July 12, 2012

File: 00216-00531 P
CD#: 00216-CORR-00531-00123
Project ID: 10-60004

Dr. Stella Swanson
Chair, Joint Review Panel
Deep Geologic Repository Project

c/o Canadian Nuclear Safety Commission
280 Slater Street
Ottawa, Ontario
K1P 5S9

Dear Dr. Swanson:


The purpose of this letter is to provide OPG’s presentation material to the Joint Review Panel (JRP) for the July 18, 2012 Technical Information Session, along with supporting written documentation as committed by OPG in Reference 1. This material has been prepared based on the information and expectations for OPG’s presentation at the Technical Information Session provided by the JRP in Reference 2.

OPG’s written submission and the slides for the oral presentation are attached to this letter as Attachments 1 and 2, respectively.
If you have questions on the above, please contact Mr. Allan Webster, Senior Manager, Licensing, at (905) 839-1151, ext. 6051.

Sincerely,

<original signed by>

Albert Sweetnam  
Executive Vice-President  
Ontario Power Generation

Attach

cc.  Dr. J. Archibald – Joint Review Panel c/o CNSC (Ottawa)  
     Dr. G. Muecke – Joint Review Panel c/o CNSC (Ottawa)  
     P. Elder – CNSC (Ottawa)  
     F. King – NWMO (Toronto)
ATTACHMENT 1

Attachment to OPG letter, Albert Sweetnam to Dr. Stella Swanson, “Deep Geologic Repository Project for Low and Intermediate Level Waste – Submission for the July 18, 2012 JRP Technical Information Session”

July 13, 2012

CD#: 00216-CORR-00531-00123

OPG’s Written Submission
for JRP’s Technical Information Session on July 18, 2012
Written Submission

Contents

1.0 INTRODUCTION .............................................................................................................. 1

2.0 CONSTRUCTION PHASES .............................................................................................. 1
  2.1 Site Preparation .......................................................................................................... 1
  2.2 Initial Construction ...................................................................................................... 2
  2.3 Shaft Sinking ............................................................................................................. 3
      2.3.1 Controlled Drill and Blast ......................................................................................... 5
      2.3.2 Alternative Methods ................................................................................................. 5
  2.4 Lateral Development ................................................................................................... 6
      2.4.1 Controlled Drill and Blast ......................................................................................... 7
      2.4.2 Alternative Methods ................................................................................................. 7
  2.5 Waste Rock Management ............................................................................................ 8
  2.6 Final Surface Facilities ............................................................................................... 8

3.0 WATER MANAGEMENT .................................................................................................. 9

4.0 VENTILATION ................................................................................................................. 10

5.0 MINE SAFETY ................................................................................................................ 11

6.0 BACKFILL .................................................................................................................. 12
  6.1 Long-term Safety ........................................................................................................ 13
  6.2 Operational Stability and Safety .................................................................................... 15
  6.3 Long-term Stability ..................................................................................................... 16

7.0 REFERENCES ................................................................................................................ 18
1.0 INTRODUCTION

This written submission is being provided to the Joint Review Panel as a summary of the material to be presented in OPG’s oral presentation on July 18, 2012, at the Technical Information Session to be held in Ottawa. Due to numerous references to the slides in the oral presentation provided in Attachment 2, this written submission should be read in conjunction with the accompanying presentation slides.

2.0 CONSTRUCTION PHASES

The DGR facility will be constructed over several phases comprising:

- Site preparation
- Initial construction
- Shaft sinking
- Lateral development
- Final surface facilities

The following sections describe the activities, key features and constraints of these phases and are supported by slides 2 through 44 of Attachment 2. OPG’s initial and supplementary responses to Information Request (IR) LPSC-01-25 also provide information relevant to these sections.

2.1 Site Preparation

Site preparation activities are covered in slides 4 through 12 of Attachment 2. The DGR project site was used as a laydown area in the 1970s and 1980s for the construction of the heavy water plant at the Bruce nuclear site. An aerial view from May 1980 (refer to slide 5) shows the extent of past land disturbance at the DGR project site. Slide 6 shows an aerial view of the current DGR project site.

Field investigations were conducted in 2011 to better characterize the subsurface conditions of the DGR project site. The scope of the program included 25 shallow boreholes, 23 test pits and 2 geophysical lines as represented in slide 7 (refer to OPG’s response to IR EIS-03-54). Analysis of the field data is documented in GOLDER (2012) and supports further facility and infrastructure design. The subsurface cross-sections on the project site are referred to in subsequent sections.

Site preparation activities are expected to be completed over approximately 9 months and will be dependent on the time of year the site is first able to be accessed. There are constraints for grubbing and clearing for the period from May 1 through July 31 because of the bird nesting season. Site mobilization will commence with the establishment of the site construction island fencing, primary and secondary controlled access and construction site offices (refer to slide 8).
Site grading to establish key site infrastructure and the construction area will commence in conjunction with grubbing and clearing. The existing 44 kV power line will be removed from the southern portion of the site. The area around the shafts will set the peak elevation of the site. The grade will be set above the maximum probable flood elevation and will take into account the water management ditches to maintain sufficient grades for gravity flow to the stormwater management pond at the north of the site. The overall site grading plan will incorporate the material excavated from the development of the shafts (shales and dolostones) and will evolve through multiple phases of construction.

The water management system is illustrated in slide 10. As mentioned above, the initial site grading is critical to establish the ditching system grades. There may be a requirement for localized intermediate sumps in the area around the shafts until after the shaft collars are constructed and final elevation established at the headframes. The shaft area oil/water separator sump will also be established as part of this phase. The stormwater management pond discharges to the north of the site through an existing ditch system to Lake Huron. Refer to Section 3.0 for further discussion of the stormwater management pond.

The temporary and permanent waste rock stockpiles will be prepared as part of site preparation (refer to slide 11). The dolostone stockpile area will also serve as a laydown area for initial construction in advance of shaft sinking. Refer to Section 2.5 for discussion of the permanent limestone waste rock management area.

Installation of site services completes the site preparation phase (refer to slide 12). Permanent service and fire water rings will be established and connected to existing Bruce nuclear site infrastructure. This provides the opportunity to establish fire hydrants to be available in the event of a fire during the construction phase. A 13.8 kV substation and main power distribution system will be completed for future tie-ins of temporary and permanent equipment. A diesel fueling station and concrete batch plant will be located at the site for the duration of the construction project. Refer to OPG’s response to IR LPSC-01-26 for additional information.

2.2 Initial Construction

Initial construction activities are primarily limited to the shaft facilities area of the site located in the south-west quadrant of the project site (refer to slide 14). The subsurface condition in this area, as defined by the 2011 site investigations, is shown in slide 15. The area around the shafts consists primarily of 12 – 14 m thick dense till with isolated and discontinuous sand and gravel and clayey silt till lenses. Refer to OPG’s initial and supplementary responses to IR EIS-01-01.

In advance of shaft collar development, ground treatment at the shafts (grouting or freezing) will be conducted (refer to OPG’s response to IR LPSC-01-31). Ground treatment will be completed from the surface prior to disturbing the till to access bedrock.
Upon completion of the ground treatment, the areas around both shafts will be excavated to bedrock for shaft collar development. The collars will be excavated to a depth of approximately 20 m below the bedrock surface. A concrete liner will be poured through this depth and the shaft headframe foundations constructed (refer to OPG’s response to IR LPSC-01-30). Shaft collar excavation requires near-surface drilling and blasting. Daytime blasting practices will be employed until the headframes are in-place, and fly-rock control practices will be exercised. The shaft sinking galloways (working stages in the shafts raised and lowered using surface mounted winches) will be constructed at the surface and lowered into the collar before the construction of the headframes.

The headframes and temporary shaft sinking hoist houses will then be constructed. The main shaft sinking hoist house will only be required for shaft development, whereas the ventilation shaft sinking hoist house will be retained and used to hoist waste rock during lateral development. The headframe and hoist houses will be equipped and commissioned for shaft sinking. Temporary shaft ventilation fans and heaters will be located at each shaft to provide fresh air for shaft sinking (refer to Section 4.0 for further discussion of ventilation through the various phases). Emergency generators will be integrated with the main substation in the event of a power loss and the need to extract workers in the shafts.

Following shaft sinking, the temporary ventilation systems will be replaced with the permanent facilities and the temporary main shaft hoist house and Galloway winches removed. The permanent main shaft hoisting systems will be commissioned and the Waste Package Receiving Building (WPRB) constructed to act as a main shaft collarhouse for the lateral development phase (refer to slide 17).

2.3 Shaft Sinking

The typical shaft sinking process is shown in slides 19 and 20. The shaft is advanced in 5 m lifts (or 5 m rounds) through the use of a drill jumbo suspended from the Galloway. The dimensions of the DGR shafts are sufficient to accommodate a dual boom drill jumbo as illustrated. The round is drilled and loaded with explosives.

Before the round is blasted, the Galloway is raised from the bottom of the shaft (face) to a nominal distance of 30 m to avoid damage from fly-rock. Fresh air is provided to the face by the temporary surface fans. Once the shaft air has cleared the blast fumes and dust, the Galloway is lowered and the workers return to the face. Travel to and from surface is typically through the use of the shaft bucket. A secondary personnel cage will be included in both the main and ventilation shafts to provide an alternative means of egress.

Initial ground support is installed from the muckpile (muckpile is a mining term for blasted rock pile) to ensure workers are working under supported ground. The type of ground support will vary through the various strata along the depth of the shaft to the specific rock type and properties.
Two-dimensional geomechanical analyses of the shaft excavations were conducted for formations that will be intersected. The analyses were conducted first without support to assess the extent of overstress, and subsequently with support in the form of bolts (immediate support) and after advancing the shaft in the form of a concrete liner (permanent support). In the analyses of the concrete liner, the bolts were removed from the model to simulate potential corrosion of the steel. Typical results are shown for the Guelph Formation in slide 21.

The analyses indicated that three types of immediate support should be considered for the shafts, depending on the formation characteristics and the analyses results (refer to slide 22).

Support Type A - for rock formations with no overstressed zones:
- 12 mm diameter rock bolts 0.5 m long at 1.8 m c/c (centre-to-centre) circumferentially and 1.67 m c/c longitudinally (3 rows per round along shaft)
- Rock bolts installed with mechanical anchors
- 102 mm x 102 mm Welded Wire Mesh (WWM) installed over exposed area of shaft

Support Type B - rock formations with overstressed zones (with no potential for slaking):
- Fully resin grouted 25 mm diameter thread bar type rock bolts 2.5 or 3.0 m long* at 1.8 m c/c circumferentially and 1.67 m c/c longitudinally (3 rows per round along shaft)
- 102 mm x 102 mm Welded Wire Mesh (WWM) installed over exposed area of shaft with the rock bolts with an additional 12 mm diameter rock bolts 0.5 m long to secure mesh in place to conform to rock surface

Support Type C – rock formations with overstressed zones and potential for slaking to occur (shales and shaly units):
- Fully resin grouted 25 mm diameter thread bar type rock bolts 2.5 or 3.0 m long* at 1.8 m c/c circumferentially 1.67 m c/c longitudinally (3 rows per round along shaft)
- 102 mm x 102 mm Welded Wire Mesh (WWM) installed over exposed area of shaft with the rock bolts with additional 12 mm diameter rock bolts 0.5 m long to secure mesh in place
- 75 mm plain shotcrete with silica fume additive

* rock bolt length is 2.5 m in the ventilation shaft and 3 m in the main shaft.

Following application of ground support, the blasted rock is removed using a mucking machine suspended from the Galloway and placed into buckets to be raised to surface to temporary stockpiles. As the muckpile recedes, additional ground support is installed as required.

The shaft liner is installed in 5 m sections with the base of the liner trailing the face by 10 m. The thickness of the shaft liner will vary depending on the location and formation adjacent the liner (refer to OPG’s initial and supplementary responses to LPSC-01-17). The upper 200 m of each shaft will be constructed with hydrostatic liners, or water-tight liners. The liners below
200 m will be a “leaky liner”, a liner that allows water to drain to a series of water rings allowing the water to drain to the shaft bottom.

As the shafts are developed, geomechanical verification activities will be conducted at key locations within the shafts to verify modeling and design predictions. Further, a geoscientific verification program will be completed in the main shaft as described in the Geoscientific Verification Plan (NWMO 2011).

It is expected that the ventilation shaft will reach the repository horizon in advance of the main shaft as the steel headframe will not take as long to complete and the geoscientific verification program in the main shaft will add additional periods for work programs at specific elevations. Initial lateral development will be completed from the ventilation shaft station and progressed until the tie-in with the main shaft. For additional information on lateral development, see Section 2.4. Once the tie-in to the main shaft is completed, the shaft development will continue to the bottom of the shafts and the loading pocket and waste pass system installed.

2.3.1 \textit{Controlled Drill and Blast}

Controlled drill and blasting techniques and practices are common in today’s mining industry. The parameters that influence the design of a drill and blast pattern include the following:

- Rock density;
- Shaft / development heading dimensions;
- In situ stresses;
- Drill configuration and capability;
- Mucking equipment available and desired muck pile profile;
- Loading equipment available;
- Wetness or dryness of the area;
- Explosive to be used; and
- Distance to and sensitivity of nearby structures and/or equipment.

Shaft development will require many different drill and blast configurations to accommodate the change in shaft excavation diameter to suit liner requirements and varying rock strata. These designs will be developed closer to construction when the above parameters are finalized. Slide 24 illustrates a typical drill and blast pattern for the main shaft with an excavation diameter of 8 m with electronic detonators to control firing sequencing.

2.3.2 \textit{Alternative Methods}

Mechanical excavation options, such as boring and roadheader technologies were considered as part of the conceptual design of the DGR. There was no evidence of successful implementation of these methods for shaft sinking. Thus, these excavation methods were deemed impractical for shaft sinking and were not pursued any further.
Blind boring (boring from surface to depth) is limited to the size of excavation that can practically be excavated. Both the main and ventilation shaft excavation diameters are beyond the practical limit. Raise boring was not further considered as it would require a shaft to be constructed by conventional methods and extensive lateral development to assemble the cutting head and establish a cuttings disposal system.

A vertical roadheader option was further examined, specifically for development through the cap shales. However, conventional drill and blast practices would be required for the upper 450 m of the shaft, as well as through the repository horizon to shaft bottom due to the strength of the rock. It would not be practical to configure the shaft sinking set-up to accommodate both approaches.

Controlled drill and blast methods can be employed to minimize the extent of the highly damaged zone through the shales. The geoscientific verification program of the main shaft will provide supporting evidence of the extent of the excavation damage zone (EDZ).

2.4 Lateral Development

Slides 28 through 33 show the evolution of lateral development through several phases. Ventilation and mine safety considerations for each of these phases are discussed in Sections 4.0 and 5.0 below. Ground support requirements are discussed in Section 6.2.

As stated in Section 2.3, the initial lateral development at the repository horizon will be completed off of the shafts. Excavation will be done using hand drills and small mucking machines to deliver rock to the same shaft bucket that is used during shaft sinking. During initial development, tunnel sizing may be limited by the size of equipment that can be delivered underground and until there is enough room to bring down larger mechanized drilling equipment. The size of equipment will also be limited to the amount of fresh air that can be supplied through the temporary shaft ventilation systems. Initial development would be limited to approximately 100 m³/day or 3.5 m advance/day (a typical drill round breaking length) and progress to approximately 250 m³/day, with the addition of a second crew advancing from the main shaft station and the ability to progress to two-boom drill jumbos and have multiple advance headings, or faces.

The connection of the main and ventilation shafts provides the opportunity to increase ventilation flow and mobilize additional equipment (refer to slide 30). At this point, lateral development would temporarily cease, the shafts would continue to shaft bottom, the waste pass and loading pocket constructed and the ventilation shaft hoisting system converted to a waste rock transfer (skipping) arrangement. The main shaft sinking hoist and associated headframe infrastructure will be removed and the main shaft Koepe and auxiliary hoists commissioned to move personnel and equipment to the repository horizon. The permanent ventilation facilities at the main and ventilation shafts will be constructed to accommodate the increase in fresh air requirements for increased development throughput.
The underground services area would be constructed upon re-initiation of lateral development to provide for service, fueling and storage areas (refer to slide 31). Advance rates would progressively increase due to the use of larger equipment and availability of open faces to reach the nominal average of 2,100 tonnes per day. Access tunnels to both emplacement panels and the return air tunnel will be progressed to provide for additional working faces and to establish flow-through ventilation in the panels as shown in slide 32.

Lateral development will continue in this manner until the repository level is completed. As emplacement rooms are completed and there is no need for access with large excavation equipment, the floors will be concreted and the ventilation end-walls constructed. Operational features including the rail embedded emplacement rooms in Panel 1 and the gantry crane would be installed. For additional supporting information, refer to OPG’s response to IRs EIS-03-61 and EIS-03-62.

2.4.1 Controlled Drill and Blast

Controlled drill and blast is the preferred approach for the lateral development phase. The key difference in lateral development versus shaft sinking is the consistency in application of drill and blast patterns through the Cobourg Formation, varying mainly with respect to excavation dimensions and orientation to the principle horizontal stress. The control of overbreak and EDZ is also less important for lateral development because there are no plans to construct engineered seals in the access tunnels and rooms (refer to OPG’s response to IRs LPSC-01-32, LPSC-03-57 and EIS-03-52).

Slide 36 illustrates a typical drill and blast pattern for an emplacement room. The layouts will be further optimized once the parameters list provided in Section 2.3.1 are finalized. A range of powder factors (commonly expressed as kg of explosives per m³ in-situ rock) and drill hole densities have been evaluated based on observations of the Darlington Cooling Water Tunnel project and industry practice for low density explosives (refer to IR EIS-03-96). The use of computer aided and laser aligned drilling jumbos and electronic detonation provides for very controlled and repeatable blasting performance.

2.4.2 Alternative Methods

As with the alternative methods considered for shaft sinking, lateral excavation by roadheader was considered as part of the conceptual design based on a limited sample set of the Cobourg Formation parameters. Based on additional information from the further site characteristic drilling DGR-1 through DGR-6, the average unconfined compressive strength (UCS) of the Cobourg Formation was reaching the practical limits of demonstrated roadheader performance.

Additionally, the use of conventional drill and blast would be required for a significant portion of initial development in order to create sufficient space to mobilize roadheaders required for the size of openings in the DGR. The risk of roadheaders meeting the development requirements, the management of dust emissions and fines/cuttings handling and the need to have
conventional drill and blast equipment available did not support selecting roadheader as the preferred excavation approach.

### 2.5 Waste Rock Management

The preparation of the waste rock management area is discussed in Section 2.1 and illustrated in slide 11. Additional information on the profile of the waste rock pile, underlying subsurface conditions and the construction approach are provided on slides 39 through 41.

Section 6.2.3 of the Preliminary Safety Report (PSR) provides a narrative description of the waste rock pile and slide 39 shows a profile of the waste rock pile exaggerated in the vertical axis to illustrate the 5 m lifts and underlying surface contact. The subsurface conditions for the A-A profile are shown on slide 40 and shows that the base elevation of the waste rock pile is on till with approximately 10 m+ of continuous till between the waste rock pile base and the bedrock surface. Grading beneath the waste rock pile will ensure gravity drainage of any water that seeps into the waste rock pile towards the perimeter ditch system. This is further discussed in Section 3.0.

The pile will be constructed from the south-west corner nearest the shafts and developed towards the north and the east in 5 m lifts. Waste rock will be transported to the pile by off-highway truck, accessed via the ramp, and placed by bulldozer. The development of the pile in this manner minimizes the impact of noise and dust on nearby receptors. The initial side slope will be 1.5:1 as the rock reaches a natural angle of repose as the bulldozer pushes the pile forward. These slopes will be battered back to the 2.5:1 final slope as part of final pile development.

### 2.6 Final Surface Facilities

As the lateral development phase approaches completion and the repository is being equipped for operational requirements, the final surface facilities will be constructed and the site prepared for operations. This will also include the dismantling of construction equipment and temporary structures such as the ventilation hoist house, loading pocket and diesel fueling station.

The key facilities established during this final construction phase are the amenities building, compressor building, permanent emergency generator (if different than that used during construction) and the crossing to the Western Waste Management Facility (refer to slide 43). Modifications will be required to the WPRB to convert from a shaft collarhouse to a waste receiving area. Final site grading, road and parking area paving and radiological zone fencing will be established around the perimeter of the Zone 2 area.

Final commissioning activities will be completed to meet operational requirements. This will also include re-commissioning of permanent equipment that was used for construction activities to ensure they meet the requirements of operations.
3.0 WATER MANAGEMENT

The DGR site water management for both the construction and operations phases are shown in slides 46 through 58 of Attachment 2. The water management system is designed such that all water produced or captured on the site is centralized and discharged from the stormwater management pond to Lake Huron at MacPherson Bay through an existing ditch system on the Bruce nuclear site. Slide 47 shows the drainage of the Bruce nuclear site off of the DGR project site and the path for the DGR discharge.

Surface drainage on the DGR site for the construction and operations phases is shown on slide 48. Surface and waste rock run-off is collected in the site ditch system and directed to the stormwater management pond (refer to OPG’s response to IR LPSC-01-12). Underground drainage is shown in slide 49. The repository tunnel grading is such that water gravity drains through the tunnels and collects at the main sump. It should be noted that this is designed to manage construction water (e.g., used for drilling and dust suppression) as the repository emplacement rooms and tunnels are expected to be dry during the operating phase. Slide 50 shows the water handling and sump arrangements underground. Pumping systems are designed with redundancy in the event of failure and are connected to the emergency back-up power.

The underground discharge and expected average water management flows for the construction and operations phases are shown in slides 51 through 53. Underground discharge during the construction phase is predominantly process water from development activities. The estimate of 21 L/s average discharge volume is conservative as it has been developed to accommodate high volume drilling equipment, does not account for potential re-circulation of waste water underground and is generous for dust suppression. Recent analysis of the infiltration of groundwater in the “leaky liner” portion of the shafts shows that inflow rates are likely to be less than 0.45 L/s as opposed to the 2 L/s as reported in the PSR.

Underground water discharge during the construction phase will be pumped to a treatment plant, discharged into the site oil/water separator system and discharged into the surface ditch system. During operations, it is not expected that water treatment at surface will be required before discharge to the oil/water separator due to the low volumes and ability to settle suspended solids at the main sump.

An illustration of the stormwater management pond, exaggerated in the vertical axis, is provided on slide 54. The underlying subsurface conditions are shown on slide 55. It should be noted that the bottom of the stormwater management pond, like the waste rock management area, is in contact with the continuous dense till subsurface with 7 – 10 m of till underlying both (refer to OPG’s response to IR EIS-03-56).

Water quality at the point of discharge from the stormwater management pond is expected to meet discharge criteria. Conservative assumptions have been used in the development of the water quality envelope, both in terms of constituent loading and discharge volumes, for purpose
of designing the stormwater management pond. Leachate analysis of the waste rock discharge indicates the potential for elevated levels of 5 metals, salinity and un-ionized ammonia above the Provincial Water Quality Objectives (PWQO), however, the levels in stormwater management pond are expected to meet discharge criteria. In the remote event that water quality of pond discharge water exceeds acceptable limits, contingency plans can be executed and discharge held until mitigated. Slides 57 and 58 provide contingency treatment options available to treat elevated ammonia and saline water conditions. These options could be applied at the source (e.g., underground sumps prior to discharge to surface) or at the stormwater management pond.

4.0 VENTILATION

Mine ventilation will adapt through the shaft sinking and lateral development phases (refer to OPG’s response to IR LPSC-01-35). As described in Section 2.2 above, temporary shaft ventilation will be constructed at each shaft. The volume of air will be dependent on the final shaft sinking equipment, but will be in the range of 25 – 40 m³/s. Temporary heaters will be required to support year-round sinking activities. Fresh air is pushed to the working face of the shaft through ducting and returns to surface through the shaft opening (refer to slide 61). For initial lateral development from the shafts, the fresh air ducting is advanced into the tunnels as shown in slide 62.

Once the main and ventilation shafts are connected, the temporary ventilation systems are removed and fresh air is supplied down the main shaft and exhausted through the ventilation shaft. Slide 63 illustrates how the air is controlled from the main shaft at the repository level through the use of a ventilation bulkhead located between the two shafts. These doors are kept closed and fans are used to direct the air from the main shaft access tunnel to the working areas. The supply of fresh air and sizing of fans on surface and underground will be dependent on the type and amount of equipment used for development. The design is flexible to allow the addition of fans to meet a wide range of development equipment options. Exhaust air is returned through the tunnels to the ventilation shaft and exits through the ventilation plenum (refer to OPG’s response to IR LPSC-01-14).

The bulkhead will remain in place until development progresses to provide flow-through ventilation through the return air tunnel and the main underground fans are established to the north of the ventilation shaft. Slide 65 shows the removal of the ventilation bulkhead from between the main shaft and ventilation shaft. Fans are still required to provide fresh air to dead-end development headings, but once connected to the return air tunnel, the ducting is removed and the airflow is controlled through the use of the main underground fans and the louvers located at the emplacement room end-walls. Airflow in the shaft services is controlled in the same manner. The fresh air requirements will vary through the various phases and the design can accommodate peak ventilation rates of 290 m³/s. The final peak and average design requirements will be determined once a contractor fleet has been identified.
Once lateral development is completed, the need for ducting is nearly eliminated and the system is controlled by the main underground fans and the end-wall louvers as described in Section 6.3.8.3 of the PSR. Slide 66 shows a series of rooms active in Panel 2 with the remaining rooms closed with no ventilation. In Panel 1, one emplacement room is ventilated to provide an alternative egress option in the event of an emergency and access is required from Panel 2.

5.0 MINE SAFETY

The key hazards and mine safety considerations during shaft sinking and lateral development phases are not considerably different from each other. The DGR project has identified several key hazards and mitigating strategies through the review of operating experience in the mining industry and international repositories, lessons learned from within Ontario Power Generation and other industries, conducting multi-disciplinary design and risk reviews with key stakeholders and technical experts; and seeking guidance from industry organizations such as Workplace Safety North.

The following provide a high-level summary of key hazard areas identified through these processes:

- Fall of ground / rock fall
- Working at heights
- Egress
- Explosives use and handling
- Falling objects
- Fire
- Equipment / Transportation
- Run-of-muck
- Body Mechanics

Some mitigation actions include, but are not limited to:

- Robust project health, safety and environment management program
- Stand-by generators to supply emergency power (to mitigate loss of grid failure)
- Use of auxiliary personnel cage to provide secondary egress from the shafts
- Project Mine Rescue capabilities and Mutual Aid Agreements with local mines
- Fire water system in-service for the construction phase
- On-board fire suppression for equipment and portable fire extinguishers installed at key work areas
- Use of non-combustible oils and fluids
- Administrative barrier in the form of approved procedures for working under suspended loads, working at heights and operating equipment / vehicles
- Proper ground support and options for remote placement (e.g. shotcrete)
• Controlled use of ignition/combustible sources
• Kick plates and fall protection system (tie-offs)
• Daily inspection of ropes and other critical safety devices
• Mandatory use of PPE and other required safety equipment

Key consideration is given to opportunities for egress and refuge through all phases of development (refer to OPG’s response to IRs EIS-03-53, EIS-03-60, LPSC-03-60). As mentioned above, the use of auxiliary personnel cages in both shafts during sinking and initial development provide a level of back-up for egress. As the shafts are connected in the initial repository development, the means of egress again increase.

The remaining development will be completed in a manner to provide the opportunity for multiple means of egress as soon as practical. However, there will be several dead-end development headings at any given time. The use of portable refuge stations and administrative barriers will be required to manage risk. Availability of trained responders and mine rescue personnel (refer to OPG’s response to IR LPSC-03-61), communication systems (refer to OPG’s response to IR LPSC-03-59) and protocols, implementation of fire protection programs, use of safe work planning and best practices will be key to providing a safe mine working environment (refer to OPG’s response to IR LPSC-01-37).

6.0 BACKFILL

Backfill is a term which refers to the process of filling an excavated part of a repository with material (e.g., excavated rock, concrete, bentonite/sand) so as to achieve a defined purpose. It also refers to the material itself. In the case of the DGR project, backfill is proposed in three instances (refer to slide 74):

a) Closure walls to provide permanent isolation of sets of waste-filled emplacement rooms from the remaining parts of the working repository;
b) Shaft Area Concrete Monolith to provide added assurance of long-term stability of the shaft seals; and
c) Shaft Seals to provide long-term isolation of the repository from the surface environment.

It is not proposed to backfill other parts of the repository because it would not be beneficial from a long-term safety point of view, and is not required for operational safety reasons or for ensuring the long-term integrity of the approximately 200 m of shale cap rock above the repository. These points are covered in more detail below.

Closure walls will be installed at defined locations in the access and return air tunnels during the operating phase of the repository. As described in Section 6.13 of the PSR, the closure walls will be designed to limit the release of gases to the working part of the repository and, in the remote event, withstand the impact of an explosion from the build-up of gases within the closed section of the repository. Closure walls will be constructed of mass concrete and grouted to fill any gaps between the closure wall and rock surface. Low heat of hydration concrete, standard
grouting techniques and conventional forming practices will be employed in their construction. Concrete will likely be delivered to the repository through the main shaft cage which has the capacity to transport transmixers directly from surface.

The shaft area concrete monolith is described in Section 13.6.2 of the PSR and is essentially mass concrete filling of all openings below the repository level and within 60 m of the shafts on the repository horizon. The concrete monolith provides a stable foundation for the shaft seal systems. As with the closure walls, low heat of hydration concrete will be used to assist in the reduction of heat generation. The concrete monolith will not require grouting or reinforcement. Due to the volume of concrete for this application, a series of slick-lines (or pipes transporting concrete from the surface to the repository) will be installed in the shafts to transport concrete.

The shaft seals are a combination of relatively simple and durable materials in a layered configuration to provide multiple barriers (refer to slide 76). The selection of materials is described further in IR response EIS-03-64. The materials are as follows:

- Concrete
- 70:30 bentonite/sand mixture
- Asphalt
- Engineered fill

The material placement methods have been proven in other applications. This will require temporary facilities constructed on surface to produce the respective materials to the required specification. Working platforms and transport equipment will replace the existing infrastructure in both shafts during this decommissioning phase.

The shaft liner will be removed from the shafts in sections, working from the repository level above the concrete monolith upwards, and a portion of the shaft wall rock removed (assumed 500 mm annulus of damaged rock – see IR response LPSC-03-58). The materials are placed in the order shown on slide 76 and compacted as required. The asphalt and bentonite/sand mixtures are placed in lifts (i.e., small vertical increments) to ensure adequate compaction. The concrete sections, or bulkheads, are designed to support the overlying materials, as well as, isolate areas of higher hydraulic conductivity. These bulkheads will be keyed into the walls of the shaft to enhance structural performance.

The shaft liner will not be removed in the upper 185 m of the shaft and the shaft will be backfilled with engineered fill, such as crushed rock. A concrete cap will be placed near the surface of the shaft to provide a restrictive barrier at the surface.

6.1 Long-term Safety

There are unique features relevant to the DGR which impact the consideration of backfill. These include:
- Potential large amount of gas generated in the DGR due to inherent organic/metal content of L&ILW and containers;
- Extremely low permeability of the rock, which is beneficial for restricting the movement of radionuclides, but also prevents significant loss of generated gas;
- High C-14 content in the DGR waste means that it is important to contain the gas, at least until the C-14 substantially decays; and
- The design of the DGR and the nature of the site and host rock allow the DGR to be stable without emplacement room or tunnel backfill.

The reference repository design therefore includes backfilling the shafts to provide a good seal, but no room or tunnel backfill. However, the case of a fully backfilled repository was analyzed. It is the NE-BF case, and it is described in the PSR and the supporting technical reports as described in slide 78.

The key result is shown in slide 79. This figure shows repository gas pressure as a function of time, with time on a logarithmic axis up to 1 million years. The two green lines are for two variants of the reference case. They show that the gas pressure increases over several thousands of years to approach the background hydrostatic pressure of the host rock (represented by the two horizontal grey lines).

The results with a fully-backfilled repository are shown in red and blue. The repository was assumed filled with gravel, such that there was about 30% porosity. The red and blue lines show the effect of two bounding assumptions on the availability of water to support corrosion and other gas-generating reactions. Reality is likely somewhere between these lines. These results show that with the smaller available volume in a backfilled repository, the gas pressures can get much higher - up to 16 MPa in these calculations.

In summary:
- The L&ILW waste and containers have significant organic and metal content. In the long-term, this is expected to largely degrade into gaseous CO₂, CH₄ and H₂. In fact, some would likely remain as non-volatile compounds, but the DGR safety assessment conservatively assumes that it all goes to gas.
- The very low permeability of the enclosing host rock and shaft seals significantly limits gas migration away from the repository. As a consequence, the gas builds up within the repository void space.
- Without backfill, the repository gas pressure will equilibrate at around the natural hydrostatic pressure of the surrounding host rock. This is a stable condition, similar to conditions in natural gas reservoirs.
- With backfill, the available void space in the repository would be reduced by about a factor of three. Reducing volume results in a corresponding higher gas pressure within the repository.
- Higher gas pressure could result in development of fractures, increased rate of gas release from repository, and higher doses due to C-14 in gas. Therefore, for long-term safety, it is better to provide more space to ensure that gas pressures remain low.
6.2 Operational Stability and Safety

Geomechanical models have been developed to assess the stability of openings and ground support design during the construction and operating phases. The modeling parameters are based on the rock mass characterization and results of field investigations and laboratory testing on samples from boreholes DGR-1 through DGR -8. These parameters were developed through a series of workshops and reviewed and agreed with the DGR technical review group experts. With these parameters, a repository wide model was developed as shown on slide 83.

The vertical stress contours on the horizontal plane through the repository show that the level of stress in the pillars between the rooms increased to about 25 MPa in the core (from an in-situ stress of 17 MPa) which is still only a fraction of the strength of the rock. The 40 m pillar between the two panels is just slightly over 20 MPa. Three areas where the emplacement rooms intersect with the access tunnels and the ventilation tunnels were identified for a more detailed 3-dimensional analysis.

A 2-dimensional model through the centre of the emplacement rooms was constructed, taking advantage of symmetry, and included the formations immediately above and below the Cobourg Formation (refer to slide 84). It should be noted that the orientation of the emplacement rooms is aligned with the direction of the major horizontal stress, which is the most favourable orientation.

Modelling the Lower Member of the Cobourg Formation as a continuum showed that the rock mass remains in the elastic state for the most part with localized overstress at the corners of the rooms. These are zones of high confinement and pose no concerns (see slide 85).

The field investigations have identified horizontal planes of tensile weakness in the Cobourg Formation at an average spacing of 0.7 m; therefore, models which explicitly included these planes in the mesh were constructed. Because the position of the weakness planes is random in nature, two realizations of the mesh were considered. The analyses show that, depending on the position of the weakness planes with respect to the roof of the rooms, one or two planes above the roof can potentially delaminate. These results factored in the design of the roof support.

The rock support is comprised of 3-m-long rock bolts with a nominal pretension of 2 tonnes to ensure that the layers immediately above the roof are ‘squeezed’ together and anchored beyond the potential delamination zone. The rock bolts will be fully grouted after the initial tension is applied. This can be done one or two rounds behind the face to help with the cycle time at the excavation face. For additional information, refer to OPG’s response to IR LPSC-01-34.
For immediate support, and to ensure the safety of the personnel, either steel-wired mesh and bolting, or fibre-reinforced shotcrete is required before advancing the excavation face.

Analyses of the intersections of the emplacement rooms with the access tunnels and with the ventilation tunnels confirm the observations from the 2-dimensional analyses, namely that the zones of overstress are limited to the corners of the rooms and tunnels, which are zones of high stress confinement. Tensile zones exceeding the rock tensile stress, when they exist, are very small and localized.

6.3 Long-term Stability
The repository system has been engineered for long term rockmass performance based on numerical stability analyses incorporating realistic but conservative predictions for:

- Time-dependent strength degradation,
- Effects of gas pressure build-up,
- Seismic ground shaking,
- Glacial loading and unloading, and
- Combinations of these effects.

The repository life span can be broken into the short term, including construction, operation and closure followed by long-term and ultra-long term states. In the long-term state of 100,000 years (after 1-2 glacial and interglacial periods) the expected repository performance is as follows:

- Damage to walls, roof & floor, increasing after each glacial cycle, and
- Pillars remain functional supporting overall stability of immediate roof.

Following ultra-long-term state of 1,000,000 years (after 8-10 glacial periods):

- Immediate roof in Cobourg Formation will slowly and progressively collapse into the repository, and
- Pillars will lose load capacity and the roof will sag.

Key to system stability over indefinite time frame (1,000,000 year +):

- Broken limestone beds and blocks will bulk (volume increase) and choke off further collapse.
- Displacements will not cause rupture of overlying or underlying shales.

All materials have a lower bound stress limit below which no damage can occur. For rocks, this is related to the crack initiation strength. This is the minimum level to which rock strength will decay over time and is detected in laboratory analysis as the stress level required for the first
occurrence of induced micro-damage (refer to slide 95). The rate of strength decay from short term levels to this limit is a function of confining stress and therefore distance to an excavation boundary. The rock strength reaches a minimum level prior to the first glacial maximum.

In order to study the long-term response of the gradually disintegrating rockmass around the excavations, a special 2D response model was used that allows the rock to separate into discrete blocks that can settle into the void (refer to slide 96). This is needed to analyze the bulking or in volume increase of the failed rock and the feedback into overall system stability. Incipient bedding planes are also modelled discretely in this model.

The advance of glacial ice (up to 3 km of thickness is assumed) involves vertical loading, as well as lateral loading, due to crustal flexure in addition to pore pressure effects. This sequence also involves a rotation of maximum stress direction from horizontal to vertical and back to horizontal, creating fatigue damage through each cycle. Analyses show that the repository horizon will sustain damage but remain stable as a system through at least 2 glaciations or 150,000 years (refer to slide 97). After the 3rd glaciation (300,000 years) and beyond, the repository will experience significant disintegration and block failure into the void. This void will “choke” as the material increases in volume during failure eventually to a steady and static state by the 8th glaciation (1,000,000 years).

The short-term excavation response will be dominated by minor damage and possible shallow spalling during the construction phase as exhibited in the Norton Limestone Mine in Ohio shown in slide 98 (similar excavation geometry and similar depth and rockmass to the DGR Cobourg Formation). Long-term behaviour will involve increasing levels of disintegration into the repository rooms accompanied by volume increase within the failed material. This volume increase will ultimately “choke” the void space and prevent further collapse over the ultra-long term. This behaviour is well understood from natural analogues and from large scale mining experience.

In order to assess the large scale effects of this local collapse within the Cobourg Formation horizon, a 3D model including all strata and the full repository footprint was analyzed. The settlement due to the predicted failure (up to the limit of choking and steady state) was applied to the Cobourg Formation. The model was analyzed with and without expanded 40 m barrier pillars between primary panels.

Without extended barrier pillars, the displacements in the overlying barrier shales are minor (45 cm at the centre and dissipating to zero over a lateral distance of 600 m) with small amounts of yield within the weaker Blue Mountain Shale (refer to slide 100). The result is significantly improved with the incorporation of the 40 m barrier pillars between panels. The geometry of these pillars is such that they maintain their functionality over the period of analysis (1 Myr+). The pillars reduce the deformations in the Georgian Bay Formation to 30 cm over 600 m and effectively eliminate engineering damage to the overlying Blue Mountain shale. The key Georgian Bay and Queenston shales see no damage from the settlement and the settlement
dissipates to zero farther up into the stratigraphy. The geosphere integrity is therefore preserved.

7.0 REFERENCES


ATTACHMENT 2

Attachment to OPG letter, Albert Sweetnam to Dr. Stella Swanson, “Deep Geologic Repository Project for Low and Intermediate Level Waste –Submission for the July 18, 2012 JRP Technical Information Session”

July 13, 2012

CD#: 00216-CORR-00531-00123

OPG’s Presentation
for JRP’s Technical Information Session on July 18, 2012
Presentation to Joint Review Panel

Technical Information Session

Ottawa, Ontario
July 18, 2012
Outline of Presentation

Part One
  • Site Preparation
  • Initial Construction
  • Shaft Sinking

Part Two
  • Lateral Development
  • Waste Rock Management
  • Final Surface Facilities

Part Three
  • Water Management

Part Four
  • Ventilation
  • Mine Safety

Part Five
  • Backfill
Construction Phases

- Discussion of each of the construction phases:
  - Site preparation
  - Initial construction
  - Shaft sinking
  - Lateral development
  - Final surface facilities
- Identify key aspects/constraints
- Identify temporary facilities for construction
Site Preparation

- Historical use of project site
- 2011 geotechnical field investigation
- Site preparation activities include:
  - Site fencing
  - Grubbing and site grading
  - Water management
  - Lay-down areas
  - Waste rock management areas (temporary and permanent)
  - Installation of services and potential tie-ins
  - Concrete batch plant
Historical Site Usage - 1980

- DGR Site Boundary
- Interconnecting Road
- DGR Shafts
Current DGR Site
Site Geotechnical Investigations (2011)
Site Preparation – Fencing and Field Offices

- Construction Fencing
- Site Access
- Field Offices
- North Marsh
- Secondary Site Access
Site Preparation – Site Grubbing and Grading

• Grubbing constraint
  May 1 - July 31

• Shaft collars determine overall site grade

• Grading for initial construction
Site Preparation – Water Management

Permanent stormwater management pond and ditch system

Pond Discharge

Oil/Water Separator
Site Preparation – Waste Rock Management Areas

Prepare temporary and permanent waste rock areas

- Shale
- Dolostones
- Overburden
- Limestone
Site Preparation – Site Services

- Service Water Connection
- 13.8 kV Electrical Substation
- Fire Water Connection
- Concrete Batch Plant
- Construction Diesel Station
- Emergency Generator
Initial Construction

- Initial construction activities include:
  - Ground treatment at the shaft locations
  - Establish shaft collars – near surface blasting practices
  - Install shaft sinking equipment (galloways, winches, etc.)
  - Erect headframes and temporary hoisting facilities
  - Temporary shaft sinking ventilation installations
  - Water treatment plant(s) for shaft development
  - Main shaft collarhouse
Initial Construction

- Dolostone stockpile area used for laydown area during initial construction

Dolostones
Current Site – Subsurface Conditions
Initial Construction

- Permanent Ventilation
- Main Shaft Collarhouse
Shaft Sinking

- Shafts will be developed in parallel
- Shaft sinking activities include:
  - Conduct geotechnical verification programs
  - Conduct geoscientific verification program in the main shaft
  - Equipping the shaft as the shaft is developed
  - Establish initial shaft stations at both shafts
  - Initial lateral development from the ventilation shaft
  - Loading pocket development
  - Establish main ventilation facilities at surface
Support and Muck

Install Liner
Shaft Modeling

Unsupported

Supported Bolt Loads

Supported Liner Loads
Shaft Support

**Support Type A** – for rock formations with no overstressed zones
- 12 mm diameter rock bolts 0.5 m long at 1.8 m c/c circumferentially and 1.67 m longitudinally (3 rows per round along shaft)
- Rock bolts installed with mechanical anchors
- 102 mm x 102 mm Welded Wire Mesh (WWM) installed over exposed area of shaft

**Support Type B** – for rock formations with overstressed zones (with no potential for slaking)
- Fully resin grouted 25 mm diameter thread bar type rock bolts 2.5 or 3.0 m long* at 1.8 m c/c circumferentially and 1.67 m c/c longitudinally (3 rows per round along shaft)
- 102 mm x 102 mm Welded Wire Mesh (WWM) installed over exposed area of shaft with the rock bolts with additional 12 mm diameter rock bolts 0.5 m long to secure mesh in place to conform to rock surface

**Support Type C** – for rock formations with overstressed zone and potential for slaking to occur (shales and shaly units)
- Fully resin grouted 25 mm diameter thread bar type rock bolts 2.5 or 3.0 m long* at 1.8 m c/c circumferentially and 1.67 m c/c longitudinally (3 rows per round along shaft)
- 102 mm x 102 mm WWM installed over exposed area of shaft with the rock bolts with additional 12 mm diameter rock bolts 0.5 m long to secure mesh in place
- 75 mm plain shotcrete with silica fume additive

* - rock bolt length is 2.5 m in the ventilation shaft and 3 m in the main shaft
Shaft Sinking – Controlled Drill and Blast

• The following parameters are required to optimize drill and blast patterns:
  • Rock density
  • Shaft / development heading dimensions
  • In situ stresses
  • Drill configuration and capability
  • Mucking equipment available and desired muck pile profile
  • Loading equipment available
  • Wetness or dryness of the area
  • Explosive to be used
  • Distance to and sensitivity of nearby structures and/or equipment

• Requirements will vary through different strata
• The following slide provides a preliminary drill and blast layout for the main shaft
• Detailed drill and blast layouts to be developed following additional design
Typical 8m Diameter Main Shaft Layout

Assumes a standard 44mm drill jumbo bit

Electronic detonator sequence shown
Shaft Sinking – Alternative Methods

- Mechanical excavation techniques considered:
  - Roadheader
  - Boring (blind and raise)
- Raise and blind boring are not practical options
- Roadheader was further evaluated, specifically for the shales
  - Still a requirement for conventional shaft sinking requiring multiple sinking configurations
  - Limited applications in shaft sinking applications
Outline of Presentation

Part One
- Site Preparation
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Part Two
- Lateral Development
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- Final Surface Facilities

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- Water Management

Part Four
- Ventilation
- Mine Safety

Part Five
- Backfill
Lateral Development

- Lateral development will be completed and commissioned in entirety prior to L&IL waste emplacement
- Initial development constrained by ventilation and efficient waste rock movement with shaft sinking equipment
- Shaft development will hold at the repository horizon until connection of the main and ventilation shafts
- Loading pocket is required for efficient waste rock transport to surface
Lateral Development – Initial Station

Ventilation Shaft
Lateral Development – Development From Shafts
Lateral Development – Tie-in Shafts

Refuge Station

Multiple Faces for Advance

Geoscience Area
Lateral Development – Shaft Services

Establish underground services area

Return air tunnel

Waste Rock Pass
Lateral Development – Full Development

Ramp to Shaft Bottom

Panel 1

Panel 2
Lateral Development – Final Development

Configure underground services area for operations

Complete rail access rooms with gantry crane

Concrete floors throughout repository
Lateral Development – Final Development
Lateral Development – Controlled Drill and Blast

- Drill and blast patterns will be consistent for similar headings
- Computer aided, laser aligned drilling equipment
- Range of powder factors and drill densities considered
- The following slide provides a preliminary drill and blast layout for an emplacement room
Typical Emplacement Room Layout

Assumes a standard 44mm drill jumbo bit

Electronic detonator sequence shown
Lateral Development – Alternative Methods

- Initially considered mechanical excavation on small sample set of Cobourg Formation properties
- The strength of the Cobourg Formation is close to practical limits of roadheader technology
- Requires drill and blast for large portion of initial development in the area of the shafts
- Unproven in similar applications to the scale of the DGR
- Health and safety consideration with dust generation in limestone
Lateral Development – Alternative Methods
Waste Rock Management
Subsurface Conditions – Profile A-A

Relative positioning of limestone waste rock pile

Nominally 10m + between base of WRMA and bedrock surface
Waste Rock Management
Final Surface Facilities

- Removal of temporary structures (e.g. ventilation hoist house)
- Amenities facility
- Establish connection to Western Waste Management Facility (WWMF)
- Establish permanent operations services
- Final paving and radiological zone fencing
- Commission facility for operational requirements
Final Surface Facilities

Temporary stockpile material has been used in final site grading.

Limestone stockpile remains

Connection to WWMF
OPG’s DEEP GEOLOGIC REPOSITORY PROJECT
For Low & Intermediate Level Waste

Final Surface Facilities

- WPRB
- Radiological Zone Fencing
- Amenities Building
- Compressor Building
- Permanent Emergency Generators
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Part One
• Site Preparation
• Initial Construction
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Part Two
• Lateral Development
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• Final Surface Facilities

Part Three
• Water Management

Part Four
• Ventilation
• Mine Safety

Part Five
• Backfill
Water Management

- Site and Repository Drainage
- Underground Dewatering
- Water Sources
- Stormwater Management Pond
- Water Quality
- Contingency Treatment Options
Drainage off the DGR Site Boundary
Site Grading and Drainage for Construction
Water Management – Repository Drainage
Water Management – Repository Sumps

DEWATERING PROCESS FLOW DIAGRAM
Underground Discharge and Constituents

- Underground process water rate of <21 L/s during construction. Groundwater contribution likely closer to 0.45 L/s than 2 L/s as per PSR.
- Underground sump has settling features prior to pump-out to surface. Minor TSS contaminants. Pumped to the surface temporary water treatment plant.
- The treated water will be discharged into a oil/water separator at the surface for further treatment of oils, grease and suspended solids, prior to discharge into the collection ditch system and to the stormwater management pond.
Water Management – Water Flows & Sources

Construction – Average 25 L/s to the pond

- Underground dewatering: 83%
- Runoff from Surface Facilities Area: 9.2%
- Runoff from WRMA: 5.7%
- Direct precipitation on pond: 1.8%
- Shales and dolostones: 2.7%
- Argillaceous limestone: 3.1%
Water Management – Water Flows & Sources

Operations – Average 6 L/s to the pond

- Runoff from Surface Facilities Area 36%
- Argillaceous limestone 13%
- Direct precipitation on pond 7.2%
- Runoff from WRMA 24%
- Rehabilitated land 11%
- Underground dewaterting 31%
Water Management – Stormwater Management Pond

Longitudinal Profile

- Incoming Swale
- RipRap Protection
- Sediment Forebay Permanent Pool
- Forebay Weir
- RipRap Protection
- Active Storage Permanent Pool
- Hickenbottom Inlet
- MH with Low Flow and High Flow Pipes
- Interconnecting Road
- Culvert 250

Elevation (masl)

- 177
- 178
- 179
- 180
- 181
- 182

- 100yr = 14,400 m³
- Overflow Weir = 10,600 m³
- 2yr = 8,240 m³

(Not To Scale)
Subsurface Conditions – Profile A-A

Relative positioning of stormwater management pond

Nominally 7 – 10m between base of pond and bedrock surface
Water Management – Water Quality

- Conservative assumptions based on leachate analysis of the waste rock. Lab testing indicates 5 metals, salinity and un-ionized ammonia may be slightly above PWQO from the waste rock discharge. Expect lower actual values in pond discharge and will be confirmed through extensive monitoring program.

- Management of ammonia through selection, handling and use of explosives. Treatment options such as aerators and reverse osmosis skids.

- Salinity results assumed 2 L/s inflow from the shafts. Potential to grout inflow locations in the shafts or have a treatment skid underground prior to main sump.

- Planning for temporary water treatment plants during construction for TSS as needed. Contingency planning for skids for ammonia and saline as needed.
Water Management – Treatment Options

- Removal of ammonia by membrane treatment or oxidation
- Relatively small equipment and can be done “in-line”
Water Management – Treatment Options for Salinity

• Removal of TDS
• Reverse osmosis, electrodialysis
• May need pre-treatment or conditioning
• Will produce a brine that will need to be disposed
• Treatment can involve a series of portable modules
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Part One
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Part Three
• Water Management

Part Four
• Ventilation
• Mine Safety

Part Five
• Backfill
Ventilation

- Ventilation design considers the following phases:
  - Shaft sinking
  - Initial development off of shafts
  - Connection of the shafts
  - Full development with large equipment
  - Operations ventilation
Ventilation – Shaft Sinking

Temporary Heater

25 – 40 m³/s

Fresh Air Ducted to the Face

Exhaust
Ventilation – Off-shaft Development

- Return Air Travels via Tunnel
- Fresh Air Ducted to the Face
Ventilation – Connection of Shafts

Fresh Air is Fan Assisted Beyond the Main Shaft Station

Ventilation Bulkhead Doors Closed with Fans
Ventilation – Initial Development

Return Ventilation Loops Back to Ventilation Shaft

Bulkhead to Eliminate Short Circuit to Ventilation Shaft

Ventilation Bulkhead Doors Closed with Fans Pulling From Main Shaft
Ventilation – Panel Development

Establish return air flow

Establish main underground fans

Install room end-walls

Retract ducting following breakthrough
Ventilation – Operations Ventilation

Panel 1

Panel 2

All Regulators Closed

Regulators Partially Open

All Regulators Closed
Mine Safety – Shaft Sinking and Off-Shaft Development

Key Hazards during Sinking Phase:

1. Rock Fall
2. Working at Heights
3. Explosives
4. Falling Objects
5. Fire
6. Equipment / Transportation
7. Body Mechanics
Mine Safety – Connect the Shafts

Key Hazards during Lateral Development Phase:

1. Equipment / Transportation
2. Rock Fall
3. Run-of-Muck
4. Working at Heights
5. Explosives Handling
6. Falling Objects
7. Fire
8. Body Mechanics
Mine Safety – Full Development (Dead-end)

Egress Routes in Main Shaft and Ventilation Shaft

Permanent Refuge Station

Portable Refuge Station

Portable Refuge Station
Mine Safety – Full Development (Multiple egress)
Mine Safety – Operational

- Permanent Refuge Station
- Portable Refuge Station
- Portable Refuge Station
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Part Three
- Water Management

Part Four
- Ventilation
- Mine Safety

Part Five
- Backfill
Backfill

- Use of backfill in the DGR
- Long-term safety
- Operational stability and safety
- Long-term stability
Backfill in the DGR

- Shaft Seals
- Closure Walls
- Concrete Monolith
Location of Closure Walls

Closure Walls - 3

Closure Walls - 2

Closure Walls - 1
Shaft Seals
(Not to Scale)

Concrete Cap (≈20 m)

Engineered Fill (≈165 m)

Bentonite / Sand (≈150 m)

Bentonite / Sand (≈50 m)

Bentonite / Sand (≈120 m)

Bentonite / Sand (≈105 m)

Concrete Monolith

Repository Horizon
Features of OPG DGR relevant to Room Backfilling

• Potential large amount of gas generated
  o Due to inherent organic/metal content of L&ILW and containers

• Extremely low permeability of the rock
  o Prevents significant loss of generated gas

• Significant C-14 content in gas phase
  o Desirable to contain gas until C-14 decays (~ 60 ka)

• Strength and depth of DGR design and host rock
  o Design is stable without backfill
  oExtent of rockfall will be limited such that shale caprock is not affected
Backfill – Long-term Safety Assessment

- Reference case is repository with no room or tunnel backfill
- Quantitative assessment of a backfilled repository is provided in the NE-BF case
- Results summarized in Section 8.8.5.2 Backfilled Repository, p.581 of Preliminary Safety Report
- More details in Section 7.3.5.2 Backfilled Repository, Postclosure Safety Assessment report (QUINTESSA et al. 2011)
- Detailed modeling further described in Section 5.14, Case NE-BF - Backfilled Repository, Gas Modelling report (GEOFIRMA and QUINTESSA 2011)
Room Backfill – Assessment Results

- Repository gas pressure as a function of time w/wo backfill:
  NE-RC - Reference Case (no backfill)
  NE-SBC – Simplified Base Case (no backfill)
  NE-BF (NW) – Backfilled repository, Non-Water-Limited
  NE-BF (WL) – Backfilled repository, Water-Limited

- Results show potential for high gas pressures with backfilling
Room Backfill Summary – Long-term Safety (1)

- The L&ILW waste and containers have significant organic and metal content
- In the long-term, this is conservatively expected to largely degrade into gaseous CO₂, CH₄ and H₂
- The very low permeability of the enclosing host rock and shaft seals significantly limits gas migration away from repository
- As a consequence, the gas builds up within the repository void space
- Without backfill, the repository gas pressure will equilibrate at around the natural hydrostatic pressure of the surrounding host rock
Room Backfill Summary – Long-term Safety (2)

- With backfill, the available void space in the repository would be reduced by about a factor of three (e.g., void space of ~30% for gravel fill). Reducing volume results in a corresponding higher gas pressure within the repository.
- Higher gas pressure could result in development of fractures, increased rate of gas release from repository, and higher doses due to C-14 in gas.
- Therefore, for long-term safety, it is better to provide more space to ensure that gas pressures remain low.
- The repository design and rock strength provide a mechanically stable environment without room backfilling, thus maximizing the space.
Repository Model

Material Models
- Generalized Hoek-Brown criterion (adopted for support design)
- Elastic-Brittle-Plastic criterion

Methodology
- Analysis of a repository wide model to assess the level of stress in the pillars between the emplacement rooms (3D)
- Analysis of the emplacement rooms in a rock mass represented as a continuum (2D)
- Analysis of the emplacement rooms in a rock mass with discrete bedding planes (2 realizations, 2D)
- Detailed analyses of selected intersections (3D)
Repository Model

areas identified for 3D analyses
2D Modeling – Geometry and Mesh

- Cobourg Formation – Lower Member
- Cobourg Formation – Collingwood Member
- Blue Mountain Formation
- Sherman Fall Formation
2D Modeling – $K_h = 1.5$; Continuum Model

Generalized Hoek-Brown Criterion – Residual Strength based on $D = 1.0$

**GENERALIZED HOEK-BROWN – RES: $D = 1$**

**Peak:**
- $\sigma_c = 114.0$ MPa
- $m_b = 7.426$
- $s = 0.2946$
- $a = 0.5$

**Residual:**
- $\sigma_c = 114.0$ MPa
- $m_{bres} = 5.014$
- $s_{res} = 0.1599$
- $a_{res} = 0.5$
2D Modeling – $K_h = 1.5$; Discrete Bedding – Realization 1 and 2

**Generalized Hoek-Brown Criterion**

Residual Strength based on $D = 1.0$

**Bedding Realization 1**

- Peak: $\sigma_c = 114.0$ MPa
- Residual: $\sigma_c = 114.0$ MPa
- Stiffness: $k_n = 1200$ GPa/m
- Residual Stiffness: $k_{sres} = 460$ GPa/m
- Tensile Strength: $\sigma_t = 1.66$
- Friction Angle: $\phi = 38^\circ$
- Inclination: $\delta = 5.0^\circ$

**Bedding Realization 2**

- Peak: $\sigma_c = 0.6$ MPa
- Residual: $\sigma_c = 0.0$ MPa
- Inclination: $\delta_{res} = 0.0^\circ$

---

**Generalized Hoek-Brown – Res: D = 1**

- Peak: $\sigma_c = 114.0$ MPa
- Residual: $\sigma_c = 114.0$ MPa
- Stiffness: $k_n = 1200$ GPa/m
- Residual Stiffness: $k_{sres} = 460$ GPa/m
- Tensile Strength: $\sigma_t = 1.66$
- Friction Angle: $\phi = 38^\circ$
- Inclination: $\delta = 5.0^\circ$

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**Inciipient Weak Planes:**

- Peak: $c = 0.6$ MPa
- Residual: $c_{res} = 0.0$ MPa
- Tensile Strength: $\sigma_t = 0.0$ MPa
- Inclination: $\delta_{res} = 0.0^\circ$
2D Modeling – $K_h = 1.5$; Discrete Bedding – Realization 1 and 2

**Maximum bolt load** = 6.7 tonnes
**Maximum bedding stress** = 68.7 MPa at the bottom corner
**Minimum bedding stress** = -0.17 MPa in the floor
**Bedding stress in the crown** = 0.012 MPa (due to bolts)

**Average bolt load** = 3 tonnes
**Maximum displacement** = 6.5 mm in the crown
**Maximum bedding aperture** = 0.1 mm in the floor
**Bedding aperture in the crown** = 0.01 mm

**GENERALIZED HOEK-BROWN – RES: D = 1**

**Peak:**
- $\sigma_c = 114.0$ MPa
- $m_b = 7.426$
- $s = 0.2946$
- $a = 0.5$

**Residual:**
- $\sigma_c = 114.0$ MPa
- $m_{bres} = 5.014$
- $s_{res} = 0.1599$
- $a_{res} = 0.5$

**INCIPIENT WEAK PLANES:**

**Stiffness:**
- $k_n = 1200$ GPa/m
- $k_s = 460$ GPa/m

**Peak:**
- $c = 0.6$ MPa
- $\phi = 38^\circ$
- $\sigma_t = 1.66$
- $\delta = 5.0^\circ$

**Residual:**
- $c_{res} = 0.0$ MPa
- $\phi_{res} = 38^\circ$
- $\sigma_{res} = 0.0$
- $\delta_{res} = 0.0^\circ$
Typical Ground Control – Emplacement Rooms
Typical Ground Control – Emplacement Rooms
3D Modeling – North Panel Connection to Ventilation Tunnel

**Boundary Element Mesh**

**Yielded Zones** (consistent with 2D analyses)

**Zones Exceeding Tensile Strength**

**GENERALIZED HOEK-BROWN:**

Peak:

\[
\sigma_c = 114.0 \text{ MPa}
\]

\[
m_b = 7.426
\]

\[
s = 0.2946
\]

\[
a = 0.5
\]

- **Yellow** Overstressed zone
- **Red** Zone exceeding tensile strength of rock
3D Modeling – South Panel Connection to Ventilation Tunnel

Boundary Element Mesh

Yielded Zones (consistent with 2D analyses)

Zones Exceeding Tensile Strength

GENERALIZED HOEK-BROWN:

Peak:
\[ \sigma_c = 114.0 \text{ MPa} \]
\[ m_b = 7.426 \]
\[ s = 0.2946 \]
\[ a = 0.5 \]

Overstressed zone

Zone exceeding tensile strength of rock
3D Modeling – Connection to an Access Tunnel

GENERALIZED HOEK-BROWN:
Peak:
\[ \sigma_c = 114.0 \text{ MPa} \]
\[ m_b = 7.426 \]
\[ s = 0.2946 \]
\[ a = 0.5 \]

Yielded Zones (consistent with 2D analyses)

Zones Exceeding Tensile Strength

Overstressed zone
Zone exceeding tensile strength of rock
Long-term Stability Assessment

Long term performance based on numerical stability analyses incorporating realistic but conservative predictions for:

- Time-dependent strength degradation
- Effects of gas pressure build-up
- Seismic ground shaking
- Glacial loading and unloading
- Combinations of these effects
Long-term State

Long-term state after 1-2 glacial and interglacial periods and 100,000 years:

- Damage to walls, roof & floor but pillars remain functional

Ultra-long-term state after 8-10 glacial periods and 1,000,000 years:

- Immediate roof will collapse and pillars will lose load capacity

Key to system stability over indefinite time frame (1,000,000 year +):

- Broken limestone beds and blocks will bulk (volume increase) and choke off further collapse
- Displacements will not cause rupture of overlying or underlying shales
Crack Initiation Stress and Lower Bound (Ultra-Long-Term) Strength in Cobourg Formation

- Conservative lower-bound rock strength is assumed (45MPa at 0 confinement)
- Advanced Long Term model:

  Rate of decay and long-term minimum strength are functions of confining stress

\[
\text{Long-term Lower Bound} = \frac{\sigma}{\sigma_{uc}}
\]

\[
\text{CI/UCS} = \frac{\sigma}{\sigma_{uc}}
\]
Detailed 2D Response Models

Bedding and “Voronoï” block model

Allows rock to disintegrate into discrete blocks to evaluate post failure response
Multiple Glaciations and End State

Glacial Cycle 1 (+50 kyr)
Glacial Cycle 2 (+150 kyr)
Glacial Cycle 3 (~300 kyr)
Glacial Cycle 8 (+1 Myr)
**Short Term**

Wall Damage Minor Spalling (arrows)

**Long Term**

Limits of Caving vs Bulking in % increase
Stability and Integrity of Overlying Formations

- 3D panel model to look at effects of caving in Cobourg Formation on stability and integrity of overlying formations
- Includes increased width of barrier pillar between panels to 40 m
Differential Displacement at base of Georgian Bay Shale

No long term yield in key overlying shales – Georgian Bay and Queenston
Thank You