APPENDIX A

REPORT ON MODELLING DELORO MINE INDUSTRIAL AREA REMEDIATION SCENARIOS

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1. Introduction

In 2003, CH2M HILL developed a groundwater flow model for the Deloro Mine Site (Deloro Mine Site Cleanup, Groundwater Flow Simulation [CH2M HILL, July 2003]). The model was developed using available information from borehole logs and test pits constructed at the mine site. The groundwater flow modelling simulated existing groundwater conditions in the Industrial Area. Groundwater levels in the monitoring wells and flows to the existing groundwater pumping stations were used as calibration data.

Several remediation scenarios for the Industrial Area were evaluated with the model, which incorporated an engineered composite cover with and without a geosynthetic clay liner (GCL) to minimize infiltration of precipitation through the waste into the groundwater. Although the addition of the GCL almost eliminated infiltration through the engineered cover, the groundwater levels did not change significantly from the scenario with the engineered cover without the GCL. As the engineered cover did not accomplish the objective of dewatering of the surficial deposits, several scenarios involving drains and pumping wells were also evaluated with the model.

A passive scenario is preferred over active pumping because it supports the site-wide closure objective of minimizing perpetual operation and maintenance. As such, a horizontal drain installed 2 m below the bedrock surface and extending from the north end of the engineered cover, south to the Moira River, was determined to be the preferred option, based on the 2003 modelling results. The modelling indicated that, if implemented, this option would dewater most of the overburden under the engineered cover. However, the modelling indicated that the overburden deposits under the southern part of the engineered cover, south of the east-west site access road, would not be completely dewatered by the drain. As a result, the modelling report recommended moving the waste and engineered cover to the north of the east-west site access road, to the area where dewatering of the overburden was determined to be more effective, based on the results of the modelling. (This has been addressed in Section 3.4.1 [see Figure 3-5] of the attached Industrial Area Closure Plan).

Evaluations undertaken subsequent to the completion of the 2003 modelling report indicated that constructing a drain 2 m below the bedrock surface would provide significant construction challenges. This is due to fluctuations in the elevation of the bedrock surface; the variable thickness of overburden above the bedrock (ranging from less than 1 m to more than 6 m along the proposed alignment of the drain);and the need for significant dewatering during construction. In addition, the available area is limited and may be insufficient to maintain stable side slopes of the drain in overburden during construction.

The previously developed model for the Deloro Mine Site was used as the basis for modelling a number of new potential groundwater diversion/control scenarios, as part of the remediation of the Industrial Area. The following sections of this report describe the technical approach for updating the model and summarize the modelling results for these new scenarios.

Conclusions and recommendations are also presented at the end of this report. An outline of follow-up work to allow the recommended scenario to be further developed is also included. This will reduce uncertainties in the predicted performance and allow Final Design of the groundwater interception system.

2. Technical Approach

2.1 Updating Model Setup

This section describes the updates/modifications to the original groundwater flow model undertaken as part of the current modelling exercise. For details on the setup and calibration of the original model, refer to CH2M HILL's 2003 modelling report referenced previously.

In the original modelling exercise completed in 2003, some of the available information on ground surface elevations and depths to bedrock at geotechnical boreholes drilled in the Industrial Area were not used (Reference: "Task 423C, Geotechnical Investigation of Subsurface Conditions for the Proposed Groundwater Cross-Flow Interceptor, Technical Memorandum, CH2M HILL, March 28, 2001"). In the 2001 report, elevation data for the ground surface and top of bedrock was derived from an air photo-generated contour map. This data was not used in the original modelling due to concerns about its accuracy. However, in the current modelling exercise, this data was used to update the model in the critical area of the equalization pond and the proposed groundwater interceptor system.

Data from the geotechnical boreholes in the above report were incorporated into the original ground and bedrock surface grid files and re-contoured on a 10-metre grid spacing. These new surfaces were subsequently imported into MODFLOW and the model re-run with the new layer configurations. Due to uncertainties in elevations, the interpolated ground and bedrock surface elevations should be viewed as approximate.

Model calibration was satisfactory (approximately 9.4 percent normalized root mean square error) with the new surface configurations.

In the original model, the preferred remediation scenario used an engineered composite cover without a geosynthetic clay liner (GCL) installed. The updated Industrial Area Closure Plan (*Deloro Mine Site Cleanup – Industrial Area Closure Plan, Final Report* [CH2M HILL, August 2004]) incorporated a GCL as part of the engineered cover in order to reduce the infiltration of precipitation through the waste and reduce leachate generation .

The model was updated to include the GCL as part of the engineered cover. To represent the effect of the GCL in the model, the overall infiltration through the cover was reduced to 1.54 mm/year as determined from the HELP modelling completed as part of the original model development.

As discussed previously, the results of modelling the shallow bedrock groundwater interceptor drain indicated that the southern part of the site under the proposed engineered cover could not be dewatered. The modelling report recommended moving the waste and engineered cover to the north of the east-west site access road. As a result, the southern limit of the engineered cover was moved northward to correspond to the northern edge of the road as shown on Figure 3-12 in the *Industrial Area Closure Plan, Draft Report* (CH2M HILL, March 2004). This revised waste area/cover configuration has been incorporated into the updated model.

In subsequent sections of the report, the updated groundwater flow model is referred to as the base model.

2.2 Modelling Approach

The various remediation scenarios were modelled as steady-state models. The results reflect the steady-state conditions that will prevail once the remedial elements of the scenario have been installed and have been in operation for a period of time. There will be significant transient effects once the remediation scenario has been implemented which will persist until the new groundwater level regime is achieved.

The modelling results were evaluated by comparing the predicted groundwater levels in the deeper overburden under the respective remediation scenario with those of the base model. In modelling the remediation scenarios, it was initially assumed that the primary objective of the remediation scenarios was to completely dewater the surficial deposits (overburden). In order to quickly determine from the modelling results if this criterion had been satisfied, the point of the deepest known bedrock surface elevation under the proposed engineered cover area was identified in the base model. This point is at row 198, column 57 of the model, under the equalization pond, and has been identified as Test Cell 1. The top of bedrock elevation in the model is 187.53 metres above sea level (masl) at this cell. The model-generated groundwater levels and saturated thickness in this cell was compared to this elevation. Saturated thickness maps of the surficial deposits were only generated if the head in the test cell was lower than the top of bedrock.

The hydraulic head in Layer 2 (deep overburden) at Test Cell 1 in the base model is 192.62 masl. This is 5.09 m above the bedrock surface. In some modelling scenarios, two other test cells in addition to Test Cell 1 were identified in Layer 2 in order to compare simulated water levels and saturated thickness with the elevation of the bedrock surface. These additional test cells are identified and evaluated in the appropriate remediation scenario.

Flows to the groundwater pumping stations were used to show the impact on flows to the pumping stations in the base model, and the effect of each respective remediation scenario, compared to existing site conditions. Less flow to the pumping stations was interpreted to represent a more effective remediation scenario. Once steady-state conditions with the best remediation scenario are achieved, the pumping stations may not need to operate. The comparisons were developed to demonstrate the relative effectiveness of the remediation scenarios.

The water budget for existing conditions, as given in the Deloro Mine Site Cleanup, Groundwater Flow Simulation (CH2M HILL, July 2003), shows the simulated flows to the pumping stations. Water budget zones were originally developed in the model as MODFLOW zone budget zones and are consistent throughout the following report. These simulated flows are duplicated in Table 1 for reference.

TABLE 1
WATER BUDGET ZONES FOR PUMPING STATIONS, EXISTING CONDITIONS

Zone	Pumping Station	Existing Conditions Flow (m^3/day)
Zone 3	PS3	112.7
Zone 7	PS1 and PS2	15.0
Zone 8	PS4	86.7
Zone 9	PS5	40.7

Groundwater pathline modelling was used to evaluate the origin of groundwater entering the wells or drains included in some of the scenarios and the relative effectiveness of the respective remediation scenario. The pathlines defined the capture zone of the well or drain, and provided a general indication of the proportion of contaminated versus uncontaminated water that would likely enter the well or drain.

The following sections of this report describe modelling results for a series of remediation scenarios with the updated groundwater flow model (i.e. base model).

3. Updated Model With Engineered Cover Including GCL (Base Model)

The piezometric surface for the updated model (base model) deeper overburden, Layer 2, is shown in Figure 1 (see Appendix A.1). This piezometric surface includes the influence of reduced recharge under the engineered cover with the GCL. This figure shows that groundwater flows from the northwest to southeast to the Moira River. Within the Industrial Area, the groundwater elevation within the deeper overburden ranges from approximately 197 masl to approximately 188 masl, indicating that a relatively steep horizontal hydraulic gradient exists beneath the Industrial Area.

The predicted groundwater flows to the pumping stations for the base model (engineered cover with GCL) are summarized in Table 2. This table shows that the engineered cover with the GCL is predicted to significantly reduce the flows to the pumping stations.

TABLE 2
PREDICTED PUMPING STATION FLOWS FOR BASE MODEL

Zone	Pumping Station	Existing Conditions Flow (m^3/day)	Predicted Base Model Flow (m^3/day)	Base Model Percent of Existing Conditions Flow
Zone 3	PS3	112.7	72.5	64.3
Zone 7	PS1 and PS2	15.0	3.0	20.0
Zone 8	PS4	86.7	38.3	44.2
Zone 9	PS5	40.7	18.1	44.5

4. Remediation Scenarios

Several different remediation scenarios were constructed and evaluated using the base model, which included the engineered cover with GCL. These include:

1. Base model with a perimeter drain with a maximum depth of 4.9 m around the north, west and south sides of the revised engineered cover footprint.
2. A grout curtain/impermeable wall extending 2 m into bedrock on the north, west and south sides of the engineered cover.
3. A grout curtain/impermeable wall extending 2 m into bedrock on the north and west sides of the engineered cover (referred to as the Partial Wall).
4. A (Partial Wall) grout curtain/impermeable wall to 2 m depth with the interceptor drain included with the same configuration as in Scenario 1.
5. A (Partial Wall) grout curtain/impermeable wall to 6 m depth with the interceptor drain included with the same configuration as in Scenario 1.
6. Deep interceptor wells on the north and west sides of the engineered cover installed to 30 m.
7. A (Partial Wall) grout curtain/impermeable wall to 2 m depth with the deep interceptor wells as in Scenario 6.
8. A horizontal well with 0.5 percent slope which mimics the shape of the engineered cover on the north and west sides and extends southward to the natural marsh approximately 175 m south of the site access road.
9. A horizontal well with 0.5 percent slope which extends north to the area west of the Tuttle Shaft with no east-west extensions.
10. Scenario 9 with (partial) grout curtain/impermeable wall to 2 m below bedrock surface.
11. A “level” horizontal well in the bedrock along the west side of the site, which extends from north of the Tuttle Shaft south to the east-west site access road. From there, a HDPE pipe conveys the water south to its outlet at the Moira River at the southwest property line.
12. Scenario 11 with eight vertical pressure relief wells spaced adjacent to the engineered cover which tap the deeper bedrock and flow to the horizontal well by artesian pressure. This horizontal well trends north-south and extends to the area just north of the Tuttle Shaft.

For the modelling scenarios, it was assumed that the horizontal wells would be constructed entirely in the bedrock.

4.1 Scenario 1: Base Model with a Perimeter Drain with Maximum Depth of 4.9 m Around the North, West, and South Sides of the Revised Engineered Cover Footprint

For Scenario 1, a perimeter drain with varying slopes was incorporated into the base model. In the model, this is represented as a drain where the elevation input for each drain cell controls the water levels outside the drain.

The drain was located just outside the limits of the engineered cover on the north, west and south sides. The maximum depth of the drain was 4.9 m below the ground surface (entirely within overburden). The location of the drain is shown in Figure 2 (see Appendix A.1).

The predicted steady state piezometric surface in Layer 2, deeper overburden, with the drain in operation is shown in Figure 2. Steady-state flow in the drain is 60.9 m^3/day. Modelling results for Scenario 1 indicate minimal lowering of the groundwater surface as a result of the drain's operation. The predicted hydraulic head in Test Cell 1 in Layer 2 is 192.34 masl, which is 0.28 m lower than the base model condition for this cell. The saturated thickness of the surficial deposits is 4.81 m in the test cell.

The predicted flows to the pumping stations under this scenario are summarized in Table 3. As shown in Table 3, the model predicts no significant reduction in flows under this scenario.

TABLE 3
PREDICTED PUMPING STATION FLOWS, SCENARIO 1

Zone	Pumping Station	Existing Conditions Flow (m^3/day)	Predicted Base Model Flow (m^3/day)	Base Model Percent of Existing Conditions Flow	Predicted Scenario 1 Flow (m^3/day)	Scenario 1 Percent of Existing Conditions Flow
Zone 3	PS3	112.7	72.5	64.3	73.0	64.8
Zone 7	PS1 and PS2	15.0	3.0	20.0	2.9	19.3
Zone 8	PS4	86.7	38.3	44.2	38.5	44.4
Zone 9	PS5	40.7	18.1	44.5	18.1	44.5

4.2 Scenario 2: A Grout Curtain/Impermeable Wall Extending 2 m into Bedrock on the North, West, and South Sides of the Engineered Cover

For Scenario 2, a grout curtain/impermeable wall extending 2 m into bedrock on the north, west and south sides of the engineered cover was incorporated into the base model. The grout curtain/impermeable wall combination was simulated using the wall package in MODFLOW. The location of the wall is shown on Figure 3 (see Appendix A.1).

The wall was assigned a permeability of 1 x 10^{-8} cm/sec and a thickness of 10 cm. In practice, the grout curtain will likely be thicker than 10 cm, especially in the bedrock where the grout will likely spread along fractures. The grout curtain was extended to the bottom of Layer 3, which is 2 m thick and is the top layer of the bedrock.

Figure 3 shows the predicted steady state piezometric surface in Layer 2, deeper overburden. Modelling results for Scenario 2 indicate installation of the wall results in minimal lowering of the groundwater surface.. The predicted hydraulic head in Test Cell 1 in Layer 2 is 192.03 masl, which represents a head drop of 0.59 m from the base model. The saturated thickness of the surficial deposits at the test cell is 4.5 m. It was determined that the south section of the wall, from where it intersects the east-west site access road, to the river, is detrimental since heads on the north side of the wall tend to be higher than heads on the south side of the wall.

The predicted flows to the pumping stations under this scenario are summarized in Table 4. These values are almost identical to flows in the base model with the exception of a 12.7 percent drop in PS3.

TABLE 4
PREDICTED PUMPING STATION FLOWS, SCENARIO 2

Zone	Pumping Station	Existing Conditions Flow (m^3/day)	Predicted Base Model Flow (m^3/day)	Base Model Percent of Existing Conditions Flow	Predicted Scenario 2 Flow (m^3/day)	Scenario 2 Percent of Existing Conditions Flow
Zone 3	PS3	112.7	72.5	64.3	63.3	56.2
Zone 7	PS1 and PS2	15.0	3.0	20.0	2.9	19.3
Zone 8	PS4	86.7	38.3	44.2	38.5	44.4
Zone 9	PS5	40.7	18.1	44.5	18.1	44.5

4.3 Scenario 3: A Grout Curtain/Impermeable Wall Extending 2 m into Bedrock on the North and West Sides of the Engineered Cover (referred to as the Partial Wall)

For Scenario 3, a grout curtain/impermeable wall extending 2 m into bedrock on the north and west sides of the engineered cover (referred to as the Partial Wall) was incorporated into the base model. This scenario is similar to Scenario 2, except that the southern extension of the wall in Scenario 2 (which bounds the south part of the engineered cover from just north of where the wall intersects the east-west site access road to the Moira River) has been removed.

Figure 4 (Appendix A.1) shows the extent of the wall/grout curtain and the predicted steady state piezometric surface in Layer 2, deeper overburden for Scenario 3. Modelling results for Scenario 3 indicate installation of the wall results in limited lowering of the groundwater surface. The predicted hydraulic head in Test Cell 1 in Layer 2 is 191.91 masl, which represents a head drop of 0.71 m from the base model. This gives a saturated thickness of 4.38 m in the surficial deposits in the test cell.

The predicted flows to the pumping stations under this scenario are summarized in Table 5. As shown in Table 5, the predicted flows are very similar to the base model.

TABLE 5
PREDICTED PUMPING STATION FLOWS, SCENARIO 3

Zone	Pumping Station	Existing Conditions Flow (m^3/day)	Predicted Base Model Flow (m^3/day)	Base Model Percent of Existing Conditions Flow	Predicted Scenario 3 Flow (m^3/day)	Scenario 3 Percent of Existing Conditions Flow
Zone 3	PS3	112.7	72.5	64.3	71.6	63.5
Zone 7	PS1 and PS2	15.0	3.0	20.0	2.8	18.7
Zone 8	PS4	86.7	38.3	44.2	38.5	44.4
Zone 9	PS5	40.7	18.1	44.5	18.1	44.5

4.4 Scenario 4: A (Partial Wall) Grout Curtain/Impermeable Wall to 2 m Depth with the Interceptor Drain Included with the Same Configuration as in Scenario 1

For Scenario 4, a grout curtain/impermeable wall extending 2 m into bedrock on the north and west sides of the engineered cover (Partial Wall), with an interceptor drain included with the same configuration as in Scenario 1, was incorporated into the base model. In this scenario, the wall has the same configuration as Scenario 3, but is extended south to the east-west site access road.

Figure 5 (see Appendix A.1) shows the extent of the wall/grout curtain, the interceptor drain and the predicted steady state piezometric surface in Layer 2, deeper overburden for Scenario 4. The predicted flow in the interceptor drain is 81.5 m^3/day. Modelling results for Scenario 4 indicate installation of the wall and drain result in limited lowering of the groundwater surface. The predicted hydraulic head in Test Cell 1 in Layer 2 is 191.76 masl, a drop of 0.86 m from the base model. The saturated thickness of the surficial deposits is 4.23 m in the test cell.

The predicted flows to the pumping stations under this scenario are summarized in Table 6. As shown in Table 6, the predicted flows are very similar to the base model.

TABLE 6
PREDICTED PUMPING STATION FLOWS, SCENARIO 4

Zone	Pumping Station	Existing Conditions Flow (m^3/day)	Predicted Base Model Flow (m^3/day)	Base Model Percent of Existing Conditions Flow	Predicted Scenario 4 Flow (m^3/day)	Scenario 4 Percent of Existing Conditions Flow
Zone 3	PS3	112.7	72.5	64.3	71.3	63.3
Zone 7	PS1 and PS2	15.0	3.0	20.0	2.7	18.0
Zone 8	PS4	86.7	38.3	44.2	38.5	44.4
Zone 9	PS5	40.7	18.1	44.5	18.1	44.5

4.5 Scenario 5: A (Partial Wall) Grout Curtain/Impermeable Wall to 6 m Depth with the Interceptor Drain Included with the Same Configuration as in Scenario 1

For Scenario 5, a grout curtain/impermeable wall extending 6 m into bedrock on the north and west sides of the engineered cover (Partial Wall), with an interceptor drain included with the same configuration as in Scenario 1, was incorporated into the base model. In this scenario, the wall has the same configuration as Scenario 4, but is installed deeper into the bedrock.

Figure 6 (see Appendix A.1) shows the extent of the wall/grout curtain, the interceptor drain and the predicted steady state piezometric surface in Layer 2, deeper overburden, for Scenario 5. The predicted flow in the perimeter interceptor drain is 86.2 m^3/day. Modelling results for Scenario 5 indicate installation of the wall and drain result in limited lowering of the groundwater surface. The predicted hydraulic head in Test Cell 1 in Layer 2 is 191.87 masl. This represents a decline of 0.75 m from the base model. The saturated thickness in the test cell is 4.34 m.

The predicted flows to the pumping stations under this scenario are summarized in Table 7. As shown in Table 7, there is a 13.8 percent decline in the flow to PS3, but the other flows are similar to the base model.

TABLE 7
PREDICTED PUMPING STATION FLOWS, SCENARIO 5

Zone	Pumping Station	Existing Conditions Flow (m^3/day)	Predicted Base Model Flow (m^3/day)	Base Model Percent of Existing Conditions Flow	Predicted Scenario 5 Flow (m^3/day)	Scenario 5 Percent of Existing Conditions Flow
Zone 3	PS3	112.7	72.5	64.3	62.5	55.5
Zone 7	PS1 and PS2	15.0	3.0	20.0	2.9	19.3
Zone 8	PS4	86.7	38.3	44.2	38.5	44.4
Zone 9	PS5	40.7	18.1	44.5	18.1	44.5

It is apparent from the modelling of Scenarios 1 to 5 that a grout curtain/wall by itself, or with a relatively shallow perimeter drain, is not sufficient to achieve dewatering of the surficial deposits.

4.6 Scenario 6: Deep Interceptor Wells on the North and West Sides of the Engineered Cover Installed to 30 m

For Scenario 6, wells were input into the base model and the number of wells and their pumping rates were varied until the piezometric surface of the deeper overburden under the engineered cover was lower than the bedrock surface. Three wells were required, one on the northeast side, one on the northwest corner of the engineered cover, and the third west of the equalization pond. The locations of the wells are shown in Figure 7 (see Appendix A.1). Pumping rates of the wells were 100, 150, and 450 m^3/day, respectively.

For this scenario, two additional test cells were added in the analysis in order to compare the hydraulic heads with the top of bedrock. Test Cell 2 is on the southwest edge of the equalization pond and Test Cell 3 is half way between the northern edge of the equalization pond and the primary treatment building.

Figure 7 shows the predicted steady state piezometric surface in Layer 2, deeper overburden, for Scenario 6. The predicted hydraulic head in Test Cell 1 in Layer 2 is 187.51 masl, which is 2 cm below the bedrock surface in the cell. The predicted hydraulic head at Test Cell 2 is 187.87 masl and the top of bedrock is 187.79 masl giving a saturated thickness of 8 cm. The predicted hydraulic head in Test Cell 3 is 188.58 masl and the top of bedrock is 188.42 masl, resulting in a saturated thickness in this area of 16 cm.

Figure 8 (see Appendix A.1) shows the predicted distribution of saturated thickness of the surficial deposits. With this configuration of wells pumping, most of the surficial deposits below the engineered cover would be dewatered. Given the uncertainty in obtaining accurate elevations of the top of bedrock, these predicted values of saturated thickness should be regarded with caution. There is a strip of overburden on the west bank of the Moira River that remains saturated under this remediation scenario. This strip of saturated overburden is present in all the remediation scenarios where complete dewatering of the overburden is obtained. Therefore, any leachable waste in this area has to be excavated and moved westward. The eastern limit of the proposed engineered cover will also have to be moved westward to correspond with the zero saturated thickness line.

Changing the depths of the wells to 15 m in the model had no effect on the resultant hydraulic heads. This is because the model is an equivalent porous media model and the deep bedrock is represented as one layer.

The predicted flows to the pumping stations under this scenario are summarized in Table 8. As shown in Table 8, there is a significant reduction in flow to all of the pumping stations compared to the base model. The pumping wells for PS4 and PS5 were made inactive in the model (no pumping) after it was determined that the overburden was dewatered in the area of these wells. The table reflects no pumping from these wells.

TABLE 8
PREDICTED PUMPING STATION FLOWS, SCENARIO 6

Zone	Pumping Station	Existing Conditions Flow (m^3/day)	Predicted Base Model Flow (m^3/day)	Base Model Percent of Existing Conditions Flow	Predicted Scenario 6 Flow (m^3/day)	Scenario 6 Percent of Existing Conditions Flow
Zone 3	PS3	112.7	72.5	64.3	52.7	46.8
Zone 7	PS1 and PS2	15.0	3.0	20.0	1.3	8.7
Zone 8	PS4	86.7	38.3	44.2	0	0
Zone 9	PS5	40.7	18.1	44.5	0	0

Reverse particle tracking was carried out to determine the origin of the groundwater entering the interceptor wells. The predicted reverse groundwater flow pathlines in the deeper overburden are shown in Figure 7. It is apparent from this figure that most of the groundwater captured by these wells originates from the west and north of the Industrial Area. A few pathlines cross the area of the engineered cover in the shallow and deep bedrock. The results of the particle tracking indicate that the groundwater pumped by the wells should be relatively uncontaminated by the wastes in the Industrial Area.

4.7 Scenario 7: A (Partial Wall) Grout Curtain/Impermeable Wall to 2 m Depth with the Deep Interceptor Wells as in Scenario 6

For Scenario 7, three pumping wells were used in the base model at the same locations as in Scenario 6. This scenario also included a grout curtain/impermeable wall (Partial Wall) to 2 m depth in the bedrock, in the same configuration as Scenario 4. Figure 9 (see Appendix A.1) shows the locations of the interceptor wells and grout curtain/wall. The pumping rate for the well just west of the equalization pond was reduced to 425 m^3/day.

Figure 9 shows the predicted steady state piezometric surface for Layer 3, the top of the bedrock, for Scenario 7. The predicted hydraulic head in Test Cell 1 in Layer 2 is 187.43 masl which is 10 cm below the bedrock surface in this cell. The predicted hydraulic head in Test Cell 2 is 187.64 masl, which is 15 cm below the bedrock surface. The hydraulic head in Test Cell 3 is 188.05 masl which is 37 cm below the bedrock surface. Thus, the difference in water levels between Scenario 6 and Scenario 7 for Test Cells 1, 2, and 3 is 8, 23, and 53 cm lower, respectively, for Scenario 7.

The modelling indicated that a strip of overburden along the west bank of the Moira River would remain saturated under this remediation scenario. Therefore, if this scenario is implemented, any leachable waste in this area that is in contact with the groundwater, should be excavated and moved westward. The eastern limit of the proposed engineered cover should also be moved westward to correspond with the zero saturated thickness line.

The predicted flows to the pumping stations under this scenario are summarized in Table 9. As shown in Table 9, there is a significant reduction in flow to all of the pumping stations compared to the base model. Flow to PS3 is slightly higher in this scenario than in Scenario 6. The pumping wells for PS4 and PS5 were made inactive in the model (no pumping) after it was determined that the overburden was dewatered in the area of these wells. The table reflects no pumping from these wells.

TABLE 9
PREDICTED PUMPING STATION FLOWS, SCENARIO 7

Zone	Pumping Station	Existing Conditions Flow (m^3/day)	Predicted Base Model Flow (m^3/day)	Base Model Percent of Existing Conditions Flow	Predicted Scenario 7 Flow (m^3/day)	Scenario 7 Percent of Existing Conditions Flow
Zone 3	PS3	112.7	72.5	64.3	58.3	51.7
Zone 7	PS1 and PS2	15.0	3.0	20.0	1.2	8.0
Zone 8	PS4	86.7	38.3	44.2	0	0
Zone 9	PS5	40.7	18.1	44.5	0	0

Reverse particle tracking was carried out for this scenario to determine the source of groundwater for the interceptor wells. Figure 9 shows the predicted reverse groundwater flow pathlines in the top of bedrock. It is apparent from this figure that the majority of groundwater flow to the interceptor wells originates from the north and west of the Industrial Area. Only a few pathlines cross under the engineered cover in the shallow bedrock indicating that the groundwater pumped by the wells should be relatively uncontaminated by the wastes in the Industrial Area.

4.8 Scenario 8: Horizontal Well with 0.5 Percent Slope Which Mimics the Shape of the Engineered Cover on the North and West Sides and Extends Southward to the Natural Marsh Approximately 175 m South of the Site Access Road

For Scenario 8, a horizontal well was incorporated into the base model in the bedrock. Initially, it was decided to model the horizontal well (drain) with approximately the same layout as in the original modelling completed in 2003 (deep interceptor trench) to evaluate the similarities between the two results. The drain in the original model was set at a varying elevation which was keyed to the bedrock surface, at a depth of 2 m below the bedrock surface. In some areas this resulted in a very deep drain.

For this scenario, the drain was placed in the same position along the northern edge of the engineered cover, but was moved west of New Westerly Creek on the west side of the engineered cover area (Figure 10 [see Appendix A.1]). A drain slope of 0.5 percent was used and the highest elevation of the drain, in this case simulating a horizontally drilled “well”, was initially at the northeast corner of the engineered cover near borehole GA12-1.

Several simulations were required to determine the optimum elevation of the drain. In order to dewater the surficial deposits, the elevation of the drain in the same model row as Test Cell 1 had to be 184.21 masl. The model ground surface in this drain cell is at 200.7 masl, while the top of bedrock is at 187.3 masl.

The modelling results also indicated that the drain needed to be extended on the northeast side of the engineered cover in a direction southeast to the vicinity of borehole GR4-1, in order to achieve sufficient lowering of groundwater levels (Figure 10). The elevation of the drain near GR4-1 is 187.08 masl. The drain slope of 0.5 percent was maintained for this segment of the drain. The end of the drain, near GR4-1, is the drain's highest elevation.

The drain extends southward to the natural marsh approximately 175 m south of the site access road. The drain’s elevation where it ends at the marsh is 182.8 masl, which is 6.0 m below the ground surface. Due to this elevation difference, the water would have to be pumped to the river at this location. The modelling results indicate that a horizontal well could be used, but the elevation of the well would have to be 184.21 masl or less to achieve dewatering of the surficial deposits.

Figure 10 shows the predicted steady state piezometric surface for Layer 3, the top of the bedrock, for Scenario 8. The predicted flow in the horizontal well is 1,557.6 m^3/day. The predicted hydraulic head in Test Cell 1 in Layer 2 is 187.43 m, which is 10 cm below the bedrock surface. The predicted hydraulic heads in Test Cells 2 and 3 are 0.84 and 1.12 m below the bedrock surface, respectively.

The modelling indicated that a strip of overburden along the west bank of the Moira River would remain saturated under this remediation scenario. Therefore, if this scenario is implemented, any leachable waste in this area that is in contact with the groundwater, should be excavated and moved westward. The eastern limit of the proposed engineered cover should also be moved westward to correspond with the zero saturated thickness line.

The predicted flows to the pumping stations under this scenario are summarized in Table 10. As shown in Table 10, there is a significant reduction in flow to all of the pumping stations compared to the base model. The pumping wells for PS4 and PS5 were made inactive in the model (no pumping) after it was determined that the overburden was dewatered in the area of these wells. The table reflects no pumping from these wells.

TABLE 10
PREDICTED PUMPING STATION FLOWS, SCENARIO 8

Zone	Pumping Station	Existing Conditions Flow (m^3/day)	Predicted Base Model Flow (m^3/day)	Base Model Percent of Existing Conditions Flow	Predicted Scenario 8 Flow (m^3/day)	Scenario 8 Percent of Existing Conditions Flow
Zone 3	PS3	112.7	72.5	64.3	43.1	38.2
Zone 7	PS1 and PS2	15.0	3.0	20.0	0.6	4.0
Zone 8	PS4	86.7	38.3	44.2	0	0
Zone 9	PS5	40.7	18.1	44.5	0	0

Reverse particle tracking was carried out for this scenario with particles placed on both sides of the drain and tracked backward to their point of origin. Figure 10 shows the predicted reverse groundwater flow pathlines in Layer 3, the top of the bedrock. As shown in Figure 10, there are few pathlines in the top of the bedrock. There are two groups of flow lines which flow in two directions as influenced by the horizontal well.

Figure 11 (see Appendix A.1) shows the pathlines in the deeper bedrock only (Layer 4). The majority of the pathlines are in the deeper bedrock. Pathlines from the shallow bedrock connect to those in the deeper bedrock. Therefore, if there is some degree of contamination in the shallow bedrock under the engineered cover, it could enter the horizontal well. However, the majority of groundwater flowing to the horizontal well originates from the west and north of the site and so is presumably uncontaminated.

As the overburden is dewatered under the engineered cover, there is no groundwater flow, and therefore, no pathlines in both overburden layers.

4.9 Scenario 9: Horizontal Well with 0.5 Percent Slope Which Extends North to the Area West of the Tuttle Shaft with no East-West Extensions

For Scenario 9, the east-west extension of the horizontal well (drain) just north of the engineered cover that was used in Scenario 8 was removed, and the drain was extended north 196 m to just north of the Tuttle Shaft (Figure 12 [see Appendix A.1]). The elevation of the drain is 186.35 masl at the end of its northern extension.

Figure 12 shows the predicted steady state piezometric surface for Layer 3, the top of the bedrock, for Scenario 9. Figure 13 (see Appendix A.1) shows the piezometric surface for Layer 4, the deeper bedrock. The predicted flow in the horizontal well is 1,916.0 m^3/day. The predicted hydraulic head in Test Cell 1 in Layer 2 is 187.76 masl, which is 23 cm above the bedrock surface. Lowering the drain 0.25 m along its length resulted in a head in Test Cell 1 of 187.46 masl, which is 7 cm below the bedrock surface. The predicted hydraulic head in Test Cell 2 is 187.03 masl which is 76 cm below the bedrock surface. The hydraulic head in Test Cell 3 is 188.04 masl which is 38 cm below the bedrock surface.

Figure 14 (see Appendix A.1) shows the predicted saturated thickness of the deeper overburden for this scenario. This figure shows that the overburden is dewatered under the western part of the engineered cover, but that there is a band of saturated overburden along the western bank of the Moira River. These results suggest that, if this scenario is implemented, the eastern boundary of the engineered cover and any leachable wastes in contact with groundwater in the area, should be moved westward.

The predicted flows to the pumping stations under this scenario are summarized in Table 11. As shown in Table 11, there is a significant reduction in flow to all of the pumping stations compared to the base model. The pumping wells for PS4 and PS5 were made inactive in the model (no pumping) after it was determined that the overburden was dewatered in the area of these wells. The table reflects no pumping from these wells.

TABLE 11
PREDICTED PUMPING STATION FLOWS, SCENARIO 9

Zone	Pumping Station	Existing Conditions Flow (m^3/day)	Predicted Base Model Flow (m^3/day)	Base Model Percent of Existing Conditions Flow	Predicted Scenario 9 Flow (m^3/day)	Scenario 9 Percent of Existing Conditions Flow
Zone 3	PS3	112.7	72.5	64.3	46.1	40.9
Zone 7	PS1 and PS2	15.0	3.0	20.0	1.2	8.0
Zone 8	PS4	86.7	38.3	44.2	0	0
Zone 9	PS5	40.7	18.1	44.5	0	0

Reverse particle tracking was carried out for this scenario with particles placed on both sides of the drain and tracked backward to their point of origin. Figure 12 shows the predicted reverse groundwater flow pathlines in the shallow bedrock. The limited number of pathlines in the shallow bedrock shows that the potential for contamination of the horizontal well due to flow of groundwater from the shallow bedrock under the engineered cover is low.

Figure 13 (see Appendix A.1) shows predicted pathlines in the deeper bedrock. These figures show that most of the groundwater flow to the horizontal well is from the deeper bedrock west of the engineered cover, with a component of flow from the north. The overall potential for contamination of the well from flow of groundwater from under the engineered cover is low.

4.10 Scenario 10: Scenario 9 with (Partial) Grout Curtain/ Impermeable Wall

For Scenario 10, the north-south horizontal well with the same configuration as in Scenario 9 (Figure 12) was incorporated into the base model, along with the partial grout curtain/wall adjacent to the north and west sides of the engineered cover, to 2 m below the bedrock surface, with the same configuration as in Scenario 4 (Figure 5).

The predicted flow in the horizontal well is 1,126.5 m^3/day. The predicted hydraulic head in Test Cell 1 in Layer 2 is 186.75 masl which is 0.78 m below the bedrock surface. The saturated thickness of the overburden in Test Cell 1 is therefore, 0.0m. The saturated thickness of the overburden in Test Cells 2 and 3 are also 0.0m.

The modelling indicated that a strip of overburden along the west bank of the Moira River would remain saturated under this remediation scenario. Therefore, if this scenario is implemented, any leachable waste in this area that is in contact with the groundwater, should be excavated and moved westward. The eastern limit of the proposed engineered cover should also be moved westward to correspond with the zero saturated thickness line.

The predicted flows to the pumping stations under this scenario are summarized in Table 12. As shown in Table 12, there is a significant reduction in flow to all of the pumping stations compared to the base model. The results of the modelling indicate that the incorporation of the grout curtain/wall did not significantly affect the flows to pumping stations 1, 2 and 3, compared to Scenario 9. The pumping wells for PS4 and PS5 were made inactive in the model (no pumping) after it was determined that the overburden was dewatered in the area of these wells. The table reflects no pumping from these wells.

TABLE 12
PREDICTED PUMPING STATION FLOWS, SCENARIO 10

Zone	Pumping Station	Existing Conditions Flow (m^3/day)	Predicted Base Model Flow (m^3/day)	Base Model Percent of Existing Conditions Flow	Predicted Scenario 10 Flow (m^3/day)	Scenario 10 Percent of Existing Conditions Flow
Zone 3	PS3	112.7	72.5	64.3	45.1	40.0
Zone 7	PS1 and PS2	15.0	3.0	20.0	0.85	5.7
Zone 8	PS4	86.7	38.3	44.2	0	0
Zone 9	PS5	40.7	18.1	44.5	0	0

Reverse groundwater flow pathline modelling showed similar results to Scenario 9 without the grout curtain. The modelling results indicate that there are no apparent advantages to installing the grout curtain, with the exception of lowered flow in the horizontal well.

4.11 Scenario 11: "Level" Horizontal Well, from North of the Tuttle Shaft, to the East-West Site Access Road, Connected to a HDPE Pipe with Outlet at the Moira River

For Scenario 11, a "level" horizontal well, from north of the Tuttle Shaft, to the east-west site access road, was incorporated into the base model (Figure 15 [see Appendix A.1]). The horizontal well was connected to a high density polyethylene (HDPE) pipe that extends to the Moira River at the southwestern property boundary. The elevation and slope of this pipe was selected to accommodate the head loss in the pipe from the horizontal well to the outlet at the Moira River, which is at an elevation of 184.66 masl where the river meets the property line.

The head loss for the HDPE pipe was calculated by two methods to be 0.6 and 0.62 m over the 373 m of pipe from the end of the horizontal well to the river. The elevation of the level horizontal well would therefore be 185.3 masl along its length. At the point opposite the centre of the equalization pond, the horizontal well is approximately 4 m below the bedrock surface.

Two additional layers were added to the base model in the deep bedrock for Scenarios 11 and 12. The horizontal well was placed in Layer 4 and the vertical relief wells (Scenario 12 – see below) were placed in the bottom layer, Layer 5.

Figure 15 shows the predicted steady state piezometric surface for Layer 3, the top of the bedrock, for Scenario 11. Figure 16 (see Appendix A.1) shows the predicted piezometric surface for Layer 5, the deeper bedrock, for Scenario 11. These figures show that the predicted hydraulic gradient along the horizontal well results in flow into the horizontal well screen and flow from north to south along the horizontal well alignment.

The predicted flow in the horizontal well is 880.5 m^3/day. The modelling results indicate that the predicted hydraulic head in Test Cell 1 in Layer 2, deeper overburden, is 188.5 masl. The bedrock surface elevation in this cell is 187.53 masl. Therefore, the saturated thickness of the overburden is 0.97 m in Test Cell 1. The saturated thicknesses of the overburden in Test Cells 2 and 3 are 0.48 m and 0.68 m, respectively.

Figure 17 (see Appendix A.1) shows the saturated thickness of the deeper overburden. A small part of the area under the equalization pond has a saturated thickness of 0.5 m or greater. As shown in Figure 17, a strip of overburden along the west bank of the Moira River would remain saturated under this remediation scenario. Therefore, if this scenario is implemented, any leachable waste in this area that is in contact with the groundwater, should be excavated and moved westward. The eastern limit of the proposed engineered cover should also be moved westward to correspond with the zero saturated thickness line.

The predicted flows to the pumping stations under this scenario are summarized in Table 13. As shown in Table 13, there is a significant reduction in flow to all of the pumping stations compared to the base model. Modelling results indicate that the predicted flows to PS3 are higher than in Scenario 9. The pumping wells for PS4 and PS5 were made inactive in the model (no pumping) after it was determined that the overburden was dewatered in the area of these wells. The table reflects no pumping from these wells.

TABLE 13
PREDICTED PUMPING STATION FLOWS, SCENARIO 11

Zone	Pumping Station	Existing Conditions Flow (m^3/day)	Predicted Base Model Flow (m^3/day)	Base Model Percent of Existing Conditions Flow	Predicted Scenario 11 Flow (m^3/day)	Scenario 11 Percent of Existing Conditions Flow
Zone 3	PS3	112.7	72.5	64.3	58.0	51.5
Zone 7	PS1 and PS2	15.0	3.0	20.0	1.49	9.9
Zone 8	PS4	86.7	38.3	44.2	0	0
Zone 9	PS5	40.7	18.1	44.5	0	0

Reverse particle tracking was carried out for this scenario with particles placed on both sides of the drain and tracked backward to their point of origin. There are virtually no pathlines in the deeper overburden under the engineered cover, indicating minimal groundwater flow from the deeper overburden to the horizontal well (not shown).

Figure 15 shows reverse groundwater flow pathlines in the shallow bedrock to the horizontal well. This figure shows almost no groundwater movement to the horizontal well from the shallow bedrock under the engineered cover. Figure 16 shows groundwater flow pathlines in the deeper bedrock, where the horizontal well is located. It is apparent from this figure, that most of the groundwater flow is in the deeper bedrock and originates from the west of the Industrial Area, with a component of flow from the north.

The potential for contamination from the shallow bedrock under the shallow engineered cover entering the horizontal well is low and, as most of the groundwater flow would likely be uncontaminated groundwater from the west and north, the dilution will be high.

The water levels in the municipal Deloro Well and the Tuttle Shaft were compared for existing conditions and Scenario 11. The modelled water levels are shown in Table 14.

TABLE 14
COMPARISON OF MODELLED WATER LEVELS IN DELORO WELL AND TUTTLE SHAFT

Well	Static Water Level Elevation (masl)	Existing Conditions Pumping Water Elevation (masl)	Scenario 11 Pumping Water Level Elevation (masl)
Deloro Well	197.36	194.60	192.94
Tuttle Shaft	194.03	189.89	189.11

The difference in drawdown between the existing conditions pumping level and the Scenario 11 pumping level is 1.66 m in the Deloro Well. This indicates there will still be adequate available drawdown to maintain the village's water needs. The available drawdown under existing conditions is 19.8 m. The available drawdown under this scenario is 18.1 m.

In the Tuttle Shaft, the predicted reduction in groundwater level from existing conditions to Scenario 11 is 0.78 m. This may be significant enough to reduce artesian flow from the Tuttle Shaft; however, more information is needed to determine the impact of this scenario on the Tuttle Shaft flow.

The relative drawdown differences are dependant on the pumping rate. However, given the same pumping rate for existing conditions and Scenario 11 for the Deloro Well and the Tuttle Shaft, the model results should be representative.

4.12 Scenario 12: Scenario 11 with Eight Vertical Pressure Relief Wells in the Deeper Bedrock

For Scenario 12, the horizontal well used for Scenario 11 was included in the base model. The model also incorporated eight pressure relief wells, placed into the deeper bedrock (Figure 18 [see Appendix A.1]) and adjacent to the engineered cover. The vertical bedrock wells were incorporated into this scenario to allow groundwater to flow from the deeper bedrock by artesian pressure to the horizontal well. The eight wells in the deeper bedrock were represented as constant heads at the same elevation of the horizontal well, 185.3 masl.

An indication of the area's available artesian pressure was obtained from the static water level in the municipal Deloro Well, in relation to where the groundwater was encountered. The static groundwater level in this well was reported to be 10.1 m below the top of the well in the well log. The well log indicates that groundwater was encountered at a depth of approximately 30 m. This indicates significant artesian pressure in the area, supporting the hypothesis that deep vertical bedrock wells can be used with a horizontal well to depressurize the deeper bedrock, thereby reducing upward groundwater flow from the bedrock into the overburden beneath the waste area.

Figure 18 shows the predicted steady state piezometric surface for Layer 3, the top of the bedrock, for Scenario 12. Figure 19 (see Appendix A.1) shows the predicted piezometric surface for Layer 5, the deeper bedrock. These figures show that the predicted hydraulic gradient along the horizontal well results in flow into the horizontal well screen and flow from north to south along the horizontal well alignment.

The horizontal drain conveyed 645.3 m^3/day, and the connected vertical wells conveyed 314.2 m^3/day, for a total flow of 959.5 m^3/day. It should be stressed that these are very preliminary numbers and should not be used as the basis for design.

The predicted hydraulic head in Test Cell 1, Layer 2 is 187.60 masl. The bedrock surface elevation in this cell is 187.53 m indicating 7 cm of saturated thickness in the deeper overburden. The hydraulic head in Test Cell 2 is 0.33 m below the bedrock surface. The hydraulic head in Test Cell 3 is 0.15 m below the bedrock surface.

Figure 20 (see Appendix A.1) shows the predicted saturated thickness of the deeper overburden, Layer 2. The figure shows that the deeper overburden under the engineered cover is dewatered, but that a strip of saturated overburden along the west bank of the Moira River remains. If this scenario is implemented, these results suggest that the eastern limit of the engineered cover, and any leachable waste in contact with the groundwater, should be moved westward to the dewatered area.

The modelling results indicate that Scenario 12 has the potential to reduce the flow of groundwater from the overburden and bedrock into the waste area. However, at this time, further modelling to lower the piezometric surface in Test Cell 1 is not warranted. It is apparent that some number of deep vertical relief wells could work. However, further field work is needed to evaluate the hydraulic characteristics of the deep and shallow bedrock along the proposed alignment of the horizontal well.

The individual flow from each of the eight vertical relief wells (assuming equal flow) is 39.3 m^3/day. These yields seem to be achievable when compared to the yield of the Deloro Well. The average pumping rate from the Deloro Well is 76.3 m^3/day, and the Permit to Take Water rate is 327.0 m^3/day, which is indicative of the well's maximum sustainable yield. The well taps a confined aquifer (fracture zone) at approximately 30 m deep, with the static water level at a depth of 10 m.

The predicted flows to the pumping stations under this scenario are summarized in Table 15. As shown in Table 15, there is a significant reduction in flow to all of the pumping stations compared to the base model. Modelling results indicate that the predicted flows to PS3 are higher than in Scenario 9 and lower than in Scenario 11. The pumping wells for PS4 and PS5 were made inactive in the model (no pumping) after it was determined that the overburden was dewatered in the area of these wells. The table reflects no pumping from these wells.

TABLE 15
PREDICTED PUMPING STATION FLOWS, SCENARIO 12

Zone	Pumping Station	Existing Conditions Flow (m^3/day)	Predicted Base Model Flow (m^3/day)	Base Model Percent of Existing Conditions Flow	Predicted Scenario 12 Flow (m^3/day)	Scenario 12 Percent of Existing Conditions Flow
Zone 3	PS3	112.7	72.5	64.3	54.7	48.6
Zone 7	PS1 and PS2	15.0	3.0	20.0	1.18	7.9
Zone 8	PS4	86.7	38.3	44.2	0	0
Zone 9	PS5	40.7	18.1	44.5	0	0

Reverse particle tracking was carried out for this scenario with particles placed on both sides of the drain (and the vertical wells) and tracked backward to their point of origin. Figure 18 shows the predicted reverse groundwater flow pathlines in the shallow bedrock, Layer 3. There are very few pathlines in the shallow bedrock, which indicates almost no groundwater movement to the horizontal well from the shallow bedrock under the engineered cover. Figure 19 shows groundwater flow pathlines in the deeper bedrock, where the horizontal well is located. It is apparent from this figure that most of the groundwater flow is in the deeper bedrock and originates from the west of the Industrial Area, with a component of flow from the north.

With this scenario, there will be some groundwater flow from shallow bedrock under the engineered cover to the deeper bedrock, and then to the horizontal well. Thus, there is some potential for contaminated water to get into the horizontal well. Given that most of the water entering the horizontal well is likely uncontaminated groundwater from the west and north of the site, the dilution factor will likely be high. Further evaluation of this dilution factor is provided below.

The modelled pumping levels in the municipal Deloro Well and the Tuttle Shaft for Scenario 12 are compared to existing conditions and Scenario 11 in Table 16. The pumping water levels are slightly lower for Scenario 12.

TABLE 16
COMPARISON OF MODELLED WATER LEVELS IN DELORO WELL AND TUTTLE SHAFT

Well	Static Water Level Elevation (masl)	Existing Conditions Pumping Water Elevation (masl)	Scenario 11 Pumping Water Level Elevation (masl)	Scenario 12 Pumping Water Level Elevation (masl)
Deloro Well	197.36	194.60	192.94	192.65
Tuttle Shaft	194.03	189.89	189.11	189.00

Figure 21 (see Appendix A.1) shows the difference in piezometric surface of the deeper overburden for existing conditions with the piezometric surface of the same layer for Scenario 12. The predicted changes in groundwater levels increase westerly from a 0.5 m reduction at the Moira River to an 8.5 m reduction at the horizontal well.

Table 17 shows a comparison of calculated water level elevations for existing conditions and Scenario 12 for selected monitoring wells. As shown in Table 17, the predicted reduction in groundwater elevations for these overburden wells ranged from 0.78 m to 6.58 m, and for the bedrock wells from 0.62 m to 5.73 m.

TABLE 17
COMPARISON OF PREDICTED GROUNDWATER ELEVATIONS FOR SCENARIO 12 TO EXISTING CONDITIONS

Groundwater Monitor	Existing Conditions Groundwater Elevation (masl)	Scenario 12 Predicted Groundwater Elevation (masl)	Scenario 12 Predicted Reduction in Groundwater Elevation (masl)
Overburden			
GA-14	188.63	187.85	0.78
GA-8	196.8	192.21	4.59
GA-7	195.29	188.71	6.58
GA-6	193.1	187.76	5.34
GA-3	191.63	187.79	3.84
GA-5	191.56	187.36	4.20
Bedrock			
GA-9	193.77	188.04	5.73
GR4-2	191.05	189.28	1.77
GR3-2	188.92	188.30	0.62
GA12-1	193.57	189.93	3.64
GA2-1	192.45	188.35	4.10
GR3-1	189.3	188.28	1.02
GR-12	191.44	187.29	4.15

The MODFLOW program, Zone Budget, was used to distinguish presumably uncontaminated groundwater flows originating from the west of the site, from groundwater flows originating under the engineered cover which may be affected by leachate from the wastes. Table 18 summarizes the flows estimated from this analysis.

TABLE 18
SUMMARY OF PREDICTED FLOWS USING ZONE BUDGET

Zone	Predicted Flow to Horizontal and Vertical Wells for Scenario 12 (m^3/day)	Predicted Flow from Shallow Bedrock to Deeper Bedrock (m^3/day)
Flow From West in Deeper Bedrock	641.2	
Flow From Under the Engineered Cover in Deeper Bedrock	309.2	
Flow from Shallow Bedrock to Deeper Bedrock From Under the Engineered Cover		73.8
Flow from Shallow Bedrock to Deeper Bedrock to West of Horizontal Well		325.1

In Table 18, the flow from the shallow bedrock to the deeper bedrock under the engineered cover, 73.8 m^3/day, becomes part of the 309.2 m^3/day flowing from under the engineered cover in the deeper bedrock to the horizontal and vertical wells. Similarly, the flow from the shallow bedrock to the deeper bedrock west of the horizontal well, 325.1 m^3/day, becomes part of the 641.2 m^3/day flowing from the deeper bedrock to the horizontal and vertical wells.

The flow from the west in the deeper bedrock is more than twice the flow from under the engineered cover in the deeper bedrock. However, flow from the shallow bedrock to the deeper bedrock is 73.8 m^3/day or 22.7 percent of the flow from the shallow bedrock to the deeper bedrock on the west side of the horizontal well. The dilution for groundwater coming from the shallow bedrock under the engineered cover to the horizontal well and vertical wells (i.e. 73.8 m^3/day) compared to the entire flow in the system (i.e. 950.4 m^3/day) is a factor of 12.9:1.

4.13 Summary of Modelling Results for All Remediation Scenarios

Table 19 provides a summary of the modelling results for each scenario.

TABLE 19
SUMMARY OF REMEDIATION SCENARIO MODELLING RESULTS

Scenario	Description	Flow (m^3/day)	Saturated Thickness Layer 2 (m)			Comments
			Cell 1	**Cell 2**	**Cell 3**	
	Base Model		5.09			
1	Base Model with Shallow Perimeter Drain	Drain 60.9	4.81			Insignificant water level lowering in the overburden
2	Grout Curtain 2 m into Bedrock Around Most of the North, West, and South Sides of the Engineered Cover		4.5			Minor water level lowering in the overburden, grout curtain on the south side is counterproductive
3	Grout Curtain 2 m into Bedrock on North and West Sides of Engineered Cover – Partial Wall		4.38			Minor water level lowering in the overburden
4	Grout Curtain as in Scenario 3 with Shallow Perimeter Drain as in Scenario 1	Drain 81.5	4.23			Minor water level lowering in the overburden
5	Partial Wall to 6 m Depth with Shallow Interceptor Drain	Drain 86.2	4.34			Slightly higher water level in Test Cell 1 than in Scenario 4
6	Deep Interceptor Wells on North and West Sides of Engineered Cover	Well Flow 700	0.0	0.08	0.16	Deeper overburden almost dewatered with this scenario, increasing well yield likely could achieve dewatering
7	Partial Wall to 2 m into the Bedrock with Deep Interceptor Wells	Well Flow 675	0.0	0.0	0.0	Complete dewatering of overburden
8	Horizontal Well 0.5% Slope, on North and West Sides of Engineered Cover, Outlet 175 m South of East-West Access Road at Natural Marsh	Horizontal Well 1,557.6	0.0	0.0	0.0	Necessary to pump water conveyed by horizontal well since outlet is 6 m below river level
9	Horizontal Well from North of Tuttle Shaft, South to Natural Marsh Lake as in Scenario 8	Horizontal Well 1,916.0	0.0	0.0	0.0	Necessary to pump water conveyed by horizontal well since the outlet is 6 m below river level
10	Scenario 9 with Partial Wall to 2 m Below Bedrock Surface	Horizontal Well 1,126.5	0.0	0.0	0.0	Less flow in horizontal well may reflect decreased flow from bedrock under the engineered cover, necessary to pump water from horizontal well to river

TABLE 19
SUMMARY OF REMEDIATION SCENARIO MODELLING RESULTS

Scenario	Description	Flow (m^3/day)	Saturated Thickness Layer 2 (m)			Comments
			Cell 1	Cell 2	Cell 3	
11	Level Horizontal Well from North of Tuttle Shaft to East-West Road on West Side of Engineered Cover, HDPE Pipe Conveys Water to River	Horizontal Well 880.5	0.97	0.48	0.68	Designed to enable gravity flow to river. Approximately 1 m saturated overburden under the equalization pond area. Reduction in pumping water level in Tuttle Shaft of 0.78 m, may reduce artesian flow, drawdown in Deloro Well is 1.66 m, not significant. Relatively little groundwater flow from overburden or shallow bedrock under engineered cover to horizontal well in deeper bedrock.
12	Scenario 11 with Eight Vertical Relief Wells in the Deeper Bedrock Which Allow Water to Flow to the Horizontal Well by Artesian Flow	Horizontal Well 645.3 Vertical Relief Wells (8) 314.2	0.07	0.0	0.0	Almost complete dewatering of overburden. Number of wells needs to be determined from field work. Discharge flows by gravity to Moira River. Pumping water levels in Deloro Well and Tuttle Shaft reduced slightly from previous scenario.

5. Conclusions

Based on the modelling results for the 12 remediation scenarios described in previous sections of the report, the following conclusions are presented:

1. The shallow perimeter drain has little effect on dewatering the overburden.
2. The full or partial grout curtains, by themselves, will not be effective in dewatering the overburden.
3. The full or partial grout curtains combined with the shallow perimeter drain are not effective in dewatering the overburden.
4. The deeper grout curtain to 6 m below the bedrock surface combined with the shallow interceptor drain is not effective in dewatering the overburden.
5. Interceptor wells on the north and west sides of the proposed engineered cover appear to be able to effect dewatering of the overburden under the engineered cover. The pumping rates of the wells for this scenario could be more than 700 m^3/day. Particle tracking indicates that most of the groundwater pumped by the wells originates from the north and west of the site, indicating that the water will likely be relatively uncontaminated.
6. Combining a partial grout curtain to 2 m below the bedrock surface with the interceptor wells will achieve dewatering of the overburden with marginally less water pumped from the wells. In both Scenario 6 and 7, the line of zero saturated thickness adjacent to the west bank of the Moira River is under the eastern edge of the proposed engineered cover. This indicates that the eastern extent of the cover and any leachable waste in contact with the groundwater may have to be moved westward. This is true for the remaining scenarios.
7. A horizontal well of 0.5 percent slope placed in the bedrock just outside the north and west sides of the proposed engineered cover will achieve dewatering of the overburden if the elevation of the well is less than 184.21 masl immediately west of Test Cell 1. Since the horizontal well outlet will be at an elevation of 182.8 masl at its outlet in the natural marsh, 175 m south of the east-west access road, water will have to be pumped to the river. If the horizontal well is level, the well would have to be at an elevation of 184.0 masl to achieve dewatering of the overburden. Pumping would still be needed at the outlet.
8. A north-south trending horizontal well in the bedrock which extends north to the area of the Tuttle Shaft will work just as well as Scenario 8. The elevation of the well at its north end is 186.1 masl and 182.3 masl at the southern extension. Adding a partial grout curtain achieves essentially the same dewatering result, but the flow in the horizontal well is 59 percent of the flow without the grout curtain. Whether the drain is horizontal or sloped, pumping would be needed at the outlet to convey water to the river.

9. A north-south horizontal well in the bedrock, capable of discharging by gravity flow to the Moira River, could achieve almost complete dewatering of the overburden. However, there is an area under the equalization pond that still has some saturated thickness, of up to 1 m in the overburden. There is also a strip of saturated overburden along the west side of the Moira River indicating that the eastern edge of the engineered cover and any leachable waste in contact with the groundwater may have to be moved west.
10. A north-south horizontal well in the bedrock, capable of discharging by gravity flow to the Moira River, combined with vertical pressure relief wells appears to be able to almost completely dewater the overburden. The overburden under the equalization pond, which was not completely dewatered in Scenario 11, is dewatered with the addition of the deep vertical relief wells. As in the previous scenario, a strip of saturated overburden on the west bank of the Moira River remains.
11. The modelling indicates that both Scenarios 11 and 12 have sound potential as preferred scenarios. Both are passive solutions requiring no pumping which could potentially dewater most, or all, of the overburden, if desired. In order to determine the feasibility of these options, and the number of vertical wells required, hydrogeologic field investigations and further modelling will be needed to determine the hydraulic characteristics of the deep and shallow bedrock in the area of the horizontal well vertical relief wells. For more details, see the section on Recommendations.
12. The modelled groundwater levels in the municipal Deloro Well predict there will be relatively low interference associated with the groundwater interception of Scenarios 11 and 12. The difference in drawdown between the existing conditions pumping level and the Scenarios 11 and 12 pumping level is up to 1.95 m in the Deloro Well. The available drawdown under existing conditions is 19.8 m. The available drawdown under these scenarios is between 17.9 m and 18.1 m for Scenarios 12 and 11, respectively. This indicates that there will still be adequate available drawdown to maintain the Village's water needs.
13. The pumping water elevation at the Tuttle Shaft is predicted to decline by up to approximately 0.9 m as a result of the groundwater interception of Scenario 12. This may be significant enough to reduce artesian flow from the Tuttle Shaft; however, more information is needed to determine the impact of this scenario on the Tuttle Shaft flow.

6. Recommendations

Modelling has been carried out to determine if a totally passive solution can be used for dewatering of the surficial deposits under the proposed engineered cover. The following recommendations are provided with a view to achieving this preferred objective.

1. The only solutions with passive elements that will almost completely dewater the surficial deposits under the engineered cover include a horizontal drilled well (Scenario 11) and a horizontal drilled well connected to vertical relief wells (Scenario 12) along the west side of the proposed engineered cover. Due to the uncertainty in hydraulic properties along the proposed alignment of the horizontal well, a hydrogeologic field investigation involving drilling and installation of pumping and observation wells, and pumping tests, needs to be carried out.
2. If possible, the field studies should include measurements of the Tuttle Shaft's static water level, in relation to the ground surface, to determine if the horizontal well can depressurize the area so there is no artesian flow.
3. Following completion of the field program and data analysis, further groundwater flow modelling will be required to confirm the original and updated modelling and to determine the effectiveness of the proposed groundwater diversion/control solution. Depending on the data obtained during the field program, this modelling should also include evaluation of an additional groundwater diversion/control scenario. This scenario should incorporate a horizontal well constructed entirely in the overburden, in conjunction with vertical pressure relief wells constructed in the bedrock, to determine if effective groundwater flow control can be achieved.
4. Conservative contaminant transport modelling should be carried out to determine the likelihood of contaminated groundwater entering the horizontal and vertical wells from under the engineered cover. (Currently, it is predicted that groundwater coming from the shallow bedrock under the engineered cover to the horizontal well and vertical wells will be diluted by a factor of approximately 12.9:1, when the entire flow into the system is considered).

Note: Scenario 12 has been adopted in the Draft Industrial Area Closure Plan, since it is a passive solution and is predicted to almost fully dewater the surficial deposits. Additional conceptual details of the horizontal well and vertical pressure relief wells are presented in the Draft Closure Plan, including a cost opinion of the capital and maintenance costs for this system.