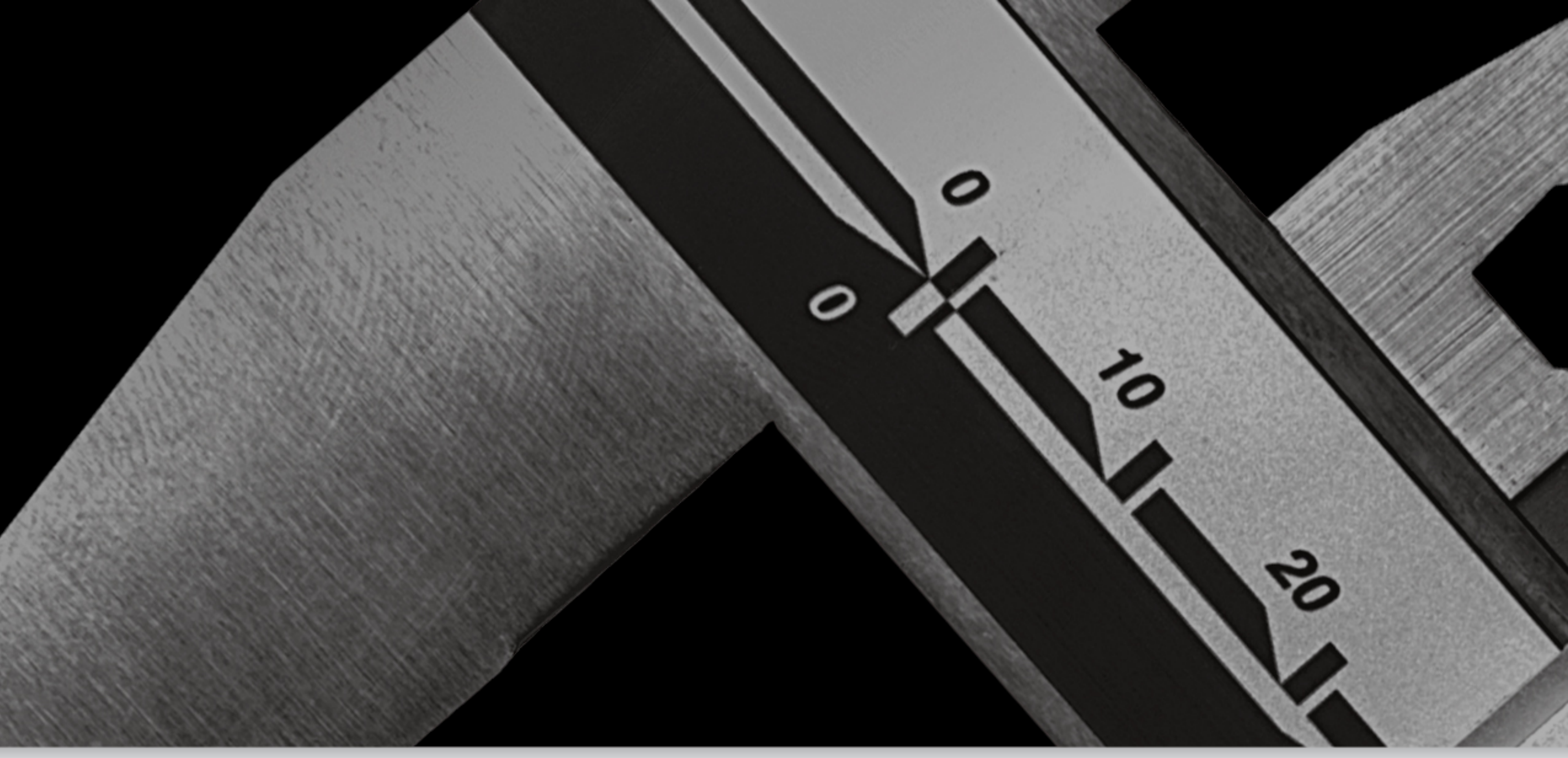
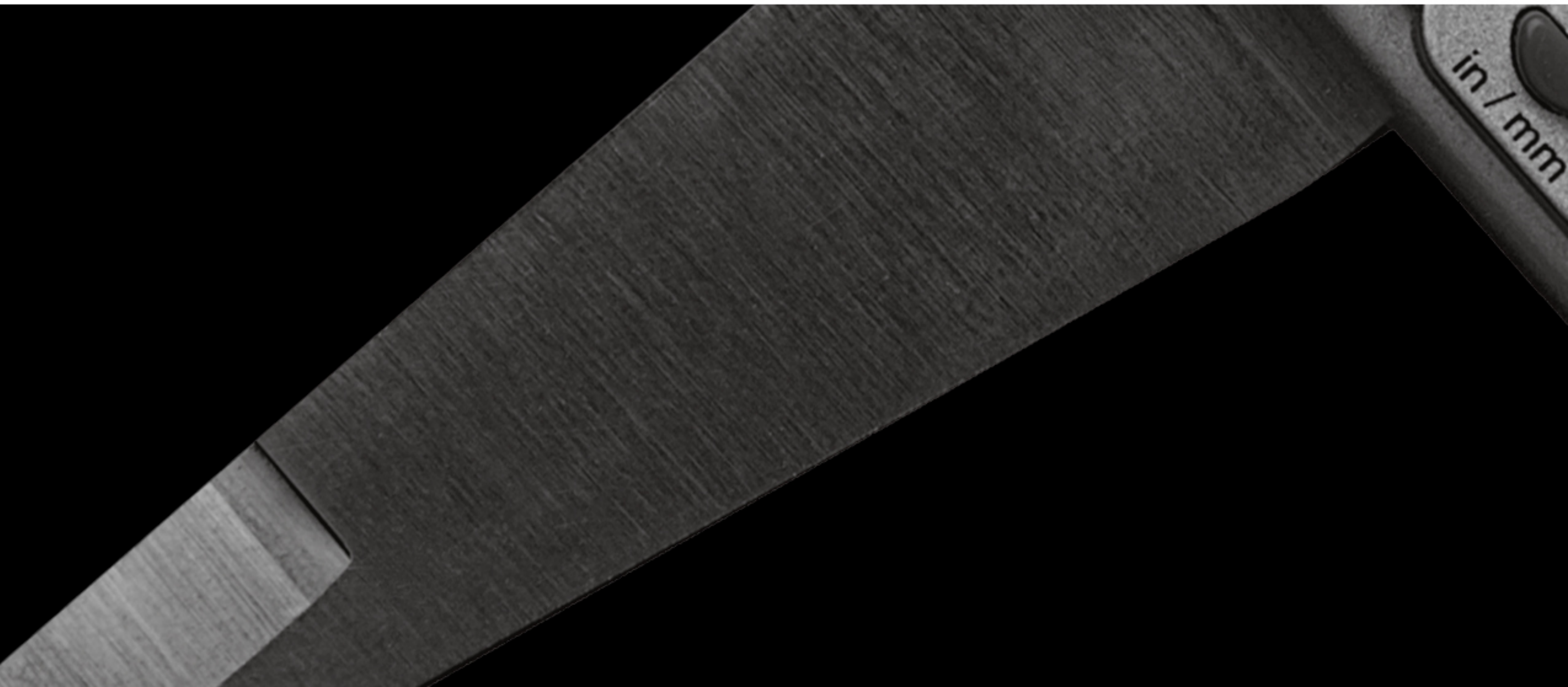


REPORT: T021056-G1

CITY OF BROCKVILLE
CONDITION INSPECTION
BROCKVILLE RAILWAY TUNNEL
BROCKVILLE, ONTARIO

November 8, 2012



Ottawa, November 8, 2012 DRAFT

Mr. Conal Cosgrove, P. Eng.
Director of Operations
City of Brockville
1 King Street West
Brockville, ON
K6V 5V1

Subject: Condition Inspection
Ref No : T021056-G1
Brockville Railway Tunnel
Brockville, Ontario

Dear Mr. Cosgrove:

In response to the Terms of Reference and our proposal no. FP3768, for the above noted project, Inspec-Sol Inc. (Inspec-Sol) has prepared the following report for your review.

We trust this provides the required information and look forward to being of continued service. If you have any questions, please do not hesitate to contact us.

INSPEC-SOL INC.

Myles A. Carter, M.Sc., P.G.
Associate
Manager, Building Science

MAC/vl

CITY OF BROCKVILLE

CONDITION INSPECTION
BROCKVILLE RAILWAY TUNNEL
BROCKVILLE, ONTARIO

Date :	**November 8, 2012**	Our Ref. :	**T021056-G1**

City of Brockville
1 King Street West
Brockville, ON
K6V 5V1

Condition Inspection
Brockville Railway Tunnel
Brockville, Ontario

Ref. No.: T021056-G1
November 8, 2012

Prepared and Approved by :

Myles Carer, M.Sc., P.G.
Associate

Distribution : Client – Conal Cosgrove, P. Eng., Director of Operations, City of Brockville
(Copy by e-mail:(ccosgrove@brockville.com) and by mail)

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1.0 Introduction

An important part of the heritage of The City of Brockville (City), the Brockville Railway Tunnel was closed after the last train passed through in 1970. Having been abandoned since, the City is considering its restoration and re-opening to the public. The Brockville Railway Tunnel Committee was commissioned by City Council to determine the feasibility of opening the entire tunnel for public access. Consequently, the City issued an RFP seeking the engineering services of a consulting firm to undertake a condition survey of the tunnel based upon the recommendations in a preliminary report issued by Stantec Consulting Ltd. The recommendations included the following three items:

1. A geotechnical/soil investigation to confirm the condition of the material(s) surrounding the tunnel in the north and south portions. Previous studies were completed in 1984 and general observations may not reflect the current (hidden) conditions.
2. A rock mechanics/geomechanical specialist to inspect the bedrock portion of the tunnel, including the vertical raise, and complete the necessary studies and test work to confirm the integrity as well as the potential requirement for long-term ground support such as rock bolts, cable bolts, screen, etc.
3. The brick-lined chimneys require inspection/test work as required throughout the entire length of each chimney to confirm the integrity as well as the potential requirement for long-term support.

This report outlines the approach and methodology to address each of the items, site limitations, field observations and recommendations, budgets for corrective work, and recommendations for further study.

2.0 Historical Background

The tunnel is Canada's oldest railway tunnel and was built between 1854 and 1860 to allow the Brockville and Ottawa Railway to connect the Brockville industrial waterfront area to the outlying areas lying between the St Lawrence and Ottawa rivers. On December 31 of 1860, the first small train, a wood-burning locomotive and two coaches came through the completed tunnel and the tunnel was officially open for traffic. The Brockville and Ottawa Railway went on to reach Sand Point on the Ottawa River in 1867, which remained the northern terminal for many years until later when the line was carried further up the Ottawa River.

The tunnel is located within a combination of bedrock and stony, sandy, silt till. The north portion is approximately 220 metres and masonry lined, as is the southern portion which is approximately 175 metres. The middle 120 metres comprises of exposed bedrock. The tunnel is approximately 515 metres long and passes under Brockville's City Hall and various City streets. The existing City Hall is also known as Victoria Hall and was built between 1862-1864, shortly after the railway tunnel was completed. Interestingly, the City Hall was constructed directly over the underlying railway tunnel.

In 1882, Town Council carried out repairs and improvements to the building based on plans prepared by Brockville architect O.E. Liston. Details are not available, but it is assumed that space was converted for town offices. The contract for the work was awarded to John Loftus for $1,814. In 1886, the space which up to that time had been used for wagon passage through the central shaft of the whole building was incorporated into the building and the entrance doors at each end were closed up.

In 1904 two additional floors were added to the rear wing. This completed the new configuration of the building as we see it today. All of the town functions were moved there, including the Police offices and jail which were housed on the ground floor of the rear portion.

2.1 Construction Methods

There is very little information available discussing the actual methods of the tunnel construction. Photographs available suggest the south end was an open cut which is likely due to the ground cover being very shallow, whereas the north end was hand driven through the thicker till deposit. The bedrock portion was a drill and blast (gunpowder) operation, and remnant traces of drilling are visible in the bedrock lining the tunnel. The tunnel is arch-shaped, measuring 14 feet 9 inches from the top of the arch to the ground and 14 feet across.

During our site visits, we noted a series of regularly spaced openings in the tunnel masonry lining along the springline from which we surmise that these were used to insert lumber to serve as shoring for the excavation work and as scaffold platforms for the masons. Construction joints were noted as distinct breaks in the masonry continuity and it is interesting that the limestone blocks terminate as butt joints against each other as opposed to continuing the coursed ashlar pattern which is more typical of masonry construction. The aforementioned openings in the masonry lining were typically reinforced with a stone lintel across the top.

The masonry portions of the tunnel have an interior wythe of limestone block and from what we can see has a more or less rubble type fill behind the interior wythe consisting of various fill materials and pieces of masonry. On the south end this fill was probably placed as the excavation was closed around the tunnel. At the north end, the fill in the over break was likely placed as the tunnel lining advanced. There are likely voids behind the masonry lining but we were unable to quantify this apart from a few visual clues where blocks had fallen, or at the openings for the shoring.

The floor of the tunnel and railways tracks have been removed with the exception of the restored portion at the south end under Water Street.

Victoria hall was constructed over the tunnel, and the masonry arch is visible at the tunnel roof near the west chimney. The arch spans over the tunnel and falls on either side onto what we suspect are the supporting footings. A brief visit into a cramped space overlying the tunnel via a hatch from a lower floor in City Hall did not reveal much additional information or construction detail.

2.2 Past Remediation Work

An article from the Canadian Engineering Heritage Record, dated 1974, highlighted some of the past remedial efforts and issues that have arisen. It was reported that the lined portion of the tunnel was grouted in June 1950, when 277 holes were drilled through the lining and sand and cement grout was injected to fill voids. Apparently, 871 bags of cement were used.

On April 18, 1956, there was a cave-in along a portion of Victoria Avenue, approximately 300 ft (our chainage +/-420m) south of the north portal where a short section of the road surface dropped. Excavation revealed a void 2 ft. deep and 3 to 4 ft. wide at a depth of 16 ft. running parallel to the tunnel axis. The full extent of the void was not determined. The tunnel was examined and considered stable and the excavation backfilled. We surmise the cavity may have been caused by washing out of fines by the circulating groundwater. Our examination of the tunnel in this area indicated some sagging of the roof at the construction joint, local fallout of bricks from the roof and a wooden bulkhead. There is also a small pile of sand and rounded gravel on the tunnel floor nearby.

It was also reported that the first 100 ft. from the south portal entrance was re-pointed in 1965, but that no other re-pointing work had been carried out for at least 25 years.

In 1985, the section of tunnel below Water Street was exposed, the tunnel lining was consolidated from the exterior, and the profile of the road adjusted to accommodate long trailers which were getting caught on the hump in the road.

Although the tunnel has remained closed for many years, restoration work comprising of the disassembling and reconstruction of the masonry wall at the north portal of the tunnel was completed in 2009, and restoration of the stone facing at the south entrance during the summer of 2012.

3.0 Investigation Approach

Inspec-Sol's approach for each of the 3 specific tasks was as follows:

3.1 Geotechnical Investigation

The borehole investigation program consisted of the advancement of six (6) boreholes (BH-C – BH-H) to 6.0 m or to practical refusal, which ever was less, with one location (BH-E) further advanced into bedrock by diamond coring techniques. In addition, three (3) piezometers were installed to monitor groundwater levels.

The methodology of the program was as follows:

- Locate the centre line of the tunnel and place demarcations along Victoria Street. This enabled the boreholes to be accurately offset to allow them to be drilled within 1 m of the sides of the tunnel.
- Boreholes were not drilled over the tunnel as part of this program.
- The boreholes were located at alternating sides of the tunnel at approximately 50 m spacing along the length.
- The boreholes were within the public right-of-way of Victoria Street.
- Standpipes were installed in three of the non-cored holes to depths of 3 m to allow monitoring of the groundwater levels, if present within the soil overburden zone. These were outfitted with flush mounted covers in the road surface to allow access but not be an obstruction.

3.2 Bedrock Condition Survey

For the rock mechanics aspect of the mandate, Inspec-Sol partnered with Dr. Mark Diederichs, professor in Geomechanics and Rock Engineering in the Department of Geological Sciences and Geological Engineering at Queen's University in Kingston, Ontario.

The field work consisted of a visual assessment and field mapping of the tunnel with attention to geological structures such as joints sets and their condition, weathered bedrock, intersection of discontinuities and potential for kinetic failures (wedges, gravity falls from roof and sidewalls, etc…), groundwater infiltration and overall stability. The tunnel was chained every 20 m and the distance marked on the east sidewall of the tunnel with white chalk, starting from 0 at the south entrance to 520 m at the north.

In addition to the classic field approach a LIDAR (Light Detection and Ranging) survey was conducted along the entire length of the tunnel to produce extremely accurate, detailed 3-D measurements of the ground. This non-destructive terrain modeling allows one to see changes in soil and bedrock spatial relationships, and allows one to understand the as-built tunnel geometry. It also provides;

- The identification and measurement of geologic structures
- Rockmass characterization
- Rockfall source detection (fallout, spalling)
- Overbreak or scaling measurement
- Tunnel deformation that may have occurred over the life of the structure and is critical to understanding the long-term stability for the proposed use.

3.3 Masonry Condition Survey

The assessment comprised of several walk-through's and visual inspections of the masonry walls to evaluate the presence of cracks and their size and depth, voids, delaminated and spalled mortar or masonry, friable joints, presence of bulges and indications of movement, efflorescence and staining indicative of water migration, and other signs of distress.

The Lidar survey was also used to detect voids in the masonry joints, bulges, and areas of higher humidity due to seepage occurring behind the masonry.

It was intended to retrieve samples of the masonry and test their mechanical properties (absorption, saturation coefficient, strength, sensitivity to efflorescence) however, it was discovered that the walls are constructed of limestone block and not red clay brick. Also, given the nature of the construction, it was not feasible to remove blocks as this could introduce instability to the walls.

The chimneys that lead from the tunnel and through the roof of City Hall are confined entry areas, and were investigated using a HD video camera.

4.0 OBSERVATIONS AND DISCUSSION

4.1 GEOTECHNICAL INVESTIGATION

The purpose of the borehole investigation was to evaluate the subsoil stratigraphy adjacent to the tunnel structure at the north and south ends of the tunnel as recommended in the Stantec report. Following our initial visit to get familiarized with site conditions, we added boreholes to verify in-situ soil and groundwater conditions along the full length of the tunnel.

4.2 FIELD INVESTIGATION

4.2.1 Field Program and Modifications to Field Program

The borehole investigation program consisted of the advancement of six (6) boreholes (BH-C – BH-H) to 6.0 m or to practical refusal, which ever was less, with one location (BH-E) further advanced into bedrock by diamond coring techniques. In addition, three (3) piezometers were installed to monitor groundwater levels. A drilling subcontractor (G.E.T Drilling) was retained by Inspec-Sol to carry out the work, which was monitored by Inspec-Sol technical field staff.

Public utility locates were obtained through the Ontario One-Call service prior to the start of field work.

Planned boreholes BH-A and BH-B were unable to be advanced during the scheduled field days, as a complete road closure would have been required to perform the work safely. The holes were unable to be relocated in the field as this issue was present in the entire work area in which underground service locates were completed.

Borehole BH-C was terminated due to the possibility of drilling into an improperly, or un-located buried service. BH-C was unable to be relocated in the field due to the proximity to other existing marked services.

4.2.2 Boreholes

The drilling program was undertaken on September 13 and 14, 2012 by means of a truck-mounted CME55 drill rig with continuous flight auger equipment. Standard Penetration Tests (SPTs) were performed at regular intervals using a 50 mm diameter split-barrel sampler and a 63.5kg hammer free falling from a distance of 760 mm. The number of drops required to drive the sampler 0.3 m was recorded as “N” value. Where applicable, the undrained shear strength of the collected samples was assessed using a pocket penetrometer. All samples were stored in air-tight containers and transported to the Inspec-Sol geotechnical laboratory in Kingston, Ontario.

Bedrock samples were retrieved from borehole BH-E using N-sized wireline diamond coring equipment, in order to confirm the existence of bedrock and comment on rock type and quality.

PVC standpipe piezometers were installed in boreholes BH-E, BH-F and BH-H.

All boreholes were backfilled with auger cuttings and a bentonite hole plug upon completion of drilling. Boreholes in public roadways were capped with a minimum of 50 mm of cold-patch asphalt.

All boreholes were laid out by Inspec-Sol personnel. The approximate locations of the boreholes are shown on the Borehole Location Plan, attached as Dwg. No. T021056-G1-2 at the end of this report.

4.3 SUBSOIL CONDITIONS

In general, soils encountered at the borehole locations consisted of a surficial covering of asphalt or topsoil, overlying a layer of fill, overlying a native glacial till. Bedrock samples retrieved from borehole BH-E were found to be a light grey, fine grained equigranular, lightly foliated gneiss.

General descriptions of the subsurface conditions are summarized in the following sections, with a graphical representation of each of the borehole locations provided on the Borehole Logs, attached as Enclosure Nos.: 1 to 6 at the end of this report. Notes on Borehole and Test Pit Logs are provided as Appendix A, at the end of this report.

4.3.1 Cover and Fill Material

A surficial covering of asphalt was encountered in boreholes BH-C through BH-F, ranging in thickness from approximately 37.5 mm to 75 mm. A sandy topsoil cover was encountered in boreholes BH-G and BH-H, ranging in thickness from approximately 100-200 mm. The topsoil was found to be generally greyish brown in colour and recovered in a damp condition.

Sand and gravel fill material (granular base course) was encountered under the asphalt cover in boreholes BH-C through BH-F. This material was found to extend to approximately 0.75 m below surface grades. A sandy fill material with some sub-round gravel and traces of silt was found to extend to approximately 3 m to 4.5 m below existing grades in BH-G and BH-H. This material was found to be generally brown in colour, had a very loose to loose consistency, and was recovered in a damp to moist condition.

4.3.2 Native Glacial Till

Native sandy with some sub-round gravel and traces of silt, containing some possible cobbles and boulders was found to underlie fill materials in all boreholes BH-C through BH-G. This material can be described as a glacial till. This material was found to extend in thickness up to 4.8 m, was found to be generally greyish brown to in colour, ranged in consistency from loose to very dense, and was recovered in damp to moist conditions. Glacial till soils are considered to contain a distributed range of particle sizes from clay to boulder sized materials.

4.3.3 Native Sand

A native silty fine sand with traces gravel was found in BH-G underlying a small layer of glacial till, and in BH-H underlying fill materials. This material was sampled up to approximately 8.0 m, and further probed with by Dynamic Cone Penetration Test (DCPT) to a refusal depth of 10.4 m on assumed bedrock. This material was found to be generally light brown in colour, ranging in consistency from loose to dense, and was recovered in a wet condition. Occasional cobbles or boulders may be possible in this material.

4.3.4 Bedrock

Bedrock samples were retrieved from borehole BH-E were described as being a light greyish white, fine grained, gneiss. RQD (rock quality in the borehole was poor due to the proximity to the surface. Rock encountered within the tunnel is also quartzite with the possibility of intersection with adjacent gneiss to the south end of the rock section. The general rock quality in the tunnel can be described in general as fair (RQD = 60 to 80, RMR 45 to 65, Q = 2 to 9). There are local areas of lower quality rock where structures intersect the tunnel.

There is a regional trend for minor shear zones and dykes trending perpendicular to the tunnel axis within the Precambrian bedrock. It is unclear if these are present in the tunnel although the early workers transitioning to rock tunnelling in the south end appears to have encountered difficulties with poor ground (now bricked over). There are minor shears (5-10 cm wide) striking (intersecting the roof) at high angles to the tunnel axis.

The predominant jointing (natural fracture patterns) are sub-vertical and striking within 30 degrees of the tunnel axis. There is a strong horizontal joint set in the roof of the tunnel. The combination of these two sets of structures results in the squaring of the tunnel profile in sections. Towards the north end of the rock segment, the vertical joints rotate away from the tunnel axis resulting in small tetrahedral wedges that have fallen from the walls, most likely at the time of construction.

There are sections of poorer rock quality, particularly at the south end of the rock segment. There will be a need for very selective scaling (removal of potentially loose rock). It will be important to scale only the most obvious loose blocks as it is possible in a shallow tunnel to manually unravel the rock blocks around the tunnel reducing the overall stability. It is important to note that there has only been one fall of rock from the roof since the tunnel was abandoned. This fall originated as rock blocks bounded above by a strong horizontal fracture. Intelligent scaling will be able to detect and remove any similar hazards in the tunn

4.4 GROUNDWATER

Three standpipe piezometers were installed in order to monitor standing water conditions in the overburden materials surrounding the tunnel. Water levels were measured on October 18, 2012 and are presented in the table below.

Table 1: Observed Groundwater Levels

Borehole	Water Level Reading (m b.g.s.)	Date
BH-E	3.44	Oct 18/12
BH-F	1.78	Oct 18/12
BH-H	5.30	Oct 18/12

It should be noted that groundwater table is subject to seasonal fluctuations and in response to precipitation events, and is anticipated to be at its highest level during wet seasons.

5.0 BEDROCK CONDITION SURVEY

The full report prepared by Dr. Diederichs is presented in Appendix B. Some of the more salient points with respect to bedrock condition and support requirements are presented below.

Advancing into the tunnel from the south, the first bedrock exposure that is encountered is a narrow 1 m window in the brickwork (deliberately created during construction) at approximately 139 m from the south portal. This rock is very blocky, fractured and weathered. Stained gouge is present inside some of the fractures. This is likely adjacent to a regional shear zone (common in the Brockville area). It would appear that this was a trial excavation finish. The brickwork resumes after this slot. This window of bedrock will require scaling. The slot is narrow enough that additional support may not be required. The brick edges on either side of the slot are in good condition and can be left as an example of the lining technique.

The next exposed bedrock section begins at 154 m. There is a partial lining in the roof or the mortar impressions of a lining that has since been removed. This rock mass is heavily jointed with a reduction in block size. There are no immediate signs of dangerous loose but this section will require scaling and possible bolting. Evidence of a fault zone appears at the end of this section (160 m). This section and the brick edge will require special attention and reinforcement.

The edge of the brickwork entering and leaving this section (to 160 m) will need careful inspection and possible stabilization. There is moderate inflow and some flow deposits in this section. There is a short section of brickwork between 160 m and the full exposed rock section at 167 m. This is likely in response to the fault zone detected at 160 m. The brick liner appears to be continuous and with limited disturbance. Horizontal offset of bricks along length of lined section at the left and right upper corners - likely due to construction techniques. The last brick section in the south ends at 167 m where more evidence of the boundaries of a fault zone are present. The rock mass at the edge of the brickwork is blocky with significant enlargement of the tunnel profile. The brick edge will require some attention (backfilling with grout and pointing) for long term stabilization although this brick edge can remain exposed.

From 168 m to 200 m the rock mass shows a more competent but still blocky nature. The walls and roof are stable but may require spot scaling. It will be important not to over-scale in such a shallow tunnel (even stable blocks may sound "loose" without confinement). Doing so may induce instabilities that do not currently exist. There is a possible fault or contact between 170 and 175 m shown on the Lidar scans, some of which are included in Appendix D. North of this location the jointing in the rock changes dramatically to include highly persistent vertical joints parallel to the tunnel and well developed horizontal jointing. This is likely the contact between the gneissic units to the south and the quartzite (all Precambrian units underlying the more recent Nepean sandstone and carbonate units for which Brockville architecture is famous). It is possible that this folded contact reappears in sections of the tunnel to the north (as predicted by regional mapping) but most of the tunnel is in quartzite.

A concentrated flow of water occurs from a point location in the roof at 180 m. This flow has been observed to be constant and does not vary with weather or precipitation history. This could possibly indicate a municipal source. Significant precipitate deposits have developed beyond 185 m.

The only recent failure (since closure in 1970) is observed at 200 m where flat blocks of quartize (4 blocks totalling less than .3 cubic metres) have peeled off the roof where a well developed horizontal joint has been intersected by a small shear structure. Careful sounding and scaling will be required to eliminate any similar blocks in the roof. Particular care should be taken where strong horizontal jointing is visible in the roof. The tunnel profile from 180 m to 220 m is entirely controlled by strong vertical jointing parallel to the walls and strong horizontal jointing in the roof. With the exception of small slabs that merit careful scaling, the tunnel, while enlarged and squared off, is stable in this section. This is the most visually impressive section of the geology within the tunnel and care should be taken to preserve this feature while maintaining safety.

After 220 m the rock becomes blocky again with some scaling requirements and possible bolting. Particular areas of concern occur at 222 m, 240 m, 255 m and 265 m (scaling and bolting possibly required). A fault crosses the tunnel at 272 m with minimal impact on stability. In general, beyond 240m the major vertical structure rotates to a trend (strike) of 15 to 20 degrees with respect to the tunnel. This leads to more blocky nature and wedge fallout in the east wall.

A shaft at 230m (presumably for rock removal) is in poor condition and will need stabilization. A timber cover appears at the top of the visible shaft. It is unknown what overlies this cover. The tunnel stability is not affected but the shaft itself poses a hazard and will need to be covered in some way (possible with plexiglass to allow viewing and lighting). The top of the visible shaft will require a new bulkhead.

The rock section continues to 295 m, with a joint controlled irregular profile and significant flow deposits, where the dug tunnel in till begins with stone brickwork. The leading edge of the brickwork at the brick-rock interface is highly irregular. The brick arch abutment climbs up on a blasted rock edge from the floor at 308 m to half height at 295 m. The leading edge at 295 m is supported by highly corroded rock bolts from the original construction. While the rock is stable, the tapered base of the brickwork and the interface at 295m requires significant work to stabilize, buttress and replace the aging rock bolts.

6.0 MASONRY CONDITION SURVEY

6.1 Tunnel Lining

A short portion of the south tunnel entrance was rehabilitated when repairs to the overlying Water Street were conducted around 1985. There is a small interpretation centre about the history of the tunnel also on display. A wood plank floor and railway track was also preserved. Past the interior locked iron gate, the ground surface is exposed gravel and soil, with running and standing water creating a muddy walking surface in many areas.

Typically, the masonry (and bedrock) surface is blackened from the creosote deposits generated by the train over the full length of the tunnel. Mortar joints are soft given their lime-based composition at the time of construction, and that the tunnel is a very humid environment. Attempts were made to retrieve intact samples of the mortar for analysis, but all attempts yielded sand or very weak intact samples that fell apart. This is typical of lime based mortars where, over the years due to constant humidity and water migration, the lime has been leached out. Lime deposits on the walls of the tunnel were observed throughout.

Although many joints are open, the masonry appears stable due to the nature of the arch design which keeps the bricks under compression. At one time, culverts were installed along the base of the wall, but most of their length has fallen into disrepair and do not function as intended. Representative photographs of our observations are included in Appendix C.

As one progresses through the tunnel from 00 m to 155 m and 160 m to 167 m (there is exposed rock mass between 155 m to 160 m) the exposed masonry is generally stable and in fair condition given its age and lack of maintenance over the years. Persistent water infiltration and light calcium deposits start to appear on the west side at a chainage of around 133 m. Very heavy flow deposits are present between 160 m-167 m. Further discussion regarding these deposits is presented in a later section.

From 295 m to 310 m (the tunnel is bedrock from 167 m to 295 m) there is a mixture of bedrock and masonry lining where the bedrock section transitions back to masonry. There are ledges of brick which rest on the bedrock at various heights along the tunnel wall and the mortar has washed out leaving a potentially unstable condition for the bricks which are marginally supported. The rock bolts that were installed sometime in the past, are highly corroded with very little cross-section remaining. This will be one area requiring stabilization.

From 310 m to 390 m the masonry lining is relatively undisturbed. There is minor to moderate mortar loss with local missing bricks in the walls. The vertical construction joint at chainage 378 m appears to have some vertical offset and the bricks at the roof appear loose. Some anchorage of the masonry using Cintec type anchors which are designed for use in stone masonry may be required at these locations. Wall deposits with streaks of the dark brown iron staining are typical.

From 395 m to 465 m there is more evidence of local disturbance of the masonry lining. Bulging in the mid and sidewalls was noted as well as local brick fall-out (1 to 3 bricks) from the sidewalls and ceiling. Mortar loss from joints is moderate to heavy. Some anchorage of the masonry using Cintec type anchors may be required at these locations. There is a wooden bulkhead at 430 m which may be remnants of the tunnel construction or possibly installed later to cover a void and prevent further collapse and inflow of overburden. At 465 m there is a small hole in the roof where wood timber is exposed. From 420 m to 456 m there is an apparent ledge of a few centimetres in the masonry which may be the result of local deformation. At 456 m there is a hole in the tunnel roof with exposed timbers, its purpose is unknown. There are numerous water infiltration points and the flow deposits are quite heavy along the tunnel in this section.

From 465 m to 490 m there is more obvious disturbance of the masonry lining with bulges in the tunnel walls and moderate to complete loss of mortar. Some anchorage of the masonry using Cintec type anchors may be required at these locations. At 470 m there is a small collapse of the brick lining into the tunnel at the base of the west wall. Repair patches of concrete have also been installed, likely where masonry blocks have fallen out exposing the surrounding glacial till. Given that the overburden is quite thick along this section of the tunnel, the potential for more groundwater inflow and freeze-thaw during the winter may be contributing to some of the observed distress. Mr. Silburn noted that the walls were coated with ice in the winter during some of his past tunnel visits.

From 490 m to 520 m the masonry appears to be more stable. There is moderate mortar loss from the brick joints along the tunnel roof. The vertical construction joint at 493 m has some offset near the middle of the roof, which may be a local instability. Cintec anchors may be required in this area. Perforated PVC piping installed during the 2009 re-construction of the portal masonry wall at the north end of the tunnel captures and direct waters to the tunnel floor. Groundwater flows along the floor of the tunnel down slope towards the south entrance where a pipe catches the water flow just before the interpretation area and directs it underground to an exterior catch basin.

6.2 Chimneys

There are two red clay brick masonry chimneys at chainage 90 m that run from the tunnel roof up through City Hall and exit at roof level. It is thought that these served as exhaust vents for the train as it passed through the tunnel. They may have been originally constructed to vent at ground level, but with the construction of Victoria Hall they were extended.

The east chimney is open to view from the tunnel, whereas the west chimney has been boarded up. Although the view from the tunnel is limited, the east chimney masonry appears in good condition. There was minor spalling of brick and the mortar joints appeared intact. Local support in the form of steel rods had been installed. On the tunnel floor, some brick debris and soil was noted.

To assess the condition of the chimneys, a video camera survey was conducted from roof level. Fall arrest procedures were followed. Images captured from the video survey are included in Appendix C. The chimneys are capped with sheet metal and plywood which has been secured to the chimney caps with concrete nails. The caps were partly pried off and the camera passed down the chimney. The interior of the east chimney was viewed down its entire length to the tunnel floor. The upper +/- 1.6 m is stone masonry with a few loose sections of red clay brick. The back (interior of the chimney) of the stone masonry has open joints and it appears that either the stones were not fully set in bedding mortar, or there has been some mortar loss over time. There were a few red clay bricks and some brick debris caught on the sides of the above-roof portion. This top section will require some restoration. Below the roof level the chimney becomes only red brick clay masonry. The immediate section below the roof (to +/-3.9m below top of chimney) is in fair to poor condition, with some erosion of the mortar joints and minor spalling of the brick. This section is probably exposed to freeze-thaw as it is close to the roof line. As one descends into the heated portion of the building, the bricks and mortar joints are in good condition. There is minor brick spalling, the mortar joints are tight, intact, and with few voids or mortar loss noted. At a depth of +/-17.0 m, there is a step in the masonry, probably where the chimney makes a bend before going straight up through the building (see Lidar images in Appendix D). At the depth of +/-18.9 m the masonry becomes a mix of stone and red brick masonry. Some mortar loss is evident but there are no flow deposits or evidence of lime leached out onto the surface of the joints or brick faces. At +/-20.7 m the supporting arch for Victoria Hall is encountered, with the open tunnel thereafter. Given the relatively good condition of the east chimney and traces of brick spalling, we suspect that the debris on the tunnel floor which includes pieces and half-bricks, landed there when the wood hoarding was removed sometime in the past.

The west chimney was also accessed from the roof; however, the opening was blocked at roof level by a concrete cap and debris. As noted in the east, the interior of the above roof portion of the chimney has open joints and will require some restoration.

It should be noted that the plywood caps on the top of the chimneys are showing signs of rot and should be replaced in the near future. The caps were re-installed using Tapcons.

7.0 Flow Deposit and Groundwater Analyses

The heavy build-up of deposits on the tunnel walls and floor indicate long-term water infiltration. The deposits are up to several cms in thickness in some areas which is unusual given that the tunnel is just over 150 years old, and that these types of build-ups that occur naturally in caves take thousands of years. The nature of the deposits in terms of their texture, colour and form add a unique character that would be of interest to the public.

In order to better understand the source of the flow deposits on the tunnel walls, samples of the deposits were taken from the east wall (chainage 180 m) and west side of the tunnel (chainage 230 m). On the west side, a sample of the dark brown deposit was retrieved whereas the east was a white sample. Both samples had very high calcium, high magnesium and relatively high sodium content, all indicative of a groundwater source, since the binder in the masonry mortar is lime. The darker sample was also analyzed for coliform due to its colour, as well as iron. The darker deposits have elevated iron which would account for the brown coloration. No coliforms were found, plus there is no unpleasant odour in the tunnel, which leads us to surmise the source does not appear to originate from a sewer.

A sample of groundwater was retrieved from a constant water source seeping through the roof of the tunnel at chainage 180 m. It is interesting to note that this water flows constantly under both wet and dry weather conditions. In addition, during a field visit with Mr. John Silburn who used to take his civil engineering students through the tunnel on field trips, he noted that it was flowing at this location even then during the 1980's. The groundwater is considered very hard (>180 mg/L CaCO3) which is reflected by the high calcium and magnesium content. Relatively high sodium content is also reflected in the flow deposit analysis; however, the deposits are low in sodium when compared to the water. It was thought that some sodium may be originating from road salt leaching through the soil.

The source of water may be a naturally occurring underground spring which follows the path of least resistance and passes over and through the various sedimentary rocks and overburden leaching out salts as it migrates. Or, possibly from a leaking potable water or fire hydrant supply line which is leaching the elements from the soils in the overlying glacial till. Flouride, measured at 0.27mg/L in the groundwater sample, also occurs as a natural background element, so it is difficult to conclude if the water is a potable source (on the assumption that Brockville adds fluoride to the water supply).

The nature and distribution of the deposits indicates there is a very active groundwater regime in the tunnel. Any repairs and restoration that are undertaken will have to be conducted in a manner that does not radically disturb the groundwater hydraulics, particularly in winter when ice lensing can cause considerable damage due to frost jacking.

The results from the analyses are included in Appendix E.

8.0 PRELIMINARY CONCLUSIONS

8.1 Geotechnical

8.2 Bedrock Stability

The following are excerpted from Dr. Diederich's report:

1. The brick-rock transition at the north end of the rock section will require significant strengthening and rehabilitation. This will include active reinforcement of the base of the bricks as they contact the rock ledge. This may involve a combination of short bolting and a concrete sill. Brickwork in the ceiling at this location will also require attention.

2. The rock chimney at 230 m will require attention. The chimney itself is stable. The bulkhead at the top is of unknown construction and it is uncertain what lies above. It may be desirable to keep the chimney exposed for historical purposes but the walls and upper bulkhead need extensive revision.

3. The rock-brick interfaces throughout will require detailed examination and will need rehabilitation and reinforcement ranging from grout backfilling and re-pointing to the construction of a light reinforced arch (steel and concrete) to protect the brick edge and provide long term stability. This can be done with sensitivity to the aesthetic and historical requirements as well as budgetary constraints.

4. Rock scaling is required in some portions of the tunnel. Rock sounding and very careful and discriminate scaling is probably advisable through the whole rock section although caution is required to avoid overscaling. This tunnel is unique in that most rock that could fall has already fallen and aggressive scaling will create rather than solve problems. Only clearly loose and potentially unstable blocks should be dislodged through scaling.

5. Spot bolting may be required. There does not appear to be any justification for pattern bolting. The rock mass quality would normally require such bolting for a new tunnel if the design arch profile is to be achieved. This requirement is moot for this tunnel as the rock has already broken back to a stable albeit irregular profile. For costing purposes, it is reasonable to assume that up to 1 bolt every 2 linear metres of tunnel may be required (60-80 bolts). In this case 1.5 to 2 m resin grouted rebar (with plates) are recommended for long term reinforcement.

6. The use of shot crete is not advised except as suggested for reinforcement of brick-rock interfaces that require stabilization. It is important to maintain the current level of tunnel drainage in both the rock and brick sections as build up of water pressure could lead to new stability issues.

7. The water seeping through the rock and precipitate formations have little or no impact on rock stability. The gneiss and quartzite are insoluble. The water and minerals are coming from the soil cover and from the Nepean rock units above and to the east.

8.3 Tunnel Masonry

1. The limestone block masonry is generally in good condition. The use of native stone from local quarries is always recommended as it tends to stands up to extreme weather conditions better than many imported materials. Local areas of instability in the form of bulges in the walls or off-sets at construction joints have been identified. Some of these may have occurred at the time of construction (i.e. construction joints) or later as the tunnel lining adjusted to in situ stresses and freeze-thaw conditions during the winter months.

2. The majority of the mortar loss has occurred above the spring line of the tunnel and mainly at the roof where the lime has been leached out and the remaining soft sand has dropped to the floor of the tunnel. Re-pointing must be done with a lime based mortar formulated for the tunnel. It must retain sufficient strength to tie the blocks together and remain in the joints while at the same time allow for expansion and contraction, and more importantly, allow moisture to migrate through the joints. Raking of the joints must be done very carefully in order to avoid over raking and loosening of the blocks.

3. Some of the larger unstable areas in the lower sidewalls which have displaced outwards will require careful dismantling and reconstruction. Smaller bulges in the walls and roof can be secured using Cintec type anchors. The system works by pre-drilling an oversized hole in the structure and inserting an anchor body surrounded by a fabric sock. A cementitious grout is injected through the middle of the anchor under low pressure. It passes through a series of grout flood holes into the fabric sock, inflating the entire assembly like a balloon and conforms to the shape of the interior cavity, binding the assembly together. The structural anchor is designed specifically for the loads and configuration of each application.

8.4 Chimneys

1. Re-pointing of the mortar joints of the above-roof portion will be required, with removal of loose debris and unstable prices of stone and brick. If the chimneys are to be left capped, the current caps should be replaced with new plywood secured with Tapcons. The metal flashing covering the plywood appears to be in good condition.

2. Below roof level to the tunnel, the east chimney appears to be in relatively good condition. Where the chimney widens and meets the arch over the tunnel, there are some open joints that will require re-pointing.

3. The west chimney could not be examined and it is recommended that an effort be made to open the chimney at the roof level to allow passage of a video camera. It is unclear why the hoarding was removed from the east chimney and not the west. Perhaps the hoarding can be carefully removed from the west side to allow a visual assessment from the tunnel.

9.0 Class D Estimates

Class "D" estimates have been developed for the remedial work. A Class D estimate is based upon a statement of requirements, and an outline of potential solutions, this estimate is strictly an indication (rough order of magnitude) of the final project cost, and should be sufficient to provide an indication of cost and allow for ranking all the options being considered. Expected precision variance: -25% to +75%.

For the rock mass stabilisation of the tunnel and vertical raise, including engineering documents, light controlled scaling under engineering supervision, spot rock bolting, support of the leading edges of the brickwork as it transitions to bedrock, a budget of $350,000 is estimated. It does not include reconstruction of the bulkhead at the top of the raise, as that will require further investigation.

In order for any work to be undertaken in the tunnel, a working platform will be required. Our initial thoughts are that the tunnel floor be scraped clean, a geotextile placed, and a 200 mm layer of clean stone be laid. The clean stone will act as a drainage layer for water to pass underneath to the pipe outlet near the south portal. This would be followed by 150 mm layer of Granular A that will provide a working platform and could also be used to support a boardwalk for the public. The sides of the tunnel could also have a culvert installed to catch water dripping down the sidewalls. Water captured by the culverts would re-direct water to the same outlet near the south portal, and could potentially reduce the amount of water flowing on the tunnel floor. Constant flows from the roof tunnel could be re-directed to the side. Estimated costs for the earthworks, not including a culvert is in the order of $75,000.00.

Masonry restoration costs…………

Appendix A

- Borehole Logs and Site Plan

Appendix B

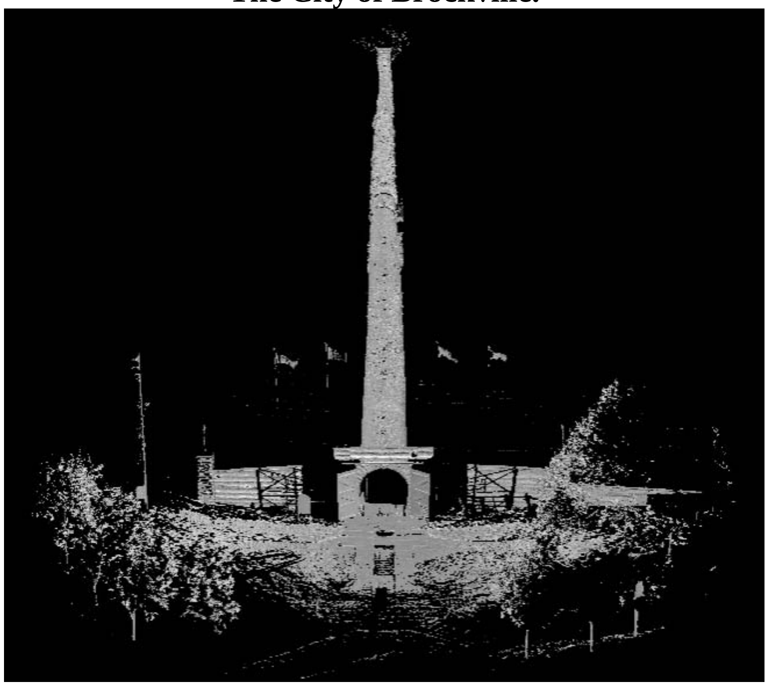
- Rock Engineering Report

MARK S. DIEDERICHS
Professional Engineer and Rock Mechanics Consultant
c/o Innovative GeoMechanics
83 Earl Street, Kingston, Ontario

October 30, 2012

ROCK ENGINEERING STUDY OF THE BROCKVILLE RAIL TUNNEL

A REPORT PROVIDED TO INSPEC-SOL INC.
for submission to
The City of Brockville.



CONTENTS

Dr. Mark S. Diederichs, Ph.D., P.Eng.

SCOPE

For the rock mechanics aspect of the mandate, Inspec-Sol partnered with Dr. Mark Diederichs, professor in Geomechanics and Rock Engineering in the Department of Geological Sciences and Geological Engineering at Queen's University in Kingston, Ontario. The services were provided under the umbrella of Innovative Geomechanics with additional assistance from Dr. Jean Hutchinson. Lidar surveying was provided at reduced rates through a collaboration between Innovative Geomechanics and the Geomechanics Research Group at Queen's.

Mark Diederichs was retained by Inspec-Sol to perform the role of rock mechanics engineer with respect to the investigations and evaluations at this stage of the project. Collaborative input was provided with respect to the brickwork sections of the tunnel and the chimneys. The findings from this work are summarized by Inspec-Sol in the main project report. This sub-report details the rock mechanics survey and findings.

APPROACH

The field work consisted of a visual assessment and field mapping of the tunnel with attention to geological structures such as joints sets and their condition, weathered bedrock, intersection of discontinuities and potential for kinetic failures (wedges, gravity falls from roof and sidewalls, etc…), groundwater infiltration and overall stability. The tunnel was chained every 20m and the distance marked on the east sidewall of the tunnel with white chalk, starting from 0 at the south entrance to 520 m at the north.

In addition to the classic field approach a LIDAR (Light Detection and Ranging) survey was conducted along the entire length of the tunnel to produce extremely accurate, detailed 3-D measurements of the ground. This non-destructive terrain modeling allows one to see changes in soil and bedrock spatial relationships, and allows one to understand the as-built tunnel geometry. It also provides;

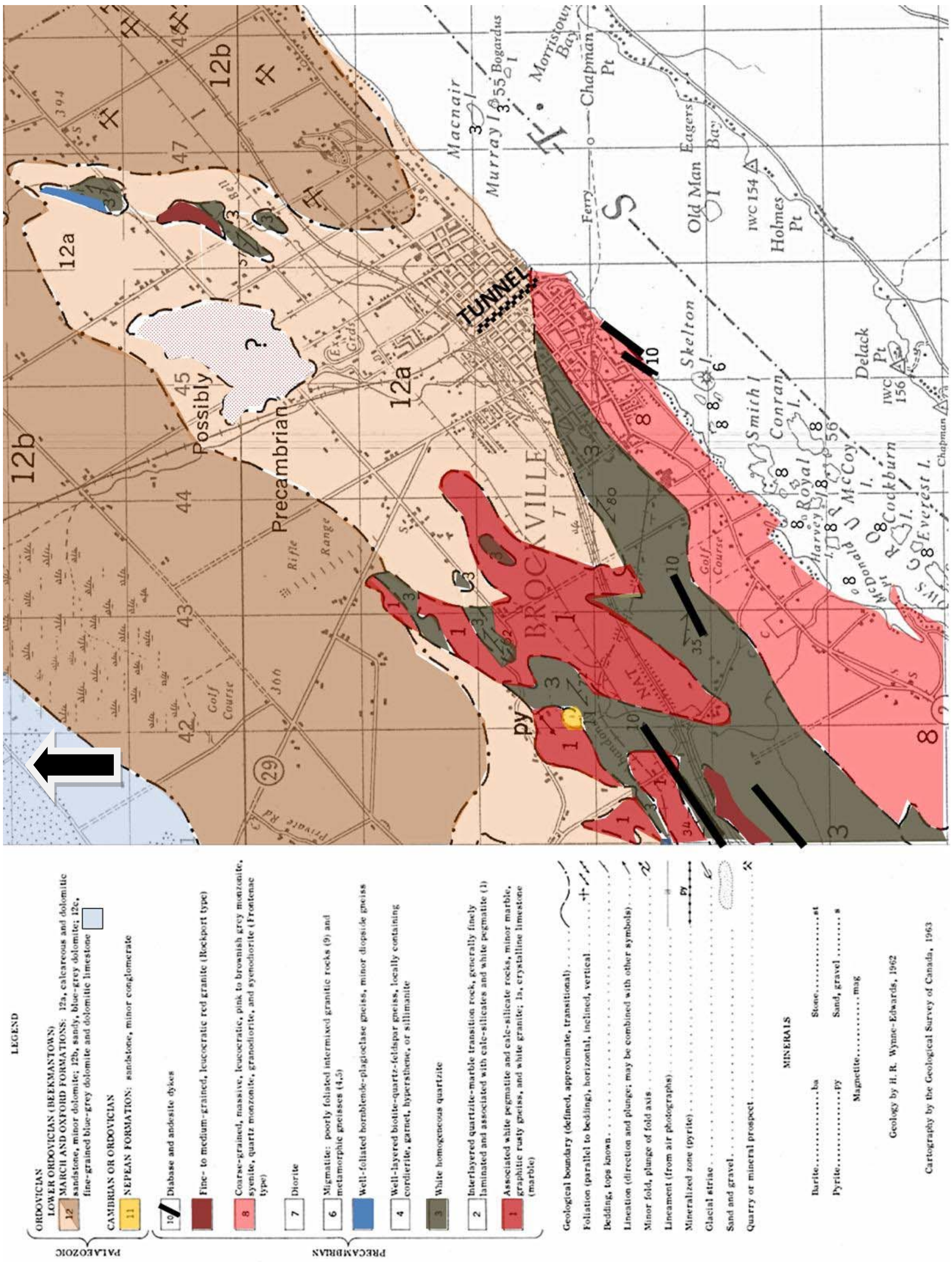
The identification and measurement of geologic structures

- Rockmass characterization
- Rockfall source detection (fallout, spalling)
- Overbreak or scaling measurement
- Tunnel deformation that may have occurred over the life of the structure and is critical to understanding the long-term stability for the proposed use.

The data is a permanent record of the tunnel and can be used for the above purposes during later stages of work.

GEOLOGY

Before embarking on a discussion of rock engineering aspects of the project it is important to understand the geology in the regional context. The map below is adapted from a 1963 Geological Survey black and white regional map.



This is a bedrock map and does not include the sediments (tills and sands) that cover the downtown area.

The south section of the tunnel from is excavated by cut and cover techniques with sand backfill. It is not clear where exactly the transition to dug tunnel begins but a change in construction method around +120m from the south portal indicates that this location may be the start of tunneling. The first exposed rock is at +138 to 139m where a window through the brick has been left open in the original construction. A brief transition to exposed rock occurs again at +155m returning to brick after a few meters, presumably as a result of stability problems. The full rock section of the tunnel resumes at 169m.

The rock in the tunnel is predominantly Precambrian quartzite (3 on the map) although the presence of a gneiss (8) or a syenite (1) is possible as the tunnel sits in a folded regime with these three lithologies. It is possible that there are later more flatly bedded Ordovician sedimentary rocks (calcareous sandstones, dolomites and limestones) lying unconformably above the tunnel or nearby to the east (above a paleo-erosion surface explaining the poor rock quality in the boreholes at the bedrock surface). These sedimentary rocks (12a and 12b) are likely the source of the calcium and magnesium deposits in the tunnel as the drainage gradient is likely southwest..

Bedrock samples were retrieved from borehole BH-E were described as being a light greyish white, fine grained, equigranular metaquartzite. RQD (rock quality in the borehole was poor due to the proximity to the surface. Rock encountered within the tunnel is also quartzite with the possibility of intersection with adjacent gneiss to the south end of the rock section.

The general rock quality in the tunnel can be described in general as fair (RQD = 65 to 85, RMR 45 to 60 , Q = 1 to 20). There are local areas of lower quality rock where structures intersect the tunnel. Rockmass behavior, however, dominated by pervasive joint sets parallel to the tunnel.

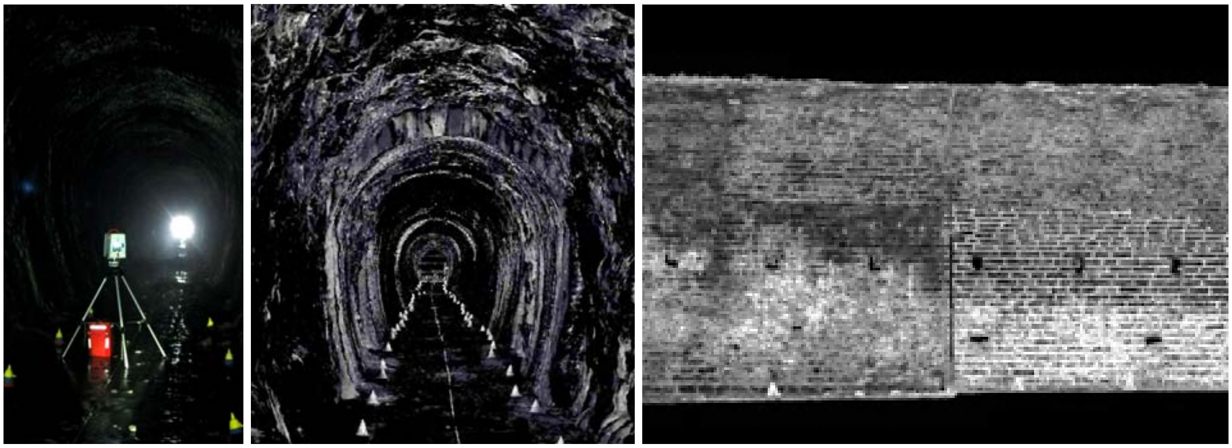
There is a regional trend for minor shear zones and dykes trending perpendicular to the tunnel axis within the Precambrian bedrock. It is unclear if these are present in the tunnel although the early workers transitioning to rock tunnelling in the south end appears to have encountered difficulties with poor ground (now bricked over). There are minor shears (5-10cm wide) striking (intersecting the roof) at high angles to the tunnel axis.

The predominant jointing (natural fracture patterns) are subvertical and striking within 30 degrees of the tunnel axis. There is a strong horizontal joint set in the roof of the tunnel. The combination of these two sets of structures results in the squaring of the tunnel profile in sections. Towards the north end of the rock segment, the vertical joints rotate away from the tunnel axis resulting in small tetrahedral wedges that have fallen from the walls, most likely at the time of construction.

There are sections of poorer rock quality, particularly at the south end of the rock segment. There will be a need for very selective scaling (removal of potentially loose rock). It will be important to scale only the most obviously loose blocks as it is possible in a shallow tunnel to manually unravel the rock blocks around the tunnel reducing the overall stability. It is important to note that there has only been one fall of rock from the roof since the tunnel was abandoned. This fall originated as rock blocks bounded above by a strong horizontal fracture. Intelligent scaling will be able to detect and remove any similar hazards in the tunnel.

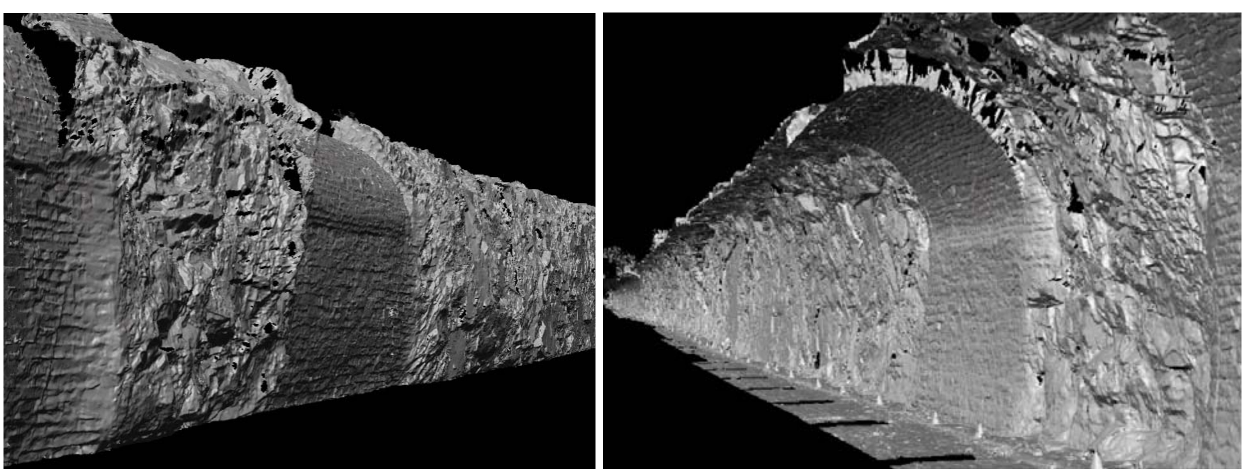
LASER SCANNING

The entire tunnel was scanned in high resolution using Lidar (Laser Detecting and Ranging). The survey produces a three dimensional scan consististing of millions of positioned points from a single tripod setup. The tunnel was scanned at 5m intervals to achieve sufficent overlap. 110 scans in total were taken over a two day period and the data sets were linked together to form a single data file. The data can be represented as point data with laser reflection intensity information as greyscale contours. This representation is useful for identifying wet areas, weathering in clean rocks and other information on the surface quality. In this tunnel, the creosote and precipitate formations make this difficult.

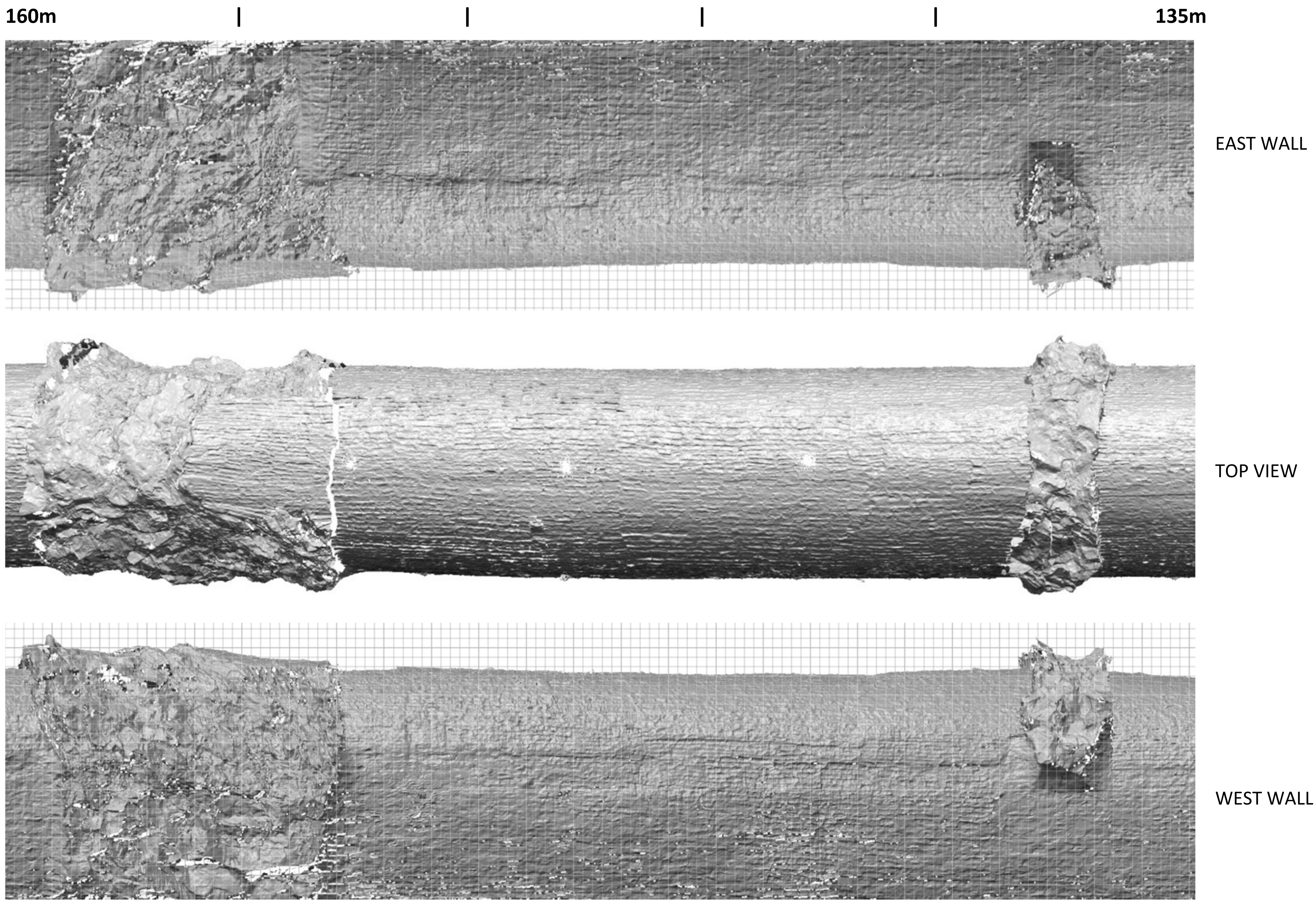


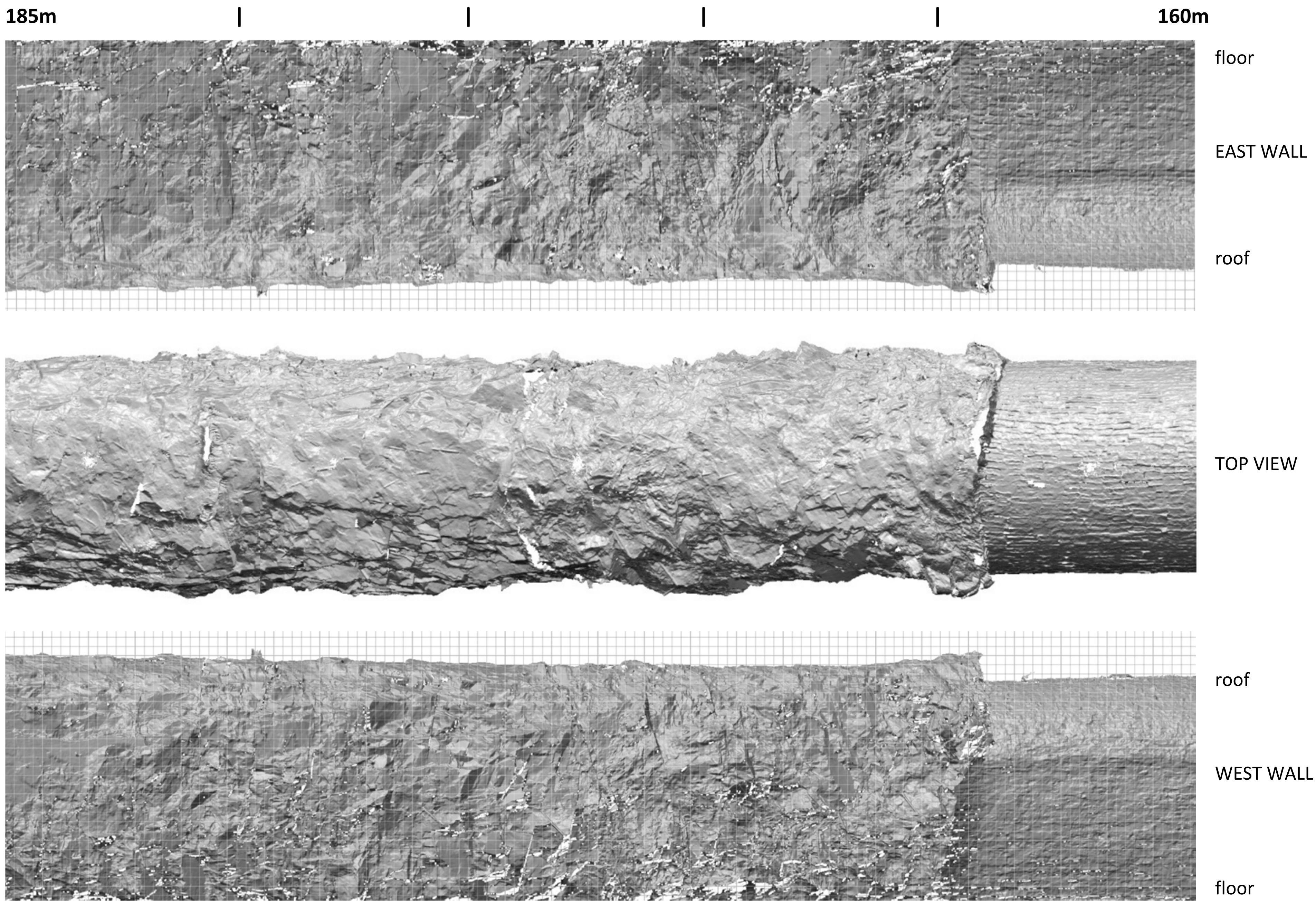
Example lidar intensity images. Left: Lidar survey instrument; (mid) inside view of rock-brick interface; Right: transition between styles of brick work in walls and in the roof.

The data can also be represented as a continuous meshed surface. This representation highlights geological structure but is also useful for identifying brick areas with missing mortar as the mesh cannot form where there is a void between the bricks. The following pages contain full meshed representations of the rock seciton of the tunnel. These contain a view from above and an outside view of both walls. The full tunnel is represented in Appendix B to the accompanying Inspec-Sol Report. In addition, fly-through videos from the model are included.

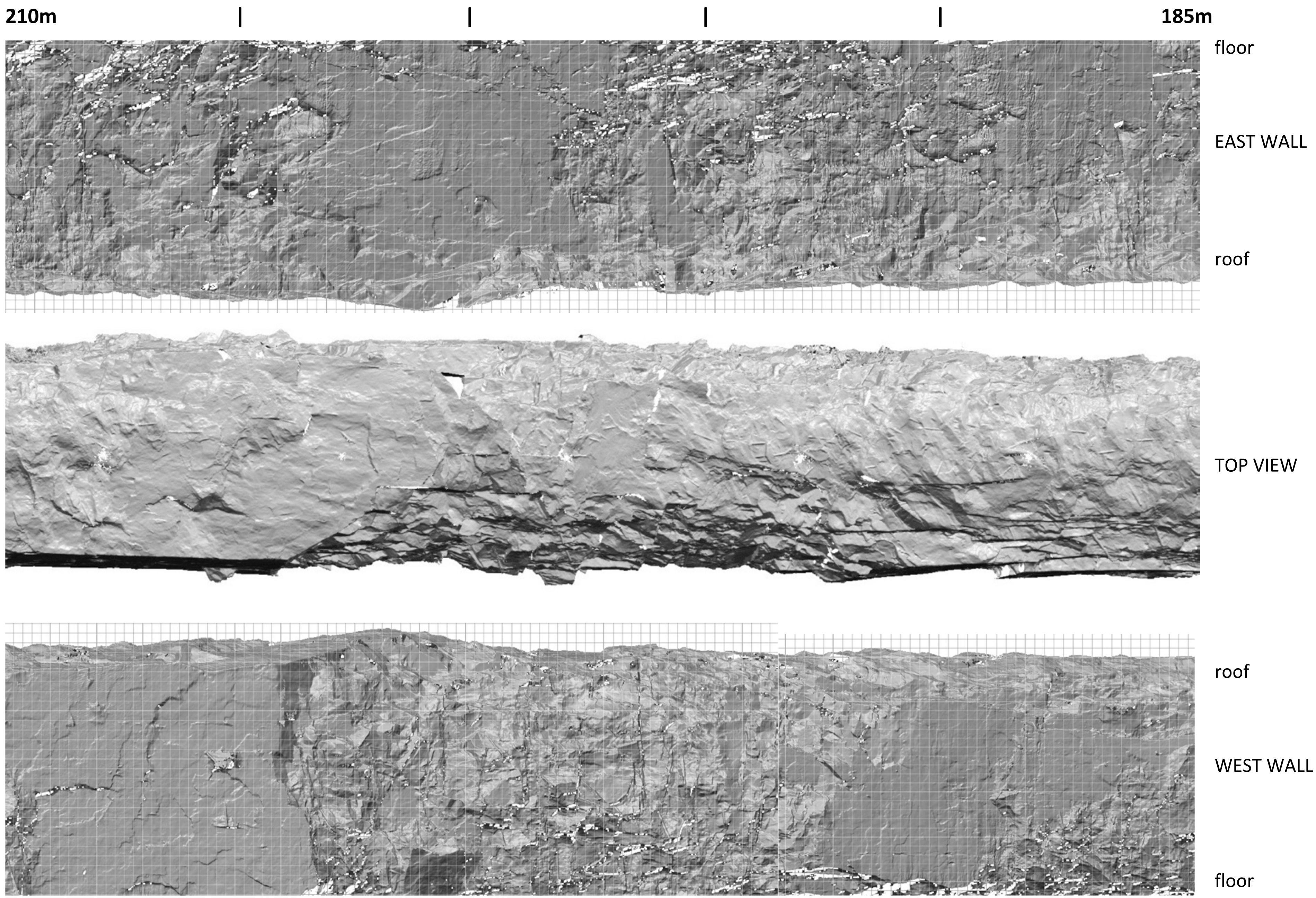


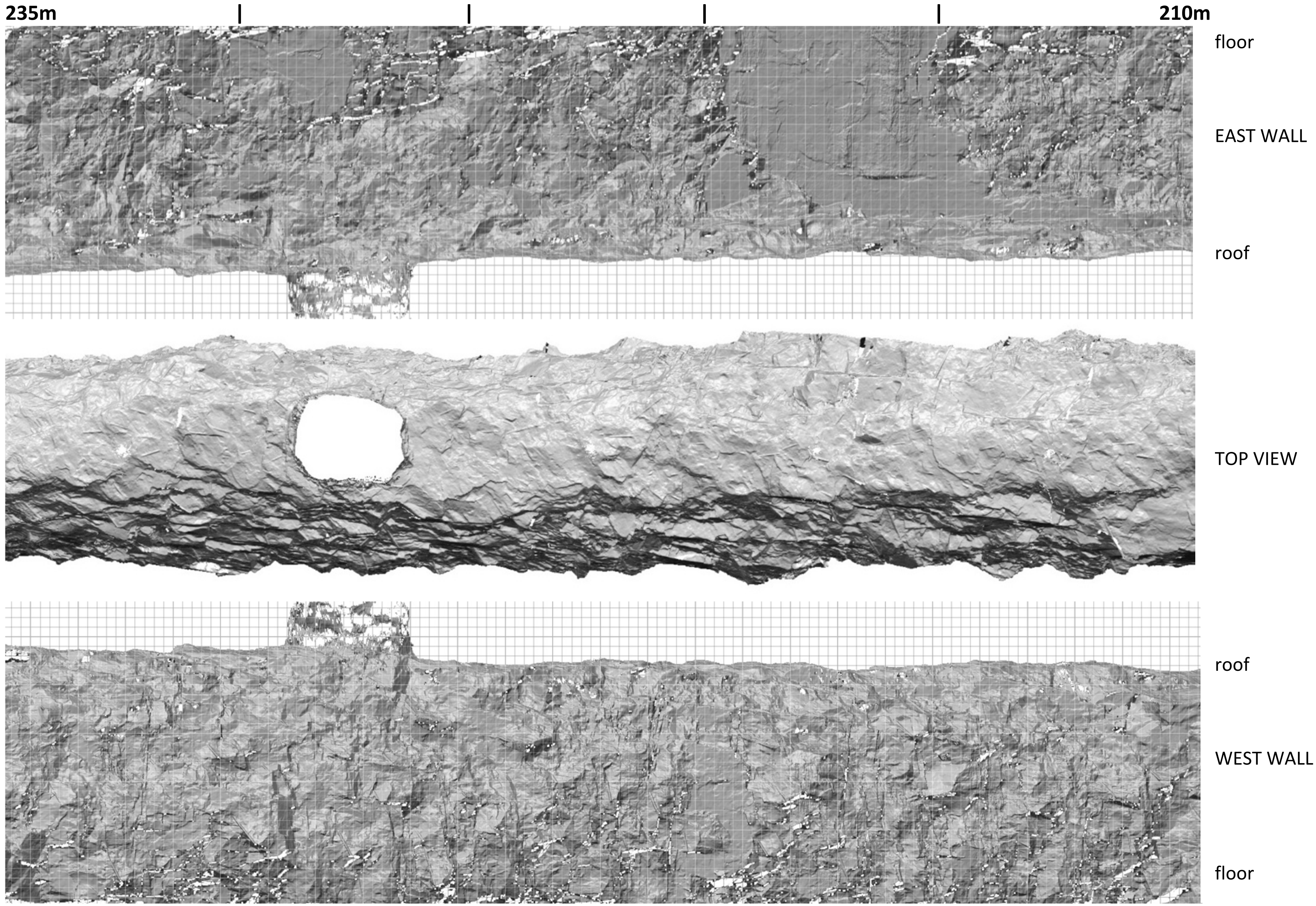
Lidar meshed models of brick/rock interface at +165m. Left image is an outside view, right image is an inside view. The 2D model can be viewed from anywhere in space.

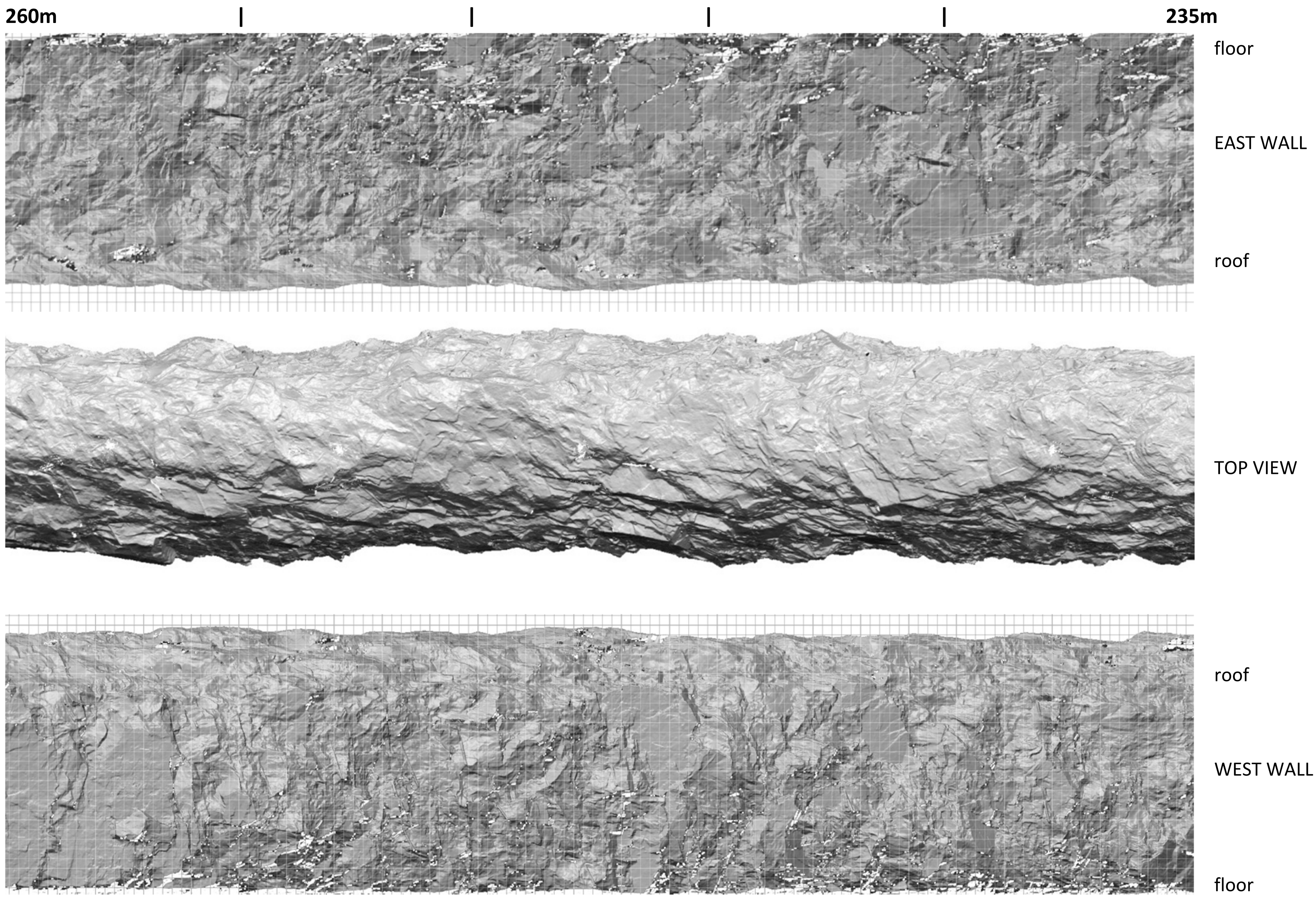

160m
135m
EAST WALL
TOP VIEW
WEST WALL

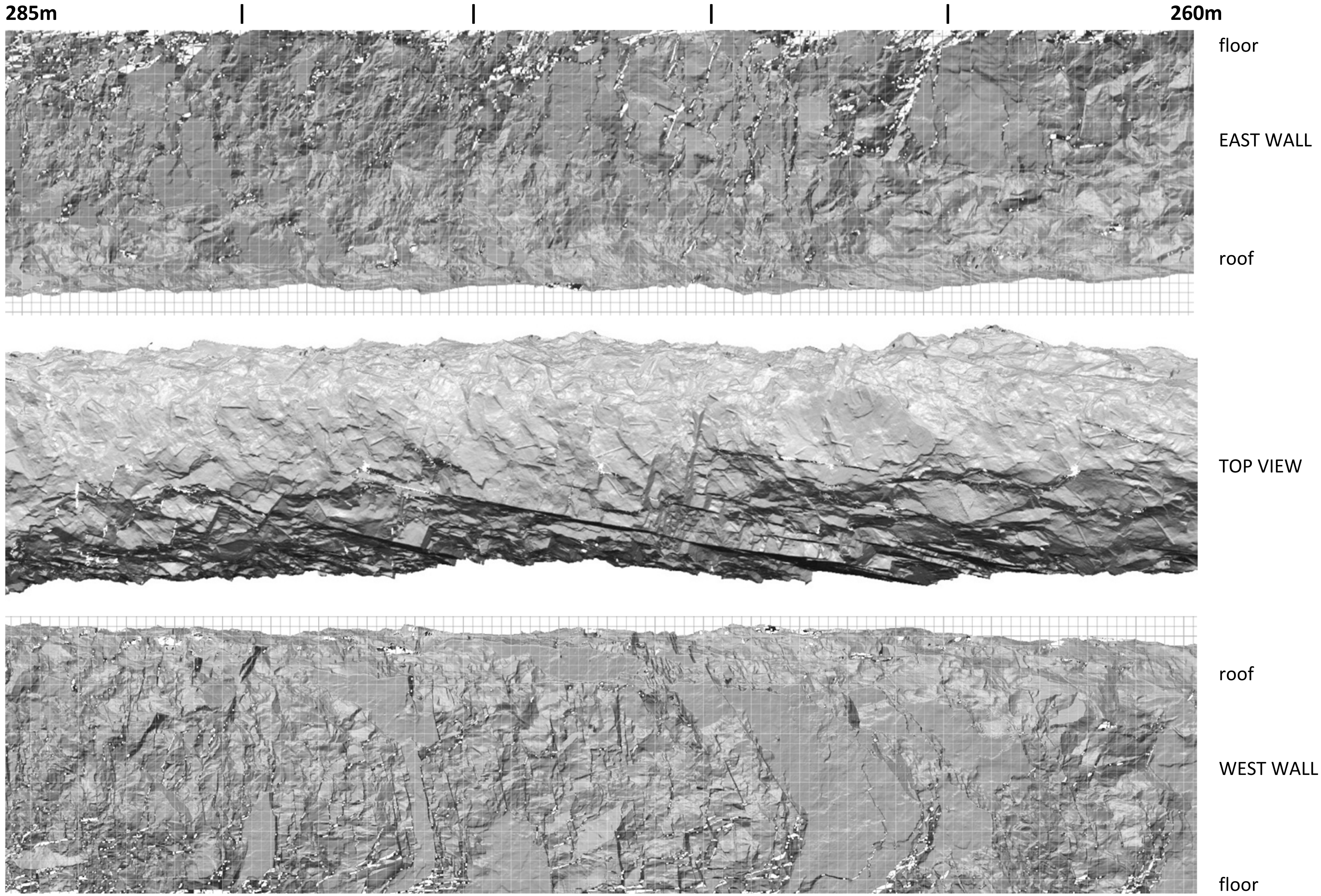


185m
160m
floor
EAST WALL
roof
TOP VIEW
roof
WEST WALL
floor

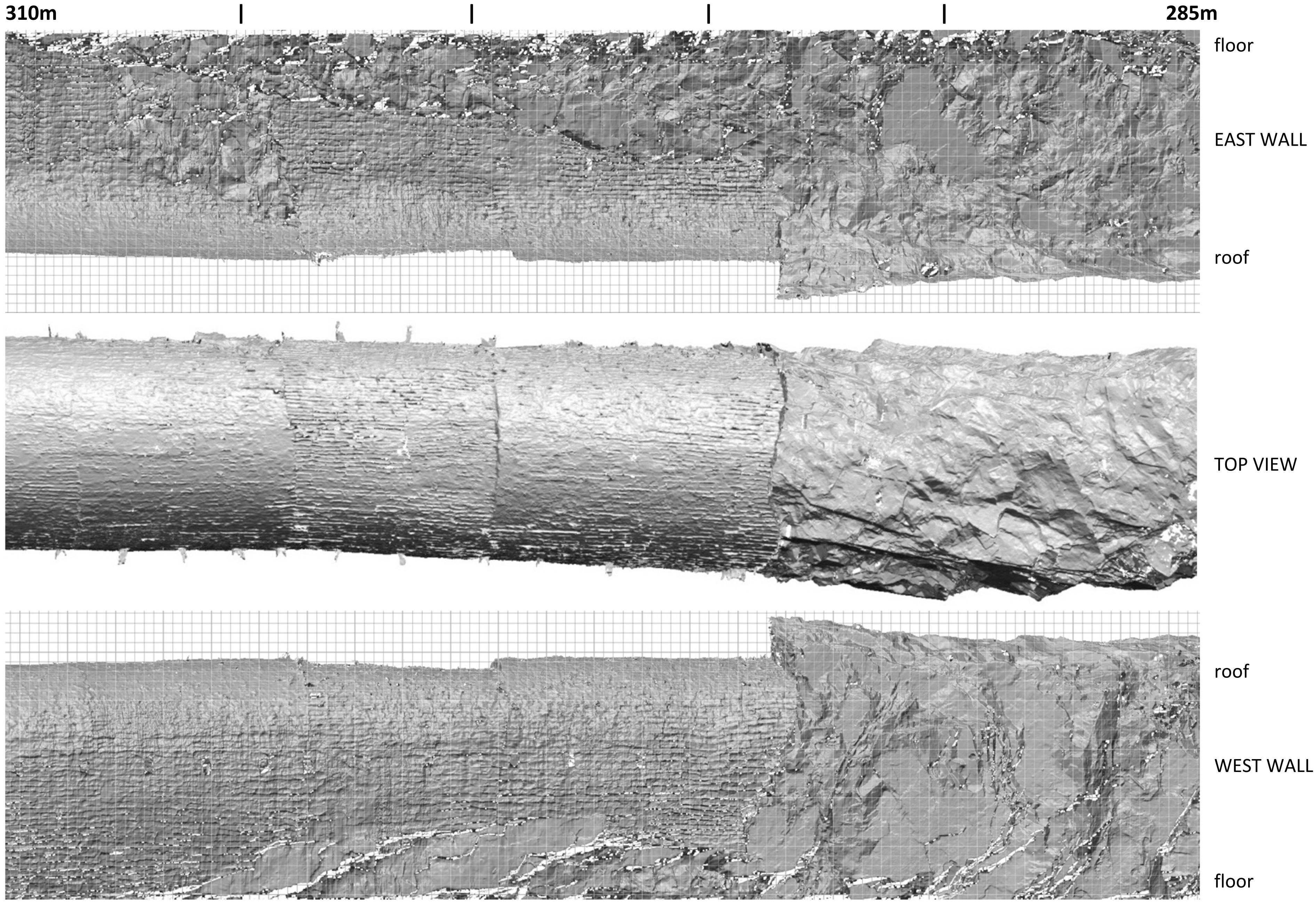

210m
185m
floor
EAST WALL
roof
TOP VIEW
roof
WEST WALL
floor


235m
210m
floor
EAST WALL
roof
TOP VIEW
roof
WEST WALL
floor


260m
235m
floor
EAST WALL
roof
TOP VIEW
roof
WEST WALL
floor



285m
260m
floor
EAST WALL
roof
TOP VIEW
roof
WEST WALL
floor

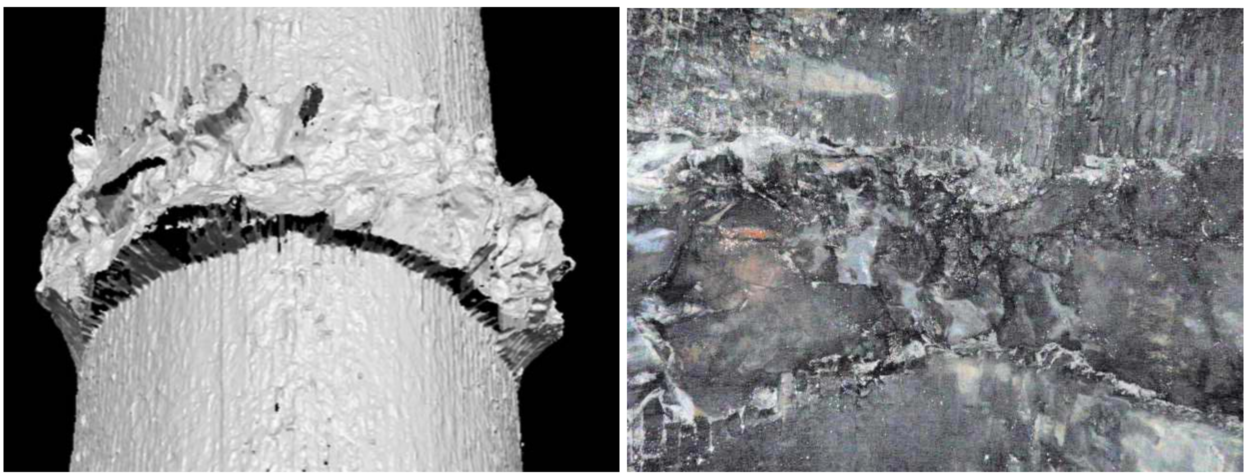


310m
285m
floor
EAST WALL
roof
TOP VIEW
roof
WEST WALL
floor

BEDROCK CONDITION SURVEY

A physical survey was performed by Dr. Diederichs and Dr. Hutchinson in addition to careful examination of the Lidar survey. Key points with respect to bedrock condition and support requirements are presented below.

Advancing into the tunnel from the south, the first bedrock exposure that is encountered is a narrow 1m window in the brickwork (deliberately created during construction) at approximately 139m from the south portal. This rock is very blocky, fractured and weathered. Stained gouge is present inside some of the fractures. This is likely adjacent to a regional shear zone (common in the Brockville area). It would appear that this was a trial excavation finish. The brickwork resumes after this slot. This window of bedrock will require scaling. The slot is narrow enough that additional support may not be required. The brick edges on either side of the slot are in good condition and can be left as an example of the lining technique.

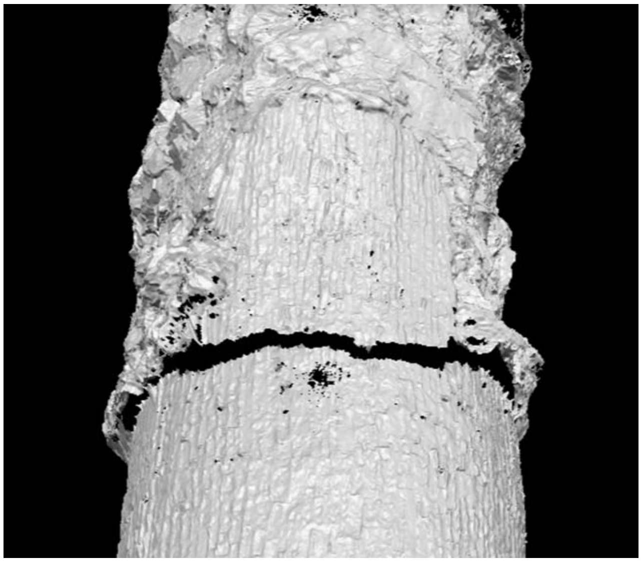


Lidar and photograph of the bedrock window at 139m into the tunnel from the south end.

The next exposed bedrock section begins at 154m. There is a partial lining in the roof or the mortar impressions of a lining that has since been removed. This rockmass is heavily jointed with a reduction in block size. There is no immediate sign of dangerous loose but this section will require scaling and possible bolting. Evidence of a fault zone appears at the end of this section (160m). This section and the brick edge will require special attention and reinforcement. The edge of the brickwork entering and leaving this section (to 160m) will need careful inspection and possible stabilization. There is moderate inflow and some flow deposits in this section.

There is a short section of brickwork between 160m and the full exposed rock section at 167m. This is likely in response to the fault zone detected at 160m. The brick liner appears to be continuous and with limited disturbance. Horizontal offset of bricks along length of lined section is likely due to construction techniques.

The last brick section in the south ends at 167m where more evidence of the boundaries of a fault zone are present. The rockmass at the edge of the brickwork is blocky with significant enlargement of the tunnel profile. The brick edge will require some attention (backfilling with grout and pointing) for long term stabilization although this brick edge can remain exposed.



(Left) lidar image of the brick-rock interface at 154m and (right) at 160m and 167m

From 168m to 200m the rockmass shows a more competent but still blocky nature. The walls and roof are stable but may require spot scaling. It will be important not to over-scale in such a shallow tunnel (even stable blocks may sound “loose” without confinement). Doing so may induce instabilities that do not currently exist. There is a possible fault or contact between 170 and 175m shown on the Lidar scans. North of this location the jointing in the rock changes dramatically to include highly persistent vertical joints parallel to the tunnel and well developed horizontal jointing. This is likely the contact between the gneissic units to the south and the quartzite (all Precambrian units underlying the more recent sandstone and carbonate units for which Brockville architecture is famous). It is possible that this folded contact reappears in sections of the tunnel to the north (as predicted by regional mapping) but most of the tunnel is in quartzite.

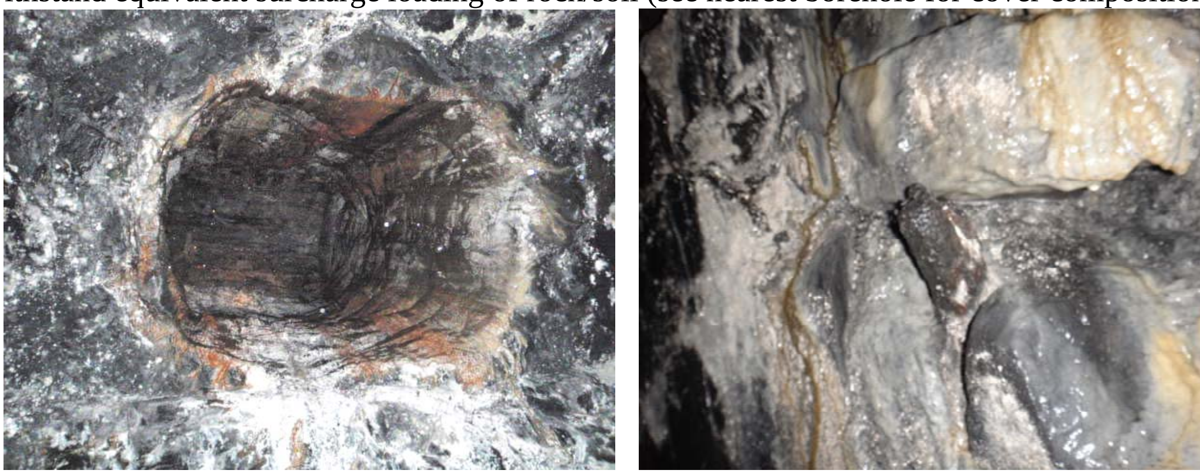
A concentrated flow of water occurs from a point location in the roof at 180m. This flow has been observed to be constant and does not vary with weather or precipitation history. This could possibly indicate a municipal source. Significant precipitate deposits have developed beyond 185m.

The only recent failure (since closure in 1970) is observed at 200m where flat blocks of quartzite (4 blocks totaling less than .3 cubic metres) have peeled off the roof where a well developed horizontal joint has been intersected by a small shear structure. Careful sounding and scaling will be required to eliminate any similar blocks in the roof. Particular care should be taken where strong horizontal jointing is visible in the roof. The tunnel profile from 180m to 220m is entirely controlled by strong vertical jointing in the parallel to the walls and strong horizontal jointing in the roof. With the exception of small slabs that merit careful scaling, the tunnel, while enlarged and squared off, is stable in this section. This is the most visually impressive section of the geology within the tunnel and care should be taken to preserve this feature while maintaining safety.

After 220m the rock becomes blocky again with some scaling requirements and possible bolting. Particular areas of concern occur at 222m, 240m, 255m and 265m (scaling and bolting possibly required). A fault crosses the tunnel at 272m with minimal impact on stability. In general beyond

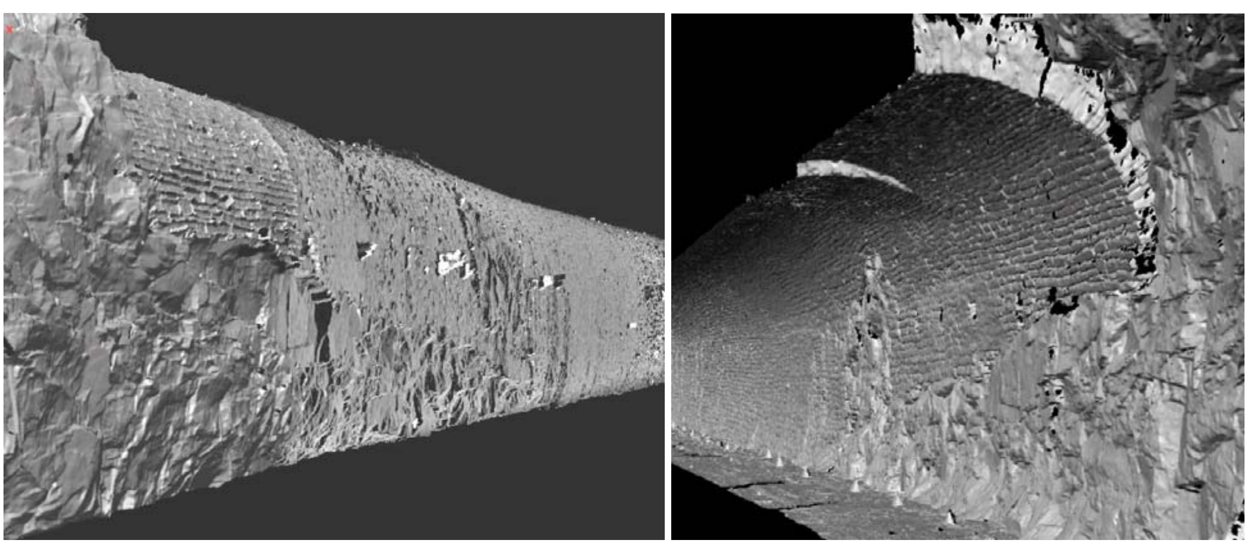
240m the major vertical structure rotates to a trend (strike) of 15 to 20 degrees with respect to the tunnel. This leads to more blocky nature and wedge fallout in the east wall.

A shaft at 230m (presumably for rock removal) is in poor condition and will need stabilization. A timber cover appears at the top of the visible shaft. It is unknown what overlies this cover. The tunnel stability is not affected but the shaft itself poses a hazard and will need to be covered in some way (possible with plexiglass to allow viewing and lighting). The top of the visible shaft will require a new bulkhead. A prudent step would be to engineer a reinforced bulkhead (cast against the existing backing) anchored and positively abutted on the shaft walls with capacity to withstand equivalent surcharge loading of rock/soil (see nearest borehole for cover composition).



(Left) Rock Hoisting Chimney at 230m. (Right) Corroded iron bar holding brickwork at 295m

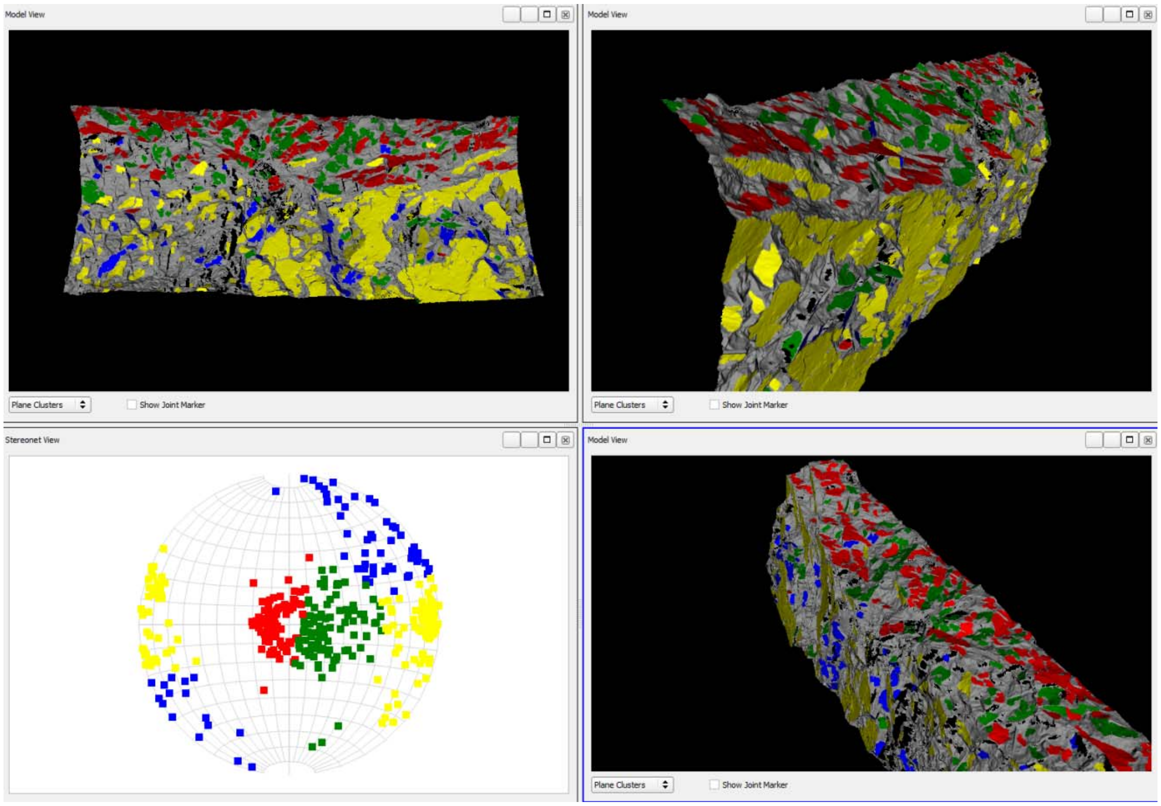
The rock section continues to 295m, with a joint controlled irregular profile and significant flow deposits, where the dug tunnel in till begins with stone brickwork. The leading edge of the brickwork at the brick-rock interface is highly irregular. The brick arch abutment climbs up on a blasted rock edge from the floor at 308m to half height at 295m. The leading edge at 295m is supported by highly corroded rockbolts from the original construction. While the rock is stable, the tapered base of the brickwork and the interface at 295m requires significant work to stabilize, buttress and replace the aging rockbolts.



Tapered interface between rock and brickwork in northern soft tunneling section.

STRUCTURAL (JOINT) ANALYSIS

The lidar scans were processed using a new algorithm from the Norwegian Geotechnical Institute (Lato et al 2011). This automated joint detection and mapping procedure was applied to three sections.



Example of mapping process using lidar scanning. (planes are identified as indicated by colours).

Results of Joint Analysis

Major planes identified from chainage 160 to 190m (from south end).

1)	Dip= 89	Dip Direction= 89 degrees from tunnel axis (north)
1)	Dip= 6	Dip Direction= 239
1)	Dip= 87	Dip Direction= 18

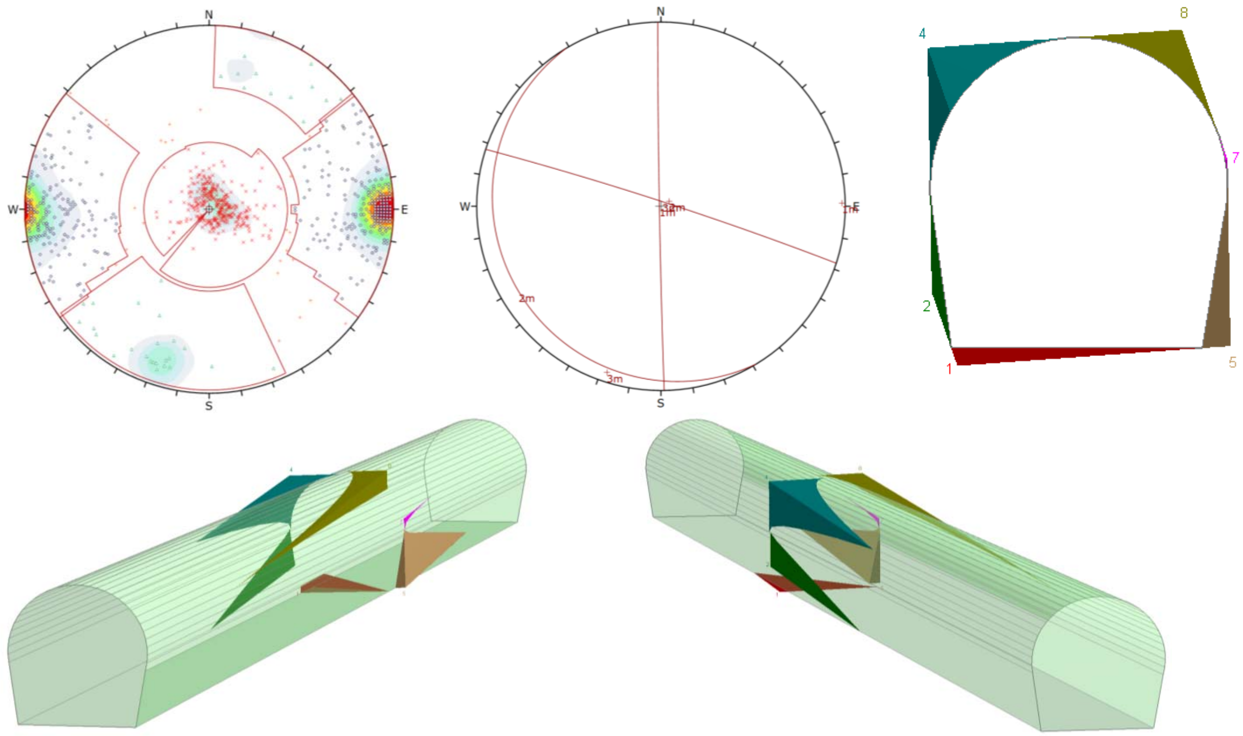
Major planes identified from chainage 160 to 190m (from south end).

1)	Dip= 87	Dip Direction= 274 degrees from tunnel axis (north)
1)	Dip= 3	Dip Direction= 155
1)	Dip= 82	Dip Direction= 196

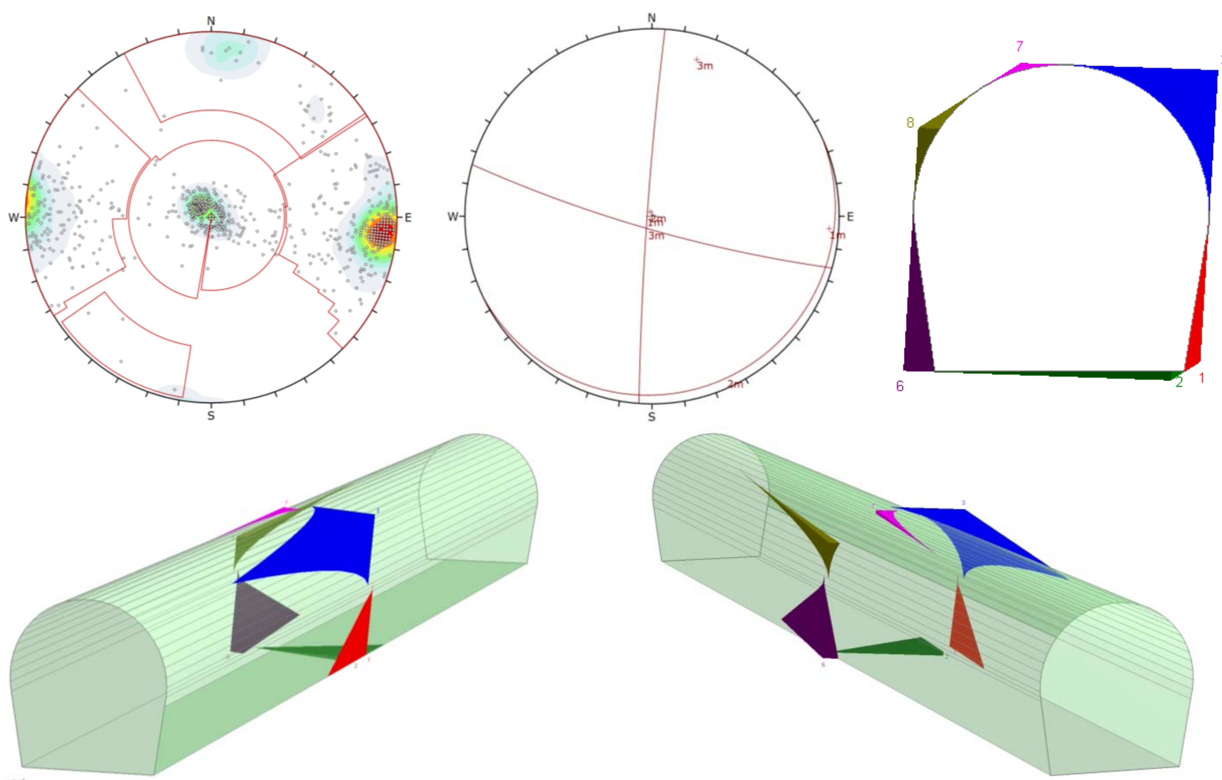
Major planes identified from chainage 160 to 190m (from south end).

1)	Dip= 86	Dip Direction= 280 degrees from tunnel axis (north)
1)	Dip= 2	Dip Direction= 155
1)	Dip= 76	Dip Direction= 210

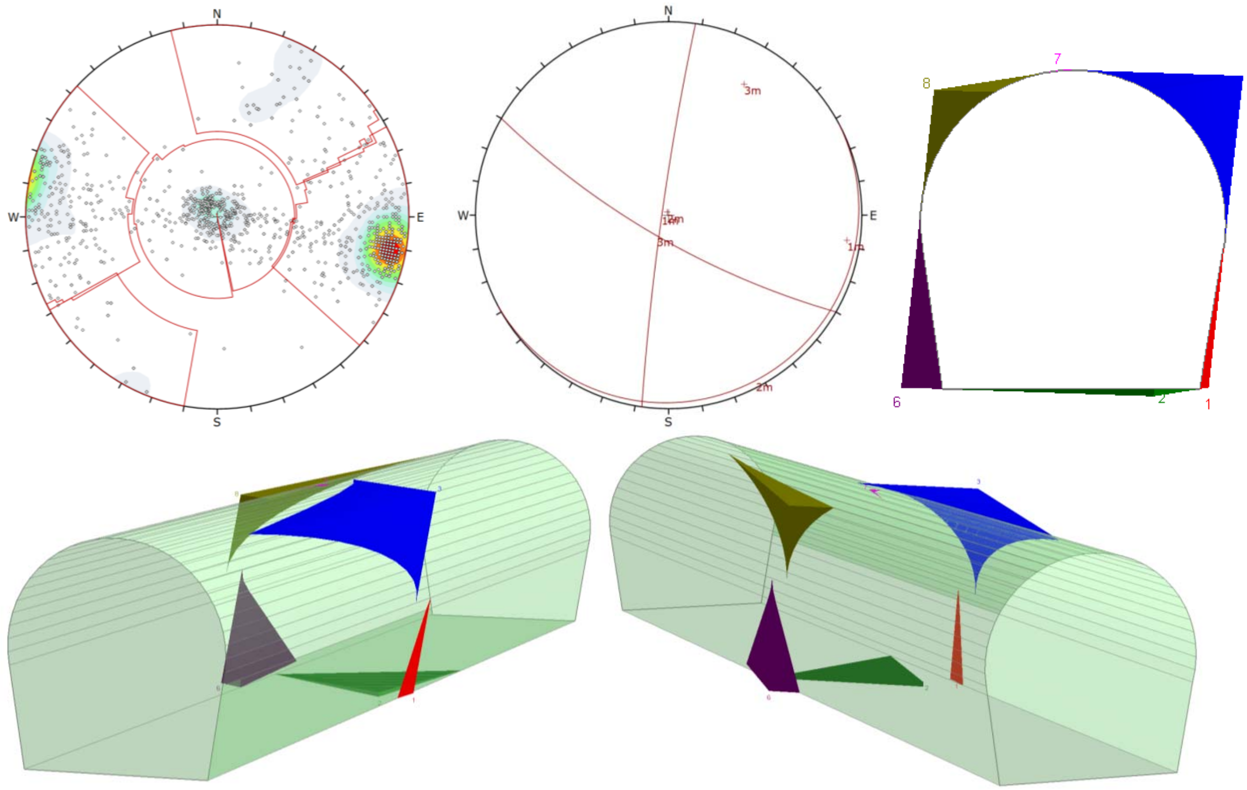
Results of Wedge and Kinematic Analysis



Joint sets and wedges (blocks) formed in an ideal tunnel profile. 160-200m (max localized bolt loading = 0.75 tonnes/m^2 of face area.)



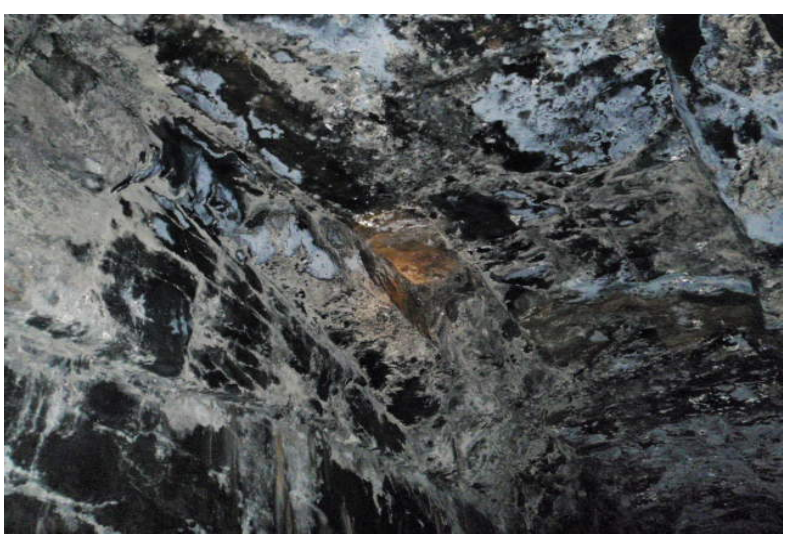
Joint sets and wedges (blocks) formed in an ideal tunnel profile.200-240m (max localized bolt loading = 0.85 tonnes/m^2 of face area.)



Joint sets and wedges (blocks) formed in an ideal tunnel profile. 240-290m
(max localized bolt loading = 0.9tonnes/m^2 of face area.)

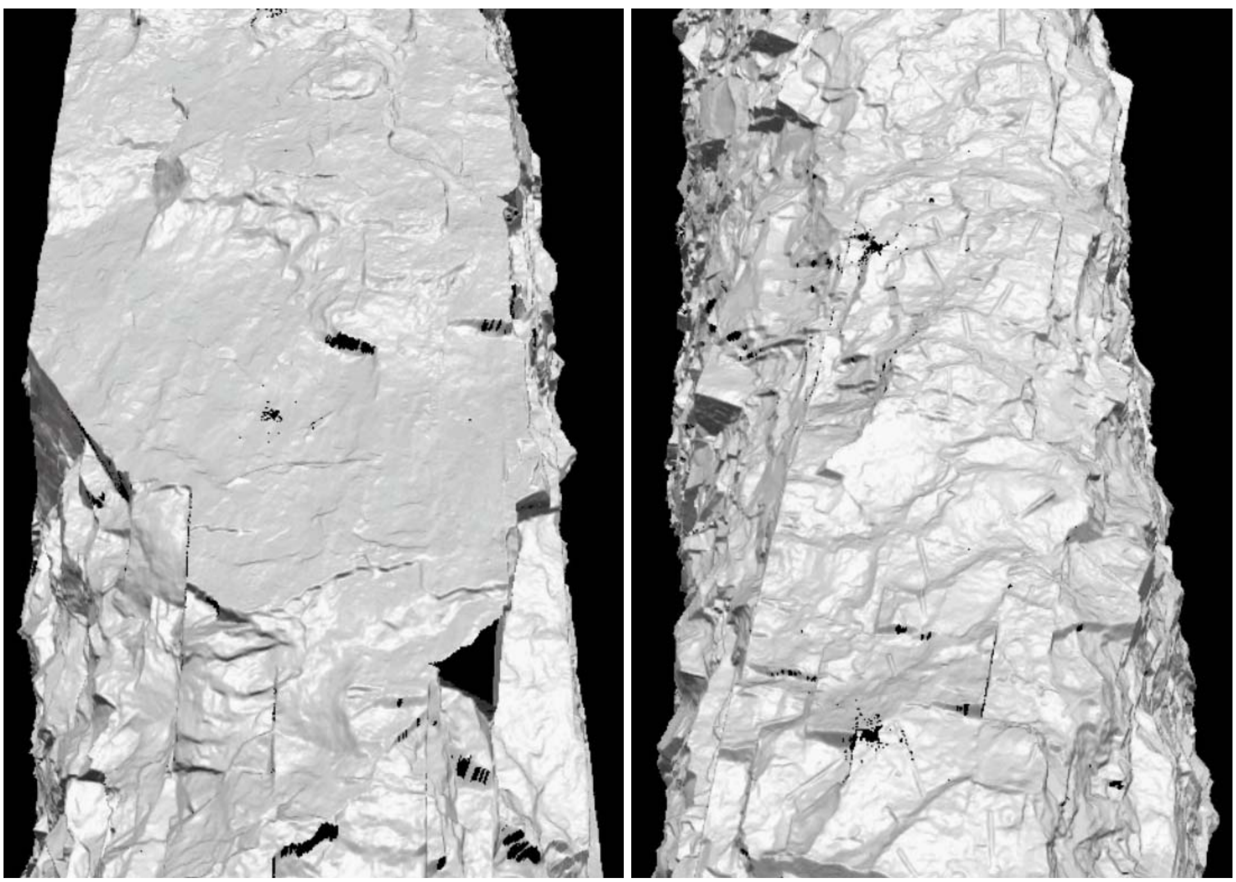
The maximum distributed bolt load to maintain the originally intended arch profile would have been less than 1 tonne per square metre. This is equivalent to a standard grouted rebar bolt on 3x3 metre spacing assuming the unstable blocks are evenly distributed. A 2x2m spacing would give a factor of safety of greater than 2. However, most of the unstable blocks formed by this profile have already fallen out (likely at excavation). Only one location has had minor block fallout since 1970. Thus only spot bolting may be required to ensure long term stability.

Examples of Structural Control



Typical horizontal and vertical structure in the roof leading to square corner.

Vertical wall joints at +220m



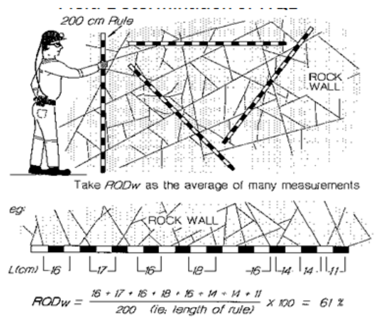
Overhead Lidar image of structural character at 205m and at 280m

ROCKMASS CLASSIFICATION

Three systems of classification are appropriate for use in this tunnel: RQD (Rock Quality Designation), RMR (Bieniawski's Classification) and Q (Barton's Classification)

RQD (65-85 for Brockville Tunnel – Mean =75):

Rock Quality Designation	RQD Value
Very Poor	0 - 25
Poor	25 - 50
Fair	50 - 75
Good	75 - 90
Excellent	90 - 100



RMR (45-60 for Brockville Tunnel – Most Common =55)

Rock Mass Class	Description	RMR
I	Very Good Rock	**81 - 100**
II	Good Rock	**61 - 80**
III	Fair Rock	**41 - 60**
IV	Poor Rock	**21 - 40**
V	Very Poor Rock	**0 - 21**

See following pages

Q (1-10 for Brockville Tunnel – Most Common = 4)

Tunnelling Quality Index, Q	Rockmass Description
0.001 - 0.01	Exceptionally Poor
0.01 - 0.1	Extremely Poor
0.1 - 1	Very Poor
1 - 4	Poor
4 - 10	Fair
10 - 40	Good
40 - 100	Very Good
100 - 400	Extremely Good
400 - 1000	Exceptionally Good

See following pages

The classification and support recommendations that arise from the Q system in particular would have been applicable at the time of construction. In order to preserve the intended arch profile, a system of pattern bolting (2m bolts on 1.5m spacing) would have been required. Half of the tunnel would likely have been stable without support if good blasting practices were employed.

However, due to blasting and due to the unsupported nature of the original, most of the rock blocks that would have fallen out of the roof have already done so. There has been one small rockfall in the last 50 years according to observations in the tunnel (assuming no cleanup effort has been mounted since then). The tunnel now is self-stable in its current profile. The only rock related hazard is the potential for additional fallout of partial slabs from the sides of the roofand small blocks tumbling from the wall. Spot bolting may be necessary in addition to careful and discriminant scaling.

Rock Mass Rating System (After Bieniawski 1989).

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Range of values							
1	Strength of intact rock material	Schmidt Hammer	>60	45-60	30-45	15-30	5-15	<5	na	
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa	
		Rating	15	12	7	4	2	1	0	12 A
2	Drill core Quality *RQD*		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%			
		Rating	20	17	13	8	3			16 B
3	Spacing of discontinuities		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm			
		Rating	20	15	10	8	5			10 C
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous			
		Rating	30	25	20	10	0			21 D

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

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

	Parameter		Range of values					
5	Ground water	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125	
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1, - 0.2	0.2 - 0.5	> 0.5	
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing	
		Rating	15	10	7	4	0	8 E

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable	
Ratings	Tunnels & mines	0	-2	-5	-10	-12	12 F
	Foundations	0	-2	-7	-15	-25	
	Slopes	0	-5	-25	-50		

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip - Dip 45 - 90°	Drive with dip - Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favourable	Favourable	Very unfavourable	Fair
Drive against dip - Dip 45-90°	Drive against dip - Dip 20-45°	Dip 0-20 - Irrespective of strike°	
Fair	Unfavourable	Fair	

RMR = A + B + C +D + E - F 45-60 **55**

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)	> 45	35 - 45	25 - 35	15 - 25	< 15

Q System for Rock Quality and Support (to preserve original profile at time of construction)

ROCK QUALITY DESIGNATION

	RQD		
		min	65
Very Poor	0-25	*RQD =*	75
Poor	25-50	max	85
Fair	50-75		
Good	75-90		
Very Good	90-100		

When RQD < 10, a value of 10 will be used for Q

JOINT SET NUMBER

	Jn		
		min	6
Massive Rock	0.5	*Jn =*	6
Random Joints Only	1	max	9
One Joint Set	2		
One Set + Random	3		
Two Joint Sets	4		
Two Sets + Random	6		
Three Joint Sets	9		
Three Sets + Random	12		
Four Joint Sets	15		
Crushed or Earthlike	20		

JOINT ROUGHNESS NUMBER

	Jr		
		min	1.5
Discontinuous	4	*Jr =*	1.5
Undulating and:		max	1.5
Rough /Irregular	3		
Smooth	2		
Slickensided	1.5		
Planar and:			
Rough /Irregular	1.5		
Smooth	1		
Slickensided	0.5		
No Rock Wall Contact	1		
Spacing Greater than 3m	(Add 1.0)		

JOINT WATER NUMBER

	Jw		
		min	0.7
Dry (< 5L/m)	1	*Jw =*	0.7
Medium	0.7	max	1
Large Inflow	0.3-0.5		
Exceptional with decay	0.1-0.2		
Exceptional no decay	0.05-0.1		

JOINT ALTERATION NUMBER

	Ja		
		min	1
Tightly Healed	0.75	*Ja =*	1.5
Unaltered; Surface Staining Only	1	max	3
Slightly Altered; Non softening Coating	2		
Silty or Sandy Clay Coatings	3		
Softening or Low Friction Mineral Coating	4		
Minor Swelling Clay <1mm thick	4		
Sandy Clay Free Gouge <1mm	4		
Non softening Fill < 5mm thick	6		
Softening Clay Fill < 5mm thick	8		
Swelling Clay Fill < 5mm thick	8-12		
Thick Infilling > 5mm	6-24		

STRESS REDUCTION FACTOR

(Stress/Intact Stength)	σ1/UCS	SRF1		
			min	1
Near Surface (Low Stress)	~0	2.5-5	*SRF1 =*	2
Low Stress	0.01 to 0.1	1	max	2.5
Medium Stress	0.1 to 0.2	0.5		
High Stress	0.2 to 0.3	1 to 2		
Mild Burst or Squeezing	0.3 to 0.4	5 to 10		
Heavy Bursting or Squeezing	>0.4	10 to 20		
Very Heavy Bursting/Squeezing	>0.6	20 to 50		
Mild Swelling		5 to 10		
Heavy Swelling		10 to 15		

(Factor for Discrete Structure)	SRF2		
		min	1
No discrete structure other than joints	1	*SRF1 =*	1
Multiple weakness zones with clay	10	max	1
Single weakness zone with clay (shallow)	5		
Single weakness zone with clay (at depth)	2.5		
Multiple shear zones (clay free)	7.5		
Single shear zone (clay free & shallow)	5		
Single shear zone (clay free & deep)	2.5		
Loose open joints (sugar cube)	5		

SRF1 vs SRF2:		
	min	1
Q System uses the (greater) value so SRF =		2
	max	2.5

OTTAWA LRT - TUNNELS: New Rideau Station to East Portal

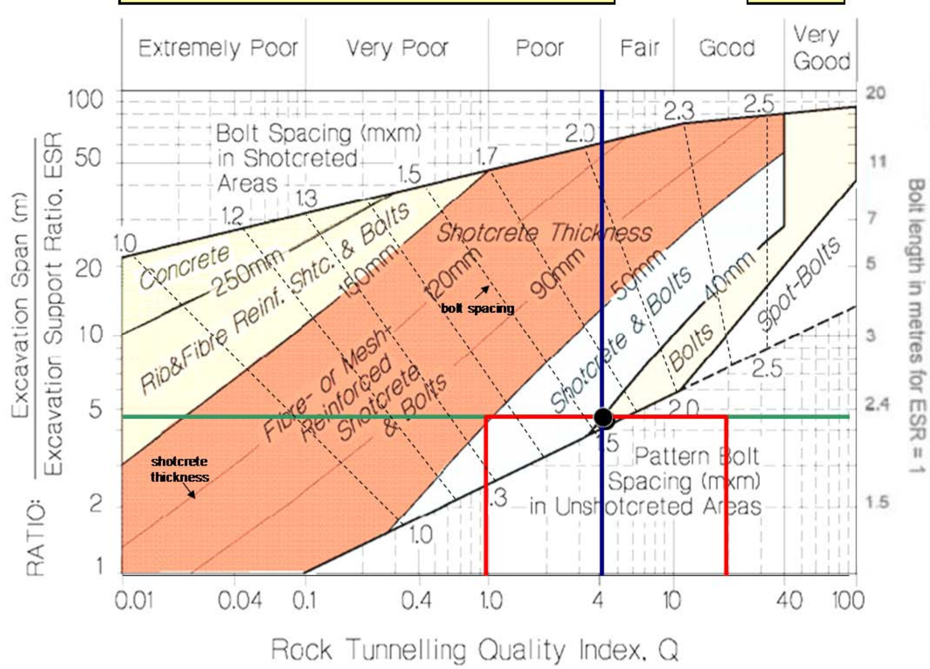
Bartons Q Value	Support Pressure		Block Size		Joint Volume	
Q= 4.38	P(roof)=	0.666	Blk Size=	12.50	Jv=	12.12
	10m span	Mpa		cm		#/m3

Innovative Geomechanics - ***Q*** *Rockmass Assessment &Support Guidelines (based on empirical Q system by Barton et al.)*

PROJECT: BROCKVILLE TUNNEL: ORIGINAL SUPPORT RECOMMENDATIONS

Date: OCT 30/2012

Survey by Mark S Diederichs



$D_e = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation Support Ratio } ESR}$

The value of *ESR* is related to the intended use of the excavation and to the degree of security which is demanded of the support system installed to maintain the stability of the excavation. Barton et al (1974) suggest the following values:

	Excavation category	*ESR*
A	Temporary mine openings.	3-5
B	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations.	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels.	1.3
D	Power stations, major road and railway tunnels, civil defence chambers, portal intersections.	1.0
E	Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

Span=	4.5
ESR =	1

min	1.011111
Q =	4.375
max	21.25

Max to Min — Q

COMMENTS: NOTE THAT THIS SURVEY IS FOR SUPPORT REQUIRED TO RETAIN THE ORIGINAL TUNNAL PROFILE. THE TUNNEL WAS UNSUPPORTED AND SO 50% OF THE TUNNEL SHOWS BLOCK FALLOUT. THE CURRENT PROFILE IS INHERENTLY STABLE AS A WHOLE WITH THE EXCEPTION OF POTENTIALLY LOOSE SMALL BLOCKS LOCALLY (MINOR SCALING AND SPOT BOLTING ONLY). Local Faults if present reduce Q by 50%.

CONCLUSIONS

There are a number of rehabilitation steps that will need to be taken in the rock section of the tunnel. In order of priority they are summarized here:

- The brick-rock transition at the north end of the rock section will require significant strengthening and rehabilitation. This will include active reinforcement of the base of the bricks as they contact the rock ledge. This may involve a combination of short bolting and a concrete sill. Brickwork in the ceiling at this location will also require attention.
- The rock chimney at 230m will require attention. The chimney itself is stable. The bulkhead at the top is of unknown construction and it is uncertain what lies above. It may be desirable to keep the chimney exposed for historical purposes but the walls need scaling and an upper bulkhead needs to be constructed just below the existing back.
- The rock-brick interfaces throughout will require detailed examination and will need rehabilitation and reinforcement ranging from grout backfilling and repointing to the construction of a light reinforced arch (steel and concrete) to protect the brick edge and provide long term stability. This can be done with sensitivity to the aesthetic and historical requirements as well as budgetary constraints.
- Rock scaling is required in some portions of the tunnel. Rock sounding and very careful and discriminant scaling is probably advisable through the whole rock section although caution is required to avoid overscaling. This tunnel is unique in that most rock that could fall has already fallen and aggressive scaling will create rather than solve problems. Only clearly loose and potentially unstable blocks should be dislodged through scaling.
- Spot bolting may be required. There does not appear to be any justification for pattern bolting. The rockmass quality would normally require such bolting for a new tunnel if the design arch profile is to be achieved. This requirement is moot for this tunnel as the rock has already broken back to a stable albeit irregular profile. For costing purposes, it is reasonable to assume that up to 1 bolt every 2 linear metres of tunnel may be required (~75 bolts). In this case 1.5 to 2m resin grouted rebar are recommended.
- The use of shotcrete is not advised except as an option for reinforcement of brick-rock interfaces. It is important to maintain the current level of tunnel drainage in both the rock and brick sections as buildup of water pressure could lead to new stability issues.
- The water seeping through the rock and precipitate formations have little or no impact on rock stability. The Precambrian gneiss and quartzite are insoluble. The minerals are coming from the soil and from the Ordovician rock units above and to the east.

In general, the rock in the tunnel is strong gneiss and quartzite. The joints are rough and tight. For the most part the major joint spacing is on the order of half a meter. This is sufficient to provide beam capacity in the roof and stable walls. Intersections between the angled joints and the tunnel profile have resulted in the current profile which, although ragged with significant apparent overbreak, has been stable for at least 50 years with only one minor rockfall incident. There is therefore little justification for a major stabilization campaign. The only potential hazards include the brick-rock interface sections (due to possible brick deterioration) and the potential for fallout of small blocks from the roof or walls due to disturbance. Careful scaling and possible spot bolting will remedy these hazards.

A preliminary estimate for preparation, access, scaling, bolting and shaft rehab is $350,000.

Appendix C

- Representative Site Photographs

Appendix D

- Lidar Images and DVD with Lidar Scans

Appendix E

- Chemical Analyses of Flow Deposits and Groundwater

Appendix B

- Rock Engineering Report