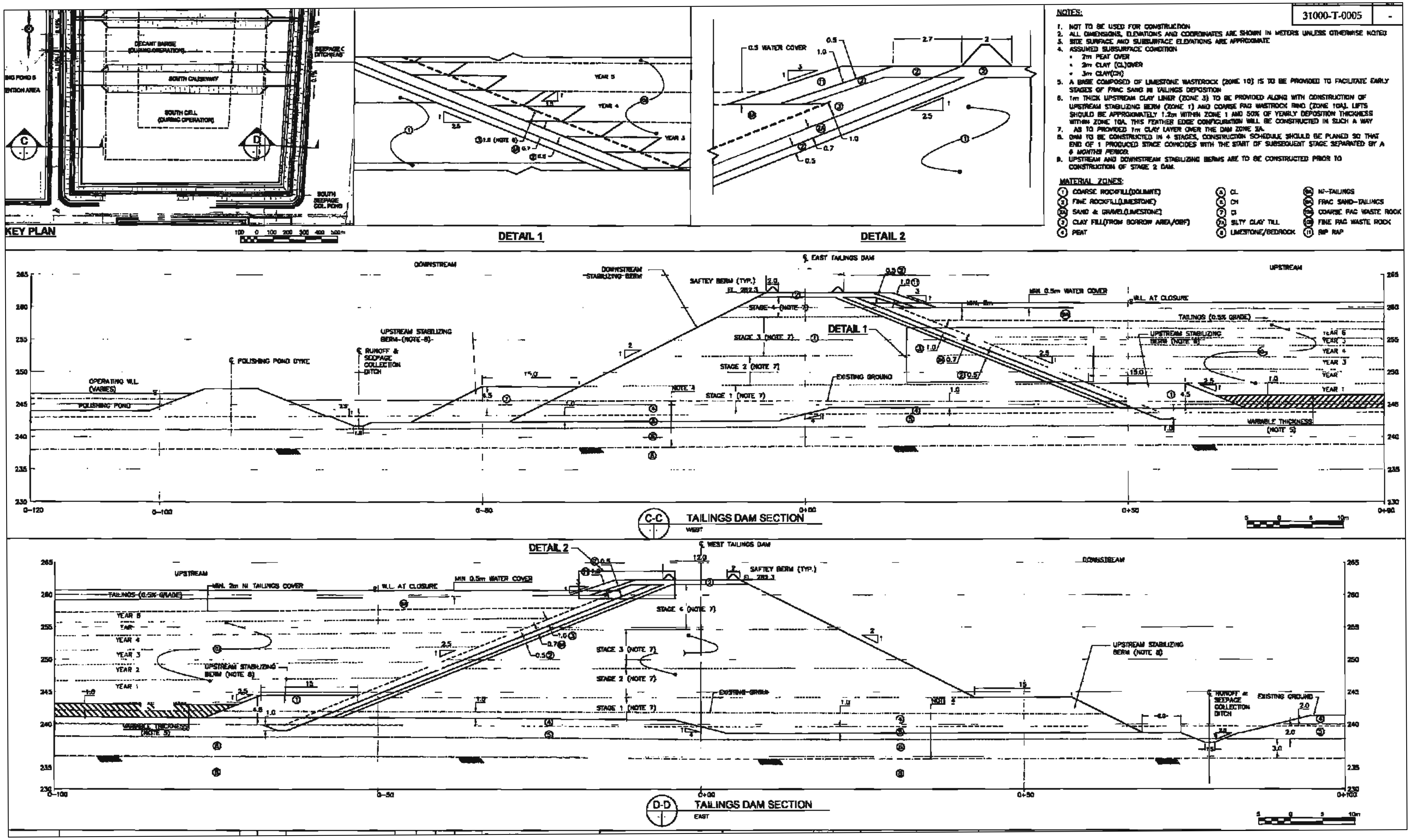
Figure 2.13-1 Deposition Plan and Profiles of theTailings and Ultramafic Waste Rock Management Facility

- The retaining structure construction will be carried out in lifts corresponding to yearly ultramafic waste rock production. A 1 m clay liner will be provided between the rind and the upstream face of the dam as depicted in Figures 2.13-2 and 2.13-3. The clay liner in between the waste rock rind and the dam will ensure full containment by minimizing seepage reporting to the downstream environment as per design criteria.
- The clay cutoff trench within the north causeway will facilitate intermittent flooding and dewatering in both of the cells (north and south cells). Maximizing PAG waste rock saturation during waste rock placement will minimize oxidation and reduce their ARD/ML potential.
- The coarse ultramafic waste rock (estimated at 80% of total PAG-waste rock production) will be deposited in a rind to be constructed immediately upstream of the dam. The rind construction will be carried out in lifts corresponding to the yearly PAG waste rock production.
- The fine ultramafic waste rock (estimated at 20% of total PAG-waste rock production) will be deposited in the north and south causeways. The north causeway will have a clay cutoff trench built in stages, also in accordance with the yearly waste rock production schedule.
- Ultramafic waste rock will be placed simultaneously in both the northern and southern cells and flooding will closely follow advancement of the ultramafic waste rock placement. Dewatering of the north cell will take place prior to the start of tailings deposition in order promote hydraulic gradients and thereby increase invasion of Ni-tailings into the void space of the ultramafic waste rock. Tailings deposition in the northern cell will take approximately 6 months. Dewatering of the southern cell will precede Ni-tailings placement and it will take another 6 months to complete the deposition in the southern cell.
- Ni-tailings deposition will be carried out from the dam crest by running feeder pipes from the main tailings supply pipe at the dam crest and down the upstream dam slope. The feeder pipes will eventually distribute tailings over the rockfill through perforated spreader pipes. Ripping of the uppermost PAG-waste rock surface might be done as an expedient to open up the uppermost fines in order to promote tailings ingress into the waste rock void space.
- Ultramafic waste rock placement and tailings deposition will alternate in the same fashion for 6 years. During this time, a decant pond will be created between the north and south causeways from which water will be pumped to the Polishing Pond (Figure 2.1-2). In 2019 and 2020, coarse ultramafic waste rock will be deposited in the area between the north and south causeways. A minimum of 1.5 m of decant water above the waste rock will be maintained to facilitate free flow and prevent potential blockage during operations of barge mounted pumps. An alternative arrangement may be pumping from perforated decant towers installed within the rockfill placed in causeways. This alternative will be examined more closely in the detailed design stage.



Source: adapted from Wardrop's drawing 0951330400-DWG-T0004 (Wardrop, 2009b)

Figure 2.13-2 Tailings and Ultramafic Waste Rock Management Facility (TWRMF) Dam Plan and Profile



Source: adapted from Wardrop's drawing 0951330400-DWG-T0005 (Wardrop, 2009b)

Figure 2.13-3 Tailings and Ultramafic Waste Rock Management Facility (TWRMF) Dam Plan and Sections

- In 2019 and 2020, the ultramafic waste rock in the rind will receive a minimum of 2 m of Ni-tainings cover by peripheral discharging from the dam crest. It is estimated that the slopes for the tailings beach and the subaqueous tailings will be 0.3% and 2%, respectively.
- The frac sand tailings deposition will be carried out from the top of causeways until the Ni-tailings deposition ceases. Thereafter, sand tailings will also be deposited from the dam crest through the main tailings supply pipe system.
- After Ni-tailings deposition will have ceased, frac sand tailings will be deposited as a final layer on top of the Ni-tailings. Frac sand tailings will be produced approximately 2 years longer than the Ni-tailings. Frac sand tailings have low metal concentrations and will leave the top surface of the TWRMF in an inert condition. On top of the Frac sand tailings, a minimum of 0.5 m of water cover will be provided on closure.

Concurrent disposal of tailings and ultramafic waste rock will ensure total encapsulation of PAG-waste rock on closure and the water cover will ensure subaqueous disposal, both of which will minimize ARD/ML concerns.

Decant from the TWRMF will be discharged to the Polishing Pond to address concerns regarding the resuspension of tailings due to wind and wave action on the water cover. Suspended solids will settle out in the Polishing Pond prior to water discharge from that facility to the receiving environment.

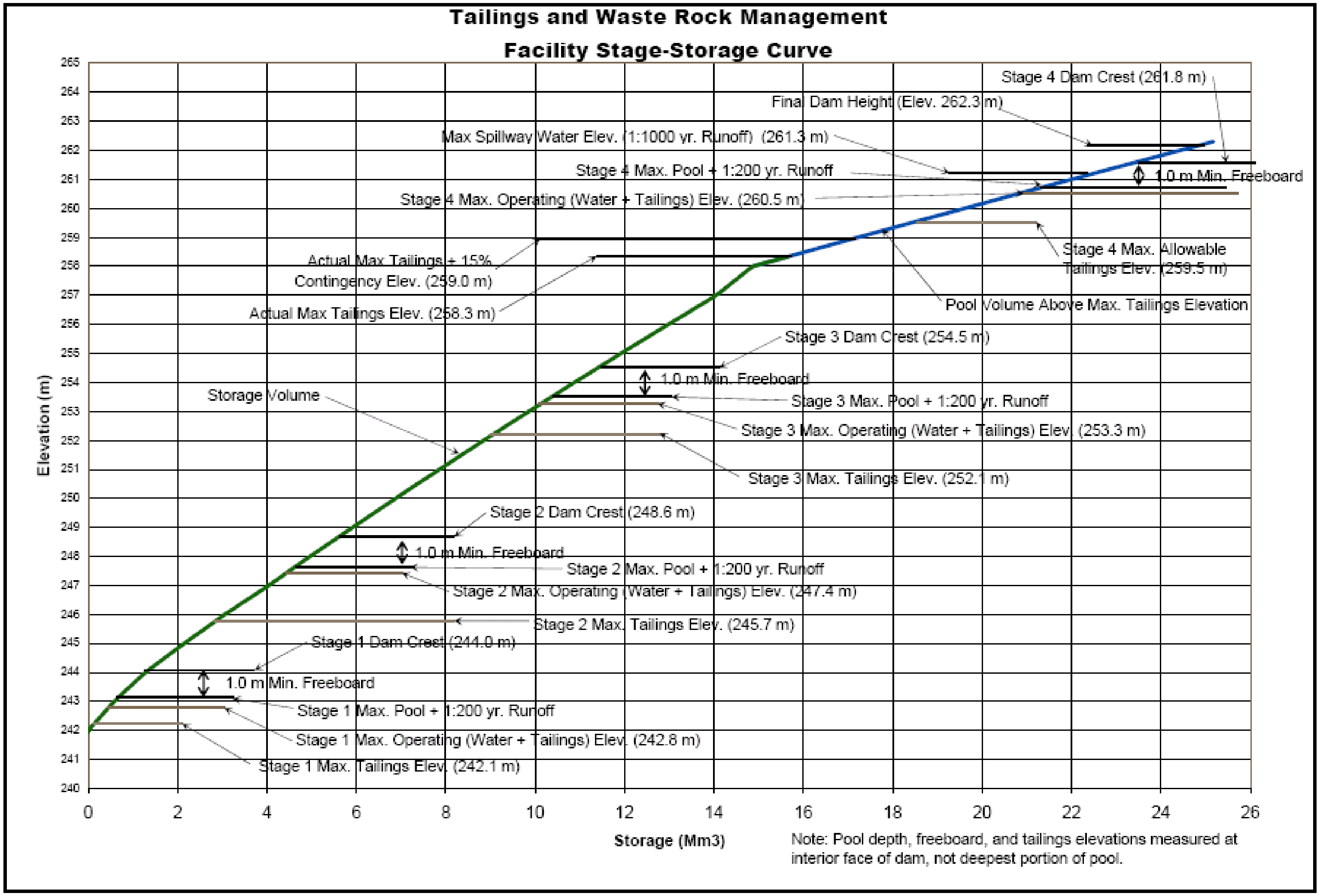
Figure 2.13-4 shows stage storage curve with critical design elevations for the TWRMF based on estimates given in Minago’s Feasibility Study (Wardrop, 2009b).

2.13.3 TWRMF Dam Options and Selections

2.13.4 TWRMF Dam Section Design

The dam will be located in an area where the geotechnical profile lends itself to a higher containment structure with a small footprint. Also, the geotechnical profile will allow for construction staging to meet the mine production schedule. To bound uncertainties related to extrapolation of confirmed shallow overburden characteristics in the southeastern part of the TWRMF, a deeper overburden was assumed to underlie the rest of the TWRMF in the geotechnical analyses considered in this report. Final confirmation of the TWRMF foundation will be part of a detailed design geotechnical investigation. The plan and profile of the TWRMF is shown in Figure 2.13-2 with typical dam sections illustrated in Figure 2.13-3.

The TWRMF dam was designed as an earth/rockfill structure varying in settled height from approximately 19 m to 21 m above the local topography. Peat will be left in place within the upstream part of the dam foundation and removed along with a 1.0 m of soft underlying clay within



Source: Wardrop, 2009b

Figure 2.13-4 TWRMF Stage Storage Curve

the downstream part. The upstream and downstream dam slopes of the rockfill dam will be 2.5H:1V and 2H:1V, respectively (Wardrop, 2009b).

Based on stability analyses, the dam will be constructed in four (4) stages to meet the consolidation requirements. The construction schedule will be planned so that the end of previous stage coincides with the start of the subsequent stage. The heights of dam fill will be up to 4.5 m and 6 m for Stages 1 and 2 and Stages 3 and 4, respectively. Stabilizing berms (4.5 m high and 15 m wide downstream and upstream) will be required prior to the start of the Stage 2 lift (Figure 2.13-3).

The construction of the dam will take two years from the start in “2012” (Year -1) to completion at the end of “2013” (Year +1). The dam shell will be constructed of coarse rockfill (Zone 1 material) comprising an estimated 800 mm minus dolomite waste rock originating from the open pit (Figure 2.13-3). The upstream side of this zone will support a 0.5 m thick zone of fine rockfill (Zone 2 material) comprised of minus 75 mm dolomite waste rock and finally a 0.5 m sand and gravel zone (Zone 2A material). The dam will have an upstream clay lining with a nominal thickness of 1 m placed over the Zone 2A in four sequences as shown in Figure 2.13-3 and briefly described below.

Sequence 1: The clay liner will extend through peat to be keyed in the native clay. The clay liner (Zone 3) will be provided in a feather-edge like gap between the top of Zone 2A on the dam upstream slope and Zone 1 within the upstream stabilizing berm. This will coincide with the completion of about 1.2 m thick lifts within the upstream stabilizing berm constructed ahead of the start of the Stage 2 lift within the main dam structure (Wardrop, 2009b).

Sequence 2: The clay liner (Zone 3) will be provided in a feather-edge like gap as depicted on Detail 1 between the top of Zone 2A on the dam upstream slope and Zone 10A within PAG-waste rockfill rind. This will coincide with the completion of about 50% of yearly lift thicknesses within Zone 10A (PAG-waste rock rind) (Wardrop, 2009b).

Sequence 3: A 1.0 m clay (Zone 3) liner above the PAG rockfill rind will be placed over Zone 2A at the dam upstream slope ahead of tailings discharge (Wardrop, 2009b).

Sequence 4: Extention of the clay (Zone 3) liner to the dam crest will be placed ahead of the water cover implementation. The thickness of the Zone 3 in this last stage will increase as dictated by a 3H:1V upstream slope. The clay liner in this uppermost zone will be protected with a 0.5 m thick fine rockfill (Zone 2), which in turn will be covered by a 1.0 m of rip rap (Zone 11) to protect the dam crest from the ice scour action. In initial stages, material for Zone 3 will be obtained from local borrow pits containing stiff clays. Subsequently, Zone 3 material may be obtained from the ODF, if it meets design specifications (Wardrop, 2009b).

A 0.3 m thick pavement surface composed of Zone 2 material will be provided on the dam crest. Appropriate safety berms composed of Zone 2 material will also be provided on the crest (Wardrop, 2009b).

2.13.5 Dam Stability and Settlement Analyses

Dam stability and settlement analyses were carried out in support of developing a dam design section that satisfies design criteria outlined in Tables 2.13-1 and 2.13-2.

Methods of Analysis

Coupled analyses using Sigma/W and Slope/W, components of GeoStudio 2007, were used in the dam stability and settlement analyses. Sigma/W uses finite element methods to solve both stress-deformation and seepage dissipation equations simultaneously. Pore water pressures generated during lift placement were calculated with Sigma/W and then incorporated into Slope/W for stability analysis. Slope/W was used to locate failures with the least factor of safety within defined search limits (Wardrop, 2009b).

In the modelling, initial pore pressure conditions were specified with an initial water table at the ground surface. Zero pressure boundary conditions were applied to the bottom of the bedrock to model dewatering wells pumping water out of the bedrock layer.

Sigma/W modelling of the dam's section assumed that the first lift will be placed on the first day, and that 6 months will pass thereafter for consolidation. Slope stability analyses were performed assuming that 10 days had passed since the lift had been placed and at the end of 182 days. All four lifts were modeled assuming no waste rock or tailings had been placed on the upstream side of the TWRMF until construction was completed (Wardrop, 2009b).

Another analysis was performed that simulated conditions six months after the completion of the facility, assuming that the waste rock and tailings had been placed at the same time. The total computed construction time of the facility was assumed to be 2 years (Wardrop, 2009b).

A small buttress with a height of 4.5 m and a width of 15 m was incorporated into the design on both the upstream and the downstream sides of the TWRMF. Construction of these buttresses was assumed to coincide with the time of placement of the Stage 2 lift to enhance stability (Wardrop, 2009b).

Pseudo static analyses were completed to simulate earthquake conditions using 0.03 g (50% of the Peak Ground Acceleration (PGA) for a 1:2,475-year return period), which is consistent with generally accepted practices adopted by the United States Army Corps of Engineers (Hynes-Griffin and Franklin, 1984).

Assumed Material Properties

Assumed material properties for the foundation materials (CL, CH and bedrock) were based on field and laboratory data. The properties of the waste rock (dolomite and PAG/ML (ultramafic), coarse and fine rockfill material were estimated based on previous experience and professional judgement (Wardrop, 2009b). Tables 2.13-3 and 2.13-4 show material properties used in Sigma/W and Slope/W, respectively.

Results of Dam Stability and Settlement Analyses

Table 2.13-5 presents results of the slope stability analyses after placement of each lift assuming that the TWRMF is filled with PAG waste rock and tailings. Except for a very short time following the completion of the Stage 1 lift (see bolded and underlined number in Table 2.13-5), the slope stability results show that the TWRMF dam satisfies the minimum requirements for static and pseudo static conditions during operations and at closure. Because of a very short duration and relatively fast increase beyond the specified factor of safety, this is considered acceptable. Detailed slope stability results are presented elsewhere (Wardrop, 2009b).

Table 2.13-3 Sigma/W Input Material Properties

Materials	Material Category	Material Model	Poisson's Ratio	Young's Modulus (kPa)	Hydraulic Conductivity (cm/s)
Dolomite Waste Rock	Effective Parameters w/PWP Change	Linear Elastic	0.35	50,000	1.00E-01
Ultramafic Waste Rock	Effective Parameters w/PWP Change	Linear Elastic	0.35	50,000	1.00E-01
Coarse Rockfill	Effective Drained Parameters	Linear Elastic	0.33	50,000	-
Fine Rockfill	Effective Drained Parameters	Linear Elastic	0.33	7,000	-
Sand and Gravel	Effective Drained Parameters	Linear Elastic	0.35	8,000	-
Peat	Effective Parameters w/PWP Change	Linear Elastic	0.35	2,000	1.00E-01
Soft Clay (CL)	Effective Parameters w/PWP Change	Soft Clay (MCC)	0.36	-	1.36E-08
Soft Clay (CH)	Effective Parameters w/PWP Change	Soft Clay (MCC)	0.37	-	4.97E-09
Bedrock	Effective Parameters w/PWP Change	Linear Elastic	0.49	100,000	6.89E-04

Source: Wardrop, 2009b

Note:

PWP Porewater pressure.

Table 2.13-4 Slope/W Input Material Properties

Materials	Model	Unit Weight (kN/m^3)	Cohesion (kPa)	Phi (º)
Dolomite Waste Rock	Mohr-Coulomb	18	0	40
Ultramafic Waste Rock	Mohr-Coulomb	18	0	40
Coarse Rockfill	Mohr-Coulomb	19	0	40
Fine Rockfill	Mohr-Coulomb	22	0	38
Sand and Gravel	Mohr-Coulomb	22	0	35
Peat	Mohr-Coulomb	13	18	0
Soft Clay (CL)	Mohr-Coulomb	21	20	29
Soft Clay (CH)	Mohr-Coulomb	18	10	25
Bedrock	Bedrock (Impenetrable)			

Source: Wardrop, 2009b

Table 2.13-5 Slope Stability Results for the TWRMF

Case	Time (days[1])	Downstream F.O.S.		Upstream F.O.S.	
		Static Required/ Computed	Pseudo static Required/Computed	Static Required/ Computed	Pseudo static Required/Computed
Lift 1	10	1.3/**1.11**	-	1.31.48	-
	182	1.3/1.59	1.05/1.46	1.3/1.56	1.05/1.42
Lift 2	192	1.3/1.32	-	1.3/1.49	-
	364	1.3/1.77	1.05/1.58	1.3/1.59	1.05/1.38
Lift 3	374	1.3/1.60	-	1.3/1.52	-
	546	1.3/1.65	1.05/1.46	1.3/1.58	1.05/1.39
Lift 4	556	1.3/1.51	-	1.3/1.65	-
	728	1.3/1.56	1.05/1.40	1.3/1.69	1.05/1.52
Full of Tailings and towards Closure	738	1.5/1.46	-	-	-
	910	1.5/1.55	1.05/1.37	-	-
	2,577	1.5/1.94	1.05/1.75	-	-

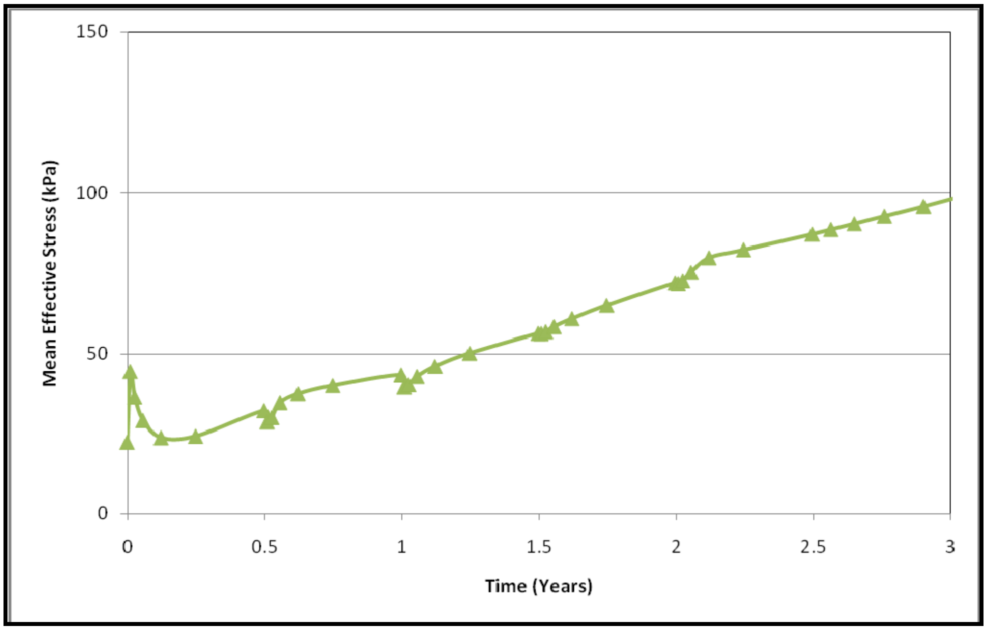
Source: Wardrop, 2009b

Note: 1 After placement of Stage 1 lift

Figure 2.13-5 through Figure 2.13-8 show the effective stress versus time, and pore water pressure versus time for the short- and long-term as computed in the foundation soils below the centerline of the dam. Figure 2.13-5 and Figure 2.13-7 illustrate how the effective stress will increase after placement of each lift and then stabilize at a later time. Figure 2.13-6 and Figure

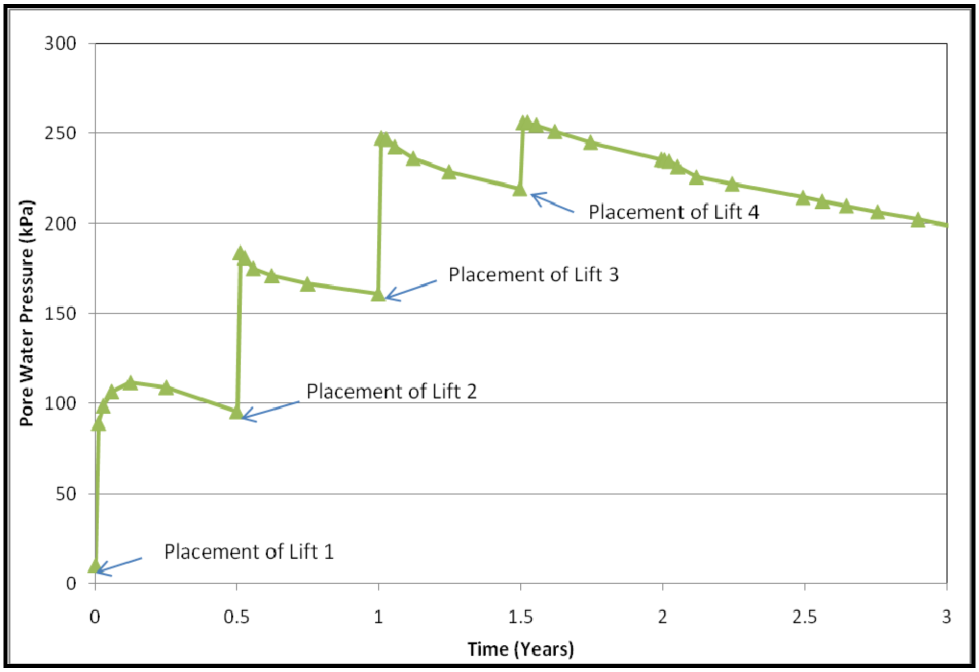
2.13-8 show estimates of pore water pressure build up after placement of each lift for dam construction for Stages 1 through 4 and its corresponding dissipation over time. The pore water pressures will dissipate in approximately 12.5 years (Wardrop, 2009b).

Figure 2.13-9 shows the settlement along the base of the TWRMF with time. The total settlement for the facility was estimated to be approximately 1 m (Wardrop, 2009b).



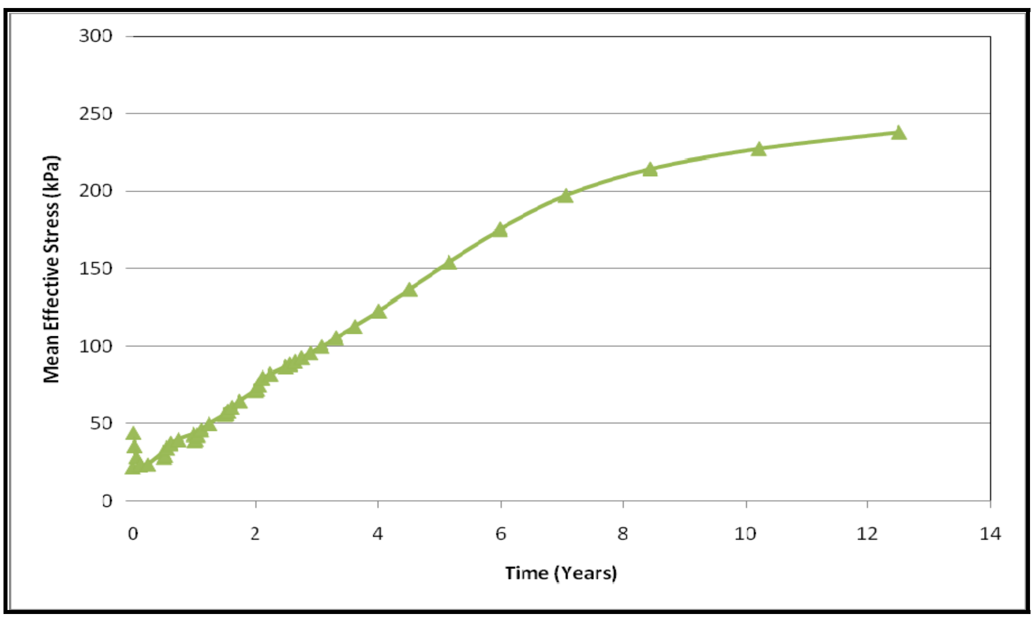
Source: Wardrop, 2009b

Figure 2.13-5 Short-term Mean Effective Stress versus Time for the TWRMF



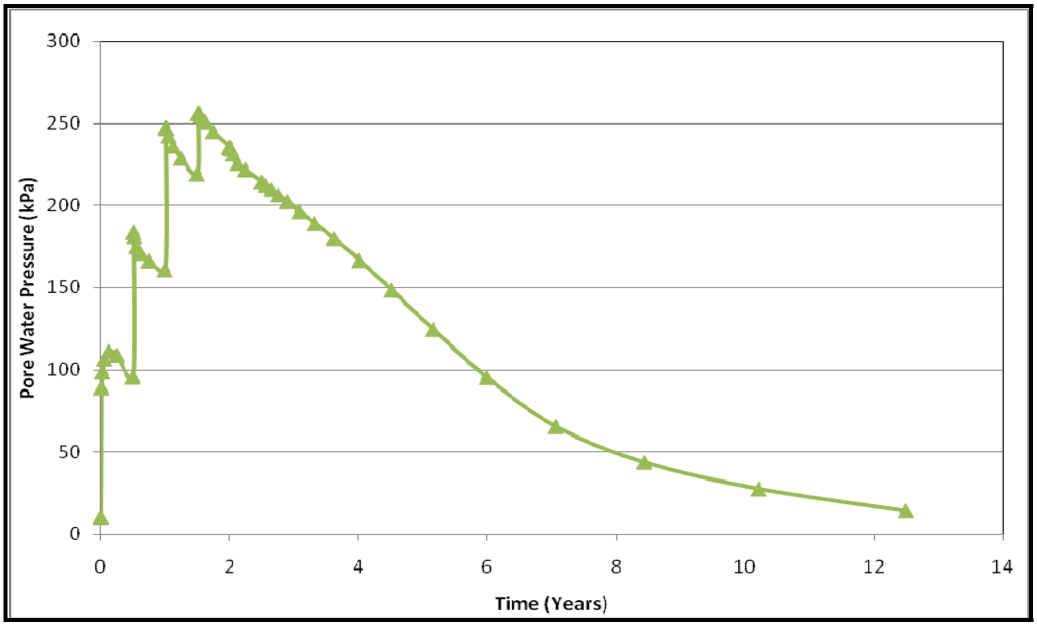
Source: Wardrop, 2009b

Figure 2.13-6 Short-term Pre Water Pressure versus Time for the TWRMF



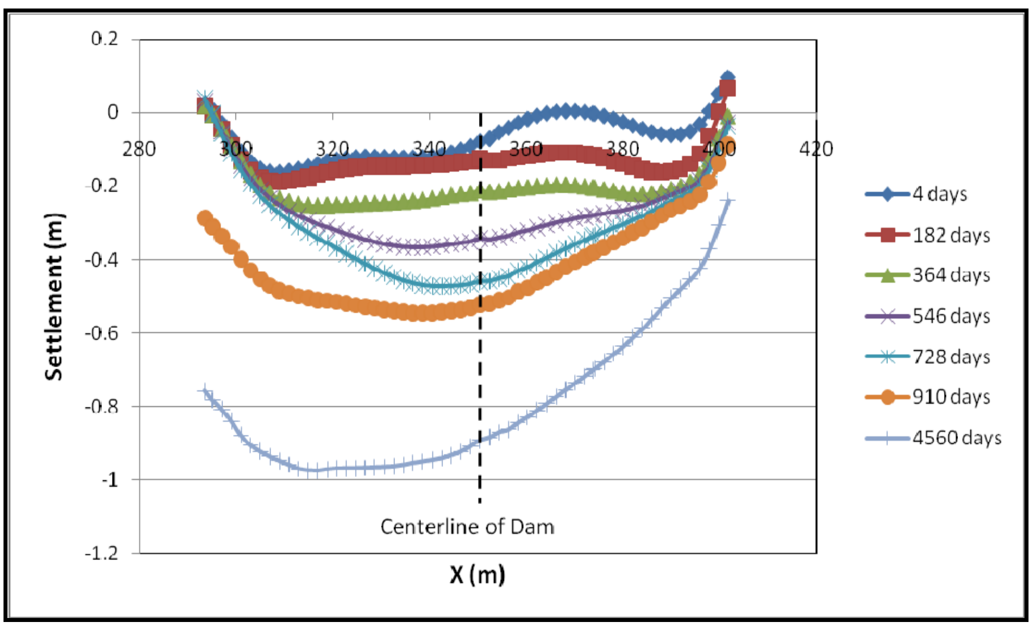
Source: Wardrop, 2009b

Figure 2.13-7 Long-term Mean Effective Stress versus Time for the TWRMF



Source: Wardrop, 2009b

Figure 2.13-8 Long-term Pre Water Pressure versus Time for TWRMF



Source: Wardrop, 2009b

Figure 2.13-9 Settlement along the Base of the TWRMF Dam

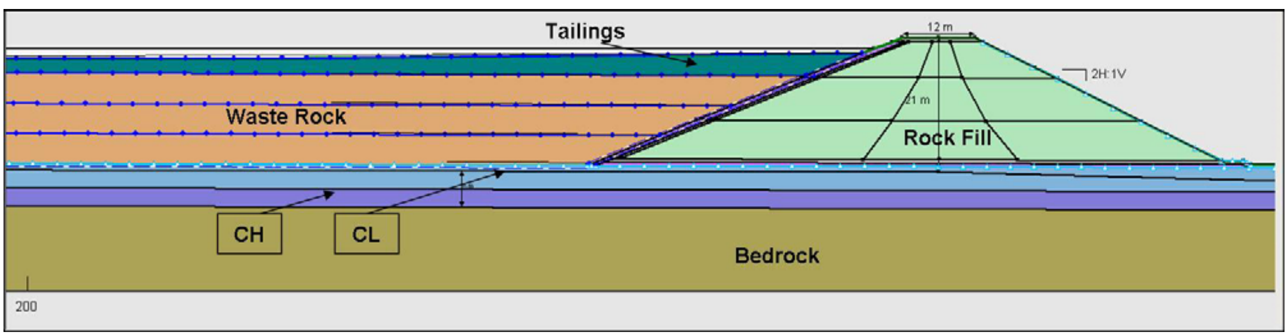
2.13.6 Seepage Analyses

Seepage analyses were critical in determining potential advantages of PAG waste rock encapsulation by Ni-tailings at closure, especially with respect to seepage water quality.

In order to develop a methodology for co-disposal of tailings and waste rock and work backwards to develop a final configuration of the containment structure, the following two scenarios were modelled (Wardrop, 2009b):

1. Dam structure comprising a gravel filter zone (Zone 2A) between the rockfill shell and the combined mass of PAG/ML waste rock and Ni-tailings. This scenario relies on attenuation of the seepage flux through the combined mass of PAG/ML waste rock and Ni-tailings.
2. Dam structure allowing for full containment of the combined mass of PAG/ML waste rock and Ni-tailings by placing clay at a variable thickness (Zone 3) over the sand and gravel filter (Zone 2A). This scenario uses the low permeability barrier to achieve better seepage control than Scenario 1.

Figure 2.13-10 illustrates the configuration of the TWRMF that was used in the seepage modeling.



Source: Wardrop, 2009b

Figure 2.13-10 Tailings Storage Facility Layout used in the Seepage Modelling

Seepage Model for the TWRMF

The seepage modeling was completed using SEEP/W (GEO-SLOPE, 2007), a two dimensional finite-element model. The modeling, which was completed in steady state, was limited to the closure conditions of the facility. The available field and laboratory data were used to estimate the hydraulic conductivity values for tailings, native clays and bedrock. The hydraulic conductivities for TWRMF dam zones and combined PAG/ML waste rock were estimated based on previous experience and professional judgement (Wardrop, 2009b). Table 2.13-6 presents a summary of the anticipated material properties and the model parameters assigned to simulate them.

Table 2.13-6 Material Properties assumed for the TWRMF Seepage Model

Material Type	Estimated Hydraulic Conductivity (m/sec)	Used in Model		
		Parameter Name	Saturated Hydraulic Conductivity (m/sec)	Saturated Volumetric Water Content (m^3/m^3)
Soft Clay (CL)	1e-10	CL	1.36e-10	0.385
Soft Clay (CH)	1e-11	CH	6.75e-11	0.3
Bedrock	1e-5 to 1e-6	Bedrock	6.90e-6	0.3
Fine Rockfill	1e-7	Fine Sand	4.30e-6	0.35
Coarse Rockfill	1e-4 to 1e-5	Uniform Sand	1.00e-5	0.3
Tailings	2e-7 to 8.2e-8	Sandy Silty Clay	1.40e-7	0.41
Clay	1e-10	CL/Well Graded High Clay	1.36e-10	0.35
Combined Waste rock and Tailings[1]	2e-7	Glacial Till (compacted)	1.00e-7	0.23

Source: adapted from Wardrop. 2009b

Note: 1 75% of void space assumed to be invaded by Ni-tailings as per design criteria.

Results of the TWRMF Seepage Analyses

The computed seepage volume reporting to the collection system immediately downstream of the 5 km long structure for Scenario 1 (leaky dam) was in the order of 2,920 m^3/day. The seepage rates for Scenario 2 were 250 m^3/day and 100 m^3/day for a 1 m and 2 m thick clay zone (Zone 3), respectively (Wardrop, 2009b).

It follows that a 1 m clay zone fulfills the seepage volume requirement towards meeting the water quality standards based on environmental concentrations and geochemistry of the seepage water. This was applied to the dam design section (Wardrop, 2009b).

2.13.7 Geotechnical Construction Considerations for the TWRMF

All peat/muskeg and the soft clay layer underneath the peat must be removed from the downstream part TWRMF dam foundation and the runoff/seepage collection ditch (Wardrop, 2009b). The muskeg/peat excavated from the downstream part of the TWRMF dam foundation will be disposed of in the Overburden Disposal Facility (ODF). The muskeg/peat removal will require prior excavation of a system of drainage ditches reporting to the collection ditch that will coincide with the future runoff/seepage collection ditch located immediately east of the eastern side of the future TWRMF dam.

The system of drainage ditches will excavated in the winter as the frozen top of the muskeg will facilitate movement of construction equipment. The rate/depth of frost penetration may also be accelerated by snow removal in the construction area (Wardrop, 2009b).

Preliminary rockfill gradation specifications for fine and coarse rockfill for the TWRMF dam are outlined below. Boundaries for rockfill and other filling materials in the TWRMF dam are illustrated in Figures 2.13-2 and 2.13-3.

2.13.7.1 Coarse Rockfill (Zone 1)

Dolomite waste rock from the open pit will be the source of coarse rockfill (Zone 1 material) for the construction of the TWRMF dam and dolomite rockfill base for the ultramafic waste rock rind and north and south causeways construction. Grading requirements for the coarse rock material are shown in Table 2.13-7.

2.13.7.2 Fine Rockfill (Zone 2)

Filter criteria were used to determine the rockfill (Zone 2 material) gradations presented in Table 2.13-8. The Zone 2 material will be obtained by primary and secondary crushing of Zone 1 dolomite waste rock.

Table 2.13-7 Gradation Requirements – Coarse Rockfill (Zone 1)

Dimension or U.S. Standard Sieve Size (mm)	% Passing by Weight
810	100
450	60-100
200	37-100
130	25-60
75	10-45
25	0-15
#4	0

Source: Wardrop, 2009b

2.13.7.3 Sand and Gravel (Zone 2A)

There are no known natural sources of sand and gravel within economic distances for the Minago project. Therefore, Zone 2A material may have to be obtained by further crushing some of the Zone 2 dolomite rockfill.

Table 2.13-8 Gradation Requirements – Fine Rockfill (Zone 2)

Dimension or U.S. Standard Sieve Size (mm)	% Passing by Weight
75	100
50	90-100
30	60-100
25	54-100
19	46-60
#4	10-22
#8	0-7
#16	0

Source: Wardrop, 2009b

2.13.7.4 Clay (Zone 3)

Clay (Zone 3 material) that will be used in the upstream TWRMF dam liner and in cut-off trenches will be initially obtained from local borrow sources within the uppermost “drier” clay. This may be replaced with clay deposited in the ODF, if it is of suitable quality.

2.13.8 TWRMF Associated Facilities

Runoff Diversion Berm

Surface water runoff will be diverted away from the TWRMF by the construction of a runoff diversion berm along its western and eastern sides. Diverting surface water will decrease the amount of water entering the system. The diversion berm will be constructed using peat and clay from the excavaton of the runoff collection system.

Runoff and Seepage Collection System

The runoff and seepage collection system will collect seepage and precipitation that falls on and near the TWRMF dam. Runoff will be collected in ditches built around the entire perimeter of the TWRMF and directed to two existing ponds, located to the northeast and southeast of the dam. Water reporting to the ponds will be pumped back to the TWRMF. Figure 2.13-2 shows the plan view of ditches and ponds that will make up the runoff and seepage collection system.

The western and eastern sides of the TWRMF will have ditch inverts sloped at 0.10%. The flow divide will be at the mid point of the western and eastern sides of the facility from where the water will be diverted north and south. The northern and southern ditches will slope at 0.15% and report to the southern and northern collection ponds (Wardrop, 2009b).

The base of eastern and western ditches will be 1.5 m wide and this will increase to 2.5 m for northern and southern ditches. Ditches will be have side slopes of 2.5H:1V and 4H:1V in native clays and peat, respectively. There will be a 0.5 m setback at the peat and clay interface. All of the ditches will be designed to have freeboard within peat without erosion protection for the design ditch invert slopes (Wardrop, 2009b).

2.13.9 Pertinent Precedents

Based on Wardrop's knowledge, there is no direct long-term precedent for a combined waste rock and tailings disposal for geographic and climatic conditions similar to Minago.

The importance of a lack of a directly-related precedent for the Co-Disposal Scheme involving PAG/ML mine waste rock and tailings in a single repository must be recognized. The handling of geochemical, environmental, and permitting issues associated with the co-disposal scheme has been developed through the incorporation of the combined experience from a variety of operations listed below (Wardrop, 2009b).

- Algoma Ore Properties, Wawa, ON (Tailings transport into unfiltered rock fill by through flow).
- Mines Gaspe Ltd., Murdochville, Que (Tailings transfer into unfiltered rock fill by static liquefaction).
- Vale Inco Limited, Sudbury, ON (Densification of tailings by blasting).
- Falcondo, Dominican Republic (Silt transport into voids of slag fill dam by through flow).
- Syncrude Canada, AB (Dredging experiments using tailings fines as the dredging fluid).
- Giant Mine, Yellowknife, NT (Tailings transport into unfiltered rock fill by through flow).

The post closure environmental considerations and costs for water treatment in perpetuity dictated the selection of co-disposal of PAG/ML waste rock and tailings in a single repository. Co-disposal of tailings and PAG-waste rock will fully contain them behind a perimeter dam to be constructed as a part of a robust operation.

2.13.10 TWRMF Dam Classification

Dam classification in accordance with the Canadian Dam Association Dam Safety Guidelines 2007 (CDA) is based on the evaluation of the consequences of dam failure in terms of risk to population, loss of life, and environmental, cultural, and economic losses. The TWRMF dam can be classified as "Significant Dam Class" and the selection of the hydrology, hydrotechnical and seismic design criteria presented in previous sections were selected in accordance with the CDA criteria considering the following:

- Dam is located in an unpopulated area of Manitoba, relatively far away from urban settlements.

- During the life of the mine, only personnel required for the operation of the mine will be temporarily resident near the mine.
- The temporary housing to accommodate the personnel of the mine and the infrastructure for the processing of the ore will be located at a distance of approximately 2 km from the TWRMF dam.
- Co-disposal of rockfill and tailings provides additional reinforcement of the dam structure which minimizes potential of a dam breach resulting in uncontrolled discharge of tailings towards to the open pit (to the southwest) or Highway 6 (to the east) of the TWRMF.

2.13.11 TWRMF Closure Considerations

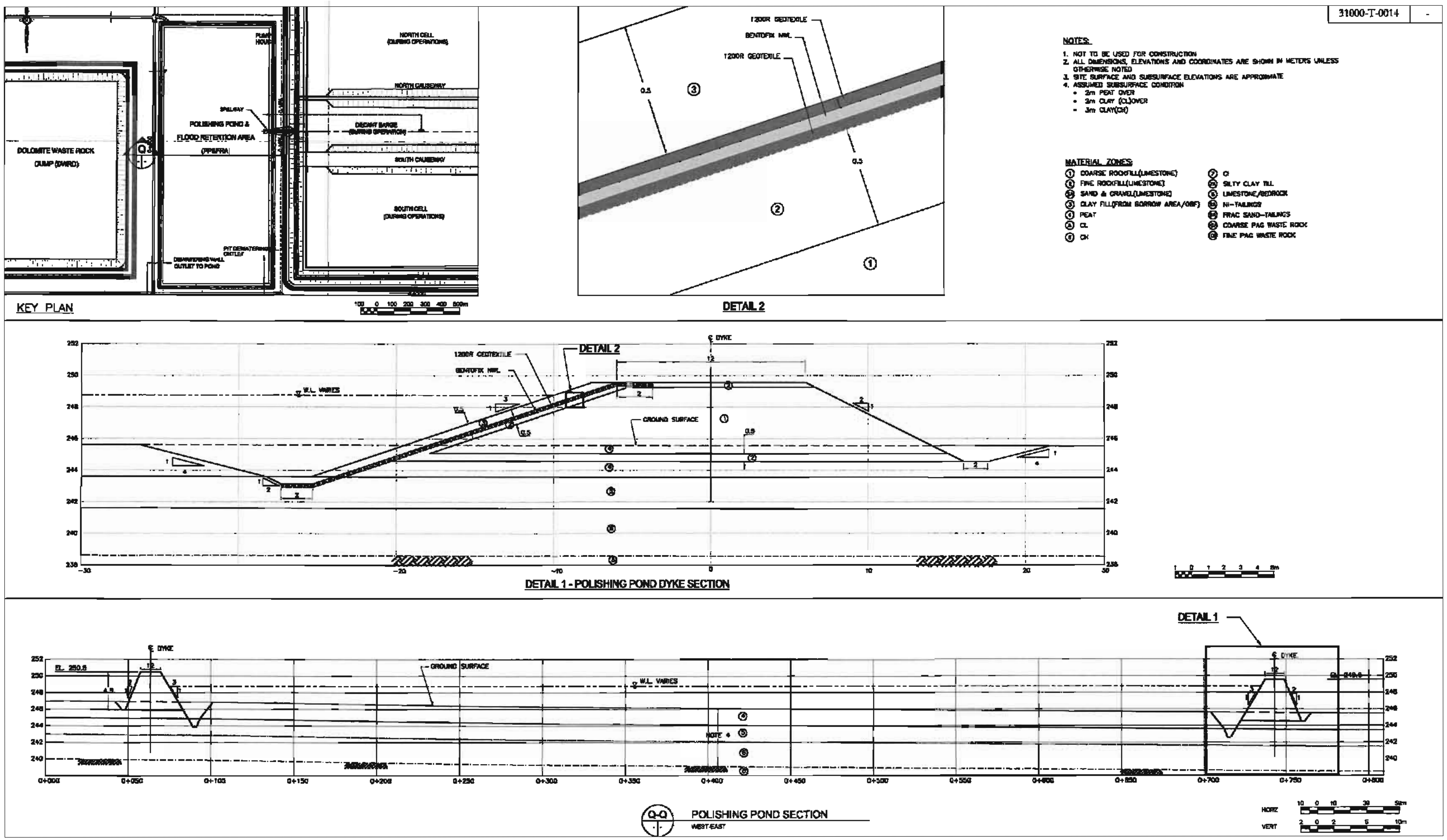
TWRMF closure aspects are covered in a separate report on closure.

2.13.12 Polishing Pond

Water in the Polishing Pond will be contained by a perimeter dyke. The plan view, section view and detail of the Polishing Pond dyke are shown in Figure 2.13-11. The dyke is designed as an earth/rock fill structure varying in height from 4.0 m to 6.0 m above the local topography. The upstream and downstream embankment slopes will be 3H:1V and 2H:1V, respectively. The dyke is scheduled to be raised in 2011 prior to the end of the dredging operations to receive water from the open pit dewatering coinciding with the last phase of the dredging operations.

The main rock fill zone (Zone 1) of the Polishing Pond dyke will be composed of 800 mm minus coarse rock, supporting a 0.5 m thick zone of 75 mm minus fine rock fill, which in turn, will support Geotextile 1200R and a Bentofix liner (Wardrop, 2009b). A 0.5 m clay cover over the Bentofix will be provided for confinement and frost protection. The geotextile at the base of the dyke along with the fine rock fill is designed to prevent migration of fines from the foundation soils into the coarse rock fill. A 0.3 m thick pavement surface, composed of fine rock fill, will be provided over the crest of the embankment (Wardrop, 2009b).

An anchor trench around the upstream toe of the Polishing Pond dyke will be extended approximately 0.5 m into the native clay to ensure full containment of the stored water. The Bentofix liner will be anchored into the trench and backfilled with locally available clay. Dewatering of the cut-off trench may be required to facilitate the installation of the Bentofix liner under dry conditions (Wardrop, 2009b). The dyke embankment will be constructed using rockfill originating from the neighboring limestone bluff located about 2 km west of the Polishing Pond.



Source: adapted from Wardrop's drawing 0951330400-DWG-T0014 (Wardrop, 2009b)

Figure 2.13-11 Polishing Pond Plan and Sections